VOLUME 2

element.

Technical Appendix A - JBS&G Environmental Assessment and Management Strategy


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Element
Environmental Assessment and Management Strategy
Forrestfield North
Transport Oriented Development Precinct
6 April 2021
57806/136,8
JBS&G Australia Pty Ltd T/A Strategen-JBS&G
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Element

Environmental Assessment and Management Strategy

Forrestfield
North
Transport Oriented Development Precinct

6 April 2021

57806/136,8
JBS&G Australia Pty Ltd T/A Strategen-JBS&G

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1. Introduction

1.1 Overview

In response to the State Government's Forrestfield-Airport Link Project, the City of Kalamunda (CoK) is facilitating the design and implementation of the development of the Forrestfield North District Structure Plan (DSP) process. The DSP area is approximately 190 ha and is bound by Roe Highway to the east, Berkshire Road to the south, Dundas Road and railway to the west, and Poison Gully Creek to the north.

The DSP is a guiding framework to maximise the opportunities, including the capacity for greater diversity in housing options, employment, and economic development options, while identifying and providing for the retention of key environmental assets.

To coordinate the development, an Activity Centre Plan (ACP) and Local Structure Plan (LSP) are being developed for two designated precincts, identified on Figure 1.1:

- ACP for the TOD Precinct (TODP) (61.02 ha)
- LSP for the Residential Precinct (123.05 ha).

The TOD ACP area is approximately 61.02 ha and is bound by Dundas Road to the west, Poison Gully Creek to the north, and the Residential Precinct to the east.

This Environmental Assessment and Management Strategy (EAMS) has been prepared to support the LSP for the TOD ACP area (the Project area). The Precinct consists of the following elements (Figure 1.2):

- residential uses
- public community purpose facility
- local open space areas
- environmental conservation areas.

1.2 Previous environmental planning amendments and considerations

In 2015, the DSP area was amended from Rural to Urban under the MRS (1282/57). As part of this process the Amendment was referred to the Environmental Protection Authority (EPA). The Amendment was not assessed, and advice given. The EPA identified the following environmental factors relevant to the scheme:

- flora and vegetation- Wavy-leaved Smokebush (*Conospermum undulatum*)
- terrestrial fauna- Remnant vegetation providing roosting, foraging and potential breeding habitat for Carnaby's and Forest Red-tailed Black Cockatoos and the Quenda/Southern Brown Bandicoot.

The EPA (2016) advised that it is expected that the local scheme text was to be amended to include provisions which would contain "specific mechanisms and adequately secure, protect and manage the significant environmental values within the amendment area".

In 2016, the CoK prepared amendment 75 to Local Planning Scheme No. 3 (the Scheme) to rezone the DSP area from Light Industry, Industrial Development and Special Rural to Urban Development.

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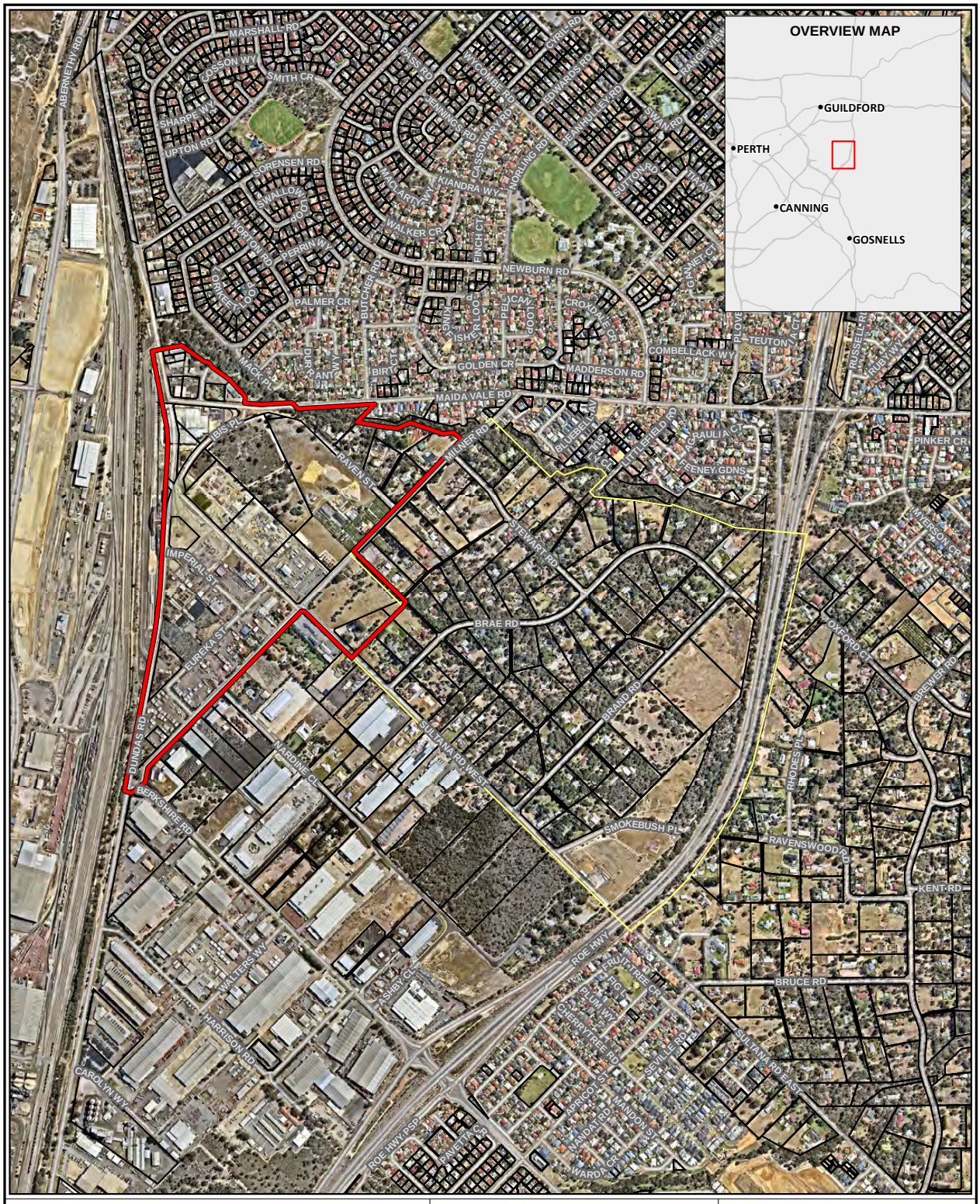
The Amendment was referred to the EPA. The scheme amendment was not assessment and advice given. The EPA (2016) provided the following advice:

- the EPA expects that further detailed management plans and scheme provisions to address flora and vegetation will be addressed prior to the finalisation of the structure plan
- the protection of declared rare flora, associated habitat, threatened fauna habitat, and low representation vegetation complexes in appropriately sized retention areas for conservation purposes. These retention areas shall be informed by Level 2 Flora and Vegetation and Fauna Surveys in accordance with EPA Guidance Statements 51 and 56 (or as revised), and targeted for Declared Rare Flora and threatened fauna, and associated habitat. The retention area size, location, protection and management mechanism shall be subject to OEPA advice prior to the WAPC endorsement of the structure plan
- all future subdivision and development proposals must be consistent with the retention areas agreed.

1.3 Purpose and scope of this document

This EAMS has been prepared to describe the environmental and heritage values within the Project area and surroundings based on existing information, and to identify potential opportunities to secure, protect, and manage the significant environmental values on site.



Legend

- Project area
- Residential precinct
- Cadastral boundary
- Roads (MRWA)

Scale 1:15,000 at A4	0 100 200 metres	**Forrestfield North TOD Precinct Perth, WA**
Coord. Sys. GDA 1994 MGA Zone 50		**PROJECT AREA**
Job No: 57806		
Client: Element		**FIGURE 1.1**
Version: A	Date: 11-Apr-2021	strategen JBS&G
Drawn By: cthatcher	Checked By: HS	

File Name: \\008PMPMR004V001.jbsg.aust\JBS Perth\Projects\1)Open\Element (TPG)\57806 - FF North TOD & Res LWMS\GIS\Maps\R05_Rev_A\57806_1_1_ProjectArea_.mxd
Image Reference: www.nearmap.com© - Imagery Date: 27 February 2021

LEGEND

Activity Centre Plan Boundary

EXISTING PLANNING FRAMEWORK

Metronet East Redevelopment Area

Project Area: Forrestfield North (RAC-0)

Ecological Conservation (Conservation Category Wetland)

Bush Forever

PROPOSED PLANNING FRAMEWORK

Region Scheme Reserves

Railway

Local Scheme Reserves and Environmental Conservation

Local Open Space

Public Purposes: Community Purpose

Environmental Conservation

Other Categories

District Integrator A (Existing / Proposed)

District Integrator B (Existing / Proposed)

Neighbourhood Connector (Existing / Proposed)

Local Street (Existing / Proposed)

Proposed Roundabout

Proposed Signalised Intersection

A Left-in, Left-out Intersection

B No Right Out Intersection

Strategic Site

Hazard seperation required at subsequent planning stages to ensure that no residential land is affected by BAL-40 or BAL-FZ

Indicative Drainage Corridor

Notes

1 Final road widths and indicative cross sections to be determined following a traffic assessment

Precincts

1 Forrestfield Station Precinct

2 Residential Precinct

Sub-Precincts

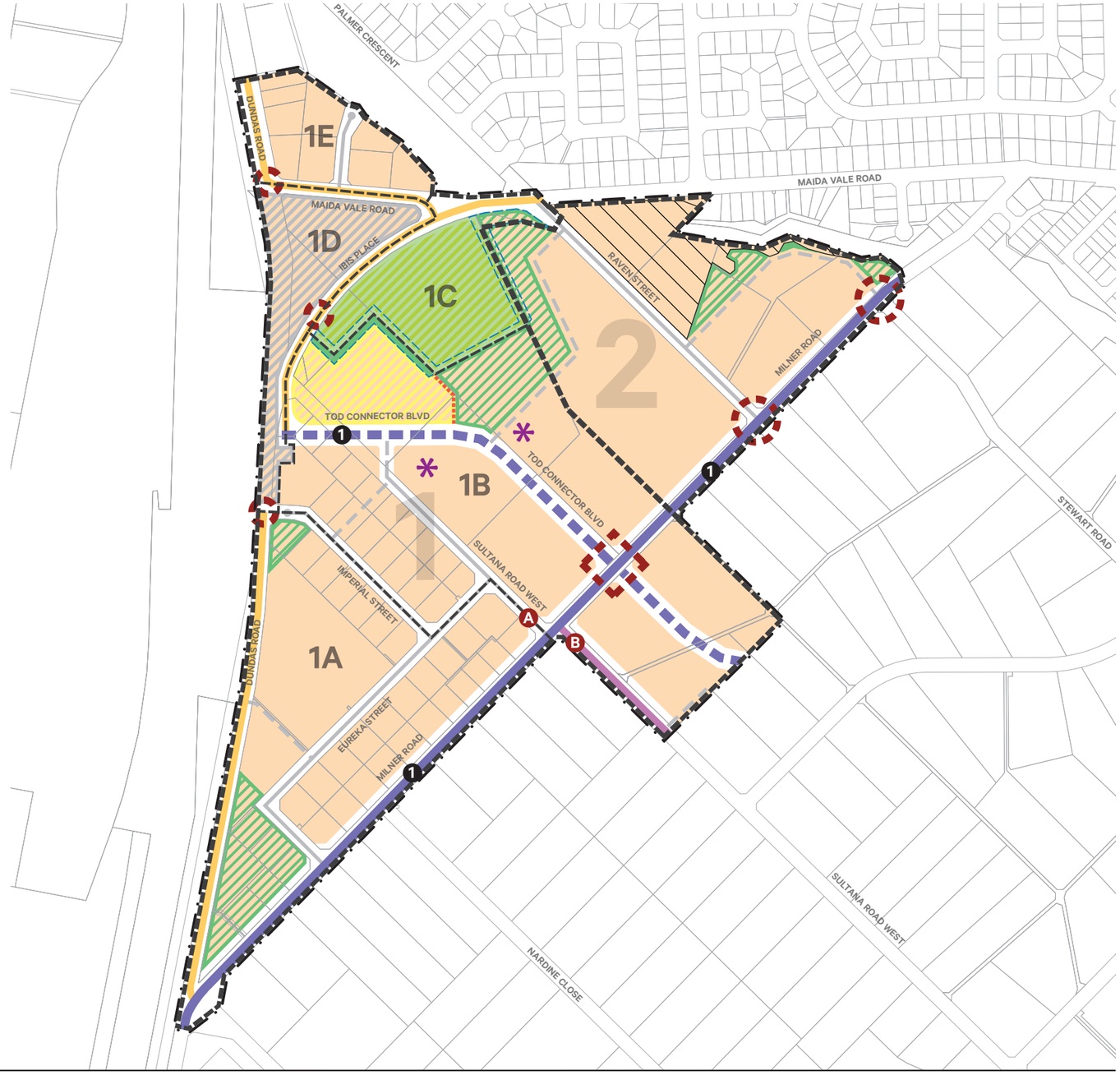
1A Eureka Sub-Precinct

1B Core Sub-Precinct

1C Conservation Sub-Precinct

1D Station Sub-Precinct

1E Maida Vale Sub-Precinct



Activity Centre Plan

Forrestfield North TOD Precinct

DRAFT

Date: 23 Mar 2020 Scale: 1:6000@ A3 1:3000 @ A1 File: **17-527 ST-1 A** Staff: MC GW Checked: MC

0 60 120m

element.

Level 18, 191 St Georges Terrace, Perth Western Australia 6000.
PO Box 7375 Cloisters Square, Perth Western Australia 6850.
T. +61 8 9289 8300 | **E.** hello@elementwa.com.au **elementwa.com.au**

1.4 Consultation

The Kalamunda Environmental Advisory Committee (KEAC) was developed to make recommendations to the Council on matters relating to the environment and social/community. In the initiation of the LSP process the committee was consulted on the major considerations of the precinct (including noise/vibration, environment, water management, sustainability, ethnographic and public realm guidelines).

A Technical Advisory Group (TAG) made up of key agency and decision makers including the Department of Biodiversity, Conservation and Attractions (DBCA) and the Office of Environmental Protection Authority (OEPA) (now Department of Water and Environmental Regulation - DWER) was established and chaired by the Department of Planning, Lands, and Heritage. During the LSP process, TAG meetings were undertaken to discuss the key elements of the project including the rationale of ACP design and the inclusion of the key environmental attributes within the precinct.

Further to the DSP, a Community and Stakeholder Engagement and Communications Strategy was developed by Element (formally TPG + Place Match) (2017) to guide community stakeholder consultation for the development of the ACP. This strategy included two main phases of engagement: The Preliminary Community and Stakeholder Engagement (May – June 2017) and the Public Advertising period (to be advised). Preliminary engagement included the following:

- City of Kalamunda council briefing,
- landowners survey,
- landowner forum,
- community and stakeholder survey, and
- community and stakeholder workshop.

DWER- EPA services branch and DBCA have been consulted in the development of the ACP design and the priority of key environmental factors and their retention within the precinct. Preliminary State biodiversity advice from these agencies were received which was incorporated within the revised ACP design (refer to Figure 1.2 incorporated the following:

- protection of wetland with conservation category ecological values within Lots 9 and 12, including a >50 m buffer area to the south and east to be designated as local open space,
- retention of Black Cockatoo foraging habitat and significant trees,
- retention of Quenda habitat, and
- maintenance of ecological linkage associated with Poison Gully Creek.

1.5 Legislation, policies, and guidelines

The following legislation, policies and guidelines and their implementation may be applicable for the redevelopment of the Residential Precinct.

1.5.1 Federal legislation

1.5.1.1 Environment and Biodiversity Conservation Act 1999

The *Environment Protection and Biodiversity Conservation Act 1999* (EPBC Act) is administered by the Department of the Environment and Energy (DEE). The EPBC Act aims to protect and manage nine Matters of National Environmental Significance (MNES) throughout Australia including:

- World Heritage Properties
- National Heritage Places
- wetlands of international importance (listed under the Ramsar Convention)

- listed threatened species and ecological communities
- migratory species protected under international agreements
- Commonwealth Marine Areas
- the Great Barrier Reef Marine Park
- nuclear actions (including uranium mines).

1.5.2 Assessment under the EBPC Act 1999

Under the EPBC Act, an action that may result in significant impact to any MNES, in accordance with the Significant Impact Guidelines 1.1 - Matters of National Significance (DoE 2013), should be referred to the DEE for assessment by the minister.

The existing MNES on the site are limited to:

- Banksia Woodlands of the Swan Coastal Plain TEC as associated with Lots 9 and 12 Ibis Place; and
- Low to moderate quality Black Cockatoo habitat (limited distribution).

1.5.3 Policy, recovery plans and conservation advice

The recovery plans and conservation advice relevant to the precinct include the following (Table 1.1: Federal policies and guidance

Policy, Recovery Plan or Advice	Description/Objective	Regulatory Authority	Relevance
Conservation advice for the Banksia Woodlands of the Swan Coastal Plain ecological community (2016)	To mitigate the risk of extinction of the Banksia Woodlands of the Swan Coastal Plain ecological community, and help recover its biodiversity and function.	DEE, OEPA, DBCA, Department of Water and Environmental Regulation (DWER) CoK	Removal and protection and/or restoration of TEC.
Carnaby's Cockatoo *(Calyptorhynchus latirostris)* Recovery Plan (2013)	To stop further decline in the distribution and abundance of Carnaby's cockatoo by protecting the birds throughout their life stages and enhancing habitat critical for survival throughout their breeding and non-breeding range, ensuring that the reproductive capacity of the species remains stable or increases.	DEE, OEPA, DBCA and DWER CoK	Removal of significant fauna habitat.
Forest Black Cockatoo (Baudin's Cockatoo *Calyptorhynchus baudinii* and Forest Red-tailed Black Cockatoo *Calyptorhynchus banksii naso*) Recovery Plan (2008)	To stop further decline in the breeding populations of Baudin's Cockatoo and the Forest Red-tailed Black Cockatoo and to ensure their persistence throughout their range in the south-west of Western Australia.	DEE, OEPA, DBCA and DWER CoK	Removal of significant fauna habitat.
Environment Protection and Biodiversity Conservation Act 1999 Environmental Offsets Policy (2012)	To compensate for the residual adverse impacts of an action on the environment.	DEE, CoK	

).

Table 1.1: Federal policies and guidance

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Environment Protection and Biodiversity Conservation Act 1999 Environmental Offsets Policy (2012)	To compensate for the residual adverse impacts of an action on the environment.	DEE, CoK	

The application of management measures and recommendations as specified within Table 1.1 are discussed in Section 4.0.

1.5.4 State legislation

Applicable legislation includes *Environmental Protection Act 1986* (EP Act), the Planning and Development Act 2005 (PD Act) *and Planning and Development (Local Planning Schemes) Regulations 2015* (PD Regulations).

Other legislation relevant to the management of the precinct are outlined in Table 1.2.

Table 1.2: State legislation and guidance

Legislation	Description	Regulatory Authority	Relevance
Aboriginal Heritage Act 1972 (AH Act)	Protection of amendment sites of Aboriginal heritage significance, both known and as yet unknown.	Department of Planning, Lands and Heritage (DPLH)	Disturbance to Aboriginal sites.
Bush Fires Act 1954	Minimising dangers resulting from bush fires, and the prevention, control and extinguishment of bush fires.	Department of Fire and Emergency Services (DFES) CoK	Fire management.
Conservation and Land Management Act 1984	Preservation and conservation of flora and fauna.	Department of Biodiversity, Conservation and Attractions (DBCA)	Removal of threatened flora or TECs/priority ecological communities (PECs).
Contaminated Sites Act 2003	Regulation of matters relating to the identification, assessment, recording, management and clean-up of contaminated land.	Department of Water and Environmental Regulation (DWER)	Contamination, acid sulfate soils.
Dangerous Goods and Safety Act 2004	Safe storage, handling and transport of dangerous goods.	Department of Mines, Industry Regulation and Safety (DMIRS)	Proposed land use within the Project area.

Legislation	Description	Regulatory Authority	Relevance
Electricity Act 1945	Licensing of persons carrying out works relating to electricity.	Department of Mines, Industry Regulation and Safety (DMIRS)	Proposed land use within the Project area.
Health Act 1911	Regulations concerning emissions disposal of sewage.	Department of Health	Sewage management.
Native Title (State Provisions) Act 1999	Recognition and protection of native title.	DPLH	Registered heritage sites.
Rights in Water and Irrigation Act 1914	Protection and licensing of water resources.	DWER	Development within the Perth Groundwater Area.
Waterways Conservation Act 1976	Conservation and management of waterways and their associated environment.	DWER	Waterways management.
Wildlife Conservation Act 1950 (WA) Note: This Act will eventually be replaced by the Biodiversity Conservation Act 2016	Conservation and protection of wildlife (flora and fauna). Special provisions and schedules apply to the protection and management of gazetted rare flora and fauna.	DBCA	Removal of threatened flora or TECs/PECs and fauna.

In addition to the above legislation, there are several policy and guidance documents which relate to specific environmental factors that are relevant to the development of the precinct. The assessment has been designed to address the recommendations of the State regulatory guidance as described in Table 1.3.

Table 1.3: Policies and guidance statements

Statement No.	Title	Application	Relevance to project
Guidance Statements			
3	Separation Distances between Industrial and Sensitive Land Uses.	Outlines generic separation distances between industrial and sensitive land uses to avoid conflict between land uses.	Management of impacts to surrounding land uses.
6	Rehabilitation of Terrestrial Ecosystems.	Outlines the principles to be applied during clearing in the areas that will be rehabilitated after construction. These areas may include batters, service infrastructure corridors and road verges.	Clearing of vegetation.
33	Environmental Guidance for Planning and Development.	Provides guidance on the environmental protection process in respect of the land use planning and development process.	Land use planning.
41	Assessment of Aboriginal Heritage.	If Aboriginal heritage values are identified on or associated with the Project area, this guidance statement will be used to assess the impact of the project on those values.	Protection and management of heritage site values.
EP Act			
	Social Surroundings.	Provides guidance on the management of noise and dust in relation to proposals and surrounding land uses.	Management of impacts to surrounding land uses.
	Flora and Vegetation.	Outlines how the factor Flora and Vegetation is considered by the EPA in the environmental impact assessment (EIA) process.	Management of impacts to flora and vegetation.
	Terrestrial Fauna.	Outlines how the factor Terrestrial Fauna is considered by the EPA in the environmental impact assessment (EIA) process.	Management of impacts to flora and vegetation.
Technical Guidance/guidelines			
Technical Guidance	Flora and vegetation surveys for Environmental Impact Assessment.	Provides guidance on the assessment on the biodiversity implications of vegetation clearing.	Clearing of vegetation, or impact to threatened vegetation or flora species.
Technical Guidance	Sampling methods for terrestrial vertebrate fauna.	Provides guidance on the assessment on the biodiversity implications of vegetation clearing on fauna habitat.	Clearing of vegetation, or impact to threatened fauna species.
Technical Guidance	Terrestrial fauna surveys.	Provides direction and information on general standards and protocols for terrestrial fauna surveys for environmental impact assessment.	Clearing of vegetation, or impact to threatened fauna species.

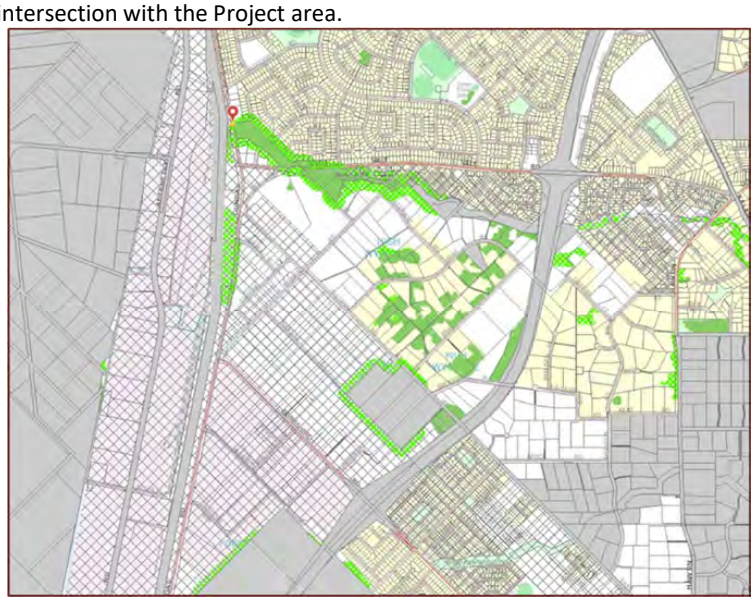
Statement No.	Title	Application	Relevance to project
Guidelines	WA Environmental Offset Guidelines.	Clarifies the determination and application of environmental offsets in Western Australia.	Environmental management.
Guidelines	Statement of Environmental Principles, Factors and Objectives, and some factor guidelines.	Communicates how, for the purposes of environmental impact assessment, the EPA: • considers the object and principles of the Environmental Protection Act 1986 • uses environmental factors and objectives to organise and systemise environmental impact assessment and reporting • takes a holistic view of the environment and a proposal or scheme's potential impact on the environment considers significance when determining whether to assess a proposal or scheme and recommend whether an assessed proposal or scheme may be implemented.	Environmental management.
Guidelines	Assessment and Management of Contaminated Sites guidelines (DER 2014).	Provides guidance on the assessment and management of contaminated sites in Western Australia (WA) within the legislative framework provided by: the *Contaminated Sites Act 2003* (CS Act) and the Contaminated Sites Regulations 2006 (CS Regulations); and the revised national site assessment framework provided in the *National Environment Protection (Assessment of Site Contamination) Measure 1999* (NEPM).	Environmental management.
Policies			
Government of Western Australia	WA Environmental Offsets Policy (EPA 2011).	Provides a framework for consistent application of environmental offsets to protect and conserve environmental and biodiversity.	Environmental management.
State Planning Policy 2	Environment and Natural Resources Policy (WAPC 2003).	Provides guidance on planning developments to conserve the natural environment.	Environmental management.
State Planning Policy 2.8	Bushland Policy for the Perth Metropolitan Region (WAPC 2010).	Provides guidance to ensure bushland protection in the Perth Metropolitan Region in the respect of land use planning.	Clearing of vegetation, or impact to threatened vegetation or flora species.
State Planning Policy 2.9	Water Resources (WAPC 2006).	Provides guidance on how to protect, conserve, enhance and manage sustainable use of water resources of environmental value.	Impact on hydrological regime Urban water management.
State Planning Policy 5.4	Road and Rail Transport Noise and Freight Considerations in Land Use Planning, Section 5.3 – Noise Criteria.	Seeks to minimise the adverse impact of transport noise, without placing unreasonable restrictions on noise-sensitive residential development or adding unduly to the cost of transport infrastructure.	Management of impacts to surrounding land uses.
State Planning Policy 4.2	Activity Centres for Perth and Peel.	Specify broad planning requirements for the planning and development of new activity centres and the redevelopment and renewal of existing centres in Perth and Peel.	The development of an activity Centre is proposed as part of the Project.
Operational Policy 4.3	Identifying and establishing waterways foreshore areas (Department of Water 2012).	Describes the process for identifying and managing foreshore areas whose values are under pressure from land-use changes in the area around them.	Project area is located immediately adjacent to Poison Gully Creek.

1.5.5 Perth and Peel Green Growth Plan for 3.5 Million

The Draft Green Growth Plan (or also known as SAPPR) indicates that there are specific commitments identified within the Project area. The Green Growth Plan does not specify which specific commitments are depicted by the Specific Commitments layer, which may include:

- threatened flora and threatened ecological communities
- conservation category wetlands and wetlands of international importance
- vegetation complexes with less than 10 per cent remaining
- Bush Forever areas (excluding those within the 'rural complementary' and 'negotiated planning solution' categories)
- Short Tongued Bee (*Leioproctus douglasiellus*) distribution (not applicable) (Department of the Premier and Cabinet 2015).

As of the 6 April 2018, the State Government suspended work and will be re-evaluating the SAPPR through an independent review. The Government has advised that "SAPPR sought to secure up front environmental approval for future development and raw materials extraction for an identified development footprint for a 30-year period. It represents the largest and most complex land use reconciliation work, attempting to balance certainty for development and long-term environmental protection" (Department of the Premier and Cabinet, 2018). While the SAPPR has been suspended, the Specific Commitments mapping was reviewed and given due regard in formulating the design of the LSP. Refer to Plate 1. As can be viewed in Plate 1, the Specific Commitments layer is limited in its intersection with the Project area.



Source: Government of Western Australia (2017)

Plate 1: Green Growth Plan for the Residential Precinct and surrounding DSP area

1.5.6 Local government policies, strategies, and guidance

The following Strategies, Local Planning Policies and By-laws are relevant to the CoK and precinct.

Table 1.4: Strategies, Local Planning Policies, and By-Laws

Reference No.	Title	Application	Relevance to project
-	Local Biodiversity Strategy (2008)	Strategically plan natural area protection so that biodiversity conservation is incorporated into the CoK's planning and decision-making processes.	Retention of Local Natural Areas (LNA).
ENV4	Flood and Stream Management	Management programs and retention of open lands and structures which do not obstruct water movement nor destroy natural public water areas within the CoK.	All lands within 15m of the banks of the Poison Creek and Crumpet Creek and areas defined as the 'Flood and Stream Management Area'.
-	Pest Plants (1979)	Council may request the owner or occupier of private land within the district to destroy, eradicate or otherwise control any pest plant on that land.	Land use management.

2. Existing environment

2.1 Topography, geology and soils

The topography of the precinct ranges from approximately 28 m Australian Height Datum (AHD) along the south-western boundary to approximately 36 m AHD in the north-eastern corner. Topographic contours for the site are shown in Figure 2.1.

Geological soil unit mapping indicates that the site is characterised by sand and pebbly silt, specifically:

- S10: Thin layer of sand – very light grey at surface, yellow at depth, fine to medium grained, sub-rounded quartz, moderately well sorted, of eolian origin over alluvial silts and sands of the Guildford formation
- MGS1: Pebbly silt – yellow, fine to medium grained, sub-angular to rounded quartz, with some feldspar, well sorted, variable silt content, of colluvial origin (Gozzard 1986).

The site is mapped as within the Pinjarra Gf5 and Gf9 phases, described as:

- Pinjarra Phase Gf5: Incised drainage channels with poorly drained gradational mottled yellow earths. Shrubland of Melaleucas and other low shrubs.
- Pinjarra Phase Gf9: Minor sandy rises (aeolian deposits) with moderately deep well drained sands overlying gravelly mottled clay.

2.2 Acid sulfate soils

Acid Sulfate Soils (ASS) are naturally occurring, iron-sulphide rich soils, sediments or organic substrates, formed under waterlogged conditions. If exposed to air, these sulphides can oxidise and release sulphuric acid and heavy metals. This process can occur due to drainage, dewatering or excavation.
Review of regional mapping indicates that the precinct has a low to moderate risk of ASS occurring within 3 m of natural soil surface (Class 2) (DER 2015). Nearest high to moderate risk of ASS occurring within 3 m of natural soil surface is approximately 200 m south east of the Project area (Figure 2.2).

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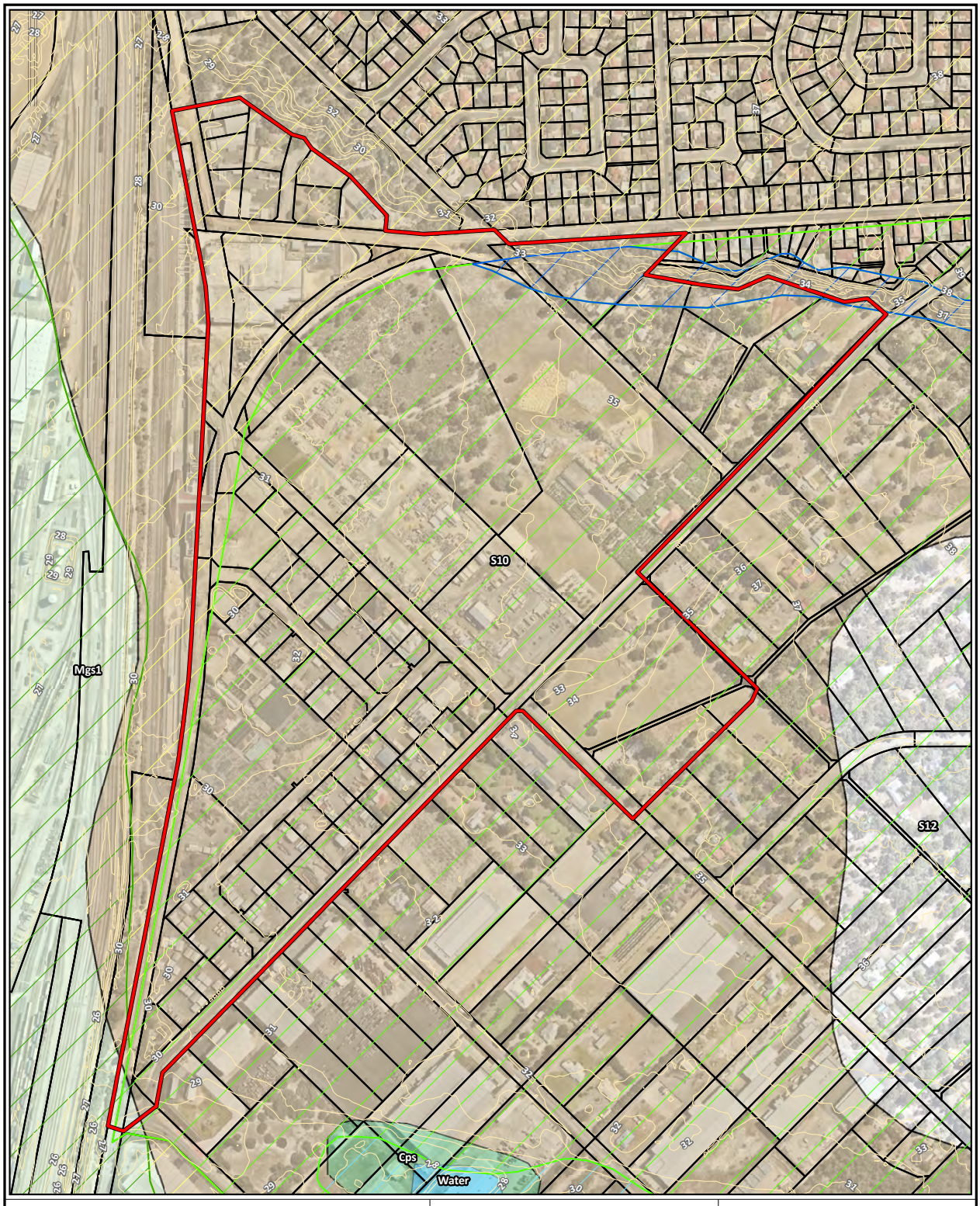


Legend

- Project area
- Cadastral boundary
- Topographic contours (mAHD)

Geology 1:50,000

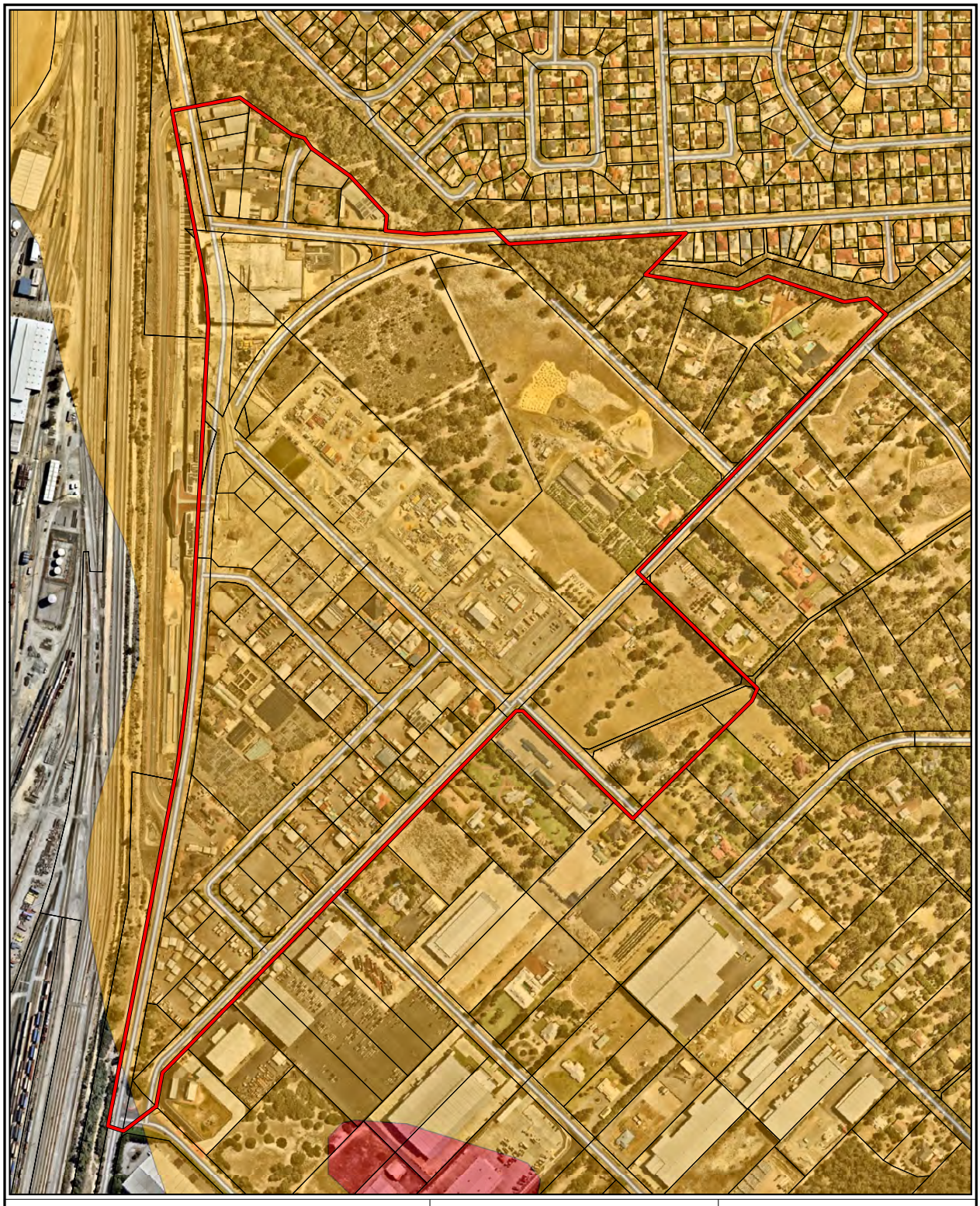
- Cps
- Mgs1
- S10
- S12
- Water

Soil landscape mapping (DPIRD)

- 213Pj__Gf5
- 213Pj__Gf9
- 213Pj__Mgs1
- 213Pj__S10
- 213Pj__Ya3

Scale 1:6,500 at A4 (0 50 100 metres)		**Forrestfield North TOD Precinct Perth, WA**
Coord. Sys. GDA 1994 MGA Zone 50		**TOPOGRAPHY, GEOLOGY AND SOILS**
Job No: 57806		
Client: Element		**FIGURE 2.1**
Version: A	Date: 11-Apr-2021	strategen JBS&G
Drawn By: cthatcher	Checked By: HS	

File Name: \\008PMPMR004V001.jbsg.aust\JBS Perth\Projects\1)Open\Element (TPG)\57806 - FF North TOD & Res LWMS\GIS\Maps\R05_Rev_A\57806_2_1_TopoGeo.mxd
Image Reference: www.nearmap.com© - Imagery Date: 27 February 2021

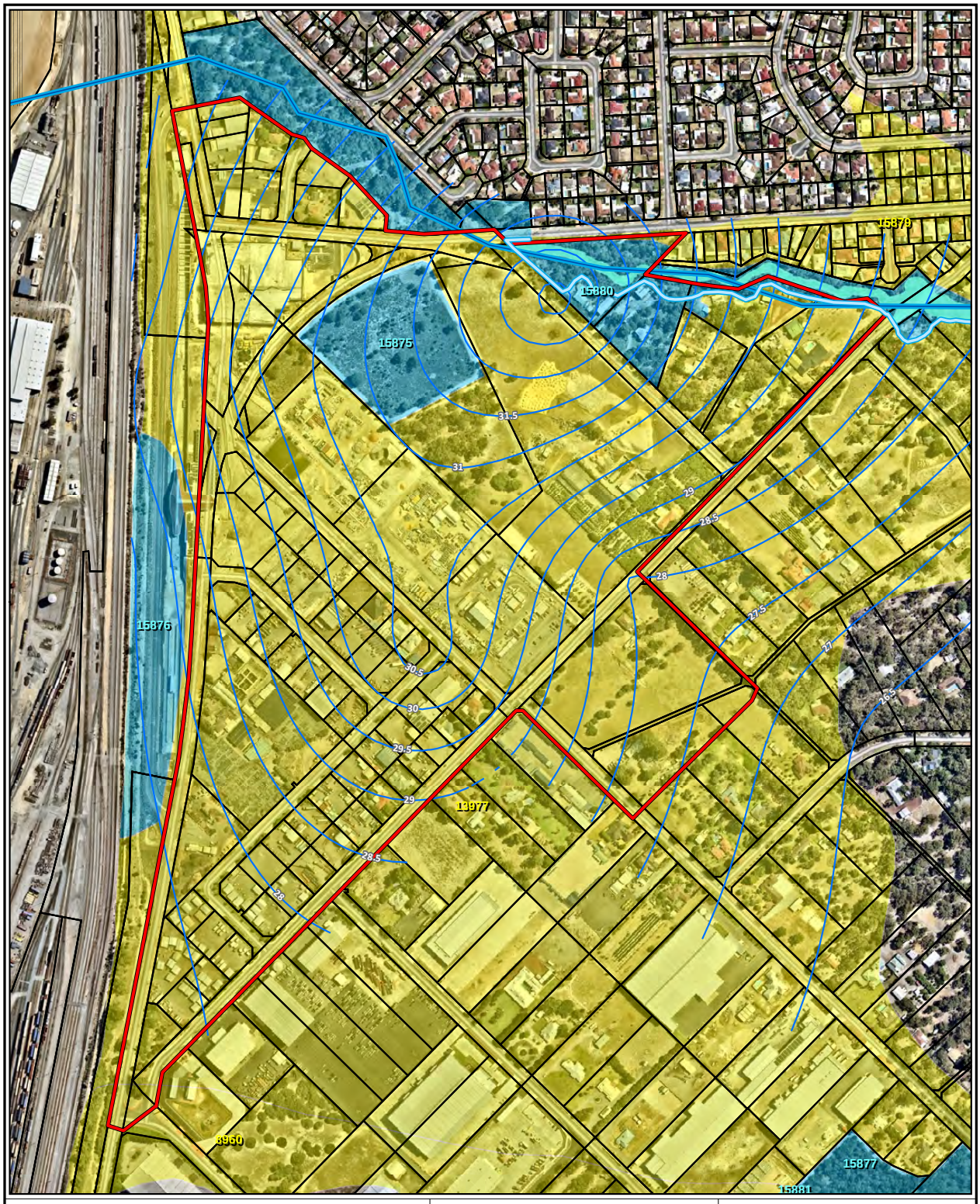


Legend:
- Project area
- Cadastral boundary

Acid sulfate soil (DWER)
- High to moderate risk
- Moderate to low risk
- Roads (MRWA)

Scale 1:6,500 at A4 (0 50 100 metres)		**Forrestfield North TOD Precinct Perth, WA**
Coord. Sys. GDA 1994 MGA Zone 50		**ACID SULFATE SOILS**
Job No: 57806		
Client: Element		**FIGURE 2.2**
Version: A	Date: 11-Apr-2021	strategen JBS&G
Drawn By: cthatcher	Checked By: HS	

File Name: \\008PMPMR004V001.jbsg.aust\JBS Perth\Projects\1)Open\Element (TPG)\57806 - FF North TOD & Res LWMS\GIS\Maps\R05_Rev_A\57806_2_2_ASS.mxd
Image Reference: www.nearmap.com© - Imagery Date: 27 February 2021



Legend:

- Project area
- Cadastral boundary
- Estimated 100 year flood area
- Geomorphic Wetlands (DBCA)
  - Resource Enhancement
  - Multiple Use
- Foreshore boundary
- Groundwater contour maximum (mAHD)
- Poison Gully Creek
- Roads (MRWA)

Scale 1:6,500 at A4 (0, 50, 100 metres)		**Forrestfield North TOD Precinct Perth, WA**
Coord. Sys. GDA 1994 MGA Zone 50		**HYDROLOGY**
Job No: 57806		
Client: Element		**FIGURE 2.3**
Version: A	Date: 11-Apr-2021	strategen JBS&G
Drawn By: cthatcher	Checked By: HS	

File Name: \\008PMPMR004V001.jbsg.aust\JBS Perth\Projects\1)Open\Element (TPG)\57806 - FF North TOD & Res LWMS\GIS\Maps\R05_Rev_A\57806_2_3_Hydrology.mxd
Image Reference: www.nearmap.com© - Imagery Date: 27 February 2021

2.3 Hydrology

A District Water Management Strategy (DWMS) was prepared by Strategen (2015) for the DSP area and approved by the Department of Water (DoW) (now Department of Water and Environmental Regulation - DWER) and the CoK. The DWMS provides guidance on groundwater management, water sources for POS and sizing of stormwater systems.

A Draft LWMS has been prepared by urbaqua (2020). The LWMS is consistent with regional and district scale urban water management planning, including the *State Water Plan* (DPC 2007) as well as *State Planning Policy 2.9 Water Resources* (WAPC 2006). The LWMS aims to meet the principles and objectives of stormwater management in Western Australia, as detailed in the *Stormwater Management Manual for Western Australia* (DoW 2007).

2.3.1 Groundwater

There are three aquifers underlying the DSP area; each assigned the name of the major geological unit in which the aquifer occurs (DWER 2017). In descending order of depth from natural surface they are:

- Superficial Aquifer (unconfined)
- Leederville Aquifer (confined)
- Yarragadee North (confined).

Groundwater flow is in a westerly direction through the Project area. Groundwater levels within the TOD Precinct range from 26.8 mAHD to 33.9 mAHD (Figure 2.3), with a depth to groundwater of over 15 m (Urbaqua 2020). The presence of low permeability material (clayey sand) results in shallow, perched groundwater in parts of the site.

Hydrogeological features of the DSP area are dominated by surface water/groundwater interactions resulting in variable groundwater gradients and flow directions across the site (EMRC 2013). The local groundwater flow appears to be influenced by a combination of geology and drainage systems to the west and northwest of the site such as localised recharge features associated with industrial areas, Perth Airport, and new developments (EMRC 2013).

Four monitoring bores are located within the Project area (MB01, MB07, MB08 and MB09), with groundwater quality monitoring undertaken by Strategen (2012).

Across the DSP area, groundwater is generally slightly acidic, with pH ranging between 5.97 to 6.35 (Strategen 2012). Groundwater is fresh with a mean electrical conductivity (EC) level of 0.345 mS/cm. The mean EC level was identified to be within the expected range of 0.3–1.5 mS/cm for slightly disturbed ecosystems in south-west Australia (Strategen 2012).

Urbaqua (2020) reported that nutrient levels slightly exceeded short and long term limits in the Swan Canning Water Quality Improvement Plan (Swan River Trust 2009). Limited sampling indicated concentrations of Copper, Chromium, Nickel, Lead and Zinc exceed Australian and NZ Guidelines for Fresh and Marine Water Quality guidelines (ANZG 2018).

Groundwater attributes within the precinct are presented in Figure 2.3

The Department of Water and Environmental Regulation's Water Register indicates there is limited groundwater available for allocation from the Superficial Aquifer within the Project area. Urbaqua (2020) have identified that there is currently a combination of temporary and standard licences

within the Project area. The temporary licences are associated with construction of the Airport rail line (160,000 lL/yr). Two standard licences also exist with a total allocation of 112,510 kL:

2.3.2 Surface water

Poison Gully Creek is located immediately north of the TOD Precinct and follows the cadastral boundary. Poison Gully Creek is an ephemeral creek that flows in a westerly direction via Limestone Creek (a tributary of Perth Airport Northern Main Drain) into the Swan River during the winter months. Previous monitoring has reported water present within September and October (Strategen 2011 and ERMC 2013).

Poison Gully Creek is part of the CoK and Water Corporation drainage network and has an extensive catchment to the east of the LSP area (Water Corporation 2010). It is estimated that Poison Gully has a catchment area of approximately 770 ha at the intersection of Maida Vale Rd and Dundas Rd. No stream gauging data is publicly available for either drainage line. It is understood that Water Corporation have previously maintained a stream gauging station for Poison Gully Creekat Littlefield Rd (Station No. 616015).

Strategen (2012) completed a Foreshore Assessment of Poison Gully Creek which was included in the DWMS. This was approved by the Department of Water (now DWER) in 2015. In accordance with *Determining Foreshore Reserves* (WRC 2001) guidelines, the assessment considered the following biophysical criteria: vegetation, hydrology, soil type, geology, topography, habitat, land use and heritage. The Creek is deeply incised, with vertical exposed banks of 3 m to 5 m high in some areas, with the steepest gradient occurring on the eastern extent of the creek. Evidence of erosion was observed along the length of the creek line to varying degrees. This is likely due to the steep gradient and degraded nature of fringing vegetation which, if intact, would serve to stabilise the banks. Heavy infestation of introduced plant species including Watsonia (*Watsonia bulbillifera*) and Spanish bamboo (*Arundo donax*) was recorded by Strategen (2012b). The foreshore boundary of Poison Gully Creek, in accordance with WRC (2001) is depicted in Figure 2.3.

2.3.3 Flood potential

Water Corporation (2010) undertook 1 in 100-year Average Return Interval (ARI) flood level modelling for Poison Gully Creek, which indicated a 1 in 100 year ARI flood level ranging between 39.81 mAHD at Roe Highway and 32.36 mAHD at Maida Vale Road (Strategen 2015). This is mostly contained within the valley area of the creekline. The flood height at Roe Highway is considered an underestimate, given the invert level of the creek at this point is greater than 40 mAHD, above the Water Corporation Flood level. Based on the Water Corporation modelling, Poison Gully will overtop the road at Milner Road in the 1 in 100-year ARI event. However, as indicated, the 1 in 100-year ARI event is contained within the foreshore boundary.

2.3.3.1 Geomorphic wetlands

The nature of the protection and management Swan Coastal Plain wetlands should be afforded is guided by the appropriate management category they have been assigned. These management categories are listed below:

Table 2.1: Management Category for Geomorphic Wetlands (DBCA 2017)

Management category	Management objective
Conservation Category Wetlands (CCW)	To preserve wetland (natural) attributes and functions
Resource Enhancement Wetlands (REW)	To restore wetlands through maintenance and enhancement of wetland functions and attributes
Multiple Use Wetlands (MUW)	To use, develop and manage wetlands in the context of water, town and environmental planning

A search of the Geomorphic Wetlands of the Swan Coastal Plain Database shows that the site contains four wetland areas, three of which are Resource Enhancement Wetlands (REW) and one is a Multiple Use Wetland (MUW) (Figure 2.3). REWs are considered as priority wetlands where they have been partially modified but still support substantial ecological attributes and function. The ultimate objective is to manage, restore and protect towards improving their conservation value. No unauthorised development is permitted within a REW. A generic wetland buffer to protect the wetlands ecosystem is usually associated with REW, site buffer assessment can be undertaken to determine the buffer widths (EPA 2008). One REW (UFI 15880) is associated with Poison Gully Creek, another (UFI 15875) is contained within lot 12, and the third (UFI 15876) is located on the western boundary of the site.

Following discussion with the City of Kalamunda and Development WA, it is considered likely that UFI 15875 will be reclassified as a Conservation Category Wetland (CCW) due to the high ecological value of the wetland. As such, a 50 m buffer has been applied to the north east and a buffer in excess of 50m to the south, which encompasses existing remnant vegetation within Lot 12 (Appendix A).

UFI 5880 is associated with the assessed Poison Gully Creek foreshore reserve and as such the conservation of this wetland has been addressed by foreshore area.

MUWs are the lowest management category assigned to wetlands by DWER, and are generally considered appropriate for development, provided the hydrological regime is not disturbed (EPA 2008).

None of the other wetlands within of adjacent to the Project area are considered to be suitable for retention such that they require the provision of land for conservation or management..

2.3.4 Public Drinking Water Source Area

The *Metropolitan Water Supply, Sewerage and Drainage Act 1909* (MWSSD Act) and the *Country Areas Water Supply Act 1947* (CAWS Act) identify and categorise public drinking water source areas as catchment areas, water reserves, or underground water pollution control areas.
According to the database available through DWER (year) the site does not intersect any Public Drinking Water Source Areas.

2.4 Contamination

The *Contaminated Sites Act 2003* (CS Act) defines contamination as having a substance present in land or water above background concentrations that presents a risk of harm to human health or the environment. The CS Act also provides for the identification, recording, management and remediation of contaminated sites. Contamination commonly occurs through accidental leakage and spillage, or poor site management practices.

The DWER (2018) Contaminated Site Database was searched and there are currently no registered contaminated sites within the precinct existing on the register. The nearest registered contaminated site is located 77 m east of the TOD Precinct within Lot 13568 on Plan 221057 which has been remediated for restricted use.

2.4.1 Previous potentially contaminating land use(s)

Land within the TOD Precinct has previously been used for market garden and orchard activities, which have been identified as potentially contaminating activities. Within the Residential Precinct, immediately adjacent to the TOD Precinct, previous land uses including Brand Road Landfill and several orchards have been identified as potentially contaminating. These are located at a minimum

distance of 765 m from the project area and are not considered to constrain development of the TOD Precinct (Figure 2.4).

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Legend:
- Project area
- Cadastral boundary
- Historical orchard activity
- Indicative landfill (source: GHD 2010)

DWER registered contaminated sites
- Remediated for restricted use
- Roads (MRWA)

Scale 1:12,000 at A4 — 0 100 200 metres

Coord. Sys. GDA 1994 MGA Zone 50

Job No: 57806

Client: Element

Version: A | Date: 11-Apr-2021

Drawn By: cthatcher | Checked By: HS

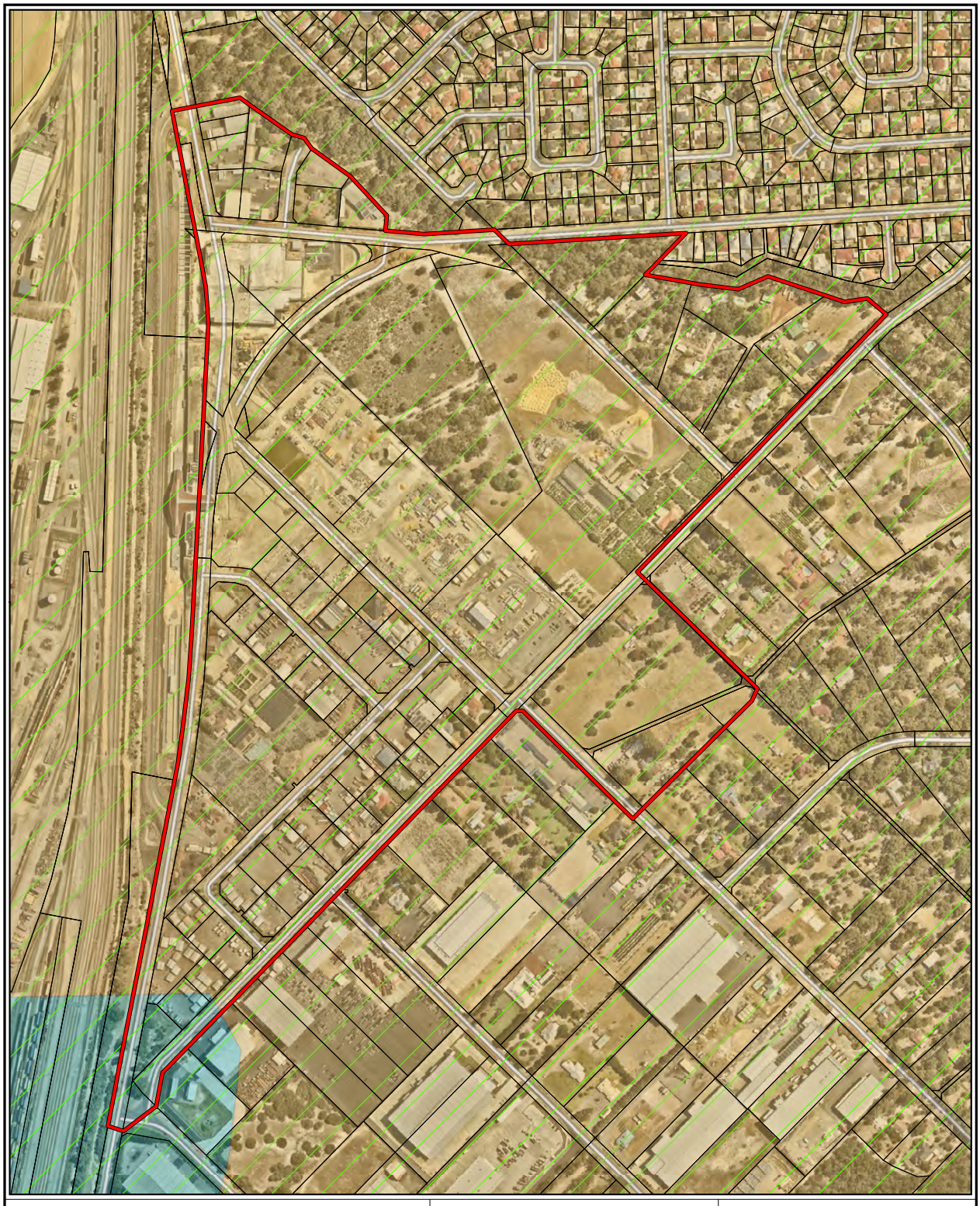
Forrestfield North TOD Precinct Perth, WA

CONTAMINATED SITES AND LOCATION OF POTENTIALLY CONTAMINATING ACTIVITIES

FIGURE 2.4

strategen JBS&G

File Name: \\008PMPMR004V001.jbsg.aust\JBS Perth\Projects\1)Open\Element (TPG)\57806 - FF North TOD & Res LWMS\GIS\Maps\R05_Rev_A\57806_2_4_ContamSites.mxd
Image Reference: www.nearmap.com© - Imagery Date: 27 February 2021



Legend:
- Project area
- Cadastral boundary

Vegetation complexes (DBCA)
- Southern River Complex

Pre-European vegetation (DPIRD)
- Bassendean 1001
- Bassendean 1018
- Roads (MRWA)

Scale 1:6,500 at A4 (0 50 100 metres)		**Forrestfield North TOD Precinct Perth, WA**
Coord. Sys. GDA 1994 MGA Zone 50		**REGIONAL VEGETATION ASSOCIATION AND COMPLEX**
Job No: 57806		
Client: Element		**FIGURE 2.5**
Version: A	Date: 11-Apr-2021	strategen JBS&G
Drawn By: cthatcher	Checked By: HS	

File Name: \\008PMPMR004V001.jbsg.aust\JBS Perth\Projects\1)Open\Element (TPG)\57806 - FF North TOD & Res LWMS\GIS\Maps\R05_Rev_A\57806_2_5_RegVegAssoc.mxd
Image Reference: www.nearmap.com© - Imagery Date: 27 February 2021

2.5 Biodiversity and natural assets

2.5.1 Flora and vegetation

2.5.1.1 Regional vegetation communities

Vegetation occurring within the region was initially mapped at a broad scale (1:1 000 000) by Beard during the 1970s. This dataset has formed the basis of several regional mapping systems, including physiographic regions defined by Beard (1981) which led to the delineation of botanical districts as described in Beard (1990): the biogeographical region dataset (Interim Biogeographic Regionalisation for Australia, IBRA) for Western Australia and System 6 Vegetation Complex mapping undertaken by Heddle et al. (1980). The site is located within the Swan Coastal Plan 2 (SWA2) bioregion.

Beard (1980) vegetation association mapping indicates that the precinct resides within Bassendean 1001 (medium, very sparse woodland or low woodland of Jarrah, Banksia, and Casuarina) and Bassendean 1018 (medium forest, low forest, or low woodland comprised of Jarrah/Marri, Banksia, Teatree, and *Casuarina obesa*). The spatial extent of these associations within the project area is displayed on Figure 2.5.

An estimated 22.05% of the pre-European extent of Bassendean 1001 currently remains within the Swan Coastal Plain IBRA region (GoWA 2019a). This is estimated at 17.39% for the Bassendean 1018 association (GoWA 2019b).

Heddle et al (1980) broadly mapped vegetation complexes across the Swan Coastal Plain. The TOD precinct is comprised of the Southern River Complex (Figure 2.5). The Southern River Complex is described as 'open woodland of *Eucalyptus calophylla* (now *Corymbia calophylla*) – *E. marginata* – *Banksia* spp. with fringing woodland of *E. rudis* – *M. rhaphiophylla* along creek beds' (Heddle et al 1980). An estimated 18.43% of the pre-European extent of the Southern River complex currently remains within the Swan Coastal Plain IBRA bioregion (GoWA 2019b).

Within the Swan Coastal Plain (SCP) area there is a focus on retaining vegetation complexes with less than 10% of their pre-European extent remaining (EPA 2015). The remaining extent of the Southern River complex is above the 10% threshold.

2.5.1.2 Vegetation types and condition

A Level 2 Flora and Vegetation assessment was completed for the DSP area, including the TOD Precinct, by AECOM (2017) and included the following:

- Ecological community mapping and vegetation condition mapping
- Targeted threatened flora searches for Conospermum undulatum
- Development of a comprehensive species list
- Threatened ecological community assessment for patches of remnant vegetation.

A total of 142 species from 96 genera and 33 families were recorded within the DSP area during the assessment (AECOM 2017). The total includes 131 (93%) locally native species. Families with the highest representation are Proteaceae (24 native taxa), Fabaceae (14 native taxa) and Myrtaceae (11 native taxa) (AECOM 2017). A total of five vegetation communities were mapped within the entire DSP area, with two occurring within the TOD Precinct. These included:

- VdCd: *Eucalyptus marginata* subsp. *marginata* mid isolated trees over *Verticordia densiflora* var. *densiflora*, *Daviesia angulata, Hypocalymma angustifolium, Stirlingia latifolia* and *Banksia bipinnatifida* subsp. *bipinnatifida* low open shrubland with *Caustis dioica*,

Mesomelaena tetragona, *Tetraria octandra* and *Schoenus brevisetis* tall open sedgeland over *Patersonia occidentalis, Lyginia barbata, Haemodorum spicatum, Tricoryne elatior* and *Anigozanthos manglesii* low open forbland.

- Trees: scattered to clumps of native trees (mostly *Eucalyptus marginata*) located in either cleared paddocks, roadsides, or over planted vegetation.

PGV Environmental (2021) subsequently undertook an assessment of the Vegetation and Flora within lots 9 and 12 Ibis Place, High Wycombe, to identify the ecological attributes within the mapped wetland area. This resulted in the resolution of the results of two previous surveys by Morgan (2015) and AECOM (2017) to confirm the values present, as well as account to changes over time. The combined, synthesised results, indicate that the TOD Precinct is comprised of three vegetation types (Figure 2.6):

- AfXpHh: *Allocasuarina fraseriana* (Sheoak) Low Open Woodland over *Xanthorrhoea preissii* Open Shrubland over *Verticordia densiflora* var. *densiflora, Hibbertia hypericoides, Stirlingia latifolia, Bossiaea eriocarpa* Low Shrubland over *Alexgeorgea nitens, Chordifex sinuosus, Caustis dioica, Dasypogon obliquifolius, Neurachne alopecuroidea* Open Sedgeland/Herbland/Grassland,
- CcVdHa: *Corymbia calophylla* (Marri) Low Open Woodland over *Verticordia densiflora* var. *densiflora, Hypocalymma angustifolium, Xanthorrhoea preissii, Stirlingia latifolia* Open to Closed Heath over *Caustis dioica, Hypolaena exsulca, Dasypogon obliquifolius, Mesomelaena tetragona* Open Sedgeland/Shrubland, and
- Trees: scattered to clumps of native trees (mostly *Eucalyptus marginata*) located in either cleared paddocks, roadsides, or over planted vegetation.

Vegetation condition within the project area ranges from Completely Degraded (1.12 ha; 28.53%) to Excellent (1.17 ha; 29.75%), according to Keighery (1994), as shown on Figure 2.7: Vegetation condition

Figure 2.8: Conservation significant ecological communities and flora

. A full breakdown of the vegetation types and condition within the project area is detailed in Table 2.2.

Table 2.2: Vegetation Types and Condition within the Project Area - adapted from AECOM (2017) and RPS (2021)

Vegetation Type	Vegetation Condition	Area (ha)	% of total
AfXpHh	Very Good	0.21	5.29
	Good	0.11	2.74
Total		**0.31**	**8.03**
CcVdHa	Excellent	1.17	29.75
	Very Good	1.11	28.37
	Good	0.14	3.60
	Degraded	0.07	1.72
	Completely Degraded	0.01	0.36
Total		**2.50**	**63.80**
Trees	Completely Degraded	1.10	28.17
Total		**1.10**	**28.17**
Grand total		**3.92**	**100**

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Legend

- Project area
- Cadastral boundary

Vegetation type

- AfXpHh (0.31 ha)
- CcVdHa (2.50 ha)
- Trees (1.10 ha)
- Roads (MRWA)

Scale 1:6,500 at A4 (0 50 100 metres)		**Forrestfield North TOD Precinct Perth, WA**
Coord. Sys. GDA 1994 MGA Zone 50		**VEGETATION TYPES**
Job No: 57806		
Client: Element		**FIGURE 2.6**
Version: A	Date: 11-Apr-2021	strategen JBS&G
Drawn By: cthatcher	Checked By: HS	

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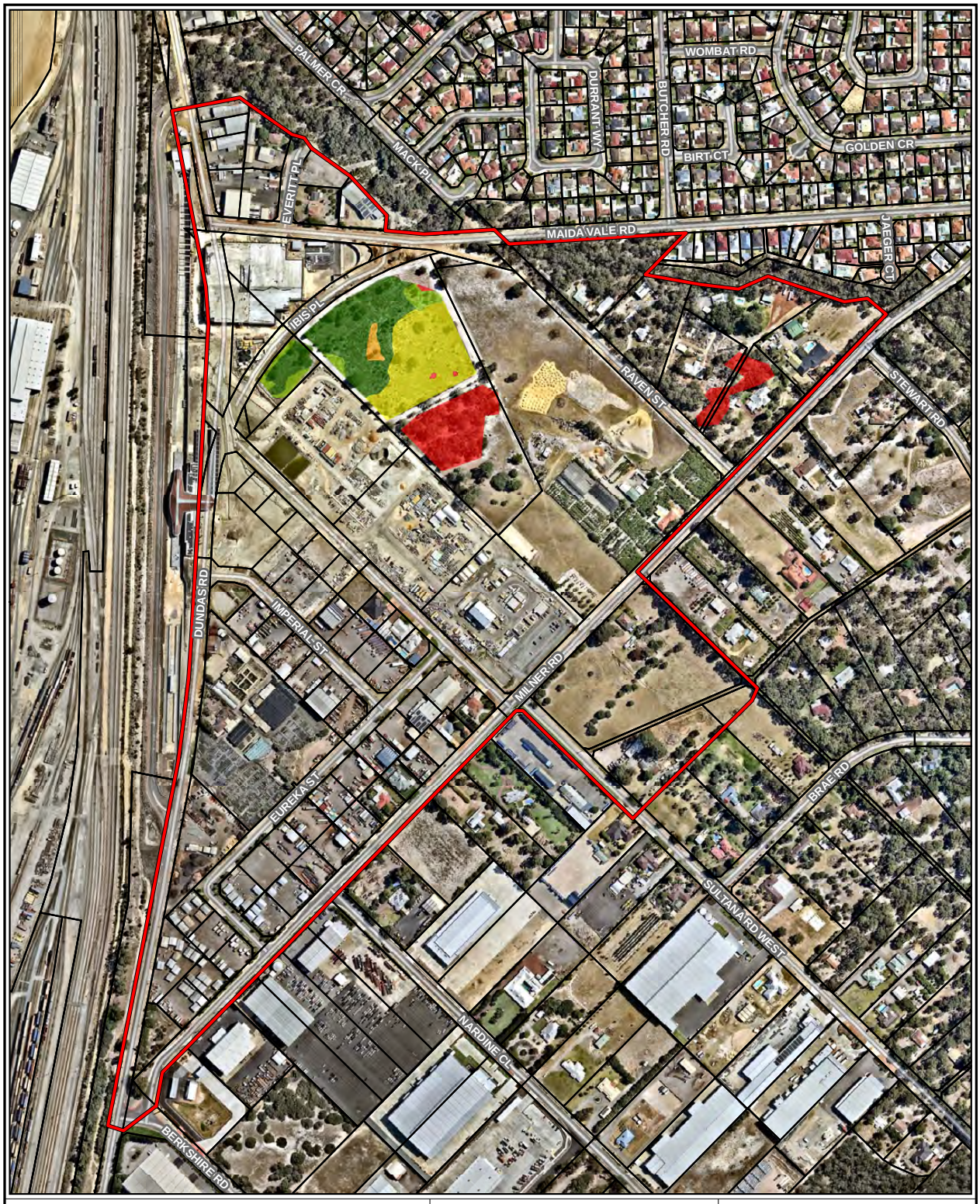

Legend:
- Project area
- Cadastral boundary

Vegetation condition
- Very good
- Good
- Excellent
- Degraded
- Completely degraded
- Roads (MRWA)

Scale 1:6,500 at A4 — 0 50 100 metres

Coord. Sys. GDA 1994 MGA Zone 50

Job No: 57806

Client: Element

Version: A | Date: 11-Apr-2021

Drawn By: ctatcher | Checked By: HS

Forrestfield North TOD Precinct
Perth, WA

VEGETATION CONDITION

FIGURE 2.7

strategen JBS&G

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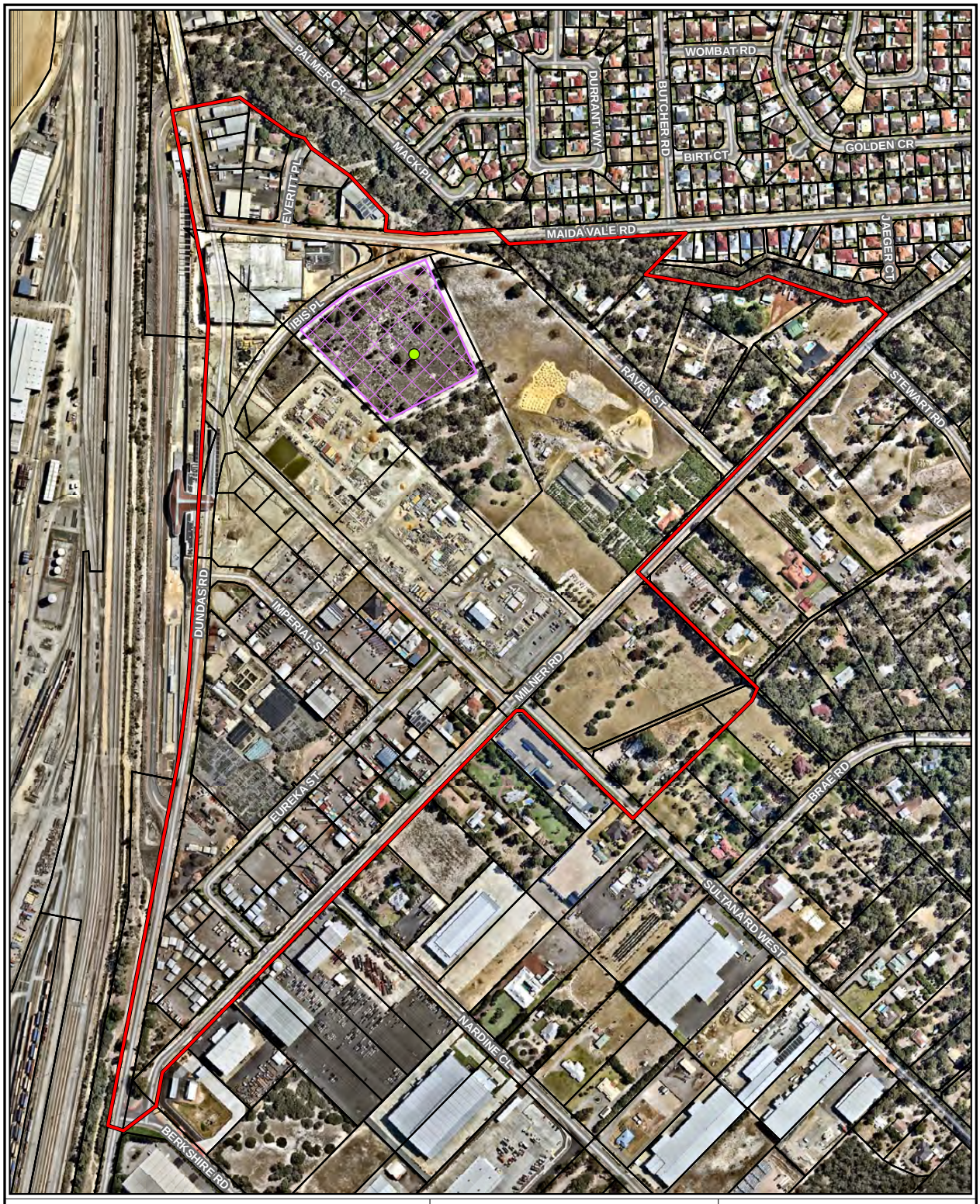

Legend

- Project area
- SCP 20a: *Banksia attenuata* woodlands over species rich dense shrublands
- Cadastral boundary
- *Platysace ramosissima* (P3)
- Roads (MRWA)

Scale 1:6,500 at A4

0 50 100 metres

Coord. Sys. GDA 1994 MGA Zone 50

Job No: 57806

Client: Element

Version: A | Date: 11-Apr-2021

Drawn By: cthatcher | Checked By: HS

Forrestfield North TOD Precinct
Perth, WA

CONSERVATION SIGNIFICANT ECOLOGICAL COMMUNITIES AND FLORA

FIGURE 2.8

strategen JBS&G

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Image Reference: www.nearmap.com© - Imagery Date: 27 February 2021

2.5.1.3 Conservation significant flora

AECOM (2017) did not record any flora species listed as Threatened under the EPBC Act at the Commonwealth level or under the *Biodiversity Conservation Act 2016* (BC Act) at a State level. One DBCA-listed 'Priority' flora species, *Platysace ramosissima* (Priority 3), was recorded by Morgan (2015) within lot 12 (Figure 2.6). *P. ramosissima* is a perennial herb found on sandy soils that grows to 0.3 m high and flowers white/cream between October and November (WAHerb 1998-). This was described as a small population by PGV Environmental (2021).

2.5.1.4 Threatened and Priority ecological communities

AECOM (2017) considered the vegetation within Lot 12, classed as VdCd at the time (see section 2.5.1.2), to be somewhat representative of FCT 20a '*Banksia attenuata* woodlands over species rich dense shrublands'. AECOM (2017) noted the results of the floristic analysis to be 'cryptic' and suggested that the area had been substantially modified through human activity. PGV Environmental (2021) describe the vegetation type, now reclassified as CcVdHa (see section 2.5.1.2) as being representative of FCT 20a, despite there being no *Banksia attenuata* present. FCT 20a is a sate-listed Threatened Ecological Community (TEC) in Western Australia. FCT 20a also has a relationship with the EPBC listed TEC Banksia Woodlands of the Swan Coastal Plain. However, given the lack of *B. attenuata* or other key Banksia species on site, the Banksia Woodland TEC is not considered to occur on site (PGV Environmental 2021). The Project area therefore contains 2.67 ha of State-listed TEC.

2.5.1.5 Introduced taxa and weeds

AECOM (2017) recorded two locations of **Zantedeschia aethiopica* within the TOD Precinct. **Z. aethiopica* is listed as a Declared Pest plant species in Western Australia pursuant to section 22 of the *Biosecurity and Agriculture Management Act 2007* (BAM Act). These locations are within and immediately south of Poison Gully Creek at the north of the TOD Precinct.

2.5.1.6 Dieback

The TOD Precinct contains species susceptible to dieback caused by the plant pathogen *Phytophthora cinnamomi*. A Phytophthora dieback survey has not been undertaken within the TOD precinct area; however, AECOM (2017) noted no visual evidence of Dieback based on the health of Xanthorrhoea, Banksia, and Eucalyptus species.

2.6 Fauna and habitat

2.6.1 Fauna survey

A Level 1 Fauna Assessment was conducted by AECOM in 2017 in accordance with the EPA Guidance Statement No. 56 (AECOM2017). Twelve fauna species were recorded within the entire DSP area. Within the TOD Precinct, two fauna habitat types were recorded, summarised in Table 2.3.

Table 2.3: Fauna habitat types within the Project Area - adapted from AECOM (2017)

Fauna Habitat Type	Description	Habitat for Conservation Significant Fauna	Area (ha)	% of total
Heath	Fairly open habitat type with isolated trees over approximately 60% heath cover. Areas of bare sand exposed to the sun (and potential predators).	Quenda	2.50	69.44
		Forest Red-tailed Black Cockatoo (foraging)		
		Carnaby's Cockatoo (foraging)		
Jarrah and planted	Native trees with planted vegetation recorded on private properties. Includes native trees over grasses, native trees with planted trees, and	Forest Red-tailed Black Cockatoo (foraging and potential breeding)	1.10	30.56

Fauna Habitat Type	Description	Habitat for Conservation Significant Fauna	Area (ha)	% of total
	native trees over planted understorey species.	Carnaby's Cockatoo (foraging and potential breeding)		
Total			**3.60**	**100**

RPS Environmental (2021) identified the fauna habitat occurring within Lots 9 and 12 Ibis Place to be of 'Good' quality, indicating that the area had previously experienced disturbance but retained many characteristics of the habitat type had it not been disturbed. The area was described as 'Good' due to its overall high vegetation quality, but small size and limited connectivity (RPS Environmental 2021).

2.6.2 Habitat for Conservation Significant Fauna Species

AECOM (2017) identified the site as containing foraging habitat for Quenda, Carnaby's Cockatoo and Forest Red-tailed Black Cockatoo. RPS Environmental (2021) also consider the site suitable for foraging by Baudin's Cockatoo.

AECOM (2017) recorded evidence of Forest Red-tailed Black Cockatoo (listed as Vulnerable under the EPBC Act and Threatened under the BC Act) and Quenda (DCBA-listed Priority 3) within the Project area. Scats and diggings by Quenda were recorded within the 'Heath' fauna habitat type, while foraging evidence of Forest Red-tailed Black Cockatoo were recorded within 'Heath' and 'Jarrah and planted' fauna habitat types (Figure 2.9) (AECOM 2017).

Black Cockatoos

Carnaby's Cockatoos, listed as Endangered under the EPBC Act, feed on the seeds, nuts, and flowers, of a variety of native and introduced plant species and insect larvae (DEE 2017b). Food plants generally occur within proteaceous genera such as Banksia, Dryandra, Hakea and Grevillea, though are known to forage on Eucalypt species in woodland areas (DSEWPaC 2012). Carnaby's Cockatoo have also adapted to feeding on exotic species such as pines and cape lilac and weeds such as wild radish and wild geranium (DEE 2017b). Carnaby's Cockatoos typically breed between July and December in the hollows of alive or dead eucalypts; primarily in Salmon Gum and Wandoo, but also within Jarrah, Marri, and other eucalypt species (Johnstone 2010). Breeding hollows utilised by the species are usually a minimum of 2 m above ground level, with a hollow depth of at least 0.25 m (DEE 2017b). Distribution mapping for the species is available through DSEWPaC (2012) which outlines the likely breeding and non-breeding range. Based on this mapping, the TOD Precinct is within the Carnaby's Cockatoo breeding range.

Forest Red-tailed Black Cockatoos, listed as Vulnerable under the EPBC Act, depend primarily on Marri and Jarrah trees for both foraging and nesting. The seeds of both eucalypts are the favoured food source of the birds, and they will nest within the hollows or alive or dead individual trees (Johnstone & Kirkby 1999). Breeding varies between years and occurs at times of Jarrah and Marri fruiting. Forest Red-tailed Black Cockatoos breed in woodland, forest, or artificial nest boxes, and may also breed in former woodland or forest that has been reduced to isolated trees (DEE 2017b).

Quenda

Preferred habitat for Quenda (*Isoodon obesulus*) usually consists of a combination of sandy soils and dense heathy vegetation (AECOM 2017). The species was recorded via scats and diggings within the TOD Precinct area, with observations constrained to within the 'Heath' fauna habitat type of Lot 12 (AECOM 2017). Based on this, 2.78 ha of Quenda habitat exists within the TOD Precinct. All observations were within the wetland area identified for retention for environmental conservation.

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2.6.3 Black Cockatoo habitat

Foraging

Foraging habitat for Black Cockatoo species within the TOD Precinct was mapped by AECOM (2017) and valued based on the foraging species present, vegetation condition, and other relevant habitat features. Scoring was based on the methodology provided by the *draft revised guideline for three threatened black cockatoo species: Carnaby's Cockatoo, Baudin's Cockatoo, Forest Red-tailed Black Cockatoo* (DotEE 2017). As a result, 3.60 ha of 'valued' habitat (i.e a habitat score of 5) for Carnaby's Cockatoo and 3.60 ha of 'quality' (i.e a habitat score of 7) habitat for Forest Red-tailed Black Cockatoo exists within the TOD Precinct. RPS Environmental (2021) consider foraging habitat within Lot 12 to be confined to Marri trees (six identified within the lot) and Proteaceous species, of which there were reported to be few (Figure 2.10).

Roosting

Black Cockatoo roosting habitat is generally found in or new riparian vegetation close to freshwater and is typically comprised of the tallest trees within suitable habitat areas (AECOM 2017). AECOM (2017) identified two confirmed roost sites used by Forest Red-tailed Black Cockatoos with the Residential Precinct area, approximately 300 m from the boundary of the TOD Precinct (Figure 2.9). No roosts were confirmed by AECOM (2017) as occurring within the TOD Precinct. Mapping available through DBCA (2018) has indicated a potential roost site to occur within the Project area. This data is compiled through the results of annual Great Cocky Count citizen science surveys and includes confirmed and unconfirmed sites. Based on the survey by AECOM (2019), roosting is not considered to occur within the Project area.

Breeding

Breeding habitat for Black Cockatoos is defined by DESWPaC (2012) as trees of a species known to support breeding, within the range of the species, that either have a suitable nest hollow or are of a suitable diameter at breast height (DBH) to develop a nest hollow (typically considered > 300 mm for salmon gum and wandoo and > 500 mm for other species). These trees are known as significant habitat trees. Trees of this size may also be large enough to provide roosting habitat.

A total of 47 potentially significant habitat trees with a DBH greater than 500 mm were recorded within the TOD Precinct area (Figure 2.10) with a further 411 located within the adjacent Residential Precinct area (AECOM 2017). None of the trees were reported to contain potentially suitable hollows and, as such, the Project area does not support breeding (RPS Environmental 2021). A summary of the potentially significant habitat trees recorded within the TOD Precinct and entire DSP area is provided by Table 2.4.

Table 2.4: Potentially significant habitat trees within the Project area - adapted from AECOM (2017)

Tree Species	Number of trees recorded within the DSP Area	Number of trees recorded within the TOD Precinct	Usage by Black Cockatoos	
			Carnaby's Cockatoo	Forest Red-tailed Black Cockatoo
Marri	54	43	Roosting and Breeding	Roosting and Breeding
Tuart	11	1	Roosting and Breeding	Breeding
Jarrah	384	2	Roosting and Breeding	Roosting and Breeding

Tree Species	Number of trees recorded within the DSP Area	Number of trees recorded within the TOD Precinct	Usage by Black Cockatoos	
			Carnaby's Cockatoo	Forest Red-tailed Black Cockatoo
Bullich	1	0	Nil	Breeding
Unknown	8	1	Unknown	Unknown
Total	**458**	**47**		

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Legend:
- Project area
- Cadastral boundary

Fauna habitat
- Heath
- Jarrah and planted

Fauna observation
- Quenda (P3)
- Roads (MRWA)

Scale 1:6,500 at A4 — 0 50 100 metres

Coord. Sys. GDA 1994 MGA Zone 50

Job No: 57806

Client: Element

Version: A | Date: 11-Apr-2021

Drawn By: cthatcher | Checked By: HS

Forrestfield North TOD Precinct
Perth, WA

FAUNA HABITAT

FIGURE 2.9

strategen JBS&G

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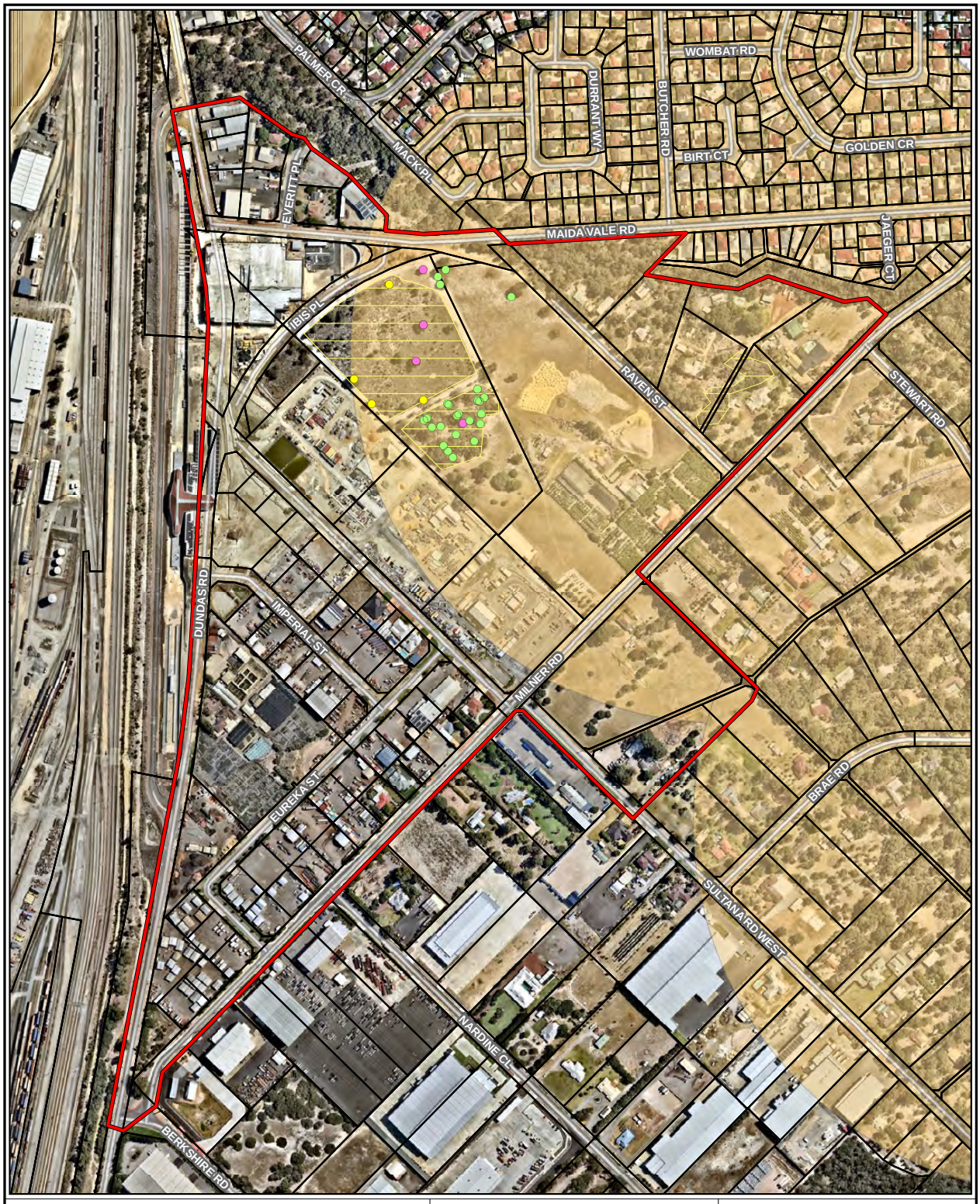
Image Reference: www.nearmap.com© - Imagery Date: 27 February 2021



Legend

- Project area
- Cadastral boundary
- Confirmed roost sites (1 km buffer) (Birdlife 2019)
- Black Cockatoo foraging habitat
- Roads (MRWA)
- Black Cockatoo potential habitat trees (AECOM 2017)
- Black Cockatoo potential habitat trees (RPS 2021)
- Forest Red-tailed Black Cockatoo foraging evidence (AECOM 2017)

Scale 1:6,500 at A4 (0 50 100 metres)		**Forrestfield North TOD Precinct Perth, WA**
Coord. Sys. GDA 1994 MGA Zone 50		**BLACK COCKATOO HABITAT**
Job No: 57806		
Client: Element		**FIGURE 2.10**
Version: A	Date: 12-Apr-2021	strategen JBS&G
Drawn By: cthatcher	Checked By: HS	

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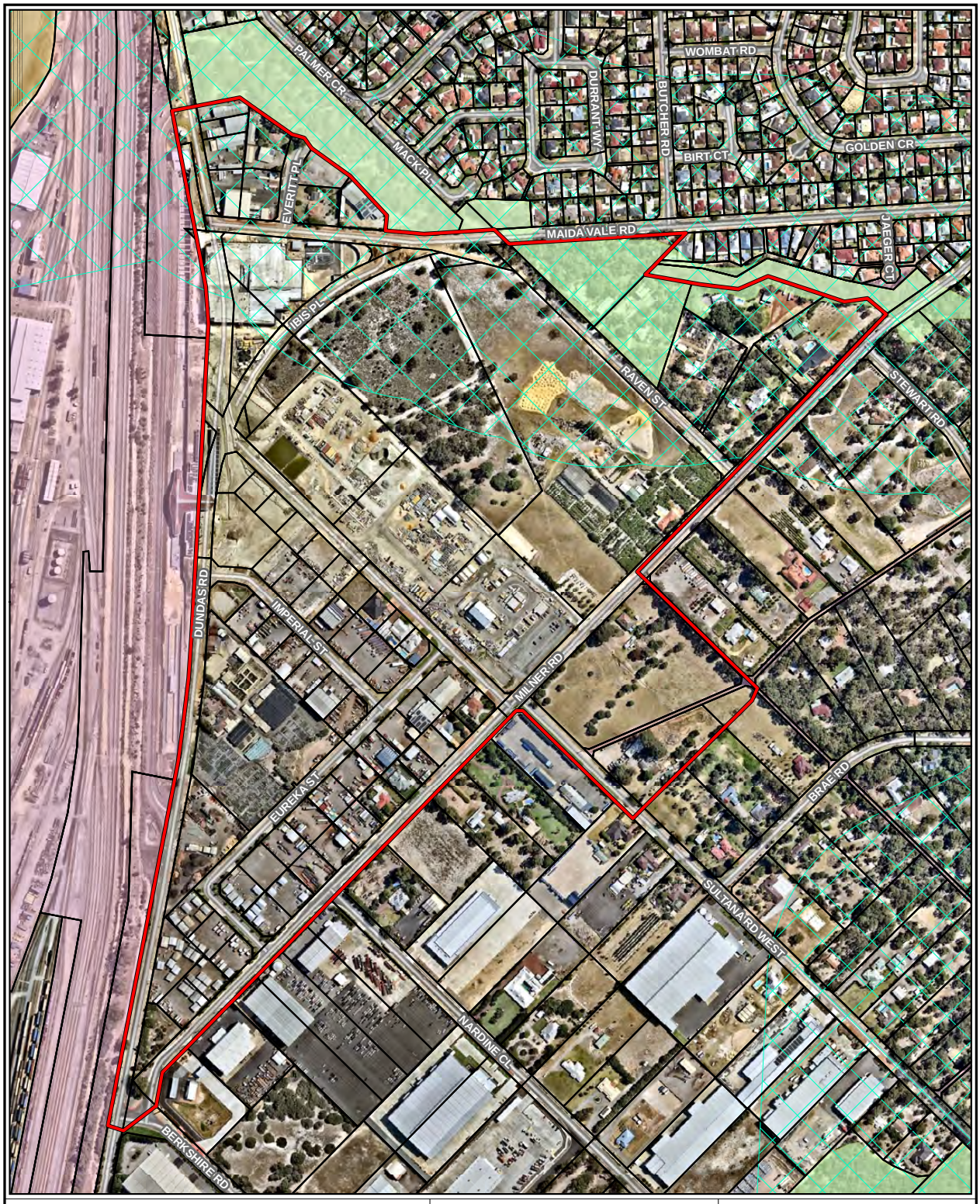
Image Reference: www.nearmap.com© - Imagery Date: 27 February 2021



Legend:

- Project area
- Cadastral boundary
- Bush Forever site (DOP, 2000)
- Other crown reserves
- Regional ecological linkages
- Roads (MRWA)

Scale 1:6,500 at A4 (0 50 100 metres)		**Forrestfield North TOD Precinct Perth, WA**
Coord. Sys. GDA 1994 MGA Zone 50		**BUSH FOREVER SITES AND REGIONAL ECOLOGICAL LINKAGES**
Job No: 57806		
Client: Element		**FIGURE 2.11**
Version: A	Date: 11-Apr-2021	strategen JBS&G
Drawn By: cthatcher	Checked By: HS	

File Name: \\008PMPMR004V001.jbsg.aust\JBS Perth\Projects\1)Open\Element (TPG)\57806 - FF North TOD & Res LWMS\GIS\Maps\R05_Rev_A\57806_2_11_BFConservation.mxd
Image Reference: www.nearmap.com© - Imagery Date: 27 February 2021

2.7 Bush Forever sites

Bush Forever site 45, Poison Gully Bushland, runs along the northern cadastral boundary of the precinct and extends into Lots 1353, 104 and 103 (Figure 2.10: Black Cockatoo habitat

Figure 2.11: Bush Forever sites and regional ecological linkages

). Additionally, Bush Forever site 123 is located approximately 500 m south east of the TOD Precinct.

2.8 Ecological linkages

The importance of ecological linkage is to connect natural areas, preferably with continuous corridors of native vegetation, which facilitates fauna movement between habitat areas (Del Marco et al. 2004). The protection, management and buffering of existing natural areas within an ecological linkage is considered a priority than revegetation of cleared portions of the link. The TOD Precinct overlaps a mapped east – west Regional Ecological Linkage (WALGA 2004) associated with Poison Gully Creek (Figure 2.11). The core, vegetated portion of the mapped linkage is within Bush Forever site 45, which will be maintained within the Poison Gully Creek foreshore reserve.

2.9 Environmentally sensitive areas

Environmentally Sensitive Areas (ESAs) are areas that have been identified for protection given their environmental significance as outlined in the Western Australian Environmental Protection (Environmentally Sensitive Areas) Notice 2005, which was gazetted on 8th April 2005. A mapped ESA intersects the TOD Precinct, which is associated with a TEC and associated buffer (EDA ID 116826). Based on the findings of AECOM (2017) and RPS Environmental (2021) this is considered associated with the occurrence of SCP 20a within Lot 12, which has been found not to meet the criteria as representative of the Banksia Woodland TEC due to the absence of key Banksia species.

2.10 Bushfire risk

According to regional Bushfire Prone Mapping available through DFES (2019) the north-eastern portion of the precinct is mapped within a bushfire prone area (Plate 2).

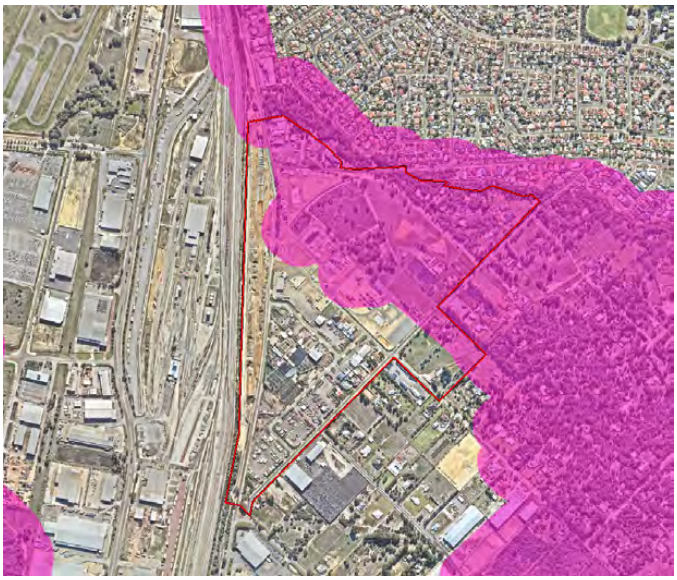


Plate 2: Bushfire Prone Areas (DFES 2019)

As a result of the bushfire prone status of the site, a Bushfire Management Plan (BMP) is required to accompany the Structure Plan to address the following requirements of *State Planning Policy 3.7 Planning in Bushfire Prone Areas* (SPP 3.7), namely Policy Measure 6.3:

- a bushfire hazard level (BHL) assessment or where lot layout is known, a Bushfire Attack Level (BAL) contour assessment to determine the indicative acceptable BAL ratings across the site
- identification of any bushfire hazard issues arising from the above assessment
- assessment against the bushfire protection criteria requirements contained within the Guidelines demonstrating compliance can be achieved in subsequent planning stages.

The BMP has been prepared by Strategen-JBS&G (2021) in accordance with *Guidelines for Planning in Bushfire Prone Areas* (the Guidelines). The BMP is a separate document and should be read concurrently with this EAMS.

2.11 Heritage

2.11.1 Indigenous heritage

The *Aboriginal Heritage Act 1972* (AH Act) aims to protect Aboriginal heritage by registering Aboriginal sites (places and/or objects) that are of cultural importance to Aboriginal people. Any proposal to use or alter an area of land, for purposes such as research or development, must first determine if Aboriginal sites occur within the proposed area. If an Aboriginal site is found to occur, permission must be sought from the Minister for Aboriginal Affairs before that land can be used or altered in any way.

The Forrestfield area surrounding the TOD Precinct holds significant value to Noongar people and is known to contain many places of major significance including Allawah Grove Reserve, Monday Swamp located at the Perth Airport and Poison Gully Creek, which is located along the northern boundary of the TOD Precinct. Ongoing heritage investigations in and around the airport indicate that Munday Swamp is still being used for hunting and foraging (Turner et al. (Ethnosciences) 2018).

Noongar families have continued to use areas in and around Forrestfield for camping, hunting, and rural-based employment. Of particular interest is the old Welshpool Reserve or Maamba (DPLH ID 3773) located in Forrestfield to the south of the DSP area. The old reserve(s), which is a Registered Aboriginal Site, is depicted on the AHIS as continuous with the boundaries of the present-day Hartfield Park, Forrestfield/Wattle Grove, Kalamunda, though the exact boundaries of the place are problematic as there seems to have been at least two Aboriginal reserves in the area with different reserve numbers and which may have overlapped (Turner et al. (Ethnosciences) 2018).

Through a search of the Department of Planning and Lands and Heritage (2019) Aboriginal Heritage Inquiry System (AHIS) one registered site (Poison Gully Creek; Site ID 25023) was found to intersect the Project area. This site is a water source and historical birthplace that has been assessed as meeting Section 5 of the AH Act. One 'Other Heritage Site', Crumpet Creek (ID 3667), also intersects the Project area. This site is listed as Artefacts/Scatter and does not meet the requirements of a Registered Heritage Site.

2.11.1.1 Ethnographic heritage assessment

An ethnographic heritage assessment was completed by Turner et al. (Ethnosciences) (2018) for the Forrestfield North DSP area, including the TOD Precinct. The assessment involved both desktop research and community consultation with the South West Aboriginal Land and Sea Council (SWALSC), a panel of Aboriginal people having an association with and knowledge of Poison Gully, and separate ethnographic consultations with on-site meetings. A summary of the assessment outcomes is provided below.

The search of the online AHIS and other archival sources confirmed Poison Gully Creek (DPLH ID 25023) to be the only Registered Aboriginal Site in the DSP study area that intersects the TOD Precinct.

Poison Gully Creek (DPLH ID 25023) was the only place within the TOD Precinct reported by the Aboriginal consultants that might reasonably be considered an Aboriginal Site within the meaning of section 5 of the AHA. Turner et al. (Ethnosciences) (2018) reported that as the site file is 'Closed', the boundaries on the AHIS are 'restricted' and therefore 'dithered'; i.e the spatial representation published on the AHIS website is broader than the actual boundary, because of the site's reported significance and hence confidentiality regarding the place's precise boundaries.

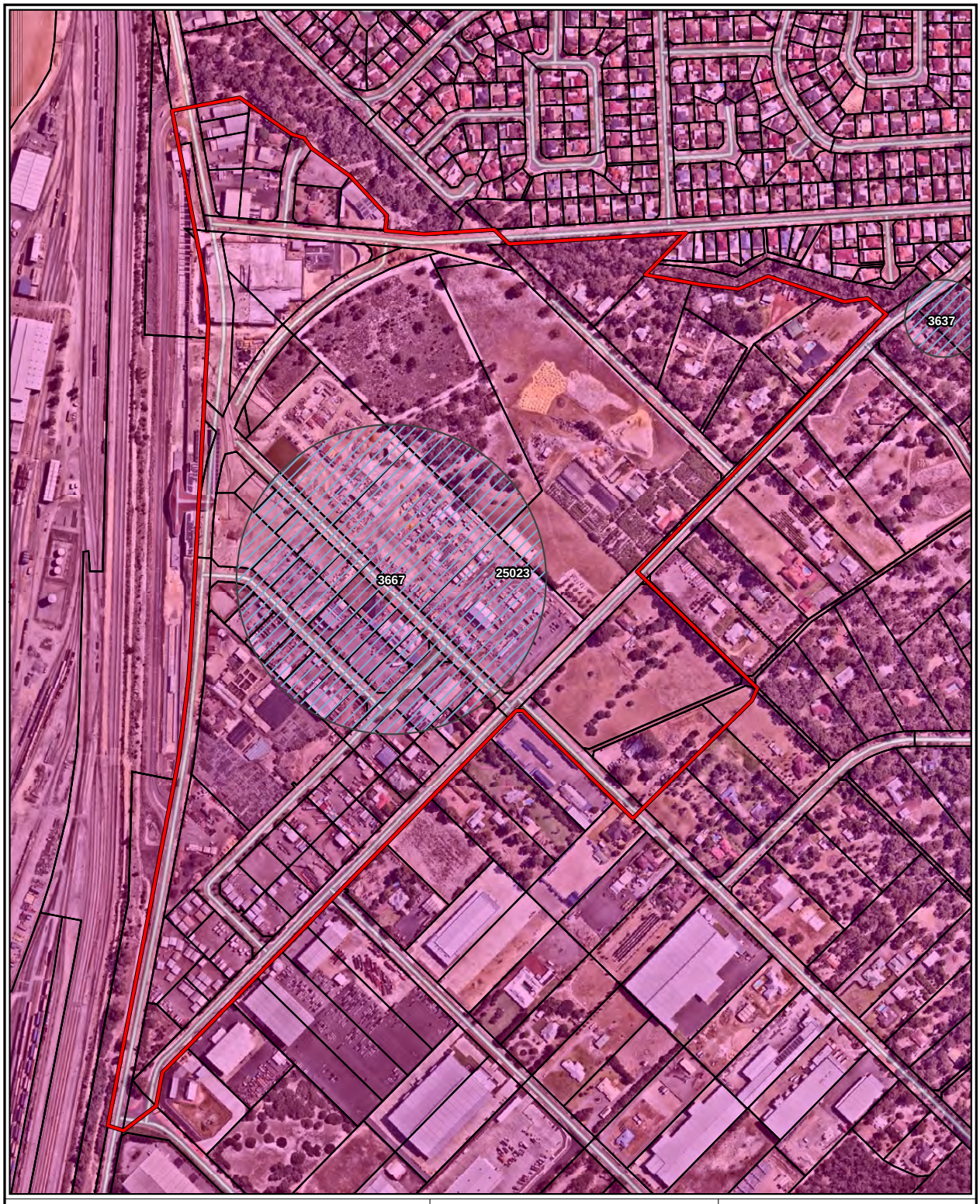
Turner et al. (Ethnosciences) (2018) discusses that areas around the TOD Precinct, such as the Perth Airport land and the Newburn Marshalling Yards, have been intensively surveyed and as a result many archaeological sites, typically artefact scatters, have been recorded. The TOD Precinct has not been surveyed in detail, forming only a part of a previous regional survey. Based on regional information it is reasonable to assume, that should an archaeological survey of the Forrestfield North DSP (including the TOD Precinct) be completed it is likely that additional archaeological sites to DPLH ID 3667 would be found. As noted above, the Forrestfield North DSP/TOD area has not been specifically surveyed archaeologically.

Consultation confirmed the cultural significance of Poison Gully Creek and highlighted its importance to Noongar women, particularly as a 'birthing place' with associated rituals and still seen as a place for teaching and learning about traditional cultural knowledge. This is specific both to the site as well as the country more generally. The key issues raised during consultation included:

- protection of remnant vegetation and creek and foreshore between Dundas Road and Milner Road and in particular the open space adjacent to the Dundas Road crossing;
- improvement and restoration of water flow and water quality within Poison Gully Creek to reflect the waterways original state;
- Poison Gully Creek as a place of significant values and is related to other significant areas such as Munday Swamp and Allawah Grove, traditionally linked by the flow of water and the bidi or tracks and part of a meshwork in which place is to be understood as an outcome of movement, practice and event, which result in the experience of 'biographical entanglements';
- continuing access to and control of the land as its traditional owners and custodians; and
- linkages between the Forrestfield North DSP/LSP area and the surrounding landscape.

2.11.2 European heritage

The Heritage Council (2017) InHerit database was searched for registered sites, which found that there are currently no sites within the precinct area.



Legend:
- Project area
- Cadastral boundary

Aboriginal heritage places (DAA-001)
- Registered site
- Other heritage place

Roads (MRWA)

Scale 1:6,500 at A4

0 50 100
metres

Coord. Sys. GDA 1994 MGA Zone 50

Job No: 57806

Client: Element

Version: A | Date: 11-Apr-2021

Drawn By: cthatcher | Checked By: HS

Forrestfield North TOD Precinct
Perth, WA

HERITAGE

FIGURE 2.12

strategen JBS&G

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3. Management Strategy

As part of the formulation of the ACP, an assessment of opportunities and constraints of the existing environment was undertaken to influence the design of the precinct, including the location of the POS areas, drainage basins/swales, environmental conservation reserves, and linkages.

3.1 Landforms

3.1.1 Policy and management objectives

Where possible, maintain the gentle sloping topography across the precinct and to retain the rural lifestyle amenity of the precinct. To also ensure that the subdivision and development of land containing ASS is planned and managed to avoid potential adverse effects on the natural and built environment.

3.1.2 Activity Centre Plan considerations

An assessment of the on-site environmental landform features has indicated there are no significant constrains to development.

3.1.3 Management requirements

Management measures have been identified to assist in achieving the Landforms management objectives which are outlined in Table 3.1.

Table 3.1: Management strategies for Landforms

Parameter	Management actions	Timing/ Responsibility
Geotechnical	Complete a geotechnical assessment including an analysis of infiltration rates Within key drainage areas.	Once drainage design has been confirmed (refer to the LWMS). City of Kalamunda/developer.
ASS	Confirm if dewatering below the water table or the excavation of more than 100 m^3 is required as part of earthworks for the ACP.	Prior to earthworks commencing within the precinct. City of Kalamunda/developer.
	Complete an ASS desktop investigation and completion of a self-assessment form to confirm whether ASS investigations are required.	Completion and finalisation precinct earth works plans (cut and fill, infrastructure, drainage services). City of Kalamunda/developer.
	Prepare an ASS Management Plan (if ASS is present, and site works will intersect ASS areas) that includes: • potential environmental impacts—groundwater drawdown, disruption to existing bore users, vegetation stress, reduction in water quality, noise and air impacts • earthwork strategy—soil extraction methods, stockpile management, soil treatment/neutralisation, calculated liming rate and disposal techniques • dewatering strategy—procedure and control measures, treatment and disposal options, contingency measures if acidification of groundwater occurs • monitoring program—soil, groundwater, vegetation, noise, and air.	Before earthworks commence onsite (Subdivision). City of Kalamunda/developer.

3.1.4 Predicted environmental outcomes

Through the implementation of the management recommendations outlined in Table 3.1, the quality of land and soils can be maintained so that landform values within the TOD Precinct are incorporated through LPS design.

3.2 Hydrology

3.2.1 Policy and management objectives

The key principles and objectives for sustainable water management for the TOD Precinct is provided in the following:

- *Liveable Neighbourhoods Edition 4* (WAPC 2009)
- *Water Resources Statement of Planning Policy 2.9* (WAPC 2004)
- *Stormwater Management Manual for WA* (Department of Water 2007)
- *Decision Making Process for Stormwater in Western Australia* (Decision Process, DWER 2017)
- *Better Urban Water Management* (WAPC 2008)
- *Interim: Developing a Local Water Management Strategy* (DoW 2008)
- *Forrestfield North District Water Management Strategy* (Strategen 2015)

3.2.2 Activity Centre Plan considerations

Key design objectives outlined to be achieved through the LSP were identified by Urbaqua (2020) are summarised in Table 3.2.

Table 3.2: Key design objectives for LSP – adapted from Urbaqua (2020)

Design element	Objectives
Water Sustainability	• Maximize local infiltration to replenish surface groundwater aquifers; and • Provide alternative water sources for domestic irrigation and in-house domestic non-drinking water demands.
Surface water management	• Maintain or restore desirable environmental flows and/or hydrological cycles. • For flood management, manage up to the 1% AEP (100-year ARI event) within the development area to pre-development flows and Water Corporation requirements (as per the Limestone Creek Capacity Review) • Ensure that the first 15 mm of rainfall receives treatment prior to discharging to a receiving environment. • Minor events should be managed to provide serviceability requirements. • Roads and public open spaces are to be designed to cater for the surface overflow for more severe storm events; and • Habitable floors at least 300 m above the 1% AEP flood level in the local drainage system and 500 m above the 1% AEP flood level in basins/sumps (where there is no overflow relief) and Poison Gully Creek.
Groundwater management	• Provide appropriate separation between finished lot levels and groundwater to provide the expected level of amenity. • Ensure soak wells devices are designed at or above maximum groundwater levels; and • Ensure that any subsoil drainage is treated prior to discharge to water was and sensitive environments.
Management of disease vectors and nuisance insects	• Limit the creation of new sits or breeding of nuisance insects. • Prevent standing water in drainage infrastructure (infiltration within 96 hours); and • Improve water quality throughout the development.

3.2.3 Management requirements

Management measures have been identified to assist in achieving the hydrology management objectives (Table 3.3). Specific management objectives and actions will be provided in the approved LWMS for the project, including specific surface water and drainage basin requirements (Urbaqua 2020).

Table 3.3: Management strategies for Hydrology

Parameter	Management actions	Timing/ Responsibility
Retention of wetland and associated buffer	Management will be undertaken in accordance with the Vegetation Management Plan (VMP) prepared by RPS Environmental (2021b). The plan specifies actions regarding: • Tree retention, • Public access restriction and fencing for Conservation Area, • Weed management, • Plant pathogens; and • Monitoring.	A Vegetation Management Plan has been prepared by RPS Environmental. Management of conservation reserves will be by the City of Kalamunda. Timing is to be throughout construction.
Works proposed within the Poison Gully foreshore area (also refer to Section 2.11 Heritage).	Development of a Construction Management Plan (pre- and during construction) which includes: • Consultation with the Noongar community, • Environmental outcomes and performance indicators, • Risk assessment and management measures, • Monitoring, • Contingency response and corrective action, and • Report and review.	Timing is to be prior to development/earth works within the creek foreshore area.

3.2.4 Predicted environmental outcomes

It is expected that through the implementation of the management measures in Table 3.3 and accordance with the LWMS, the hydrology of the wetland on Lot 12 and Poison Gully Creek will be maintained.

3.3 Biodiversity and natural assets

3.3.1 Potential impact

The following potential impacts have been identified associated with the redevelopment of the TOD Precinct in accordance with the ACP:

- Removal of quality foraging habitat for Forest Red-tailed Black Cockatoo
- Removal of valued quality foraging habitat for Carnaby's Cockatoo
- Removal of potentially significant habitat trees for Black Cockatoo species
- Removal of Quenda habitat

3.3.2 Policy and management objectives

To retain and conserve viable good condition riparian vegetation and fauna habitat and create a planning outcome which will ensure the long-term protection and management of the proposed ECRs. In regard to the retention of these matters (i.e black cockatoo habitat trees) within proposed

POS areas is subject to drainage, landscaping, bushfire and engineering requirements and will be finalised in subsequent planning processes i.e. subdivision.

3.3.3 Activity Centre Plan considerations

As part of the opportunity and constraints assessment of environmental matters within the precinct area, the outcomes of AECOM (2017) and previous assessments was analysed, including the following key matters:

- SCP 20a Banksia attenuata over species-rich dense woodlands ecological community,
- Riparian vegetation associated with conservation category wetland,
- Black cockatoo habitat trees, and
- Black cockatoo foraging habitat.

Given historic disturbance within the precinct area, these values were largely spatially confined to the mapped wetland area. The application of at least a 50 m buffer on the east and south of the wetland area encompasses the majority of mapped values within the precinct. Consultation with stakeholders (list) confirmed that the key natural area containing conservation category wetland is of state significance.

As a result, an area of environmental conservation has been delineated within Lots 9 and 12, with a buffer of local open space immediately adjacent which extends 50 m to the east and up to 112 m in the south (Appendix A).

A summary of the matters to be retained within the environmental conservation area is provided in Table 3.4.

Table 3.4: Proposed retention of environmental matters within conservation reserves and local open space within the TOD Precinct

Environmental Matter	Existing extent identified in TOD ACP area	Extent proposed to be retained in conservation area	Extent impacted
Remnant native vegetation in 'Good' or better condition.	2.73	2.73	N/A
Black Cockatoo habitat trees	47	45	2
TEC SCP20a	2.81	2.81	N/A
Black Cockatoo foraging habitat	3.60	2.81	0.78

3.3.4 Management requirements

Management measures have been identified to assist in achieve in the Biodiversity and Natural Assets management objectives (Table 3.5).

Table 3.5: Management measures for Biodiversity and Natural Assets

Parameter	Management actions	Timing/responsibility
Bushfire risk	Implementation of Bushfire Management Plan.	Once BMP approved by relevant government agencies – ongoing. City of Kalamunda / developer.
Protection and enhancement of vegetation	Implementation of Vegetation Management Plan	Ongoing City of Kalamunda / developer

Parameter	Management actions	Timing/responsibility
	Development of a Construction Environmental Management Plan to incorporate environmental elements during pre-construction and during construction encompassing: • Identification of potential threats/impacts and risks associated with construction activities within and adjacent to the conservation area (e.g dieback, fauna, and habitat management • Environmental outcomes and performance indicators • Management measures and monitoring • Contingency response and corrective actions • Reporting and review	

3.3.5 Predicted environmental outcomes

The environmental conservation areas have been committed to by the City of Kalamunda and have been reflected in the ACP. Within the TOD precinct, these retention areas conserve:

- 100% of good or better condition vegetation
- 95% Black Cockatoo habitat trees
- 78% Black Cockatoo foraging habitat.

The percentages above represent the minimum retention of each environmental matter; that is, included within the conservation area itself. The area to the south and east of the conservation area will be designated as local open space and permit the retention of the Black Cockatoo values.

3.3.6 Secondary approval requirements

Whilst MNES is present within the precinct area, given the retention proposed by the ACP, it is considered unlikely that future development projects within the Project area will require referral or assessment under the EPBC Act referral.

Under the DP Act, application for subdivision to implement the development areas will be required. It is likely that there will be subdivision conditions applied, associated with key environmental matters identified in Table 3.4.

3.4 Bushfire risk

3.4.1 Policy and management objectives

Ensure that the bushfire risks to future life, property and environmental assets are minimised through appropriate development design and implementation of bushfire management measures.

3.4.2 Activity Centre Plan considerations

A BMP for the precinct will be required, including the following:

- proposal details
- environmental considerations

- bushfire assessment results, including results of site assessment (classified vegetation, effective slope and exclusions) and bushfire hazard level assessment for pre-and post-development conditions
- identification of bushfire hazard issues
- assessment against the bushfire protection criteria of the Guidelines
- responsibilities for implementation and management.

3.4.3 Management requirements

Management measures have been identified to assist in achieving the Bushfire management objectives (Table 3.6).

Table 3.6: Management measures for bushfire risk

Parameter	Management actions	Timing/responsibility
Bushfire risk	Implementation of Bushfire Management Plan.	Once BMP approved by relevant government agencies – ongoing. City of Kalamunda / developer.

3.4.4 Predicted environmental outcomes

Vegetation retention and any revegetation within the POS areas including conservation areas particularly along the urban interface (development area) will need to incorporate bush fire requirements and management measures (i.e. low threat status). Further detail will be provided in the BMP.

3.5 Heritage and culture

3.5.1 Policy and management objectives

Ensure that the heritage values associated with Poison Gully Creek are preserved and where possible incorporated into the POS landscape design.

3.5.2 Activity Centre Plan considerations

The ACP is reflective of the heritage values associated with Poison Gully Creek including:

- incorporating existing crossing (Maida Vale Road, Milner Road and Littlefield Road). Littlefield road will be a pedestrian/cycle access linking Maida Vale area with the DSP area,
- control access along the interface of the precinct and Poison Gully foreshore area through designated paths, bollards, and fencing,
- local open space has been allocated where the creek foreshore area impedes the precinct boundary, while potential drainage storage and outflow (via upgrades to existing culverts may be required), the focus of these areas will be passive recreation and revegetation to compliment the natural assets of the creek,

3.5.3 Management requirements

Management measures have been identified to assist in achieving the Heritage management objectives (Table 3.7).

3.5.4 Predicted environmental outcomes

Table 3.7: Management measures for heritage

Parameter	Management actions	Timing/responsibility
Preservation of heritage values	• CoK and key stakeholders to continue their consultations with the Noongar community in respect of the Forrestfield North DSP, particularly as this might impact on Poison Gully Creek (DPLH ID 25023).	Ongoing.
	• Noongar community invited to engage in any works associated with Poison Gully Creek, including the rehabilitation of the creek and revegetation with native vegetation. • This is relevant to implementation to the SCMP (refer to Section 4.4.4) for ECRs which have an interface with Poison Gully Creek i.e. ESR 10, 11, 12 and 13.	The SCMP will be implemented by the COK Subdivision phase by the Developer
Any required construction works within Poison Gully Creek and/or foreshore	• Determine whether a s.18 permit is likely to be required under the *Aboriginal Heritage Act 1972* or approval under Regulation 10 from the Register of Aboriginal Site: • consultation with DPLH and relevant Aboriginal groups to confirm the significance of existing Aboriginal Heritage sites, and to confirm whether an Aboriginal Heritage survey and/or s 18 permit is likely to be required. • an Aboriginal Heritage survey of the Project to provide further detail on the significance and location of Aboriginal Heritage sites and prepare an s18 notice form under the AH Act, if required.	Prior to disturbing works- process approximately 6 months. CoK and Developer.
Heritage education and opportunities	• In conjunction with Noongar women, examine ways in which the natural bush and creek between Dundas Road and Milner Road can be enhanced as a Noongar women's place. • The potential incorporation of public art and interpretative/education signs within POS to reflect the heritage importance of the area.	Subdivision phase CoK and Developer.
Further archaeological survey	• Archaeological survey be undertaken for the Residential Precinct area.	Prior to the commencement of development CoK and/or developer.

4. Limitations

This report has been prepared for use by the client who has commissioned the works in accordance with the project brief only, and has been based in part on information obtained from the client and other parties.

The advice herein relates only to this project and all results conclusions and recommendations made should be reviewed by a competent person with experience in environmental investigations, before being used for any other purpose.

Strategen-JBS&G accepts no liability for use or interpretation by any person or body other than the client who commissioned the works. This report should not be reproduced without prior approval by the client, or amended in any way without prior approval by Strategen-JBS&G, and should not be relied upon by other parties, who should make their own enquires.

Sampling and chemical analysis of environmental media is based on appropriate guidance documents made and approved by the relevant regulatory authorities. Conclusions arising from the review and assessment of environmental data are based on the sampling and analysis considered appropriate based on the regulatory requirements.

Limited sampling and laboratory analyses were undertaken as part of the investigations undertaken, as described herein. Ground conditions between sampling locations and media may vary, and this should be considered when extrapolating between sampling points. Chemical analytes are based on the information detailed in the site history. Further chemicals or categories of chemicals may exist at the site, which were not identified in the site history and which may not be expected at the site.

Changes to the subsurface conditions may occur subsequent to the investigations described herein, through natural processes or through the intentional or accidental addition of contaminants. The conclusions and recommendations reached in this report are based on the information obtained at the time of the investigations.

This report does not provide a complete assessment of the environmental status of the site, and it is limited to the scope defined herein. Should information become available regarding conditions at the site including previously unknown sources of contamination, Strategen-JBS&G reserves the right to review the report in the context of the additional information.

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Appendix A Forrestfield North TOD ACP

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LEGEND

Activity Centre Plan Boundary

EXISTING PLANNING FRAMEWORK

Metronet East Redevelopment Area

Project Area: Forrestfield North (RAC-0)

Ecological Conservation (Conservation Category Wetland)

Bush Forever

PROPOSED PLANNING FRAMEWORK

Region Scheme Reserves

Railway

Local Scheme Reserves and Environmental Conservation

Local Open Space

Public Purposes: Community Purpose

Environmental Conservation

Other Categories

District Integrator A (Existing / Proposed)

District Integrator B (Existing / Proposed)

Neighbourhood Connector (Existing / Proposed)

Local Street (Existing / Proposed)

Proposed Roundabout

Proposed Signalised Intersection

A Left-in, Left-out Intersection

B No Right Out Intersection

Strategic Site

Hazard seperation required at subsequent planning stages to ensure that no residential land is affected by BAL-40 or BAL-FZ

Indicative Drainage Corridor

Notes

1 Final road widths and indicative cross sections to be determined following a traffic assessment

Precincts

1 Forrestfield Station Precinct

2 Residential Precinct

Sub-Precincts

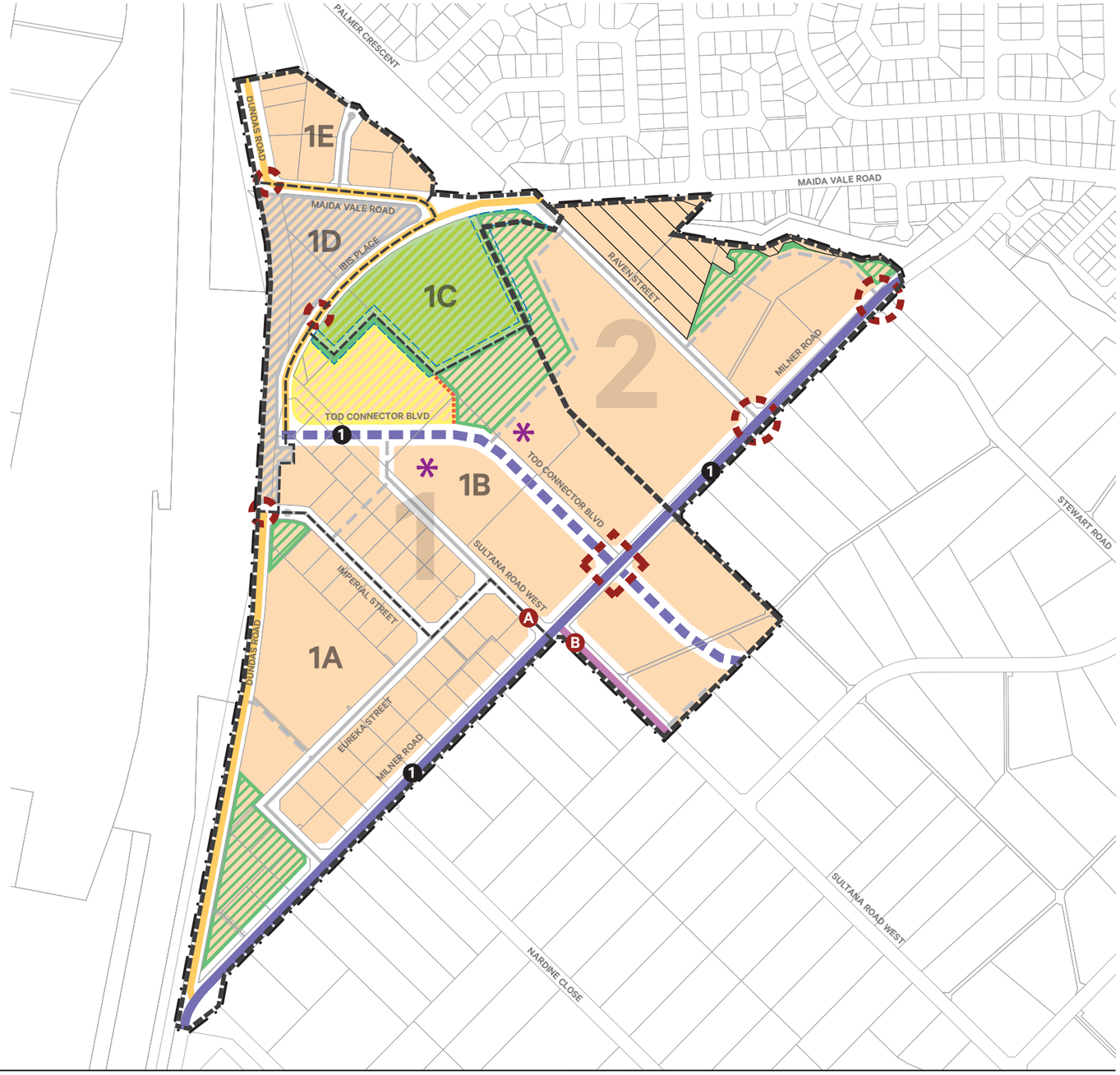
1A Eureka Sub-Precinct

1B Core Sub-Precinct

1C Conservation Sub-Precinct

1D Station Sub-Precinct

1E Maida Vale Sub-Precinct



Activity Centre Plan

Forrestfield North TOD Precinct

DRAFT

Date: 23 Mar 2020 Scale: 1:6000@ A3 1:3000 @ A1 File: **17-527 ST-1 A** Staff: MC GW Checked: MC

0 60 120m

element.

Level 18, 191 St Georges Terrace, Perth Western Australia 6000.
PO Box 7375 Cloisters Square, Perth Western Australia 6850.
T. +61 8 9289 8300 | **E.** hello@elementwa.com.au **elementwa.com.au**


www.jbsg.com.au

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Technical Appendix B - JBS&G Bush Fire Management Plan

FPA AUSTRALIA | Fire Protection Association Australia | Life. Property. Environment.

BPAD Bushfire Planning & Design

Bushfire Management Plan Coversheet

This Coversheet and accompanying Bushfire Management Plan has been prepared and issued by a person accredited by Fire Protection Association Australia under the Bushfire Planning and Design (BPAD) Accreditation Scheme.

Bushfire Management Plan and Site Details

Site Address / Plan Reference: Forrestfield North TOD Precinct

Suburb: High Wycombe **State:** WA **P/code:** 6057

Local government area: City of Kalamunda

Description of the planning proposal: Activity Centre Plan application

BMP Plan / Reference Number: 57806/137,446 **Version:** R06 Rev 0 **Date of Issue:** 04/10/2021

Client / Business Name: Element on behalf of City of Kalamunda

Reason for referral to DFES	Yes	No
Has the BAL been calculated by a method other than method 1 as outlined in AS3959 (tick no if AS3959 method 1 has been used to calculate the BAL)?	☐	☒
Have any of the bushfire protection criteria elements been addressed through the use of a performance principle (tick no if only acceptable solutions have been used to address all of the BPC elements)?	☐	☒
Is the proposal any of the following special development types (see SPP 3.7 for definitions)?		
Unavoidable development (in BAL-40 or BAL-FZ)	☐	☒
Strategic planning proposal (including rezoning applications)	☒	☐
Minor development (in BAL-40 or BAL-FZ)	☐	☒
High risk land-use	☐	☒
Vulnerable land-use	☐	☒

If the development is a special development type as listed above, explain why the proposal is considered to be one of the above listed classifications (E.g. considered vulnerable land-use as the development is for accommodation of the elderly, etc.)?

Activity Centre Plan application

Note: The decision maker (e.g. local government or the WAPC) should only refer the proposal to DFES for comment if one (or more) of the above answers are ticked "Yes".

BPAD Accredited Practitioner Details and Declaration

Name	Accreditation Level	Accreditation No.	Accreditation Expiry
Louisa Robertson	Level 3	BPAD 36748	28/02/2022
Company		**Contact No.**	
Strategen-JBS&G		08 9792 4797	

I declare that the information provided within this bushfire management plan is to the best of my knowledge true and correct

Signature of Practitioner LMRobertson **Date** 04/10/2021

Element on behalf of City of Kalamunda

Bushfire Management Plan (Activity Centre Plan)

Forrestfield North
Transport Oriented Development Precinct

4 October 2021

57806/137,446 (Rev 0)

JBS&G Australia Pty Ltd T/A Strategen-JBS&G

BPAD Bushfire Planning & Design Accredited Practitioner Level 3

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Appendix A APZ standards (Schedule 1 of the Guidelines)
Appendix B Vegetation plot photos and description
Appendix C City of Kalamunda Fire Hazard Reduction Notice 2021/2022
Appendix D Vehicular access technical standards of the Guidelines
Appendix E Water technical standards of the Guidelines

1. Proposal details

1.1 Background

Element, on behalf of City of Kalamunda (the City), intends to lodge an Activity Centre Plan (ACP) over a 61.02 ha area of land known as the Forrestfield North Transport Oriented Development Precinct (TOD Precinct; the project area).

The ACP is depicted in Figure 1 and identifies:

- two proposed Precincts:
    - Forrestfield Station Precinct, with five sub-precincts
    - Residential Precinct
- proposed Ecological Conservation (EC) reserve, incorporating a Resource Enhancement Wetland (REW)
- proposed Railway Reserve
- proposed Local Open Space (LOS)
- proposed Public Purposes: Community Purpose cell
- proposed lot and road layout
- existing Bush Forever sites.

The TOD Precinct forms part of the overall Forrestfield North District Structure Plan (DSP), which also includes an adjacent 123.05 ha parcel of land known as the Forrestfield North Residential Precinct (which is separate to the TOD residential precinct discussed above). Both precincts are depicted on the Site Overview map at Figure 2.

1.2 Site description

The TOD Precinct is located in the suburb of High Wycombe and is bound by Dundas Road and rail reserve to the west, Poison Gully Creek and Maida Vale Road to the north and Milner Road to the east/southeast (Figure 2).

The project area comprises many lots with various tenure, with the predominant land uses being commercial and industrial. Rural-residential properties also exist along Milner Road and Raven Street.

A Resource Enhancement Wetland (REW) is located within the north/central portion of the project and is proposed to be retained within a conservation reserve (EC Reserve). The existing forest vegetation bordering Poison Gully Creek is proposed to be largely retained as Bush Forever.

The northern portion of the project area is designated as bushfire prone on the *Map of Bush Fire Prone Areas* (DFES 2019; see Plate 1).

1.3 Purpose

This Bushfire Management Plan (BMP) has been prepared to address requirements under *Policy Measure 6.3 of State Planning Policy 3.7 Planning in Bushfire-Prone Areas* (SPP 3.7; WAPC 2015) and *Guidelines for Planning in Bushfire-Prone Areas* (the Guidelines; WAPC 2017).

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1.4 Other plans/reports

Other relevant reports that have been prepared for the project area include:

- Bushfire Management Plan in respective of the adjacent Forrestfield North Residential Precinct (Strategen-JBS&G 2020a).
- Flora, Vegetation and Fauna Assessment in relation to Lot 9 and 12 Ibis Place, High Wycombe (PGV Environmental 2021a).
- Vegetation Management Plan in relation to Lot 9 and 12 Ibis Place, High Wycombe (PGV Environmental 2021b).
- Environmental Assessment and Management Strategy in respect of:
  - Forrestfield North Residential Precinct (Strategen-JBS&G 2020b); and
  - Forrestfield North TOD Precinct (Strategen-JBS&G 2021a).
- Local Water Management Strategy in respect of:
  - Forrestfield North Residential Precinct (Urbaqua 2020); and
  - Forrestfield North TOD Precinct (Strategen-JBS&G 2021b).



Plate 1: Map of Bush Fire Prone Areas (DFES 2019)

LEGEND

Activity Centre Plan Boundary

EXISTING PLANNING FRAMEWORK

Metronet East Redevelopment Area

Project Area: Forrestfield North (RAC-0)

Ecological Conservation (Conservation Category Wetland)

Bush Forever

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Local Street (Existing / Proposed)

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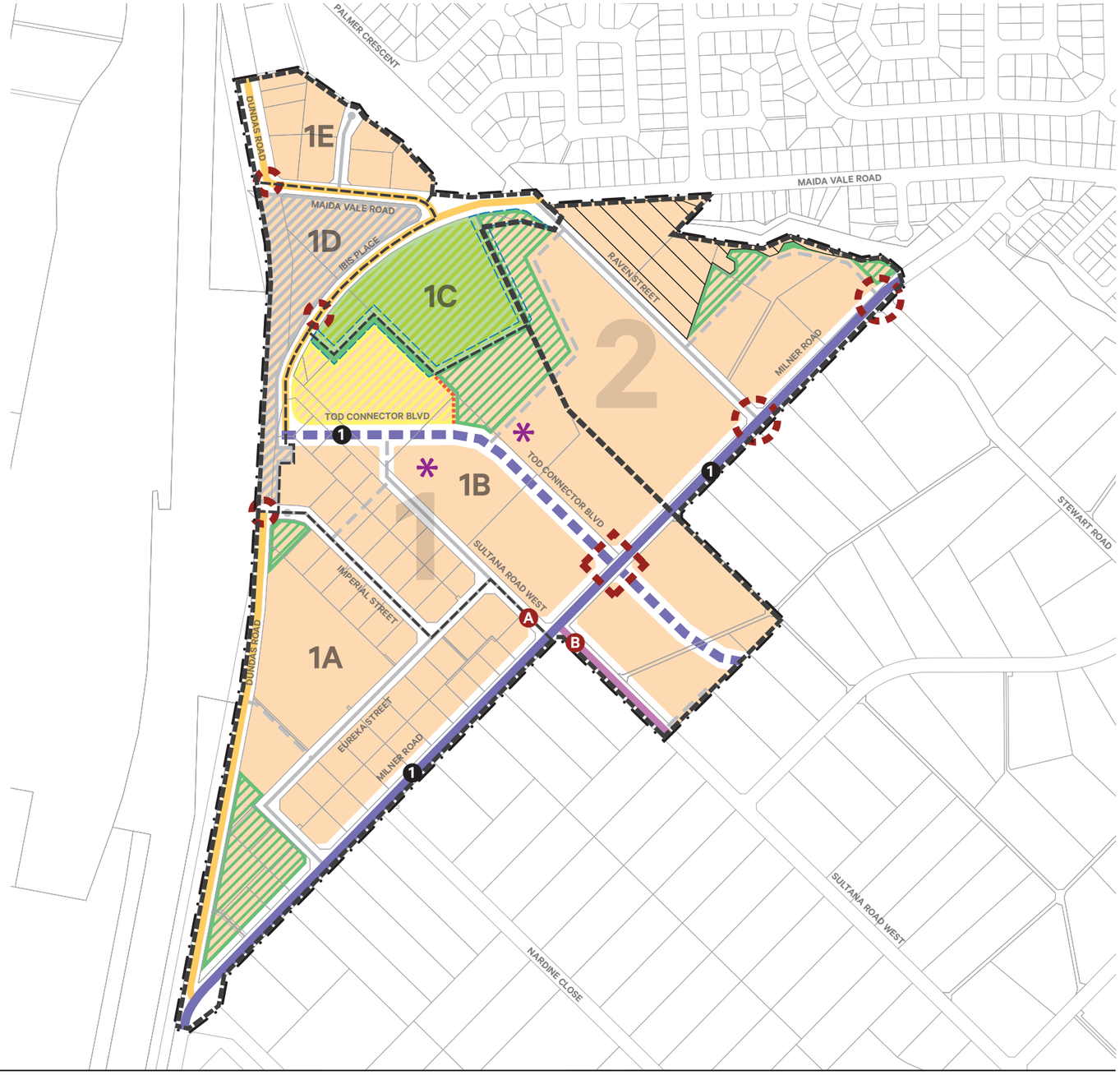
1D Station Sub-Precinct

1E Maida Vale Sub-Precinct



Activity Centre Plan

Forrestfield North TOD Precinct

DRAFT

Date: 23 Mar 2020 Scale: 1:6000@ A3 1:3000 @ A1 File: **17-527 ST-1 A** Staff: MC GW Checked: MC

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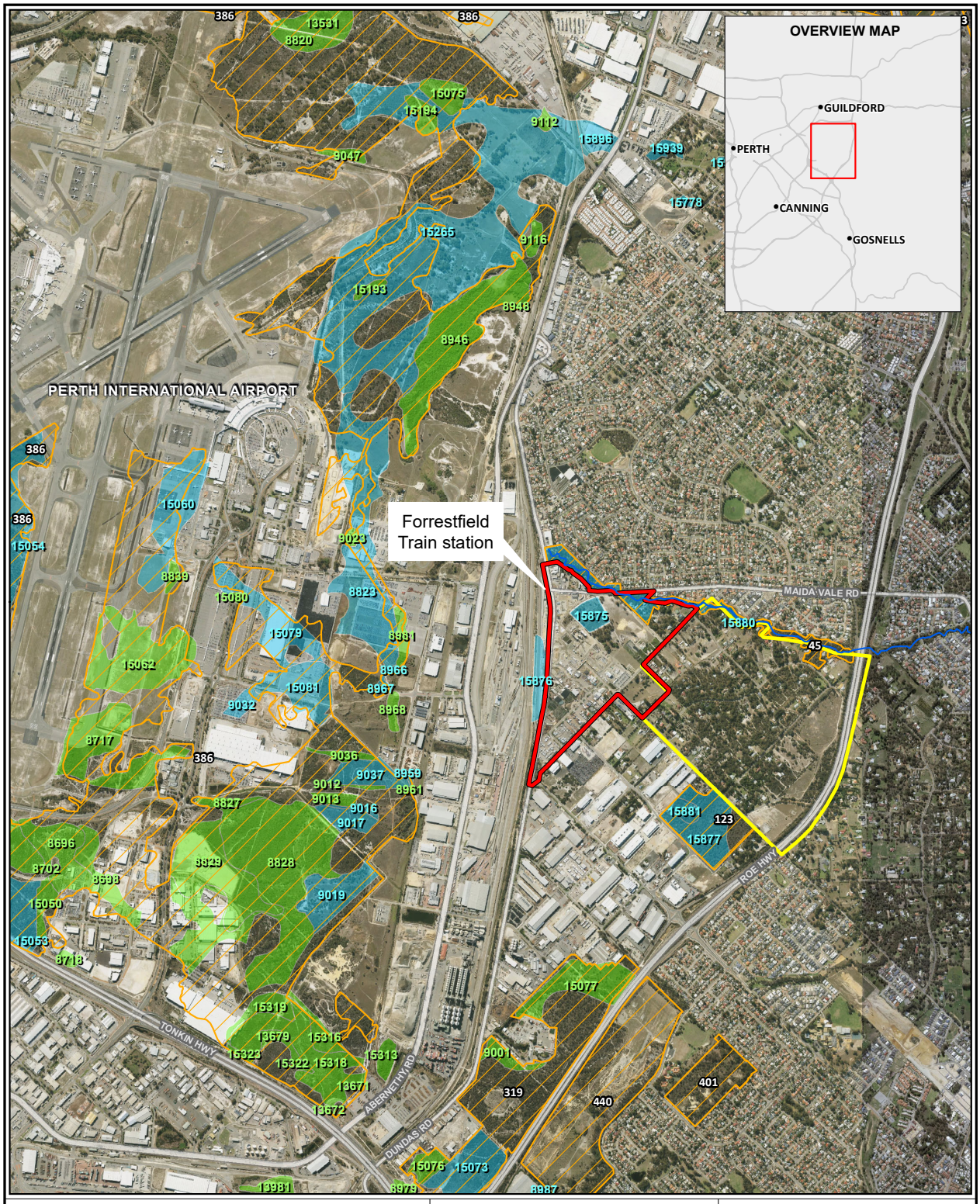
Level 18, 191 St Georges Terrace, Perth Western Australia 6000.
PO Box 7375 Cloisters Square, Perth Western Australia 6850.
T. +61 8 9289 8300 | **E.** hello@elementwa.com.au **elementwa.com.au**



Legend

- Project area
- Residential precinct
- Bush Forever sites
- Geomorphic Wetlands (DBCA)
  - Conservation
  - Resource enhancement
- Poison Creek Gully
- Major roads

Scale 1:30,000 at A4 (0 250 500 metres)		**Forrestfield North TOD Precinct Perth, WA**
Coord. Sys. GDA 1994 MGA Zone 50		**SITE OVERVIEW**
Job No: 57806		
Client: Element		**FIGURE 2**
Version: A	Date: 04-May-2021	strategen JBS&G
Drawn By: jcrute	Checked By: CT	

File Name: W:\Projects\1)Open\Element (TPG)\57806 - FF North TOD & Res LWMS\GIS\Maps\R06_Rev_A\57806_02_SiteOverview.mxd
Image Reference: www.nearmap.com© - Imagery Date: 27 February 2021

2. Environmental considerations

2.1 Native vegetation - modification and clearing

The project area is predominantly cleared and developed for commercial and industrial land uses. Remnant vegetation exists within the proposed EC Reserve in the central-north portion of the project area; within the Bush Forever site associated with Poison Gully Creek, along the northern site boundary and within rural-residential properties also at this interface. As vegetation within the EC Reserve and Bush Forever sites is proposed to be retained as part of development design, clearing of remnant native vegetation will be limited to within the rural-residential lots at the corner of Raven Street and Milner Road, although there is potential for some treed vegetation to be retained within the LOS proposed at this location.

A search of publicly available environmental data has been undertaken, along with a review of recent environmental assessments relating to the proposed EC Reserve. This information is summarised in Table 1.

Strategen-JBS&G understands that environmental impacts resulting from implementation of the proposal will be/have been addressed under standard State and Federal environmental assessment and referral requirements under the *Environmental Protection Act 1986* and *Environment Protection and Biodiversity Conservation Act 1999*.

Table 1: Summary of environmental values

Environmental value	Mapped as occurring within or adjacent to the project area		Description
	Within	Adjacent	
Environmentally Sensitive Area	✓	✓	A large proportion of the project area is mapped as being an Environmentally Sensitive Area.
Swan Bioplan Regionally Significant Natural Area	✗	✗	N/A.
Ecological linkages	✓	✓	Vegetation along Poison Gully Creek and land buffered to the south (within the project area) is mapped as forming part of a Perth Regional Ecological Linkage.
Wetlands	✓	✓	Resource Enhancement Wetlands (REW) occur within the Bush Forever land adjoining Poison Gully Creek and within the proposed EC Reserve. A Multiple Use Wetland is also present within the proposed EC Reserve. Vegetation will be retained within these wetland areas as part of the development. The Banksia Woodlands TEC was identified within the proposed EC Reserve and it has been recommended that the REW and MUW be upgraded to Conservation Category Wetlands (CCW) (PGV 2021). REWs also occur directly to the west (within the existing rail corridor) and to the southeast (adjacent to the Residential Precinct area). CCWs and further REWs also occur within the wider landscape, including within the Perth Airport landholdings to the west.
Waterways	✓	✓	Poison Gully Creek extends within and alongside the northern portion of the project area.
Threatened Ecological Communities listed under the EPBC Act	✓		Vegetation within the proposed EC Reserve is representative of the Banksia Woodlands of the Swan Coastal Plan, Threatened Ecological Community (TEC; PGC 2021).

Environmental value	Mapped as occurring within or adjacent to the project area		Description
	Within	Adjacent	
Threatened and priority flora	✓		No threatened flora species were recorded within the proposed EC Reserve during a site survey; however, a small population of a Priority 3 species was recorded on Lot 12 Ibis Place (PGV 2021).
Fauna habitat listed under the EPBC Act	✓		The proposed EC Reserve contains habitat that has been identified as being suitable for three species of Black Cockatoo and Quenda. Evidence of foraging on Marri seeds by Forest Red-Tailed Black Cockatoo was observed during a site investigation. Six marri trees are also considered as being potential Black Cockatoo breeding trees, however, no hollows were observed (PGV 2021).
Threatened and priority fauna	✓		As per above comments.
Bush Forever Site	✓	✓	Bush Forever site 196 is located within the northern portion of the project area and along the northern boundary, in association with Poison Gully Creek vegetation. There are multiple other Bush Forever sites within the surrounding area.
DBCA managed lands and waters (includes legislated lands and waters and lands of interest)	✗	✗	There are no DBCA managed lands and waters within the project area or immediately surrounding land.
Conservation covenants	✗	✗	There are no known conservation covenants over the project area or immediately surrounding land.
Heritage	✗	✗	There are no Aboriginal or other heritage sites mapped as occurring within the project area or immediately surrounding land.

2.2 Revegetation / Landscape Plans

The full extent of native vegetation retention and revegetation/rehabilitation within the project area will be determined at future planning stages through the development of a Landscape Plan.

The ACP provides the location of proposed EC Reserves and LOS but does not provide any detail regarding the likely future vegetation within these areas. For the purposes of this BMP, it has been assumed that the entire extent of the EC Reserves and LOS will constitute Class A Forest vegetation. This provides for a worst-case measure of future bushfire risk within the project area; however, it is likely that LOS will also comprise areas of non-vegetated or low threat vegetation (excluded under Clauses 2.2.3.2 [e] & [f] of AS 3959). Future vegetation within the EC Reserve and LOS may also constitute other classifications of vegetation at maturity, such as Class B Woodland, or Class D Scrub, to be determined at future planning stages in accordance with a landscape plan.

Other areas of future landscaping within the proposed development cells is expected to consist of low threat and managed gardens and street scaping in accordance with AS 3959 Clause 2.2.3.2 (f) and Schedule 1 of the Guidelines (refer to Appendix B).

As mentioned above, the extent of classified vegetation and low threat/non-vegetated areas will be demonstrated within a Landscape Plan prepared at future planning stages.

3. Bushfire assessment results

3.1 Assessment inputs

3.1.1 Vegetation classification

Strategen-JBS&G assessed classified vegetation and exclusions within 150 m of the project area through on-ground verification on 12 September 2017.

Vegetation classifications have been assigned in accordance with *AS 3959—2018 Construction of Buildings in Bushfire-Prone Areas* (AS 3959; SA 2018) and the *Visual Guide for Bushfire Risk Assessment in Western Australia* (DoP 2016). As the site photos were taken in 2017, the current extent of vegetation has been determined in conjunction with a review of Nearmap satellite imagery dated 20 April 2021 given on-ground conditions are not expected to have changed since the original assessment date.

Georeferenced site photos and a description of the vegetation classifications and exclusions are contained in Appendix B and outlined in Table 2/Figure 3 (for pre-development conditions) and Table 3/Figure 4 (for anticipated post-development conditions).

3.1.2 Effective slope

Strategen-JBS&G assessed effective slope under classified vegetation through on-ground verification on 12 September 2017 in accordance with AS 3959. Results were cross-referenced with DPIRD 2m contour data and are outlined in Table 2/Figure 3 (for pre-development conditions) and Table 3/Figure 4 (for anticipated post-development conditions).

Site observations indicate that the project area is relatively flat, varying in elevation from 30 m AHD in the northwest and south to 36 m AHD in the northeast over approximately 1 km.

Class A Forest vegetation within the Poison Gully Creek foreshore that transects the northern cadastral boundary has an effective downslope ranging from >0 to 10°. The remainder of the external classified vegetation is located on land that is predominantly flat in relation to the project area.

On completion of the development, the retained areas of classified vegetation within the project area would be predominantly flat in relation to areas supporting habitable development.

3.1.3 Pre-development inputs

A summary of the assessed pre-development classified vegetation, exclusions and effective slope within the assessment area and adjacent 150 m are listed in Table 2 and illustrated in Figure 3.

Table 2: Pre-development vegetation classifications/exclusions and effective slope

Vegetation plot	Vegetation classification	Effective slope	Comments
1	Class A Forest	Flat/upslope (0°)	Areas of remnant forest vegetation within rural-residential properties and within the Poison Gully Creek foreshore area and rail reserve – flat or upslope of future development areas.
2	Class A Forest	Downslope >0–5°	Areas of forest vegetation within the Poison Gully Creek foreshore area – downslope of future development areas.
3	Class A Forest	Downslope >5–10°	Areas of forest vegetation within the Poison Gully Creek foreshore area – with a steeper downslope in relation to future development areas.
4	Class B Woodland	Flat/upslope (0°)	Land in a state of revegetation within the REW. Treed vegetation with a grassy understorey within rural-residential property on Milner Road.
5	Class D Scrub	Flat/upslope (0°)	Scrub vegetation within the rail reserve, and vacant land within the south of the project area, and vacant land on Berkshire Road (external to project area).
6	Class G Grassland	Flat/upslope (0°)	Area of unmanaged grassland (>100 mm height within and external to the project area.

Vegetation plot	Vegetation classification	Effective slope	Comments
7	Excluded – Non-vegetated and Low threat (Clause 2.2.3.2 [e] and [f])	N/A	All currently developed, cleared and managed land within and surrounding the project area.

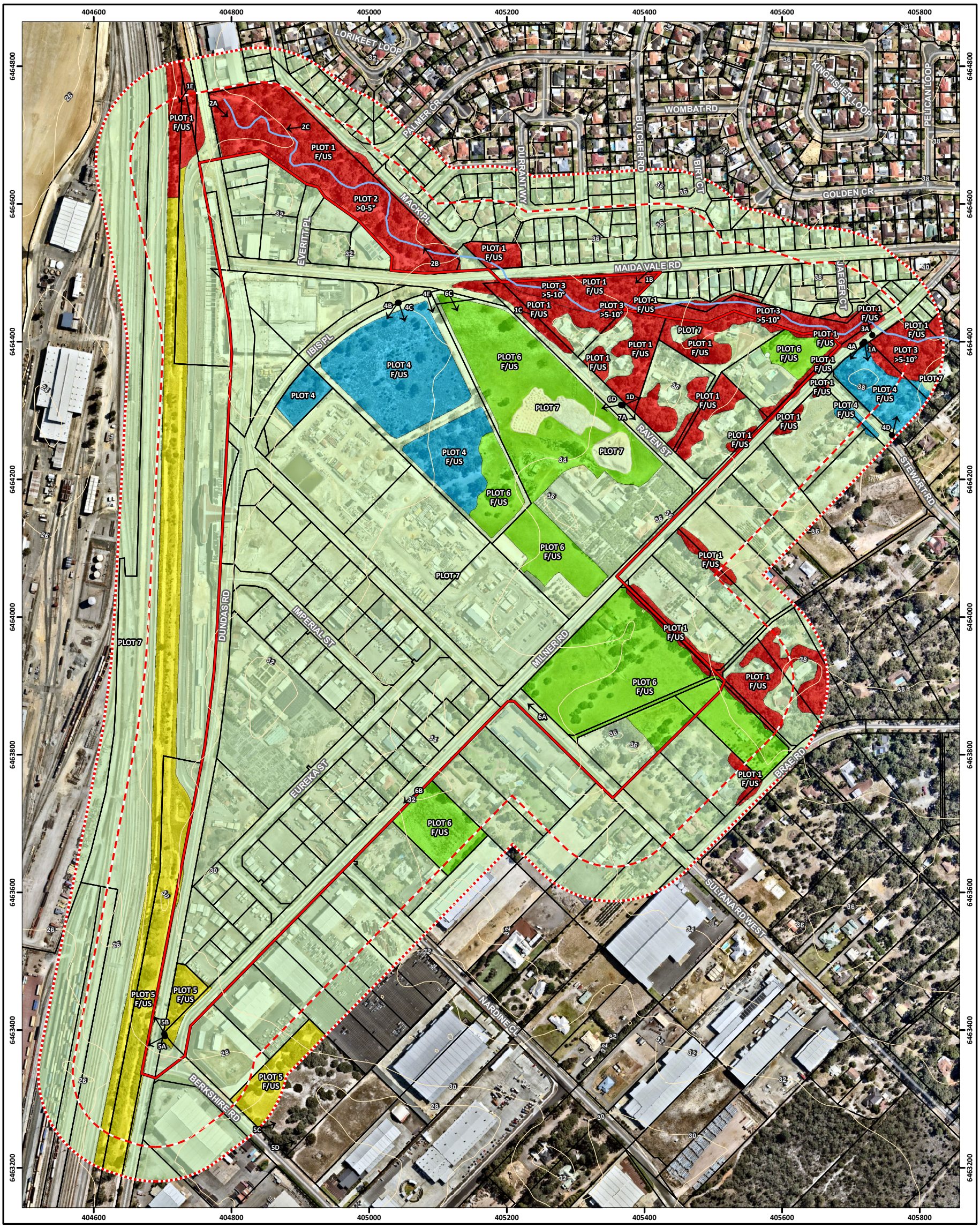
3.1.4 Post-development inputs

A summary of the potential post-development classified vegetation, exclusions and effective slope within the assessment area and adjacent 150 m are listed in Table 3 and illustrated in Figure 4.

The post-development vegetation classifications for all land external to the project area are expected to remain the same as for the pre-development classifications. If external vegetation is altered prior to future planning stages, the change in vegetation condition is to be captured through a future BHL assessment or BAL contour map assessment prepared at that stage.

Table 3: Post-development vegetation classifications/exclusions and effective slope

Vegetation plot	Vegetation classification	Effective slope	Comments
1	Class A Forest	Flat/upslope (0°)	Areas of remnant forest vegetation within rural-residential properties and within the Poison Gully Creek foreshore area and rail reserve – flat or upslope of future development areas. Also includes proposed LOC and EC Reserves, which have been classified as Class A Forest as a precautionary measure in the absence of a Landscape Plan.
2	Class A Forest	Downslope >0–5°	Areas of forest vegetation within the Poison Gully Creek foreshore area – downslope of future development areas.
3	Class A Forest	Downslope >5–10°	Areas of forest vegetation within the Poison Gully Creek foreshore area – with a steeper downslope in relation to future development areas.
4	Class B Woodland	Flat/upslope (0°)	Treed vegetation with a grassy understorey within rural-residential property on Milner Road. The EC reserve is assumed to be Class A Forest post-development.
5	Class D Scrub	Flat/upslope (0°)	Scrub vegetation within the rail reserve and vacant land on Berkshire Road (external to project area).
6	Class G Grassland	Flat/upslope (0°)	Area of unmanaged grassland (>100 mm height external to the project area. It has been assumed that all grassland within the project area will be managed in a low threat state as per future land use requirements.
7	Excluded – Non-vegetated and Low threat (Clause 2.2.3.2 [e] and [f])	N/A	All currently developed, cleared and managed land within and surrounding the project area that is assumed to remain managed for the life of the proposed development.
8	Excluded – Non-vegetated and Low threat (Clause 2.2.3.2 [e] and [f])	N/A	Areas that will be modified to and managed in a low threat state as part of the proposed development.



Legend

- Project area
- 100m assessment area
- 150m assessment area
- Cadastral boundary
- Poison Creek Gully
- Topographic contours (mAHD)
- Photo point directions

Vegetation classification

- Class A Forest
- Class B Woodland
- Class D Scrub
- Class G Grassland
- Clause 2.2.3.2 (e) & (f)

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0 150 metres

Job No: 57806		Scale 1:5,000 at A3	
Client: Element		Coord. Sys. GDA 1994 MGA Zone 50	
Drawn By: jcrute	Checked By: CT	Version: A	Date: 07-May-2021

Forrestfield North TOD Precinct
Perth, WA

PRE-DEVELOPMENT VEGETATION CLASSIFICATION AND EFFECTIVE SLOPE

FIGURE: 3

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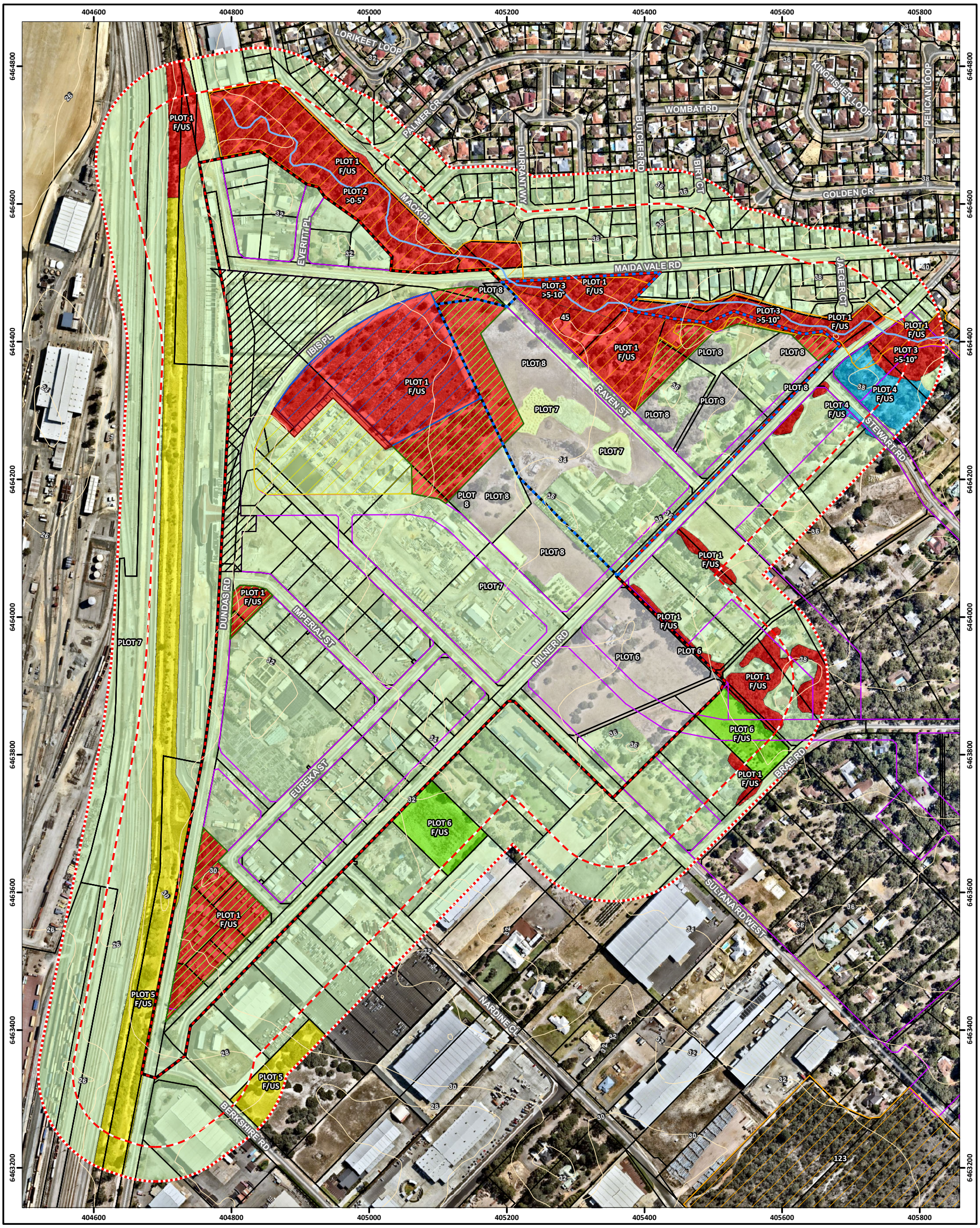
Image Reference: www.nearmap.com© - Imagery Date: 27. February 2021.



Legend

- Project area
- 100m assessment area
- 150m assessment area
- Cadastral boundary
- Poison Creek Gully
- Proposed lot layout and roads
- Topographic contours (mAHD)

Vegetation classification

- Class A Forest
- Class B Woodland
- Class D Scrub
- Class G Grassland
- Clause 2.2.3.2 (e) & (f)
- Area to be modified to non-vegetated and low threat state

- Bush Forever sites
- ROS
- Public Purposes: Community Purpose
- Environmental Conservation Reserve
- Local Open Space
- Railway
- Forrestfield Station Precinct
- Residential Precinct

strategen JBS&G

0 150 metres

Job No: 57806		Scale 1:5,000 at A3	
Client: Element		Coord. Sys. GDA 1994 MGA Zone 50	
Drawn By: jcrute	Checked By: CT	Version: A	Date: 11-May-2021

Forrestfield North TOD Precinct
Perth, WA

POST-DEVELOPMENT VEGETATION CLASSIFICATION AND EFFECTIVE SLOPE

FIGURE: 4

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Image Reference: www.nearmap.com© - Imagery Date: 27. February 2021.

3.2 Assessment outputs

3.2.1 Bushfire Hazard Level (BHL) assessment

Pre and post-development vegetation extents have been assigned a bushfire hazard level in accordance with the methodology detailed in Appendix Two of the Guidelines, as outlined in Table 4.

Table 4: Bushfire hazard levels and characteristics

Bushfire hazard level	Characteristics*
Extreme	• Class A Forest • Class B Woodland (05) • Class D Scrub • Any classified vegetation with a greater than 10° slope.
Moderate	• Class B Low Woodland (07) • Class C Shrubland • Class E Mallee/Mulga • Class G Grassland, including sown pasture and crops • Class G Grassland: Open woodland (06), Low open woodland (08), Open shrubland (09) • Vegetation that has a low hazard level but is within 100 metres of vegetation classified as a moderate or extreme hazard, is to adopt a moderate hazard level.
Low	• Low threat vegetation may include areas of maintained lawns, golf courses, public recreation reserves and parklands, vineyards, orchards, cultivated gardens, commercial nurseries, nature strips and windbreaks • Managed grassland in a minimal fuel condition (insufficient fuel is available to significantly increase the severity of the bushfire attack). For example, short-cropped grass to a nominal height of 100 millimetre • Non-vegetated areas including waterways, roads, footpaths, buildings and rock outcrops.
*Vegetation classifications from AS 3959-2018 Table 2.3.	

3.2.1.1 Pre-development

Strategen-JBS&G has mapped the pre-development bushfire hazard levels within the project area and adjacent 150 m wide assessment area. The bushfire hazard levels have been assessed on the basis of the vegetation discussed in Section 3.1.3 (i.e. the current pre-development extent of vegetation within and surrounding the project area).

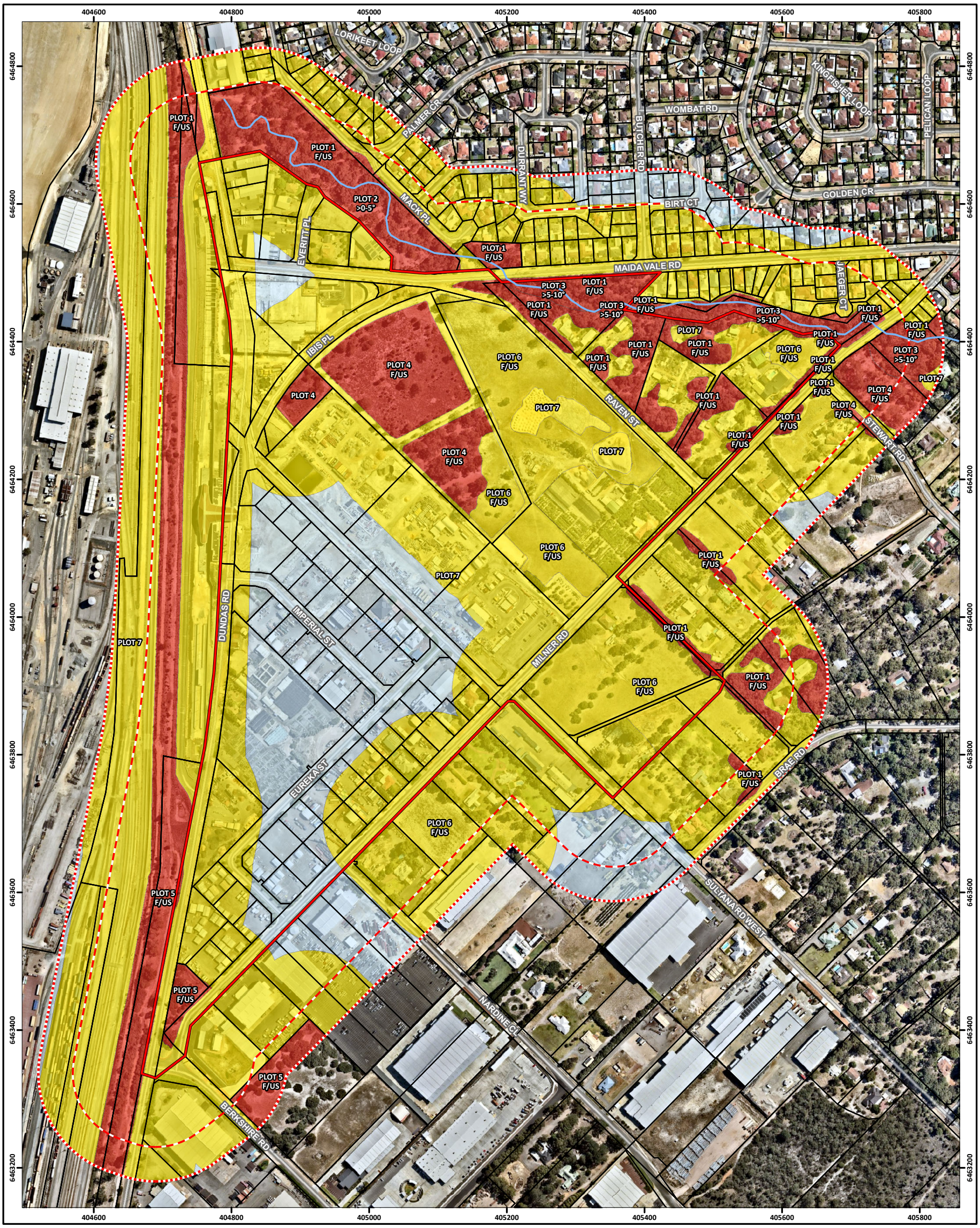
The pre-development BHL assessment (refer to Figure 5) shows that based on the existing vegetation, the project area contains land with Low, Moderate and Extreme bushfire hazard levels.

3.2.1.2 Post-development

Strategen-JBS&G has mapped the potential post-development bushfire hazard levels to demonstrate that the future bushfire hazard levels will be acceptable for future development to occur within the project area. The bushfire hazard levels have been assigned on the basis of the vegetation discussed in Section 3.1.4 and the future expected vegetation extent within and surrounding the project area.

The post-development BHL assessment (refer to Figure 6) demonstrates that all future habitable development will be located on land with either a Low or Moderate bushfire hazard level.

Land within the project area having an Extreme BHL is limited to the Bush Forever site, proposed EC Reserves and proposed LOS, where habitable development is not proposed to occur.



Legend

- Project area
- 100m assessment area
- 150m assessment area
- Cadastral boundary
- Poison Creek Gully

Hazard level
- Extreme
- Moderate
- Low

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0 150 metres

Job No: 57806		Scale 1:5,000 at A3	
Client: Element		Coord. Sys. GDA 1994 MGA Zone 50	
Drawn By: jcrute	Checked By: CT	Version: A	Date: 04-May-2021

Forrestfield North TOD Precinct
Perth, WA

PRE-DEVELOPMENT
BHL ASSESSMENT

FIGURE: 5

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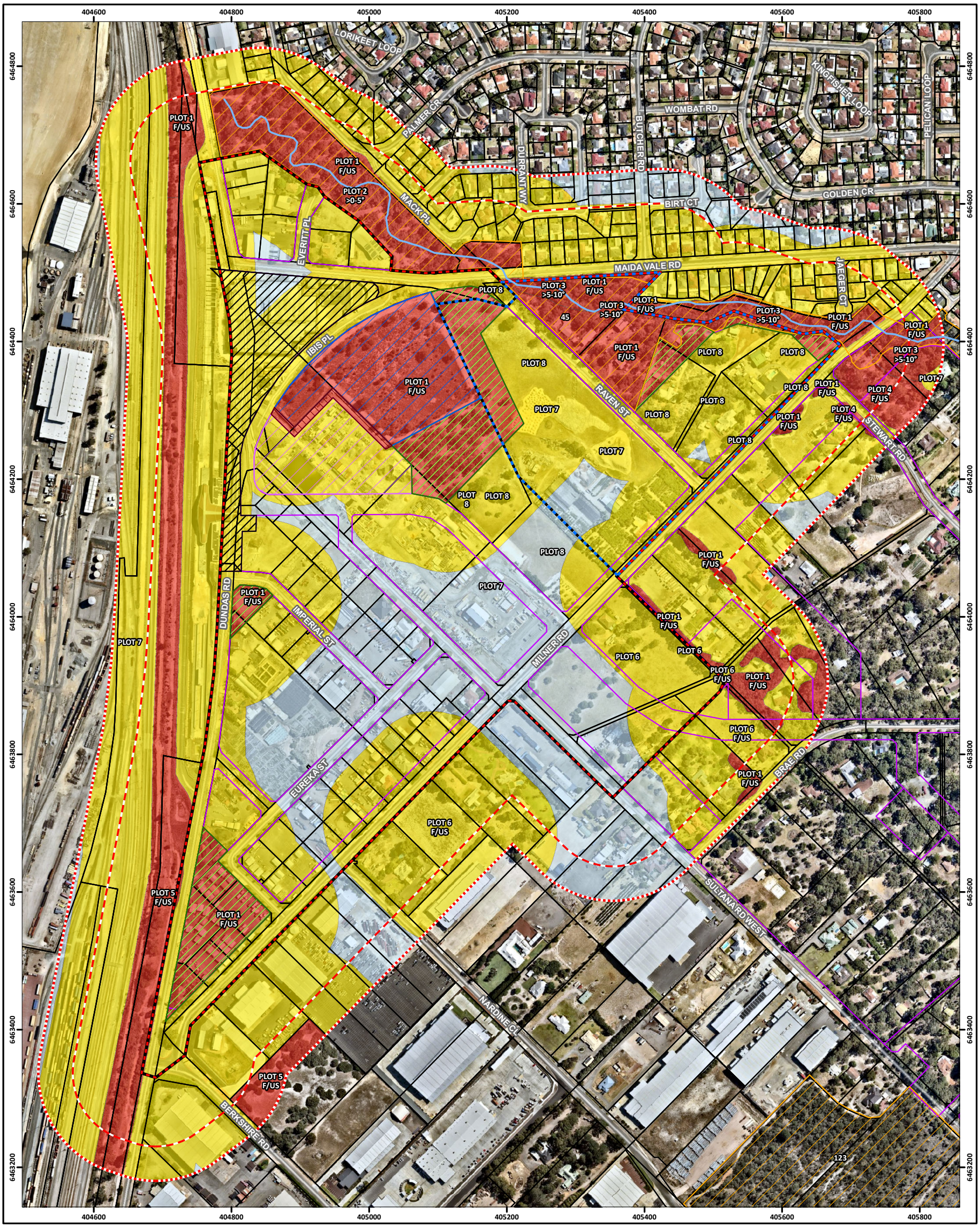
Image Reference: www.nearmap.com© - Imagery Date: 27. February 2021.



Legend

- Project area
- 100m assessment area
- 150m assessment area
- Cadastral boundary
- Poison Creek Gully
- Proposed lot layout and roads

Hazard level
- Extreme
- Moderate
- Low

- Bush Forever sites
- ROS
- Public Purposes: Community Purpose
- Environmental Conservation Reserve
- Local Open Space
- Railway
- Forrestfield Station Precinct
- Residential Precinct

strategen JBS&G

0 150 metres

Job No: 57806		Scale 1:5,000 at A3	
Client: Element		Coord. Sys. GDA 1994 MGA Zone 50	
Drawn By: jcrute	Checked By: CT	Version: A	Date: 11-May-2021

Forrestfield North TOD Precinct
Perth, WA

POST-DEVELOPMENT BHL ASSESSMENT

FIGURE: 6

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Image Reference: www.nearmap.com© - Imagery Date: 27. February 2021.

4. Identification of bushfire hazard issues

4.1 Bushfire context

Strategen-JBS&G considers that the TOD Precinct is unlikely to be impacted by a landscape scale bushfire as the surrounding vegetation is largely fragmented and not of a sufficient size/fire run to support landscape scale bushfire behaviour.

When considering the clearing that will occur within the adjoining Forrestfield North Residential Precinct to the southeast, intact vegetation adjacent to the project area will be limited to the narrow band of forest vegetation associated with Poison Gully Creek in the north.

Under predominant afternoon summer weather conditions for the location, the likely prevailing winds are from the east in the morning and southwest in the afternoon. Under these conditions, the project area may be affected by ember attack from a bushfire occurring within land to the southeast of Perth Airport.

There is potential for bushfire spread to occur from within the Forrestfield North Residential Precinct, to the east, through vegetation retained within the EC Reserves and POS, which serve the purpose of linking remnant native vegetation with the adjacent Bush Forever sites. However, this is expected to be limited as the narrow width and likely fragmented nature of the vegetation corridor would not support extended bushfire behaviour.

Notwithstanding, due to the presence of on-site bushfire prone vegetation, sufficient building separation and bushfire construction measures will need to be considered for all applicable future development to reduce the potential bushfire impacts on buildings and their occupants.

4.2 Bushfire hazard issues

Examination of strategic development design as per the ACP and existing and post-development bushfire hazard levels has identified the following bushfire hazard issues to be considered at future planning stages:

1. The project area contains Extreme bushfire hazard level areas not suitable for habitable development. However, the post-development BHL assessment demonstrates that on completion of development, all habitable development areas will be located on land with a Low or Moderate bushfire hazard level. A BAL contour assessment will be commissioned at future planning stages to demonstrate at a more detailed level that all development will be located within areas of BAL-29 or below. Therefore, the development can comply with Acceptable Solution A1.1 of the Guidelines.
2. Development design will include retention of native vegetation within the Bush Forever site, EC reserves and potentially LOS, which have been classified as Extreme hazard level vegetation. Where habitable development is proposed within proximity to the retained vegetation, setbacks and/or APZs sufficient to achieve BAL-29 are to be employed as part of a compliant subdivision design. Lot size and orientation will need to be considered at future stages to ensure appropriate setbacks can be achieved.
3. The applicable habitable building setbacks from internal and external bushfire prone vegetation required to achieve BAL-29 are to be determined through site-specific BAL Contour map analysis at future planning stages, but are likely to be:

Vegetation classification	Effective slope	Separation to achieve BAL-29
Class A Forest	Flat/ Upslope	21 m
	Downslope >0 to 5 degrees	27 m
	Downslope >5 to 10 degrees	33 m
Class B Woodland	Flat/ Upslope	14 m
Class D Scrub	Flat/ Upslope	13 m

Vegetation classification	Effective slope	Separation to achieve BAL-29
Class G Grassland	Downslope >0 to 5 degrees	9 m

4. The ACP identifies a location adjacent to the vegetated portion of the central LOS where hazard separation is required to avoid BAL-40/FZ impacts on the Community Purposes Lot. While this is indeed the case, a direct vegetation interface also has potential to exist around the north/northwest boundary of the Community Purposes lot. The width and location of the hazard separation required to be established in the form of an APZ will be determined through preparation of a BAL contour map at future planning stages.
5. The BMP has classified the proposed Local Open Space (LOS) cells as Class A Forest (Extreme bushfire hazard level) in the absence of a Landscape Plan, as a precautionary measure. It is acknowledged that LOS is also likely to include areas of open space and managed vegetation which may be excluded as non-vegetated and low threat areas in accordance with Clauses 2.2.3.2 [e] & [f] of AS 3959. It is recommended that LOS is strategically designed to incorporate these areas with low bushfire risk at the interface with future lots.
6. The post-development BHL assessment has classified vegetation to the southeast, which is located within the adjacent Forrestfield North Residential Precinct. On completion of development within this adjoining precinct, a portion of this classified land will be low threat/non-vegetated, which will reduce the bushfire risks at this interface.

   In addition to the above, there is an existing lot to the southwest of the project area which has been classified as Class G Grassland but is not located within the Forrestfield North Residential Precinct. It is acknowledged, however, that the Firebreak Notice provides for the management of grassland below 100 mm, in which case, there is grounds for this land to be excluded within the BAL contour assessment prepared at future planning stages (providing the grassland is actively being managed). The effect of excluding this grassland vegetation will be that there will be no setback requirements from this land.
7. The ACP layout includes perimeter roads at several vegetation interfaces, which will provide separation between bushfire hazards and future habitable development. However, there may be instances where the roads are not of a sufficient width for the lots to achieve BAL-29 or lower and habitable building setbacks may be required within lots.

   No perimeter road has been provided to the portion of Bush Forever north of Maida Vale Road. Provision of an FSAR should be considered at this interface to provide additional separation to developable land, as well as direct firefighter access around the perimeter to the bushfire prone vegetation.
8. The ACP includes a Community Purpose lots that, depending on the future land use, may be a Vulnerable Land Use under Policy Measure 6.6 of SPP 3.7. If considered to be a vulnerable land use, the Development Application (DA) will need to be accompanied by an updated BMP that includes a Bushfire Attack Level (BAL) assessment and a Bushfire Emergency Evacuation Plan (BEEP). Any other Vulnerable Land Uses in an area greater than BAL-LOW will need to address Policy Measure 6.6 at the DA stage of planning.
9. Development of the ACP is expected to occur in stages with each cell being constructed independently. Staging of development is to ensure that at least two access routes are always provided in accordance with acceptable solution A3.1 of the Guidelines. This may require provision of temporary staging measures such as temporary compliant cul-de-sacs and/or Emergency Access Ways (EAWs) to deliver compliant access outcomes for individual stages.

10. As the project area contains some areas of bushfire prone vegetation, staged construction at the subdivision stage of planning is to consider the BAL impacts from adjacent future stages that have not yet been developed. Low threat staging buffers may need to be implemented around the current stage of development to ensure there is no residual impact from vegetation that has not yet been cleared or landscaped to achieve a low threat state.
11. Fire service access routes (FSARs) may be required to provide access around the perimeter of the retained/rehabilitated vegetation where public roads are not proposed. Temporary FSARs may also be required during staged development to ensure firefighters are able to gain access to the site perimeter prior to future subdivision roads being constructed.

Strategen-JBS&G considers the bushfire hazards within and adjacent to the TOD Precinct and the associated bushfire risks are readily manageable through standard management responses outlined in the Guidelines and AS 3959 and the strategic level bushfire management strategies outlined in this BMP. These responses will be factored in to proposed development early in the planning process to ensure a suitable, compliant and effective bushfire management outcome is achieved for protection of future life and property assets.

5. Assessment against the bushfire protection criteria

5.1 Compliance table

An acceptable solutions assessment against the bushfire protection criteria is provided in Table 5.

Table 5: Compliance with the bushfire protection criteria of the Guidelines

Bushfire protection criteria	Method of compliance Acceptable solutions	Proposed bushfire management strategies	Compliance achievable at future planning stages
Element 1: Location	A1.1 Development location	The post-development BHL assessment (Figure 6) identifies that on completion of development, all developable land will comprise either a Low or Moderate bushfire hazard level. All habitable development is to be avoided on land with an extreme bushfire hazard level.	✓
Element 2: Siting and design	A2.1 Asset Protection Zone	APZs required for future habitable development to achieve BAL—29 are to be implemented for all relevant lots. The required APZs will be identified at future planning stages based on future subdivision/development design and following a BAL contour assessment. APZs are to be implemented and maintained in accordance with Schedule 1 of the Guidelines (Appendix A) and the City's Firebreak Notice (see Appendix C).	✓
Element 3: Vehicular access	A3.1 Two access routes	On completion of development, the TOD Precinct will be serviced by multiple access route leading to destinations in all directions. The proposed TOD Connector Boulevard will provide connectivity to the adjacent Forrestfield North Residential Precinct and will form an arterial linkage to the existing and future surrounding public road network to the east. Milner Road will provide a linkage to the suburb of High Wycombe to the north. In this regard, the proposed development is provided with more than two access routes which meets and exceeds the requirements of Acceptable Solution A3.1. During staged development, each stage is to be provided with two access routes.	✓
	A3.2 Public road	All public roads are to be constructed to the relevant technical requirements of the Guidelines (see Appendix D).	✓
	A3.3 Cul-de-sac (including a dead-end-road)	The ACP includes a single compliant cul-de-sac, Everitt Place, which is existing and a legacy of previous development. Everitt Place is compliant in nature being approximately 100 m in length with a compliant turning circle, having a radius of at least 17.5 m. Everitt Place is located in the north-western corner of the project area, and due to proximity to the Poison Gully	✓

Bushfire protection criteria	Method of compliance Acceptable solutions	Proposed bushfire management strategies	Compliance achievable at future planning stages
		Creek Bush Forever site and existing development within the adjoining lots to the east and west, there is limited potential for this road to be extended through to the public road network as part of this development. All other existing and proposed roads within the ACP are provided with through access to a public road. Any temporary cul-de-sacs constructed during stages development will be required to comply with Guideline requirements (see Appendix D), including a maximum length of 200 m and minimum 17.5 m diameter turn-around head.	
	A3.4 Battle-axe	The ACP includes an approximately 125 m long existing battle-axe leg serving existing Lot 104 Raven Street, which is located within the Residential Precinct of the TOD Precinct. This battle-axe will be extinguished through future residential lot allocation and the provision of a road at the northern boundary of the existing lots. Future battle-axe configuration should be avoided for lots adjoining or in proximity to land with an extreme BHL. Any battle-axes incorporated into future development design will need to be less than 600 m in length and constructed to the relevant technical requirements of the Guidelines (see Appendix D), including passing bays at 200 m intervals and turn-around areas for fire appliances where battle-axes are longer than 500 m.	✓
	A3.5 Private driveway longer than 50 m	Any future private driveways longer than 50 m are to be constructed to the relevant technical requirements of the Guidelines (see Appendix D), including turn-around areas within 50 m of each building, passing bays if driveways are longer than 200 m and additional turn-around areas for fire appliances every 500 m.	✓
	A3.6 Emergency access way	Based on the conceptual development design, the proposed development would not require any permanent Emergency Access Ways (EAWs) to provide through access to a public road. All roads (except for Everitt Place, which, as previously discussed, is a legacy compliant cul-de-sac) will have connections to the surrounding public road network. Any temporary EAWs required as a result of staged development are to be constructed to the relevant technical requirements of the Guidelines (see Appendix D).	✓
	A3.7 Fire service access routes (perimeter roads)	Fire service access routes (FSAR) should be considered to provide direct firefighter access to the perimeter of areas where vegetation is proposed to be retained, including adjacent to the EC Reserve, Bush Forever site/Poison Gully Creek foreshore and LOS (where vegetated). The FSARs may be located within LOS, within the identified hazard separation areas adjoining the EC Reserve/LOS and any other interfaces whether a public road is not proposed. FSARs are to be constructed to the relevant technical requirements of the Guidelines (see Appendix D).	✓
	A3.8 Firebreak width	Each stage of development is required to comply with the requirement of the City's Firebreak and Fuel Load Notice as amended (refer to Appendix C).	✓

Bushfire protection criteria	Method of compliance Acceptable solutions	Proposed bushfire management strategies	Compliance achievable at future planning stages
		On completion of the development, it is expected that the majority of lots will be fully developed and would not require individual lot firebreaks. However, any lots >4000 m^2 containing remnant or grassland vegetation will require 3 m wide x 4 m high mineral earth firebreaks inside the internal lot perimeter. Consultation with the City is required to determine whether perimeter firebreaks would be required around the proposed the EC reserve (noting some firebreaks are already in place) and Bush Forever, or whether perimeter access will be provided by way of FSARs, or a combination of both.	
Element 4: Water	A4.1 Reticulated areas	The project area is serviced by an existing reticulated hydrant network. Future development will be provided with a reticulated water supply compliant with Water Corporation DS-63 requirements (refer to Appendix E).	✓
	A4.2 Non-reticulated areas	N/A – the proposed subdivision is located within an existing reticulated area.	**N/A**
	A4.3 Individual lots within non-reticulated areas (Only for use if creating 1 additional lot and cannot be applied cumulatively)	N/A – the proposed subdivision is located within an existing reticulated area.	**N/A**

6. Responsibilities for implementation and management of the bushfire measures

This BMP has been prepared as a strategic guide to demonstrate how development compliance will be delivered at future planning stages in accordance with the Guidelines. Aside from the preparation of future BMPs to accompany future subdivision and development applications where appropriate, there are no further items to implement, enforce or review at this strategic stage of the planning process.

Future BMPs prepared for subsequent subdivision and development applications are to meet the relevant commitments outlined in this strategic level BMP, address the relevant requirements of SPP 3.7 (i.e. Policy Measures 6.4 and 6.5 respectively) and demonstrate in detail how the proposed development will incorporate the relevant acceptable solutions or meet the performance requirements of the Guidelines. Future BMPs are to include the following detailed information:

- proposed lot layout and detailed POS/LOS, reserve and drainage basin design
- post-development classified vegetation extent, effective slope and separation distances
- post-development BAL application requirements
- BAL contour map demonstrating that proposed development areas will achieve a rating of BAL–29 or lower
- width and alignment of compliant APZs, including any APZ setback requirements into lots
- confirmation of how bushfire management will be addressed during development staging
- confirmation of how bushfire management will be addressed regarding temporary bushfire hazards on adjacent future development stages, including low threat staging buffers or temporary quarantining of lots where required
- proposed approach to fuel management or AS 3959 application in response to on-site POS or easements (if and where required)
- vehicular access provisions, including demonstration that a minimum of two access routes will be achieved for each stage of development in accordance with acceptable solution A3.1 (may require consideration of temporary compliant access provisions such as cul-de-sacs and EAWs)
- provision of FSARs around the perimeter of vegetation hazards
- water supply provisions regarding reticulated water
- future requirements for any identified vulnerable land uses, such as provision of a Bushfire Emergency Evacuation Plan at the DA or building permit stage for the proposed school site
- future requirements for any future high-risk land uses, such as provision of a Bushfire Risk Management Plan
- provisions for notification on Title for any future lots with a rating of BAL-12.5 or greater as a condition of subdivision
- compliance requirements with the annual City firebreak notice
- assessment against the bushfire protection criteria
- proposed audit and compliance program outlining all measures requiring implementation and the appropriate timing and responsibilities for implementation.

Based on the information contained in this BMP, Strategen-JBS&G considers the bushfire hazards within and adjacent to the project area and the associated bushfire risks are readily manageable through standard management responses outlined in the Guidelines and AS 3959. Strategen-JBS&G considers that on implementation of the proposed management measures, the project area will be able to be developed with a manageable level of bushfire risk whilst maintaining full compliance with the Guidelines and AS 3959.

7. References

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Strategen-JBS&G 2021a, *Environmental Assessment and Management Strategy, Forrestfield North TOD Precinct*, Strategen-JBS&G, Perth/Bunbury.

Strategen-JBS&G 2021b, *Local Water Management Strategy, Forrestfield North TOD Precinct*, Strategen-JBS&G, Perth/Bunbury.

Urbaqua 2020, *Local Water Management Strategy, Forrestfield North Residential Precinct, Urbaqua, Perth.*

Western Australian Planning Commission (WAPC) 2015, *State Planning Policy 3.7 Planning in Bushfire Prone Areas*, Western Australian Planning Commission, Perth.

Western Australian Planning Commission (WAPC) 2017, *Guidelines for Planning in Bushfire Prone Areas*, Version 1.3 August 2017, Western Australian Planning Commission, Perth.

8. Limitations

Scope of services

This report ("the report") has been prepared by Strategen-JBS&G in accordance with the scope of services set out in the contract, or as otherwise agreed, between the Client and Strategen-JBS&G. In some circumstances, a range of factors such as time, budget, access and/or site disturbance constraints may have limited the scope of services. This report is strictly limited to the matters stated in it and is not to be read as extending, by implication, to any other matter in connection with the matters addressed in it.

Reliance on data

In preparing the report, Strategen-JBS&G has relied upon data and other information provided by the Client and other individuals and organisations, most of which are referred to in the report ("the data"). Except as otherwise expressly stated in the report, Strategen-JBS&G has not verified the accuracy or completeness of the data. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations in the report ("conclusions") are based in whole or part on the data, those conclusions are contingent upon the accuracy and completeness of the data. Strategen-JBS&G has also not attempted to determine whether any material matter has been omitted from the data. Strategen-JBS&G will not be liable in relation to incorrect conclusions should any data, information or condition be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to Strategen-JBS&G. The making of any assumption does not imply that Strategen-JBS&G has made any enquiry to verify the correctness of that assumption.

The report is based on conditions encountered and information received at the time of preparation of this report or the time that site investigations were carried out. Strategen-JBS&G disclaims responsibility for any changes that may have occurred after this time. This report and any legal issues arising from it are governed by and construed in accordance with the law of Western Australia as at the date of this report.

Environmental conclusions

Within the limitations imposed by the scope of services, the preparation of this report has been undertaken and performed in a professional manner, in accordance with generally accepted environmental consulting practices. No other warranty, whether express or implied, is made.

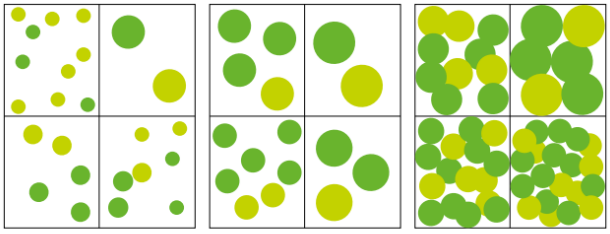
The advice herein relates only to this project and all results conclusions and recommendations made should be reviewed by a competent person with experience in environmental investigations, before being used for any other purpose.

Strategen-JBS&G accepts no liability for use or interpretation by any person or body other than the client who commissioned the works. This report should not be reproduced without prior approval by the client, or amended in any way without prior approval by Strategen-JBS&G, and should not be relied upon by other parties, who should make their own enquiries.

Appendix A APZ standards (Schedule 1 of the Guidelines)

Schedule 1: Standards for Asset Protection Zones

- **Fences:** within the APZ are constructed from non-combustible materials (e.g. iron, brick, limestone, metal post and wire). It is recommended that solid or slatted non-combustible perimeter fences are used.
- **Objects:** within 10 metres of a building, combustible objects must not be located close to the vulnerable parts of the building i.e. windows and doors.
- **Fine Fuel load:** combustible dead vegetation matter less than 6 millimetres in thickness reduced to and maintained at an average of two tonnes per hectare.
- **Trees (> 5 metres in height):** trunks at maturity should be a minimum distance of 6 metres from all elevations of the building, branches at maturity should not touch or overhang the building, lower branches should be removed to a height of 2 metres above the ground and or surface vegetation, canopy cover should be less than 15% with tree canopies at maturity well spread to at least 5 metres apart as to not form a continuous canopy.



15% 30% 70%

- **Shrubs (0.5 metres to 5 metres in height):** should not be located under trees or within 3 metres of buildings, should not be planted in clumps greater than 5m2 in area, clumps of shrubs should be separated from each other and any exposed window or door by at least 10 metres. Shrubs greater than 5 metres in height are to be treated as trees.
- **Ground covers (<0.5 metres in height):** can be planted under trees but must be properly maintained to remove dead plant material and any parts within 2 metres of a structure, but 3 metres from windows or doors if greater than 100 millimetres in height. Ground covers greater than 0.5 metres in height are to be treated as shrubs.
- **Grass:** should be managed to maintain a height of 100 millimetres or less.

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Appendix B Vegetation plot photos and description

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Photo ID: 1a



Photo ID: 1b



Photo ID: 1c



Photo ID: 1d



Photo ID: 1e

Plot number	Plot 1
Vegetation classification	Class A Forest
Description / justification	**Pre-development:** Class A Forest is located within areas of remnant forest vegetation within rural-residential properties and within the Poison Gully Creek foreshore area and rail reserve – flat or upslope of future development areas. **Post-development:** Class A Forest is located within areas of remnant forest vegetation within rural-residential properties and within the Poison Gully Creek foreshore area and rail reserve – flat or upslope of future development areas. Also includes proposed LOC and EC Reserves, which have been classified as Class A Forest as a precautionary measure in the absence of a Landscape Plan (pre-development classification was Class G Grassland, Class D Scrub, Class B Woodland and excluded). **Vegetation:** Trees 10-30 m high at maturity, dominated by Eucalypts, multi-tiered structure comprising tall canopy layer, shrubby middle layer and grass/herb/sedge understorey.

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Photo ID: 2a



Photo ID: 2b



Photo ID: 2c

Plot number	Plot 2
Vegetation classification	Class A Forest
Description / justification	Areas of forest vegetation within the Poison Gully Creek foreshore area – downslope of future development areas. Trees 10-30 m high at maturity, dominated by Eucalypts, multi-tiered structure comprising tall canopy layer, shrubby middle layer and grass/herb/sedge understorey.

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Photo ID: 3a

Plot number	Plot 3
Vegetation classification	Class A Forest
Description / justification	Areas of forest vegetation within the Poison Gully Creek foreshore area – with a steeper downslope in relation to future development areas. Trees 10-30 m high at maturity, dominated by Eucalypts, multi-tiered structure comprising tall canopy layer, shrubby middle layer and grass/herb/sedge understorey.

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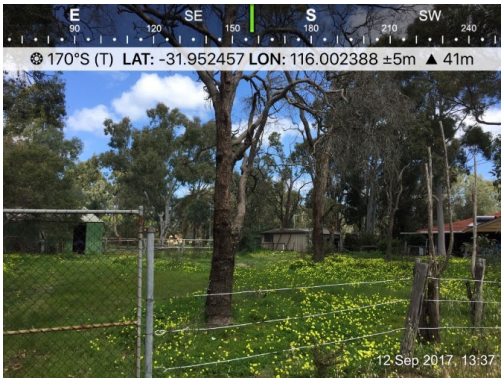

Photo ID: 4a



Photo ID: 4b



Photo ID: 4c



Photo ID: 4d



Photo ID: 4e

Plot number	Plot 4
Vegetation classification	Class B Woodland
Description / justification	**Pre-development:** Class B Woodland is located within land in a state of revegetation within the REW. Treed vegetation with a grassy understorey within rural-residential property on Milner Road. **Post-development:** Class B Woodland is located within treed vegetation with a grassy understorey within rural-residential property on Milner Road. The EC reserve is assumed to be Class A Forest post-development. **Vegetation:** Trees 2-30 m at maturity, dominated by trees with a grassy understorey (lacks shrubby middle layer and deep surface litter).

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Photo ID: 5a



Photo ID: 5b



Photo ID: 5c



Photo ID: 5d

Plot number	Plot 5
Vegetation classification	Class D Scrub
Description / justification	**Pre-development:** Class D Scrub is located the rail reserve, and vacant land within the south of the project area, and vacant land on Berkshire Road (external to project area). **Post-development:** Class D Scrub is located the rail reserve, and vacant land within the south of the project area, and vacant land on Berkshire Road (external to project area). Pre-development scrub within the south of the project area is located within proposed LOS and is assumed to be Class Forest. **Vegetation:** Vegetation with a continuous horizontal and vertical structure, 2 to 6 m high at maturity.

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Photo ID: 6a



Photo ID: 6b



Photo ID: 6c



Photo ID: 6d

Plot number	Plot 6
Vegetation classification	Class G Grassland
Description / justification	**Pre-development:** Area of unmanaged grassland (>100 mm height within and external to the project area. **Post-development:** Area of unmanaged grassland (>100 mm height external to the project area. It has been assumed that all grassland within the project area will be managed in a low threat state.

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Photo ID: 7a

Plot number	Plot 7
Vegetation classification	Excluded – Non-vegetated and Low threat (Clause 2.2.3.2 [e] and [f])
Description / justification	**Pre-development:** All currently developed, cleared and managed land within and surrounding the project area. **Post-development:** All currently developed, cleared and managed land within and surrounding the project area that is assumed to remain managed for the life of the proposed development.

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Appendix C City of Kalamunda Fire Hazard Reduction Notice 2021/2022

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City of Kalamunda

FIRST AND FINAL NOTICE

FIRE HAZARD REDUCTION NOTICE 2021/2022

COMPLIANCE DUE BY: 1 NOV 2021

ASSESSMENTS COMMENCE FROM: 1 NOV 2021

Important: The works outlined below must be completed by 1 November 2021 and maintained in this state up to and including 31 March 2022.

NOTICE TO ALL OWNERS AND OCCUPIERS OF LAND WITHIN THE CITY OF KALAMUNDA

Pursuant to Section 33 of the *Bush Fires Act 1954* (WA), the City of Kalamunda (City) gives written notice to act as specified in this notice to land that you own and/or occupy and with respect to any matter which is upon the land that you own and/or occupy within the City. Failure or neglect to comply with this notice is an offence and can result in a penalty of up to $5000. The City advises that its officers, servants, workmen, contractors, vehicles, machinery, and appliances (as the officers deem fit) may carry out the requisitions of this notice that are not complied with by the date specified above, and any costs and expenses incurred may be recovered from you as the owner and/or occupier of the land.

ALL LAND WITH A TOTAL AREA OF 4000m² OR MORE

❑ FIRE BREAKS

Install or upgrade a **3-metre-wide** by **4-metre-high clearance**, **bare mineral earth**, **continuous** (no dead ends) **trafficable** fire break as close as possible inside the entire perimeter of the land. Maintained, reticulated living lawns that are kept completely green is acceptable in conjunction with, or in lieu of, a mineral earth fire break/s (see definition FIRE BREAK).

❑ DEAD FLAMMABLE MATERIAL (DFM)

Reduce and/or maintain all dead flammable material below 8 tonne per hectare (see definition FUEL LOAD).

❑ SLASHING

Slash, mow, or trim dead grasses, dead shrubs, and dead plants to a height no greater than 50mm and remove cuttings/swath across the entire property.

❑ ASSET PROTECTION ZONE (APZ)

Maintain a fuel reduced zone around all buildings or assets which extends 20 metres from the outermost point of the building or asset.

- Gutters, roofs, and walls of all buildings to be free of flammable matter and maintained.
- Fuel load within the 20-metre zone is reduced and maintained to no more than 2 tonne per hectare.
- Trees over 5 metres in height within the 20-metre zone to be under pruned up to 2 metres.
- Trees or shrubs within 2 metres of the asset shall be pruned to a height no greater than 2 metres and/or pruned away from the asset to a distance no greater than 2 metres.

ALL LAND WITH A TOTAL AREA LESS THAN 4000m²

❑ DEAD FLAMMABLE MATERIAL

Reduce and/or maintain all dead flammable material below 8 tonne per hectare (see definition FUEL LOAD).

❑ SLASHING

Slash, mow, or trim dead grasses, dead shrubs, and dead plants to a height no greater than 50mm and remove cuttings/swath across the entire property.

❑ CLEAN GUTTER DEBRIS

Gutters, roofs, and walls of all buildings to be free of flammable matter and maintained.

VARIATION TO THE FIRE HAZARD REDUCTION NOTICE

If you consider it impractical to meet a requirement/s of this Notice, you may apply to the City for a variation **no later than 1 October 2021.**

Note: A variation is not an exemption but an application to employ other methods of property preparedness to land that you own and/or occupy.

An administration fee of $150 applies for applications received after 1 October 2021.

If your application is not granted you must comply with all requirements outlined in the Fire Hazard Reduction Notice 2021/2022.

Variation request application forms are available before 1 October 2021 on the City's website at:

www.kalamunda.wa.gov.au/fire

ADDITIONAL WORKS

You may be required to carry out further bushfire property preparedness works on your land to reduce any fire hazards considered necessary by a Fire Officer. If required, these will be outlined in a 'work order' and sent to the address of the owner and/or occupier.

EMERGENCY MANAGEMENT PLANS AND CITY APPROVED TREATMENT PLANS

All properties and/or land subject to a Bushfire Management Plan, Emergency Management Plan, or an approved Bushfire Attack Level assessment (BAL), as a result of subdivision, development application or a City approved treatment plan, must comply with the listed requirements in their entirety. **Compliance with any additional plans does not constitute compliance with this Notice**.

ENVIRONMENTAL CONDITIONS

Any property subject to environmental value such as, but not limited to, Threatened Ecological Communities (TEC), Bush Forever sites, Declared Rare Flora and Fauna (DRF) sites etc, should seek further information about what can or cannot be done prior to carrying out requirements under this Notice.

DEFINITIONS

FIRE BREAK: A strip of land free of all flammable material with the intention of minimising the spread or extension of a bushfire and provide safe access on the property for emergency vehicles and other firefighting operations.

- Clearance must be no less than 3-metres wide and 4-metres in height inside and along all boundaries (including boundaries adjacent to roads, rail and drain reserves and all public open space reserves).
- Must not be more than 4-metres wide (further width extensions may be considered upon written application for approval to the City).
- Maintained, reticulated living lawns are lawns considered to be kept completely green. Driveways may be acceptable in conjunction with, or in lieu of, mineral earth fire breaks. Contact a City Fire Officer for further assistance.
- Must have a corner turning radius of up to 10 metres.
- Must be a mineral earth break with a continuous trafficable surface for a 4WD vehicle, be clear of any obstructions and must not terminate in a cul-de-sac (dead end).

FUEL LOAD: Can be live and dead vegetation that accumulates over time.

This Notice refers only to dead vegetation.

Fine fuels include = leaf litter, grasses, twigs (up to 6mm diameter), bark etc.

Heavy (course) fuels include = branches, logs, stumps etc.

- A fuel load depth of 15mm (fine fuels) to the mineral earth is indicative of approximately 8 tonne per hectare. The more fuel load, the higher the flame height and increased fire intensity.
- Mulch piles, stored firewood and burn piles can contribute to fuel loading on land and must be stored safely away from assets, removed from the property, or actioned as directed by a Fire Officer.

If you require any further information, please contact the City of Kalamunda Community Safety team on (08) 9257 9999, email at enquiries@kalamunda.wa.gov.au or visit www.kalamunda.wa.gov.au/fire

Rhonda Hardy
CHIEF EXECUTIVE OFFICER

kalamunda.wa.gov.au

T 9257 9999 F 9293 2715 E enquiries@kalamunda.wa.gov.au
2 Railway Road KALAMUNDA WA 6076 PO Box 42, KALAMUNDA WA 6926
ABN 60 741 095 678

Appendix D Vehicular access technical standards of the Guidelines

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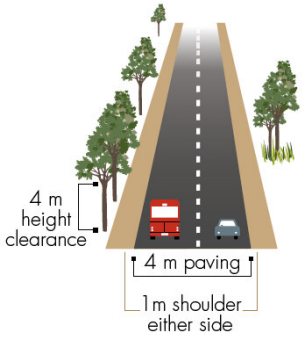
Public roads	
Acceptable solution A3.2	A public road is to meet the requirements in Table 1, Column 1.
Explanatory note E3.2	Trafficable surface: Widths quoted for access routes refer to the width of the trafficable surface. A six metre trafficable surface does not necessarily mean paving width. It could, for example, include four metre wide paving one metre wide constructed road shoulders. In special circumstances, where eight lots or less are being serviced, a public road with a minimum trafficable surface of four metres for a maximum distance of 90 metres may be provided subject to the approval of both the local government and Department of Fire and Emergency Services. Public road design: All roads should allow for two-way traffic to allow conventional two-wheel drive vehicles and fire appliances to travel safely on them. 

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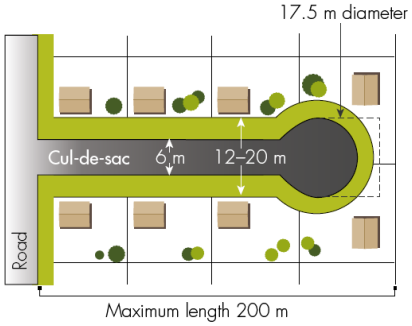
Cul-de-sac (including a dead-end road)	
Acceptable solution A3.3	A cul-de-sac and/ or a dead end road should be avoided in bushfire prone areas. Where no alternative exists (i.e. the lot layout already exists and/ or will need to be demonstrated by the proponent), the following requirements are to be achieved: • Requirements in Table 1, Column 2 • Maximum length: 200 metres (if public emergency access is provided between cul-de-sac heads maximum length can be increased to 600 metres provided no more than eight lots are serviced and the emergency access way is no more than 600 metres) • Turn-around area requirements, including a minimum 17.5 metre diameter head.
Explanatory note E3.3	In bushfire prone areas, a cul-de-sac subdivision layout is not favoured because they do not provide access in different directions for residents. In some instances, it may be possible to provide an emergency access way between cul-de-sac heads to a maximum distance of 600 metres, so as to achieve two-way access. Such links must be provided as right of ways or public access easements in gross to ensure accessibility to the public and fire services during an emergency. A cul-de-sac in a bushfire prone area is to connect to a public road that allows for travel in two directions in order to address Acceptable Solution A3.1. 

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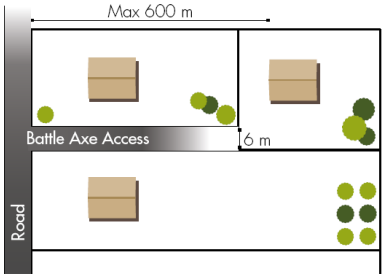
Battle-axe	
Acceptable solution A3.4	Battle-axe access leg should be avoided in bushfire prone areas. Where no alternative exists, (this will need to be demonstrated by the proponent) all of the following requirements are to be achieved: • Requirements in Table 1, Column 3 • Maximum length: 600 metres • Minimum width: six metres.
Explanatory note E3.4	In bushfire prone areas, lots with battle-axe access legs should be avoided because they often do not provide two-way access and egress for residents and may be easily blocked by falling trees or debris. In some instances, however; it may be appropriate for battle-axe access to be used to overcome specific site constraints. Where used, they should comply with the minimum standards for private driveways. Passing bays should be provided at 200 metre intervals along battle-axe access legs to allow two-way traffic. The passing bays should be a minimum length of 20 metres, with the combined width of the passing bay and the access being a minimum of six metres. Turn-around areas should allow type 3.4 fire appliances to turn around safely (i.e. kerb to kerb 17.5 metres) and should be available at house sites and at 500 metre intervals along the access leg. 

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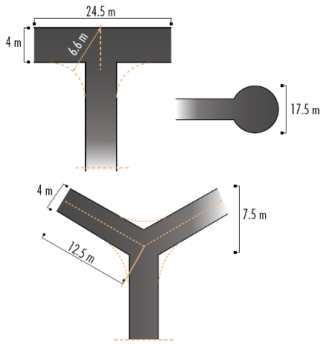
Private driveway longer than 50 metres	
Acceptable solution A3.5	A private driveway is to meet all of the following requirements: • Requirements in Table 1, Column 3 • Required where a house site is more than 50 metres from a public road • Passing bays: every 200 metres with a minimum length of 20 metres and a minimum width of two metres (i.e. the combined width of the passing bay and constructed private driveway to be a minimum six metres) • Turn-around areas designed to accommodate type 3.4 fire appliances and to enable them to turn around safely every 500 metres (i.e. kerb to kerb 17.5 metres) and within 50 metres of a house • Any bridges or culverts are able to support a minimum weight capacity of 15 tonnes • All-weather surface (i.e. compacted gravel, limestone or sealed).
Explanatory note E3.5	For a driveway shorter than 50 metres, fire appliances typically operate from the street frontage however where the distance exceeds 50 metres, then fire appliances will need to gain access along the driveway in order to defend the property during a bushfire. Where house sites are more than 50 metres from a public road, access to individual houses and turnaround areas should be available for both conventional two-wheel drive vehicles of residents and type 3.4 fire appliances. Turn-around areas should be located within 50 metres of a house. Passing bays should be available where driveways are longer than 200 metres and turn-around areas in driveways that are longer than 500 metres. Circular and loop driveway designs may also be considered. These criteria should be addressed through subdivision design. Passing bays should be provided at 200 metre intervals along private driveways to allow two-way traffic. The passing bays should be a minimum length of 20 metres, with the combined width of the passing bay and the access being a minimum of six metres. Turn-around areas should allow type 3.4 fire appliances to turn around safely (i.e. kerb to kerb 17.5 metres) and should be available at the house sites and at 500 metre intervals along the driveway. 

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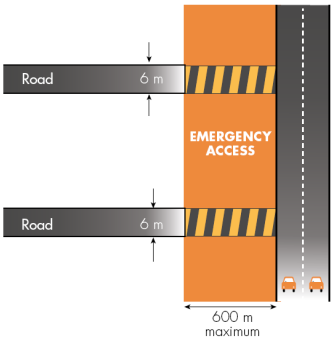
Emergency access way	
Acceptable solution A3.6	An access way that does not provide through access to a public road is to be avoided in bushfire prone areas. Where no alternative exists (this will need to be demonstrated by the proponent), an emergency access way is to be provided as an alternative link to a public road during emergencies. An emergency access way is to meet all of the following requirements: • Requirements in Table 1, Column 4 • No further than 600 metres from a public road • Provided as right of way or public access easement in gross to ensure accessibility to the public and fire services during an emergency • Must be signposted.
Explanatory note E3.6	An emergency access way is not a preferred option however may be used to link up with roads to allow alternative access and egress during emergencies where traffic flow designs do not allow for two-way access. Such access should be provided as a right-of-way or easement in gross to ensure accessibility to the public and fire emergency services during an emergency. The access should comply with minimum standards for a public road and should be signposted. Where gates are used to control traffic flow during non-emergency periods, these must not be locked. Emergency access ways are to be no longer than 600 metres and must be adequately signposted where they adjoin public roads. Where an emergency access way is constructed on private land, a right of way or easement in gross is to be established. 

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Fire service access routes (perimeter roads)	
Acceptable solution A3.7	Fire service access routes are to be established to provide access within and around the edge of the subdivision and related development to provide direct access to bushfire prone areas for fire fighters and link between public road networks for firefighting purposes. Fire service access routes are to meet the following requirements: • Requirements in Table 1, Column 5 • Provided as right of ways or public access easements in gross to ensure accessibility to the public and fire services during an emergency • Surface: all-weather (i.e. compacted gravel, limestone or sealed) • Dead end roads are not permitted • Turn-around areas designed to accommodate type 3.4 appliances and to enable them to turn around safely every 500 metres (i.e. kerb to kerb 17.5 metres) • No further than 600 metres from a public road • Allow for two-way traffic • Must be signposted.
Explanatory note E3.7	Fire service access routes should be established to separate bushfire prone areas from developed areas, and to provide access within and around the edge of subdivisions and related development. Fire service access is used during bushfire suppression operations but can also be used for fire prevention work. Fire service access routes should: • Link up with the road network at regular intervals - the development and road network forms part of the fire service access system • Be adequately signposted • Allow for two-way traffic - that is, two fire appliances must be able to safely pass each other • Have an all-weather surface (i.e. compacted gravel, limestone or sealed) • Have erosion control measures in place. Driveways may be used as part of the designated fire service access system, provided they meet the minimum standard for fire service access routes. It is beneficial to link the fire service access routes with individual driveways to allow quick access to properties and houses during fire emergencies. Where gates are used, these should be wide enough to accommodate type 3.4 fire appliances (minimum width of 3.6m) with the design and construction to be approved by the relevant local government. Gates on fire service access routes may be locked to restrict access provided that a common key system is used and such keys are made available for fire appliances and designated fire officers within the local government area and/or surrounding district. Gates should be installed where fences cross fire service access routes. Management and access arrangements should be in place to ensure that the maintenance of fire service access routes will occur in the long term after an area has been subdivided. A number of options can be used to achieve this, including but not limited to: • Individual property owners being responsible for maintaining fire service access routes where these fall on their property • Providing such access as a right-of-way or easement in gross to ensure accessibility to fire services during an emergency; and/or • A levy system administered by local government to cover the cost of maintaining fire service access routes. Such arrangements should be documented in the relevant planning application (such as a structure plan, subdivision plan or development plan) and should be agreed to by local government.

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Technical requirement	1	2	3	4	5
	Public road	Cul-de-sac	Private driveway longer than 50 m	Emergency access way	Fire service access routes
Minimum trafficable surface (m)	6*	6	4	6*	6*
Horizontal distance (m)	6	6	6	6	6
Vertical clearance (m)	4.5	N/A	4.5	4.5	4.5
Maximum grade <50 m	1 in 10	1 in 10	1 in 10	1 in 10	1 in 10
Minimum weight capacity (t)	15	15	15	15	15
Maximum crossfall	1 in 33	1 in 33	1 in 33	1 in 33	1 in 33
Curves minimum inner radius	8.5	8.5	8.5	8.5	8.5
* Refer to E3.2 Public roads: Trafficable surface					

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Appendix E Water technical standards of the Guidelines

Reticulated areas	
Acceptable solution A4.1	The subdivision, development or land use is provided with a reticulated water supply in accordance with the specifications of the relevant water supply authority and Department of Fire and Emergency Services.
Explanatory note E4.1	Water supply authorities in Western Australia include the Water Corporation, Aqwest and the Busselton Water Board. The Water Corporation's 'No. 63 Water Reticulation Standard' is deemed to be the baseline criterion for developments and should be applied unless local water supply authorities' conditions apply.

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element.

Technical Appendix C - Lloyd George Transportation Noise Assessment

Lloyd George Acoustics
PO Box 717
Hillarys WA 6923
T: 9401 7770
www.lgacoustics.com.au

Transportation Noise Assessment

Forrestfield North Transit Oriented Development (TOD)

Reference: 17064031-03.docx

Prepared for:
element

Report: 17064031-03.docx

Lloyd George Acoustics Pty Ltd

ABN: 79 125 812 544

PO Box 717
Hillarys WA 6923

www.lgacoustics.com.au

T: 9401 7770

Contacts	Daniel Lloyd	Terry George	Matt Moyle	Olivier Mallié	Ben Hillion	Rob Connolly
E:	daniel@lgacoustics.com.au	terry@lgacoustics.com.au	matt@lgacoustics.com.au	olivier@lgacoustics.com.au	ben@lgacoustics.com.au	rob@lgacoustics.com.au
M:	0439 032 844	0400 414 197	0412 611 330	0439 987 455	0457 095 555	0410 107 440

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Date:	Rev	Description	Prepared By	Verified
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Appendices

A Quiet House Packages

B Terminology

1 INTRODUCTION

This report has been prepared to consider the potential noise impacts to the area referred to as Forrestfield North TOD Precinct, located as shown in *Figure 1-1* (red outline). Sources of potential noise impacts are:

- Aircraft: Perth Airport – Future parallel runway to the west of the site (shown in blue);
- Freight Trains: Railway – Immediately west of the site, east of the new runway;
- Road Traffic: Roe Highway – Immediately east of the site as well as other secondary roads.



Figure 1-1 Site Locality

Other noise sources may also be generated as a result of the structure plan such as noise sensitive uses near light industry and activity centres or the like.

The structure plan is provided in *Figure 1-2*.

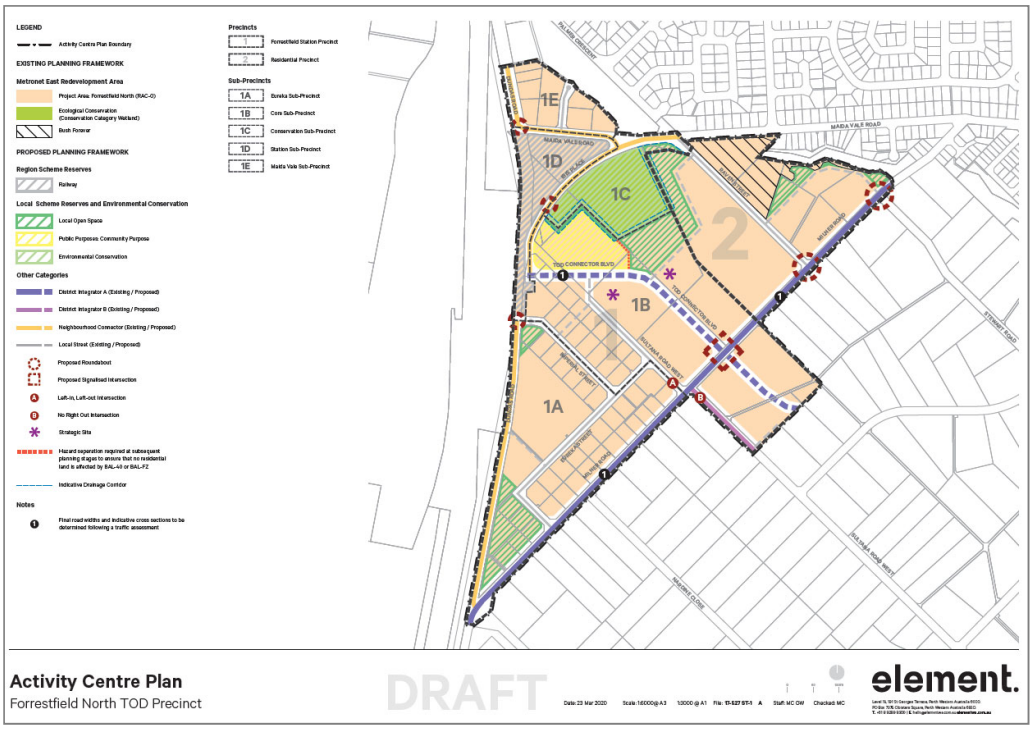


Figure 1-2 Structure Plan

Appendix B contains a description of some of the terminology used throughout this report.

2 CRITERIA

2.1 State Planning Policy 5.1

The relevant planning policy in Western Australia in relation to aircraft noise is *State Planning Policy 5.1: Land Use Planning in the Vicinity of Perth Airport*; July 2015, Western Australian Planning Commission (hereafter referred to as SPP 5.1). SPP 5.1 applies to any land within Aircraft Noise Exposure Forecast (ANEF) 20 and separates land into three zones:

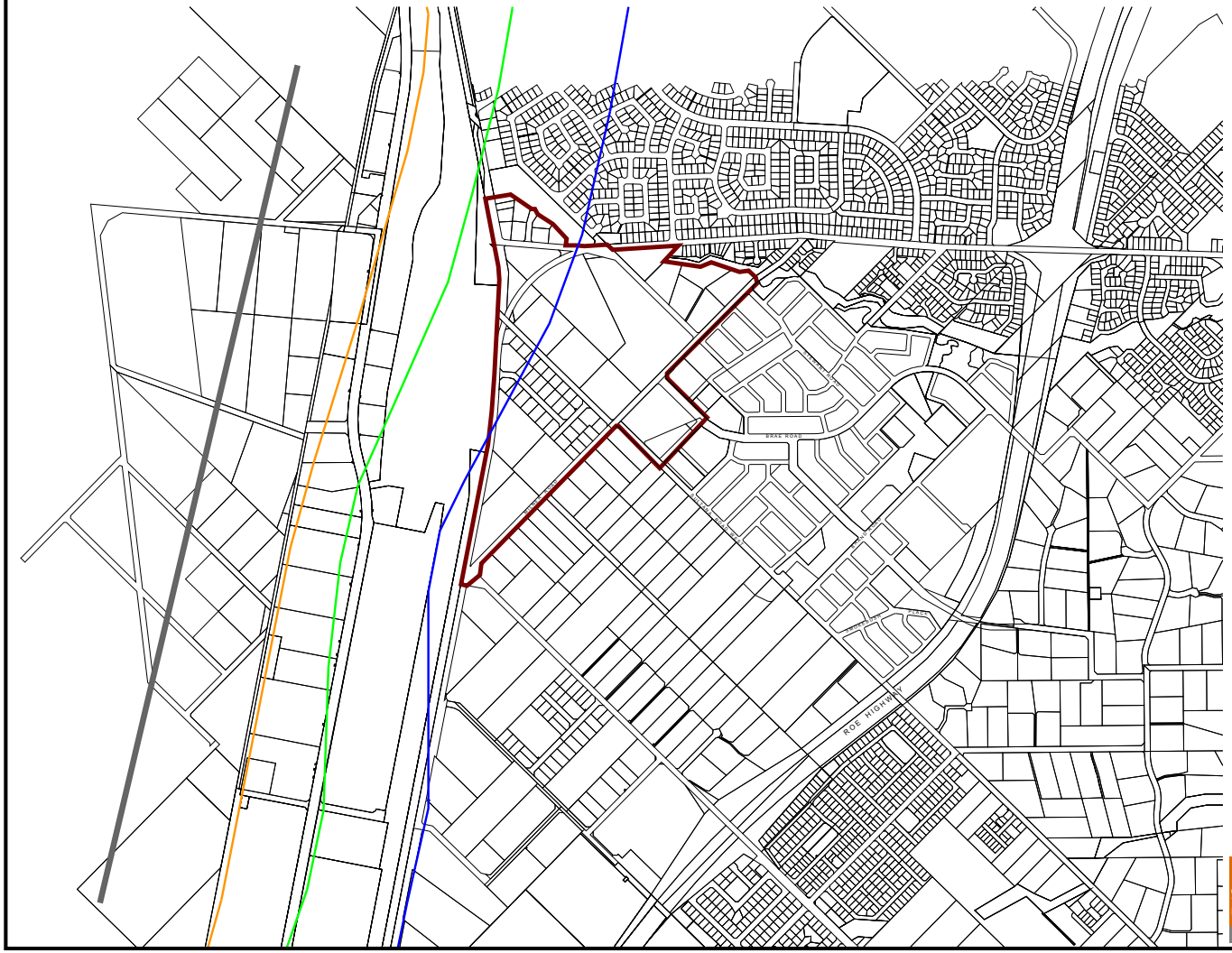
- Areas below 20 ANEF;
- Areas between 20 ANEF and 25 ANEF; and
- Areas above 25 ANEF.

Figure 2-1 shows the locations of the ANEF contours with *Figure 2-2* showing the N65 contours, both taken from the 2020 Master Plan for Perth Airport. The N65 contours indicate the average expected number of daily aircraft events above 65 dB L_{Amax} outside a building.

Forrestfield North Local Structure Plan: TOD Precinct

Aircraft Noise Exposure Forecast (ANEF) Contours

Figure 2-1



Signs and symbols

- ANEF 30
- ANEF 25
- ANEF 20
- Study Area
- New Runway

26 Mar 2021

Length Scale 1:20000

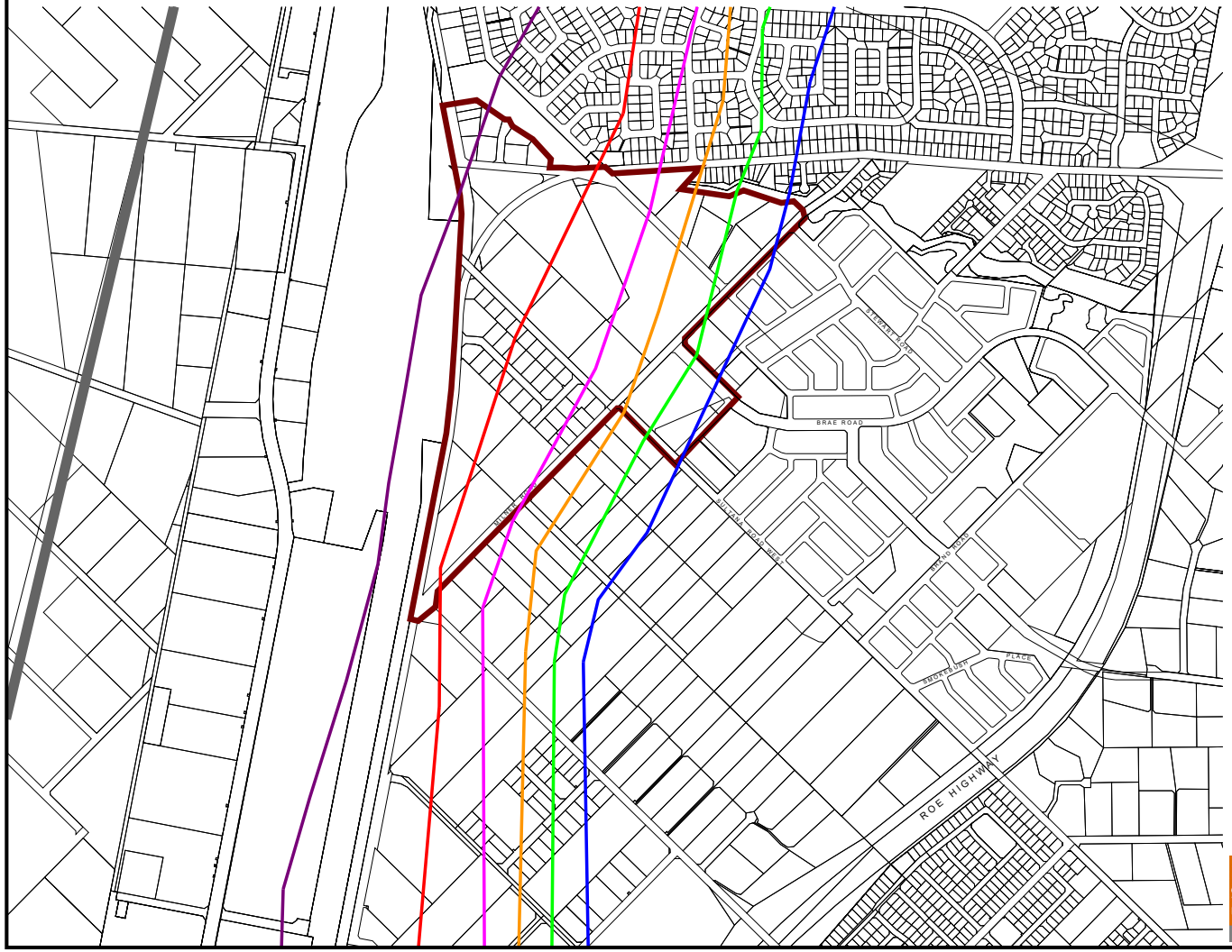
0 100 200 400 600 m

Lloyd George Acoustics
by Terry George
terry@lgacoustics.com.au
(08) 9401 7770

Forrestfield North Local Structure Plan: TOD Precinct

Aircraft N65 Contours - Average Number of Daily Events Above 65 dB(A)

Figure 2-2



Signs and symbols

- 200 Events
- 100 Events
- 50 Events
- 20 Events
- 10 Events
- 5 Events
- Study Area
- Future Runway

N

26 March 2021

Length Scale 1:15000

0 100 200 400 600 m

Lloyd George Acoustics
by Terry George
terry@lgacoustics.com.au
(08) 9401 7770

There is no restriction on zoning or development below 20 ANEF, which the majority of the TOD Precinct is outside of. The northwest portion of the TOD Precinct is between 20 & 25 ANEF, at which point SPP 5.1 recommends a maximum dwelling density of R20 except where:

- Land is identified for more intensive development through strategic planning instruments such as regional or sub-regional structure plan;
- A higher density coding is desirable to facilitate redevelopment or infill development of an existing residential area; and
- It can be demonstrated that the public benefits of higher density coding outweigh the negative impacts of exposing additional residents to aircraft noise.

Between 20 & 25 ANEF, the following developments are conditionally acceptable:

- Dwellings and caravan parks;
- Educational establishments;
- Child-care premises
- Hospitals and nursing homes; and
- Places of worship; and
- Cinemas, theatres and exhibition centres.

There are no developments that are unacceptable between 20 & 25 ANEF.

No parts of the Forrestfield North Area are at or above ANEF 25.

2.2 State Planning Policy 5.4

The criteria relevant to road and rail traffic noise is provided in *State Planning Policy No. 5.4 Road and Rail Noise* (hereafter referred to as SPP 5.4) produced by the Western Australian Planning Commission (WAPC). The objectives of SPP 5.4 are to:

- Protect the community from unreasonable levels of transport noise;
- Protect strategic and other significant freight transport corridors from incompatible urban encroachment;
- Ensure transport infrastructure and land-use can mutually exist within urban corridors;
- Ensure that noise impacts are addressed as early as possible in the planning process; and
- Encourage best practice noise mitigation design and construction standards

Table 2-1 sets out noise targets that are to be achieved by proposals under which SPP 5.4 applies. Where the targets are exceeded, an assessment is required to determine the likely level of transport noise and management/mitigation required.

The application of SPP 5.4 is to consider anticipated traffic volumes for the next 20 years from when the noise assessment is undertaken.

Table 2-1 Noise Targets for Noise-Sensitive Land-Use

Outdoor Noise Target		Indoor Noise Target	
55 dB $L_{Aeq(Day)}$	50 dB $L_{Aeq(Night)}$	40 dB $L_{Aeq(Day)}$ (Living and Work Areas)	35 dB $L_{Aeq(Night)}$ (Bedrooms)

Notes:

- Day period is from 6am to 10pm and night period from 10pm to 6am.
- The outdoor noise target is to be measured at 1-metre from the most exposed, habitable[1] facade of the noise sensitive building.
- For all noise-sensitive land-use and/or development, indoor noise targets for other room usages may be reasonable drawn from Table 1 of Australian Standard/New Zealand Standard AS/NZS 2107:2016 Acoustics – Recommended design sound levels and reverberation times for building interiors (as amended) for each relevant time period.
- Outdoor targets are to be met at all outdoor areas as far as is reasonable and practicable to do so using the various noise mitigation measures outlined in the Guidelines.

In the application of the noise targets, the objective is to achieve:

- indoor noise levels specified in *Table 2-1* in noise-sensitive areas (e.g. bedrooms and living rooms of houses and school classrooms); and
- a reasonable degree of acoustic amenity for outdoor living areas on each residential lot. For non-residential noise-sensitive developments, for example schools and childcare centres, the design of outdoor areas should take into consideration the noise target.

It is recognised that in some instances, it may not be reasonable and/or practicable to meet the outdoor noise targets. Where transport noise is above the noise targets, measures are expected to be implemented that balance reasonable and practicable considerations with the need to achieve acceptable noise protection outcomes.

2.3 Environmental Protection (Noise) Regulations 1997

Existing industry and new premises, as part of the overall Forrestfield North development (e.g. Activity Centres), will be required to control their noise emissions to comply with the prescribed standards of the *Environmental Protection (Noise) Regulations 1997*. These Regulations have not been explained in detail in this report, due to the early stage of this development. However, as each of these developments submits Development Applications, these are to be accompanied by an acoustic assessment, undertaken by a suitably qualified acoustical consultant, being a member firm of the Association of Australasian Acoustic Consultants (AAAC).

[1] A habitable room is defined in State Planning Policy 3.1 as a room used for normal domestic activities that includes a bedroom, living room, lounge room, music room, sitting room, television room, kitchen, dining room, sewing room, study, playroom, sunroom, gymnasium, fully enclosed swimming pool or patio.

2.4 AS2670.2-1990

Exposure limits for vibration are normally defined in terms of a multiplying factor that is applied to the base curves defined in AS 2670.2:1990 *Evaluation of Human Exposure to Whole Body Vibration, Part 2: Continuous and Shock Induced Vibration in Buildings (1 to 80 Hz)*. The base curve is the point at which adverse comment is considered rare. The Standard states that at levels above the base curve, vibration may or may not give rise to adverse comment depending on circumstances. The measure of human annoyance within this report is a velocity (mm/s) root mean squared (rms). The multiplying factors are given in Appendix A of AS 2670.2-1990 as follows:

- Residential
  - Night-time – Curve 1.4;
  - Daytime – Curve 2 to Curve 4
- Commercial – Curve 4;
- Industrial – Curve 8.

It is noted that within residential areas there are wide variations in vibration tolerance. Specific values are dependent upon social and cultural factors, psychological attitudes and expected interference with privacy.

Previous projects for residential use within Western Australia have adopted Curve 2, which has been accepted by the Department of Water and Environmental Regulation (DWER), with a preference to achieve Curve 1.4 where practicable.

3 METHODOLOGY

Combinations of literature review, noise measurements and noise modelling have been undertaken as described in the following sections.

3.1 Previous Studies

Noise and vibration associated with the new Forrestfield train station (Airport Link project) will be considered by others, however it is noted the new railway will be below ground.

As part of the Airport Link project, a study of the existing noise and vibration levels from the Forrestfield Freight Rail Terminal (FFRT) has been undertaken by SLR (*Forrestfield Freight Rail Terminal, Environmental Noise and Vibration Survey*; Reference: 675.10594-R3, 24 April 2015) and made available by the Public Transport Authority (PTA). The findings of this study have been utilised within this report.

3.2 Site Measurements

To supplement the work undertaken by SLR, noise monitoring was undertaken at four (4) locations across the site in order to:

- Quantify the existing noise levels;
- Determine the differences between different acoustic parameters ($L_{A10,18hour}$, $L_{Aeq(Day)}$ and $L_{Aeq(Night)}$); and
- Calibrate the noise model for existing conditions.

The instruments used were ARL Type 316 & Ngara noise data loggers, located as shown in *Figure 3-1* and *Figure 3-2*, with the microphones 1.4 metres above ground level. The loggers were programmed to record hourly L_{Amax}, L_{A1}, L_{A10}, L_{A90}, and L_{Aeq} levels. This instrument complies with the instrumentation requirements of *Australian Standard 2702-1984 Acoustics – Methods for the Measurement of Road Traffic Noise*. The loggers were field calibrated before and after the measurement session and found to be accurate to within +/- 1 dB. Lloyd George Acoustics also holds current laboratory calibration certificate for the loggers.



Maida Vale Road Logger



Freight Railway Logger



Roe Highway North Logger



Roe Highway South Logger

Figure 3-1 Photographs of Noise Data Loggers



Figure 3-2 Locations of Noise Data Loggers

Noise loggers were set-up on 25 October 2017 and collected on 3 November 2017. All data was reviewed and it was noted that the battery failed on 2 November 2017 at the Roe Highway South location, so that this day is not reported for the location.

The weather conditions during the measurement period were obtained from the Bureau of Meteorology's, Perth Airport site. This data was compared against the Main Roads Western Australia (MRWA) specifications for measurement conditions, resulting in recorded noise on 26 & 27 October 2017 being excluded. Also note that weekend data (28 & 29 October 2017) are also considered atypical and also excluded.

3.3 Noise Modelling

3.3.1 General

The computer programme *SoundPLAN 7.4* was utilised to predict noise levels.

Road traffic noise modelling incorporated the *Calculation of Road Traffic Noise* (CoRTN) algorithms, modified to reflect Australian conditions. The modifications included the following:

- Vehicles were separated into heavy (Austroads Class 3 upwards) and non-heavy (Austroads Classes 1 & 2) with non-heavy vehicles having a source height of 0.5 metres above road level and heavy vehicles having two sources, at heights of 1.5 metres and 3.6 metres above road level, to represent the engine and exhaust respectively. By splitting the noise source into three, allows for less barrier attenuation for high level sources where barriers are to be considered.
- Corrections are applied to the exhaust of –8.0 dB (based on Transportation Noise Reference Book, Paul Nelson, 1987) and to the engine source of –0.8 dB, so as to provide consistent results with the CoRTN algorithms for the no barrier scenario;
- An adjustment of –1.7 dB has been applied to the predicted levels based on the findings of An Evaluation of the U.K. DoE Traffic Noise Prediction; Australian Road Research Board, Report 122 ARRB – NAASRA Planning Group 1982.

Rail traffic modelling used the *Nordic Prediction Method for Train Noise* (NMT) algorithms. The following options were selected with the model then calibrated against the noise measurements:

- The rail head is assumed to be 0.6 metres above natural ground;
- Temperature, humidity and air pressure are assumed to be 15°C, 70% and 1013.3 mbar respectively;
- Air absorption is in accordance with ANSI 126;
- L_{max} for diesel powered trains option is selected.

Predictions are made at heights of 1.4 metres above floor level and at 1.0 metre from an assumed building façade (resulting in a + 2.5 dB correction due to reflected noise).

Various input data are included in the modelling such as ground topography, road design, traffic volumes etc. These model inputs are discussed below.

3.3.2 Ground Topography

Topographical data was provided for the project by element and also combined with that already on file for work undertaken on the Gateway WA project, which includes the intersection design at Berkshire Road. For the existing scenario, Roe Highway consists of two lanes in each direction whereas in the future, this is assumed to be 3 lanes in each direction. Other roads and land within the study area are based on existing topography, as this is all that is currently available. As the project progresses to subdivision stage, the noise impacts would need to be reconsidered as final design and lot levels will have an impact on the noise assessment.

Existing buildings have been included in order to model the existing scenario and model calibration. For the future scenario, existing buildings within the study area are removed and these have been replaced with future building outlines and building heights provided by element (Reference: Indicative Built Form Plan, dated 28 April 2020) and modified to suit the *Figure 1-2* Structure Plan.

3.3.3 Road Traffic Data

Traffic data includes:

- Road Surface – The noise relationship between different road surface types is shown in *Table 3-1*.

Table 3-1 Noise Relationship Between Different Road Surfaces

Road Surfaces							
Chip Seal				Asphalt			
14mm	10mm	5mm	Slurry	Dense Graded	Novachip	Stone Mastic	Open Graded
+3.5 dB	+2.5 dB	+1.5 dB	+1.0 dB	0.0 dB	-0.2 dB	-1.5 dB	-2.5 dB

For noise modelling, the assumed road surfaces are:

  - Berkshire Road current DGA and staying as such in the future.
  - Maida Vale Road currently 5mm chip seal, changing to dense graded asphalt in the future.
  - Roe Highway from Berkshire Road, to just north of the logger (Roe Highway South) is OGA and then changes to a worn 14mm chip seal (approximated as 5mm chip seal). In the future, it is assumed the entire road surface will be OGA.
  - Dundas Road current DGA and staying as such in the future.
  - Milner Road – Currently modelled as 5mm chip seal to be DGA in the future.
  - Future TOD Connector Road to be DGA.

- Vehicle Speed – The existing and future posted speeds used in the modelling are:
  - Berkshire Road – 70km/hr
  - Maida Vale Road – 60km/hr
  - Roe Highway – 100km/hr
  - Dundas Road – 70 km/hr
  - Milner Road – Currently 70km/hr to reduce to 60km/hr in the future
  - Future TOD Connector Road – 60km/hr

- Traffic Volumes – Information used in the modelling has been sourced from KCTT and shown in *Tables 3-2 & 3-3*.

Table 3-2 Traffic Information Used in the Modelling - Existing

Road Name	Section	Daily Traffic Volume	Heavy Vehicles %
Berkshire Road	West of Roe Highway (Mar 16)	4,470	23.0
	West of Walters Way (Aug 2016)	5,054	15.9
Maida Vale Road	West of Roe Highway (Apr 2010)	4,757	6.4
	West of Butcher Road (Mar 17)	2,214	8.4
Roe Highway	North of Berkshire Road (Dec 15)	47,250	16.6
	South of Kalamunda Road (Dec 15)	37,956	19.4
Dundas Road	South of Maida Vale Road (Jul 16)	2,708	22.3
	North of Maida Vale Road (Jun 16)	6,091	13.7

Note: All roads are assumed to have a 50/50 traffic split in each direction.

Table 3-3 Traffic Information Used in the Modelling – Future 2050

Road Name	Section	Daily Traffic Volume	Heavy Vehicles %
Berkshire Road	West of Roe Highway	35,000	15.0
	West of Walters Way	33,000	10.0
Maida Vale Road	West of Roe Highway	19,100	6.5
	West of Milner Road	12,000	8.5
Roe Highway	North of Berkshire Road	186,000	16.5
	South of Kalamunda Road	164,000	20.0
Dundas Road	South of Maida Vale Road	27,500	20.0
	North of Maida Vale Road	47,000	8.0
Milner Road	South of TOD Connector	27,000	8.0
	North of TOD Connector	17,800	6.5
TOD Connector	West of Milner Road	14,000	6.5
	East of Milner Road	11,000	6.5

Note: All roads are assumed to have a 50/50 traffic split in each direction and 2050 heavy vehicles are based on those provided for 2031. Some heavy vehicle % are estimated.

3.3.4 Rail Traffic Data

A detailed analysis of existing number of train movements was not undertaken, however is expected to be at the minimum requirement of SPP 5.4 Guidelines being 1 train movement per hour. To allow for future growth, a 2 dB increase has been assumed in the future. Where there are 24 movements per day, the 2 dB increase would represent around 38 movements per day.

3.3.5 Ground Attenuation

The ground attenuation has been assumed to be 0.0 (0%) for the road, 0.5 (50%) throughout the subdivision, except for the public open space, which was set to 1.00 (100%). Note 0.0 represents hard reflective surfaces such as water and 1.00 represents absorptive surfaces such as grass.

3.3.6 Parameter Conversion

The CoRTN algorithms used in the *SoundPlan* modelling package for road traffic were originally developed to calculate the $L_{A10,18hour}$ noise level. SPP 5.4 however uses $L_{Aeq(Day)}$ and $L_{Aeq(Night)}$. The relationship between the parameters varies depending on the composition of traffic on the road (volumes in each period and percentage heavy vehicles).

As noise monitoring was undertaken, the relationship between the parameters is based on the results of the monitoring – refer *Section 4.2.2*.

4 RESULTS

4.1 Aircraft Noise Impacts

As discussed in *Section 2.1*, no parts of the Forrestfield North Area will be developed at or above 25 ANEF. Parts of the TOD, will be between ANEF 20 & 25. For information, the following developments are conditionally acceptable between ANEF 20 & 25:

- Dwellings and caravan parks;
- Educational establishments;
- Child-care premises
- Hospitals and nursing homes; and
- Places of worship; and
- Cinemas, theatres and exhibition centres.

There are no developments that are unacceptable between 20 & 25 ANEF. AS2021 notes that the 20 ANEF contour is difficult to accurately define due to variations in actual flight paths. It must also be remembered that ANEF contours do not represent a noise level and below 20 ANEF does not mean that aircraft noise will not be audible and can still be considered annoying by some residents.

Figure 2-2 showed the locality of the N65 contour lines. A level of 65 dB(A) is the approximate level at which aircraft noise may start to interfere with normal conversation. Given that residents moving into the area are likely to be unaccustomed to aircraft noise, it is recommended that:

- As a minimum, all developments (residential, commercial etc) incorporate 6mm thick glass in external façades.
- Any development containing noise sensitive parts, located west of the 20 ANEF, is to have an individual assessment undertaken by a suitably qualified acoustical consultant ensuring compliance with the internal design levels specified within AS2021.
- Any noise sensitive premises shall incorporate a notification on title as follows:

  This property is situated in the vicinity of Perth Airport and is currently affected, or may be affected in the future by aircraft noise. Noise exposure levels are likely to increase in the future as a result of an increase in aircraft using the airport, changes in aircraft type or other operational changes.

4.2 Road and Rail Noise & Vibration

4.2.1 SLR Study

The SLR Study determined the following distances in order to achieve the various vibration criteria:

- Land within 85 metres may receive vibration levels above 0.14mm/s (Curve 1.4);
- Land within 70 metres may receive vibration levels above 0.2mm/s (Curve 2);
- Land within 45 metres may receive vibration levels above 0.4mm/s (Curve 4).

The above are presented on *Figure 4-1*. Given the separation distance between the freight railway and TOD, vibration is unlikely to be a significant issue.

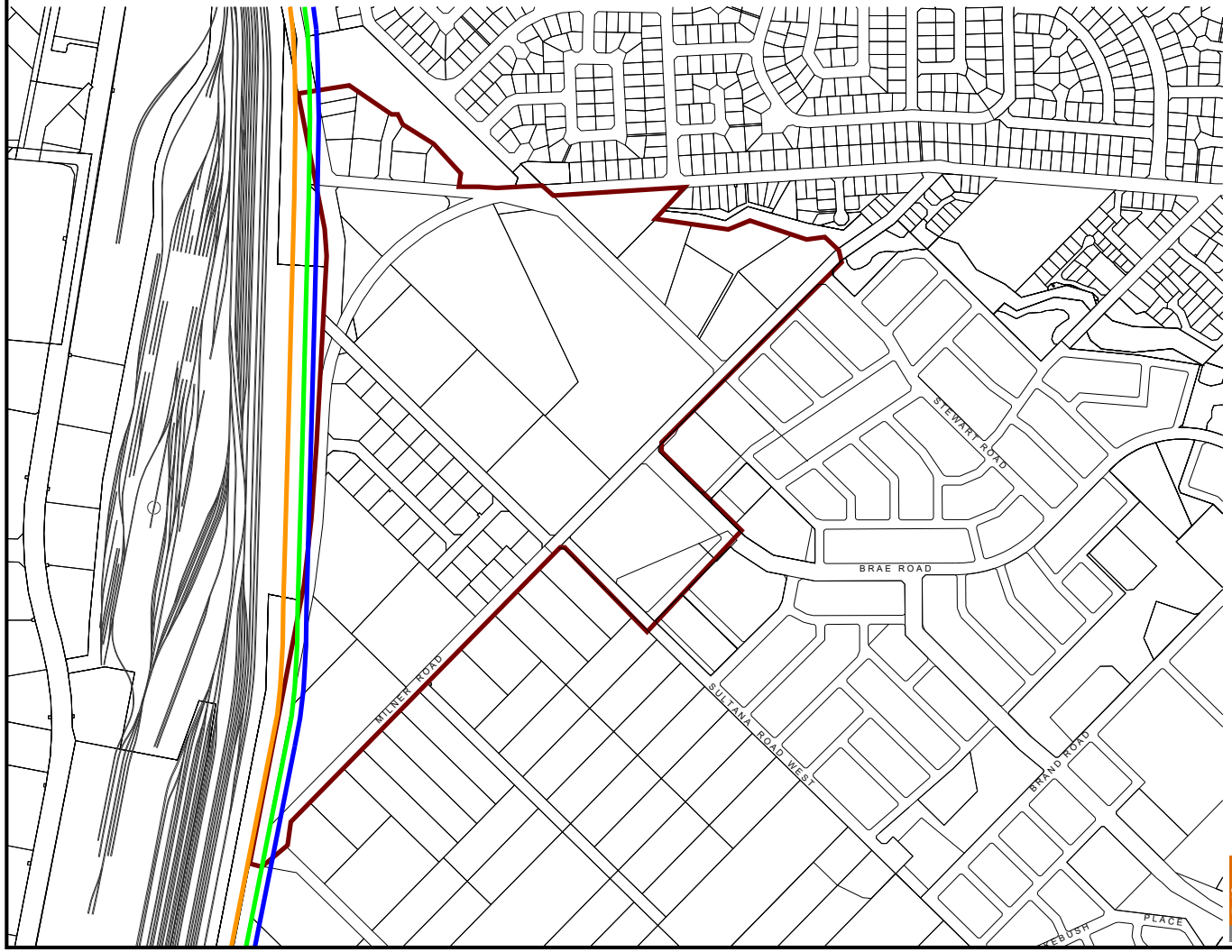
With regards to noise, the SLR contours are for ground level only and combine both intermittent freight train movements with other more permanent noises such as idling locomotives, workshop noise and the like. As the SLR contours were for ground level, these are affected by buildings and show attenuation that may not be apparent at upper floors so that the contours have been 'smoothed' as shown on *Figure 4-2*.

From *Figure 4-2*, it can be seen that the $L_{Aeq(Night)}$ parameter is more stringent than the $L_{Aeq(Day)}$ parameter. The 45 dB $L_{Aeq(Night)}$ noise contour is approximately 300 metres from the railway track.

Forrestfield North Local Structure Plan: TOD Precinct

Vibration From Forrestfield Freight Rail Terminal (SLR Study)

Figure 4-1



N

Signs and symbols

- Curve 4
- Curve 2
- Curve 1.4
- Study Area
- Freight Line

26 March 2021

Length Scale 1:10000

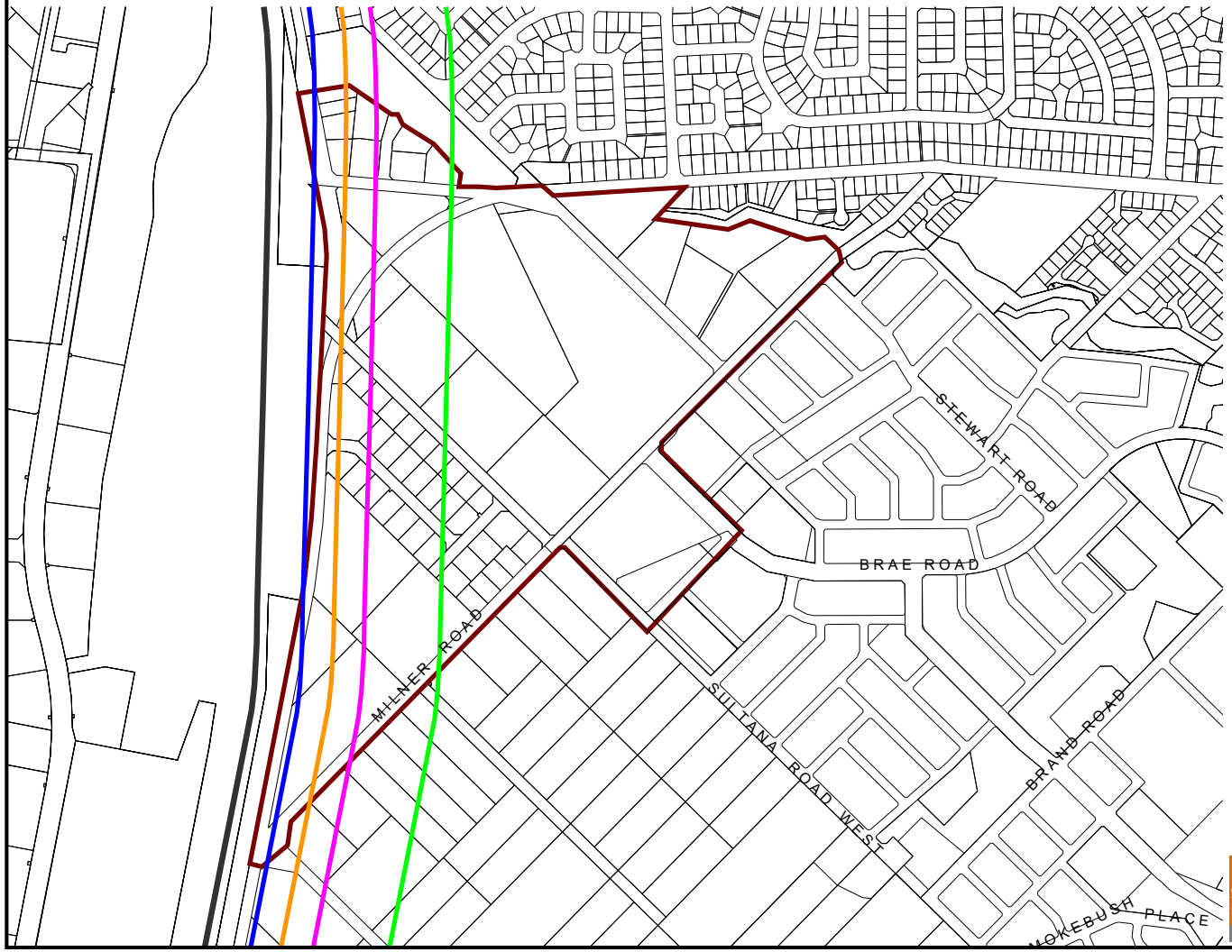
0 50 100 200 300 m

Lloyd George Acoustics
by Terry George
terry@lgacoustics.com.au
(08) 9401 7770

Forrestfield North Local Structure Plan: TOD Precinct

LAeq Noise Levels from FFRT (SLR Study)

Figure 4-2



Lloyd George Acoustics
by Terry George
terry@lgacoustics.com.au
(08) 9401 7770

4.2.2 Project Noise Logging

The results of the noise monitoring are summarised in *Table 4-1* to *Table 4-4* and shown graphically in *Figure 4-3* to *Figure 4-6*.

Table 4-1 Measured Average Noise Levels – Maida Vale Road

Date	Average Weekday Noise Level, dB			
	$L_{A10,18hour}$	$L_{Aeq,24hour}$	$L_{Aeq\ (Day)}$	$L_{Aeq\ (Night)}$
Monday 30 October 2017	59.9	58.8	60.1	53.5
Tuesday 31 October 2017	60.4	58.8	59.9	54.9
Wednesday 1 November 2017	61.0	58.4	59.5	55.2
Thursday 2 November 2017	60.9	58.8	59.6	56.4
Weekday Average	**60.5**	**58.7**	**59.8**	**55.0**

The average differences between the weekday $L_{Aeq(Day)}$ and $L_{Aeq(Night)}$ is 4.8 dB, which shows that neither the day nor night noise levels are more critical than the other, with an approximate 5 dB differential.

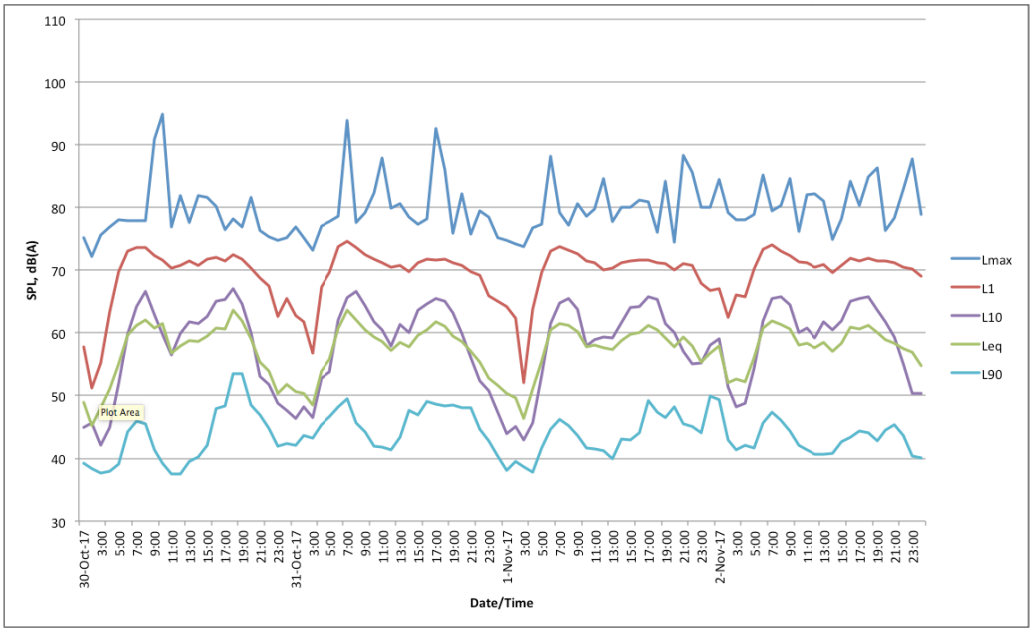


Figure 4-3 Maida Vale Noise Logging Results

Table 4-2 Measured Average Noise Levels – Freight Railway

Date	Average Weekday Noise Level, dB			
	$L_{A10,18hour}$	$L_{Aeq,24hour}$	$L_{Aeq\ (Day)}$	$L_{Aeq\ (Night)}$
Monday 30 October 2017	68.1	66.2	66.3	65.9
Tuesday 31 October 2017	68.3	66.9	67.2	66.1
Wednesday 1 November 2017	69.1	66.7	67.1	65.8
Thursday 2 November 2017	68.8	66.9	67.3	65.9
Weekday Average	**68.6**	**66.7**	**67.0**	**65.9**

The average difference between the weekday $L_{Aeq(Day)}$ and $L_{Aeq(Night)}$ is 1.0 dB, which shows that freight train movements are reasonably consistent across a 24-hour period.

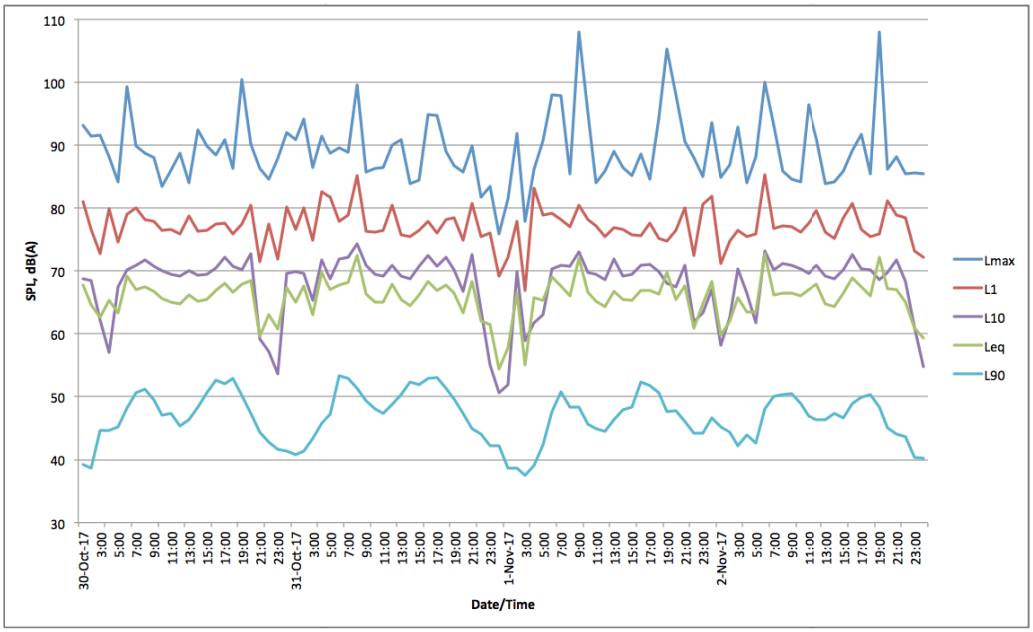


Figure 4-4 Freight Railway Noise Logging Results

Table 4-3 Measured Average Noise Levels – Roe Highway (North)

Date	Average Weekday Noise Level, dB			
	$L_{A10,18hour}$	$L_{Aeq,24hour}$	$L_{Aeq\ (Day)}$	$L_{Aeq\ (Night)}$
Monday 30 October 2017	74.4	71.1	72.3	66.3
Tuesday 31 October 2017	74.5	71.1	72.3	66.3
Wednesday 1 November 2017	74.1	70.7	71.9	66.6
Thursday 2 November 2017	74.0	70.8	71.9	67.0
Weekday Average	**74.2**	**70.9**	**72.1**	**66.5**

The average difference between the weekday $L_{Aeq(Day)}$ and $L_{Aeq(Night)}$ is 5.6 dB showing the $L_{Aeq(Day)}$ is slightly more critical than the $L_{Aeq(Night)}$.

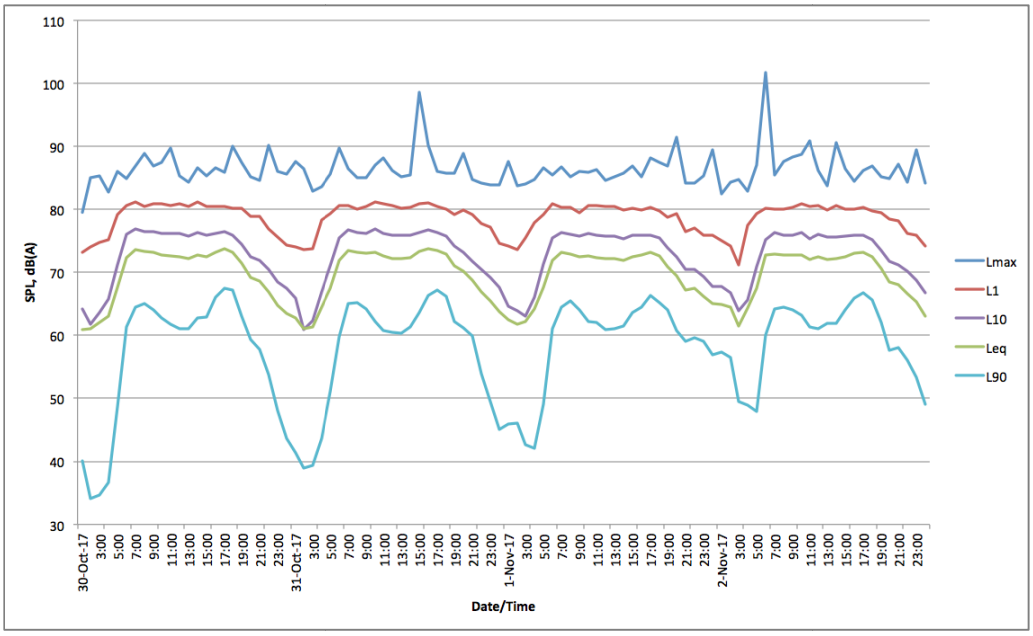


Figure 4-5 Roe Highway (North) Noise Logging Results

Table 4-4 Measured Average Noise Levels – Roe Highway (South)

Date	Average Weekday Noise Level, dB			
	$L_{A10,18hour}$	$L_{Aeq,24hour}$	$L_{Aeq\ (Day)}$	$L_{Aeq\ (Night)}$
Monday 30 October 2017	65.9	63.0	64.2	58.8
Tuesday 31 October 2017	66.0	62.9	64.0	59.0
Wednesday 1 November 2017	66.2	63.2	64.2	59.4
Weekday Average	**66.1**	**63.0**	**64.1**	**59.1**

The average difference between the $L_{Aeq(Day)}$ and $L_{Aeq(Night)}$ is 5.0 dB, such that again, neither the day nor night noise levels are more critical.

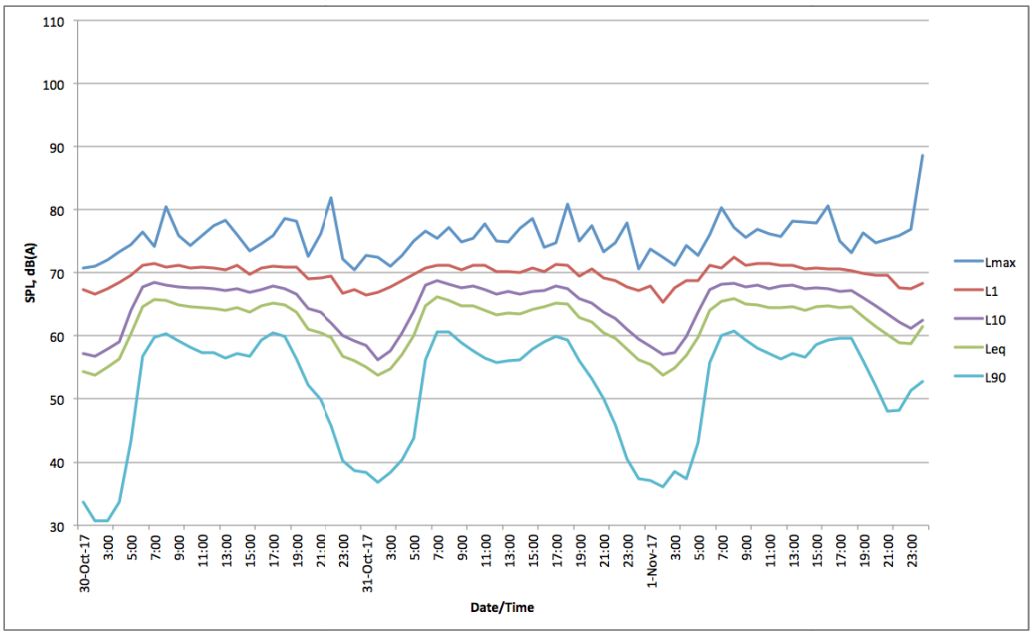


Figure 4-6 Roe Highway (South) Noise Logging Results

4.2.3 Noise Modelling

Noise modelling was originally undertaken for existing conditions and calibrated against the noise logging described in *Section 4.2.2*. The model calibrations were then maintained for future volumes as described in *Section 3.3.3* (road traffic) and *Section 3.3.4* (rail traffic) and including the future development of the site.

The results of the future noise modelling are provided in *Figure 4-7* to *Figure 4-12*, representing noise from road traffic and rail traffic to the ground, first and second floor levels. Higher floors are proposed, however the three contour plots for each noise source are considered representative.

- *Figure 4-7* Future Road Traffic $L_{Aeq(Day)}$ Noise Contours: Ground Level
- *Figure 4-8* Future Road Traffic $L_{Aeq(Day)}$ Noise Contours: First Level
- *Figure 4-9* Future Road Traffic $L_{Aeq(Day)}$ Noise Contours: Second Level
- *Figure 4-10* Future Rail Traffic $L_{Aeq(Night)}$ Noise Contours: Ground Level
- *Figure 4-11* Future Rail Traffic $L_{Aeq(Night)}$ Noise Contours: First Level
- *Figure 4-12* Future Rail Traffic $L_{Aeq(Night)}$ Noise Contours: Second Level

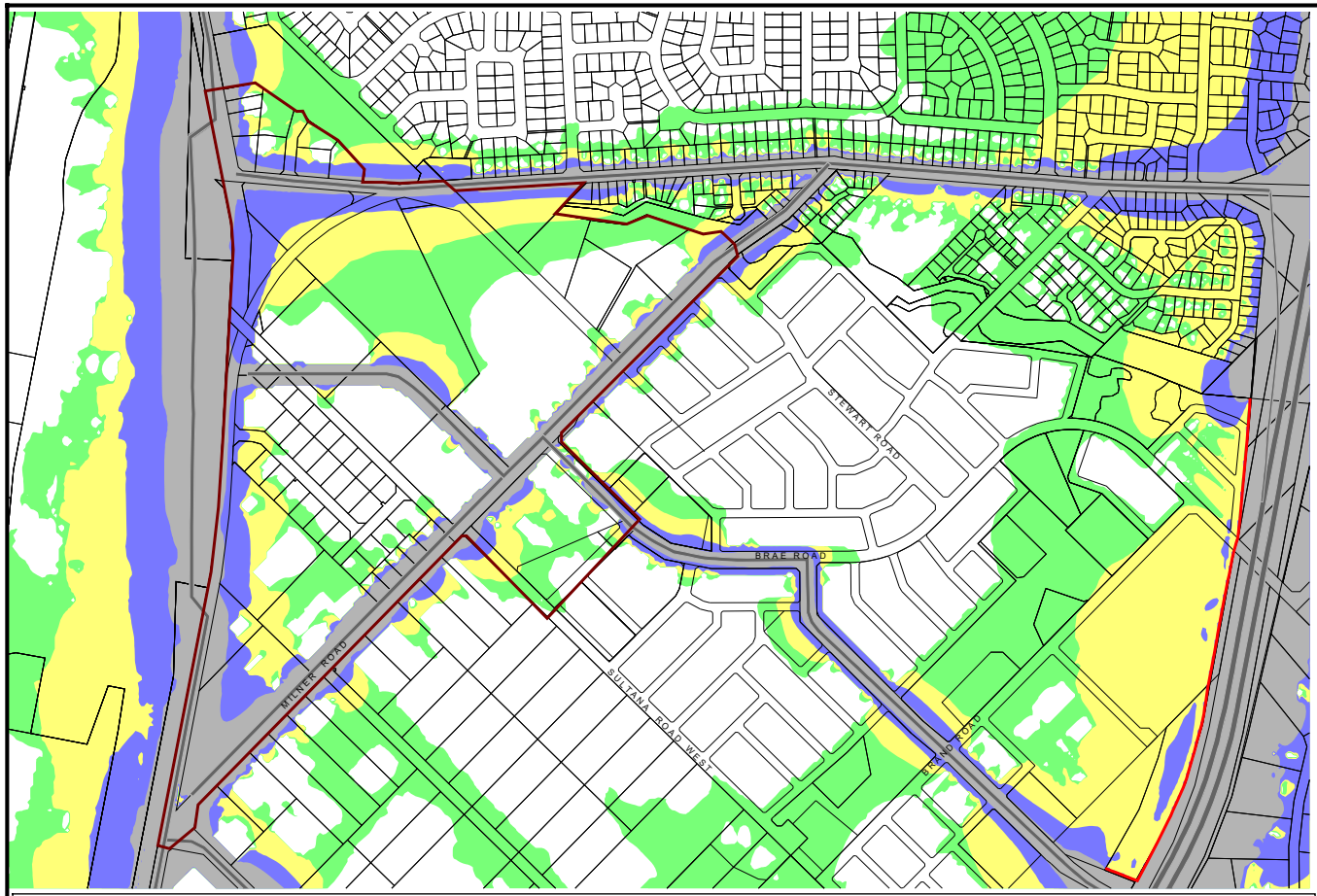


Figure 4-7

Noise levels
$L_{Aeq,Day}$ dB

<= 55	
<= 56	Exposure A
<= 57	
<= 58	
<= 59	Exposure B
<= 60	
<= 61	
<= 62	
<= 63	Exposure C
<= 64	
<= 65	
<= 66	
> 66	Exposure D

SPP 5.4 (Sep 2019)

Signs and symbols
- Road
- Study Area
- Nominal 5m High Wall

N

Length Scale 1:11000
0 50 100 200 300 400 m

Forrestfield North Local Structure Plan: TOD Precinct -
Future $L_{Aeq(Day)}$ Road Traffic Noise Level Contours

Ground Floor Level
Allows for future Built Form

SoundPlan v7.4
CoRTN Algorithms

27 March 2021

Lloyd George Acoustics
PO Box 717
HILLARYS WA 6923
(08) 9401 7770

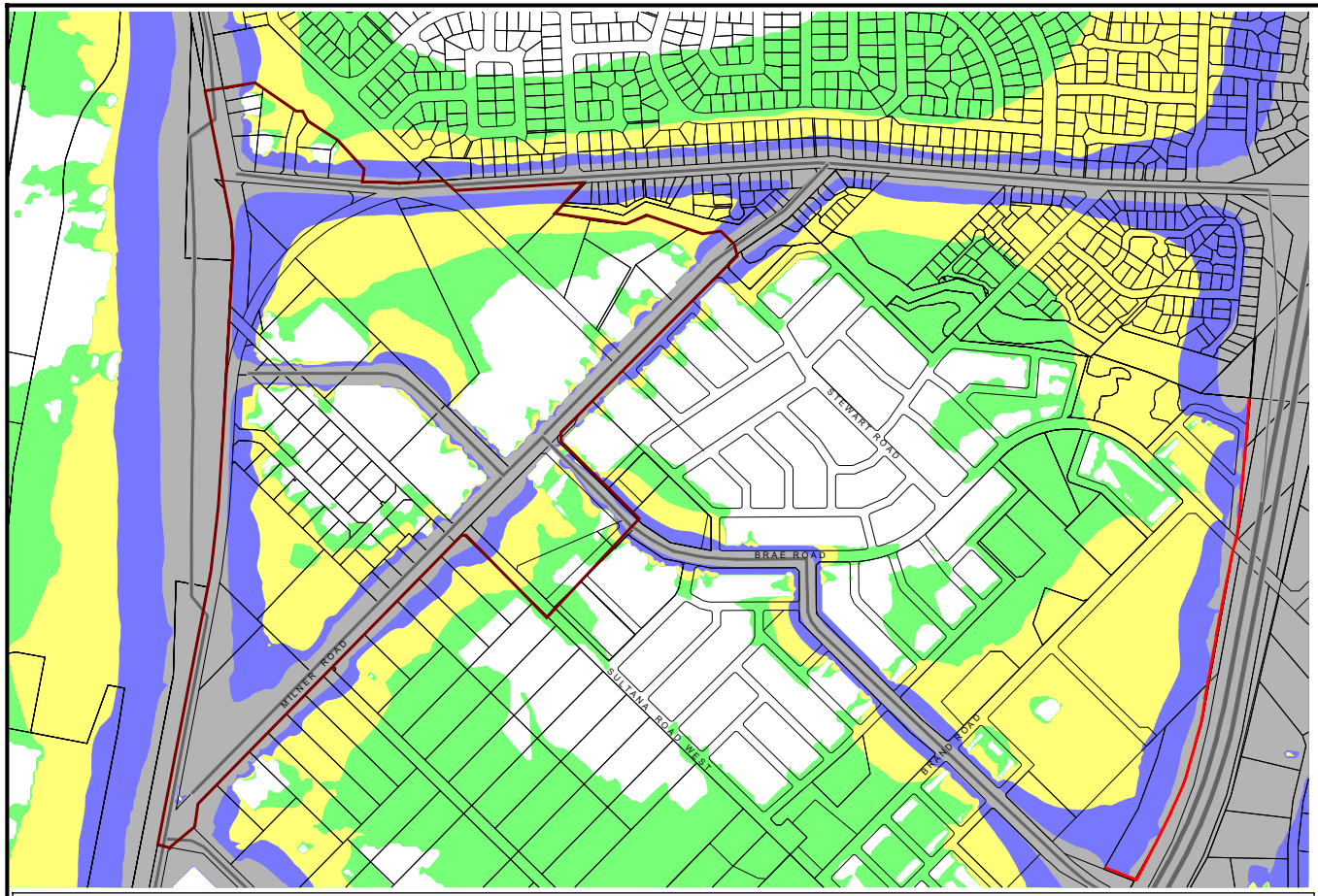


Figure 4-8

Noise levels
$L_{Aeq,Day}$ dB

<= 55	
<= 56	Exposure A
<= 57	
<= 58	
<= 59	Exposure B
<= 60	
<= 61	
<= 62	
<= 63	Exposure C
<= 64	
<= 65	
<= 66	
> 66	Exposure D

SPP 5.4 (Sep 2019)

Signs and symbols
- Road
- Study Area
- Nominal 5m High Wall

N

Length Scale 1:11000
0 50 100 200 300 400 m

Forrestfield North Local Structure Plan: TOD Precinct -
Future $L_{Aeq(Day)}$ Road Traffic Noise Level Contours

First Floor Level
Allows for future Built Form

SoundPlan v7.4
CoRTN Algorithms

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(08) 9401 7770

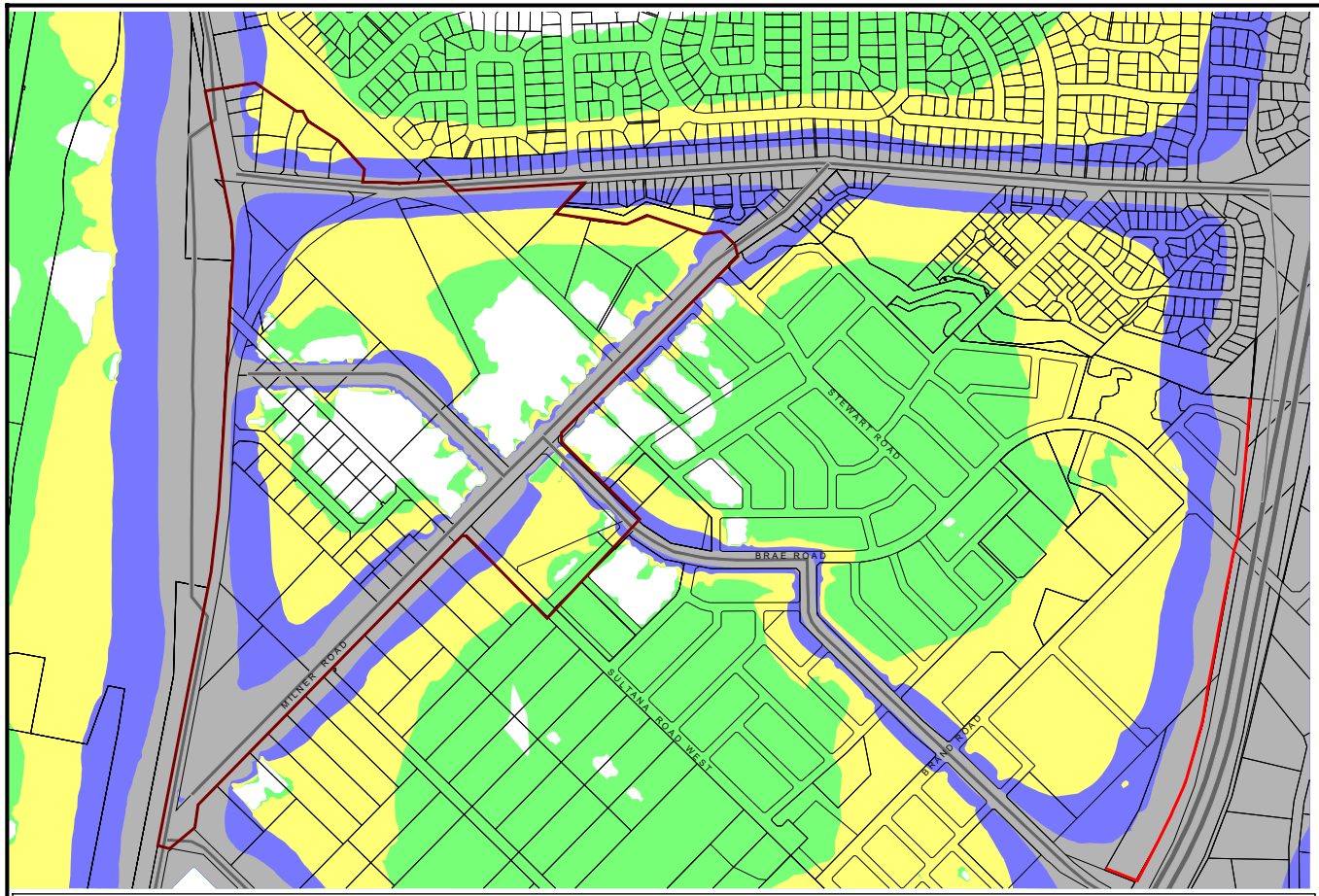

Figure 4-9

Noise levels
$L_{Aeq,Day}$ dB

<= 55	
<= 56	Exposure A
<= 57	
<= 58	
<= 59	Exposure B
<= 60	
<= 61	
<= 62	
<= 63	Exposure C
<= 64	
<= 65	
<= 66	
> 66	Exposure D

SPP 5.4 (Sep 2019)

Signs and symbols
- Road
- Study Area
- Nominal 5m High Wall

N

Length Scale 1:11000
0 50 100 200 300 400 m

Forrestfield North Local Structure Plan: TOD Precinct -
Future $L_{Aeq(Day)}$ Road Traffic Noise Level Contours

Second Floor Level and Above
Allows for future Built Form

SoundPlan v7.4
CoRTN Algorithms

27 March 2021

Lloyd George Acoustics
PO Box 717
HILLARYS WA 6923
(08) 9401 7770

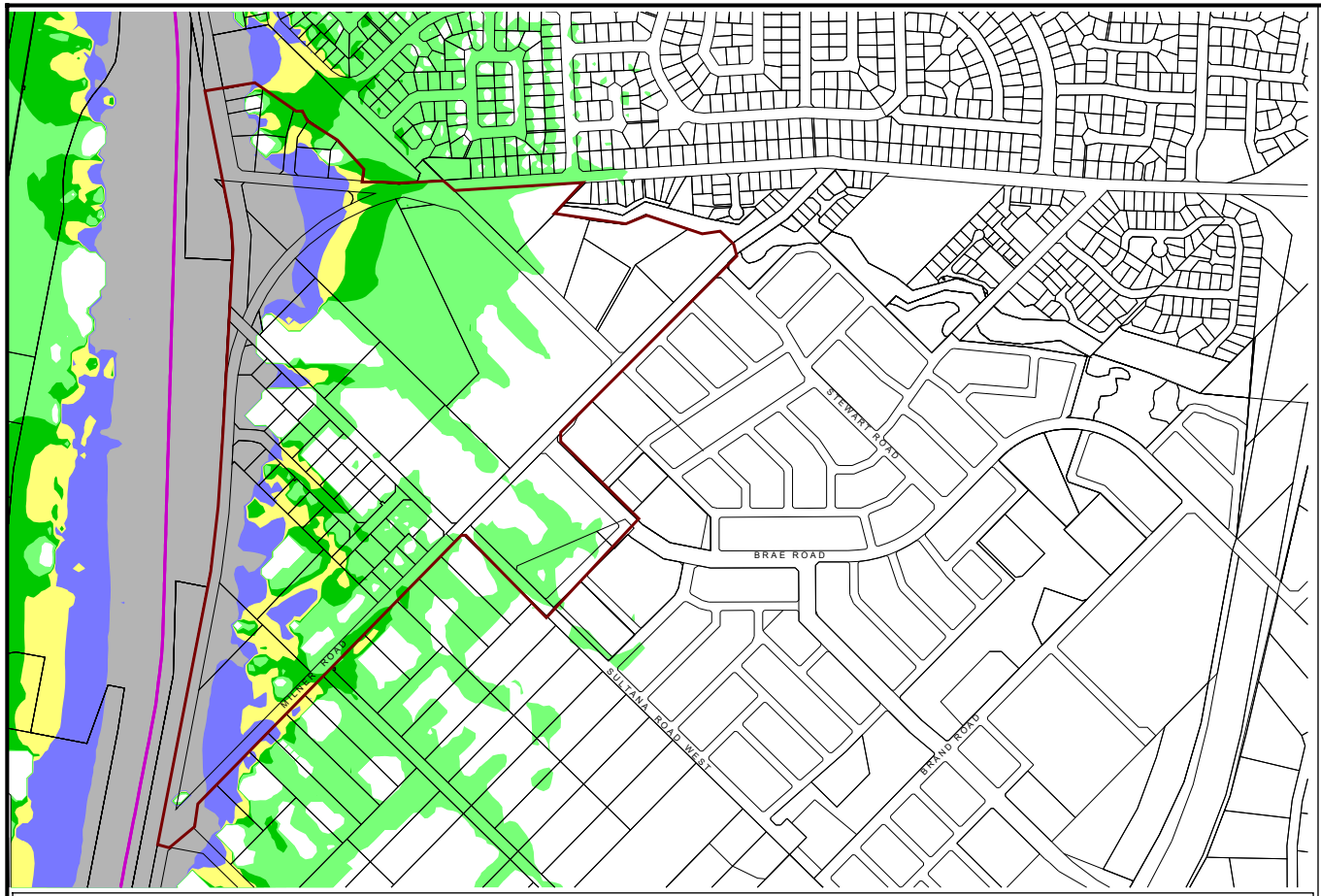

Figure 4-10

Noise levels
$L_{Aeq,Night}$ dB

<= 50	
<= 51	Exposure A
<= 52	
<= 53	
<= 54	
<= 55	
<= 56	
<= 57	Exposure A+
<= 58	
<= 59	Exposure B+
<= 60	
<= 61	Exposure C+
<= 62	
<= 63	
> 63	Exposure D

SPP 5.4 (Sep 2019)

Signs and symbols
Railway
Study Area

N

Length Scale 1:11000
0 50 100 200 300 400 m

Forrestfield North Local Structure Plan: TOD Precinct -
Future $L_{Aeq(Day)}$ Rail Traffic Noise Level Contours

Ground Floor Level
Allows for future Built Form

SoundPlan v7.4
CoRTN Algorithms

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Lloyd George Acoustics
PO Box 717
HILLARYS WA 6923
(08) 9401 7770

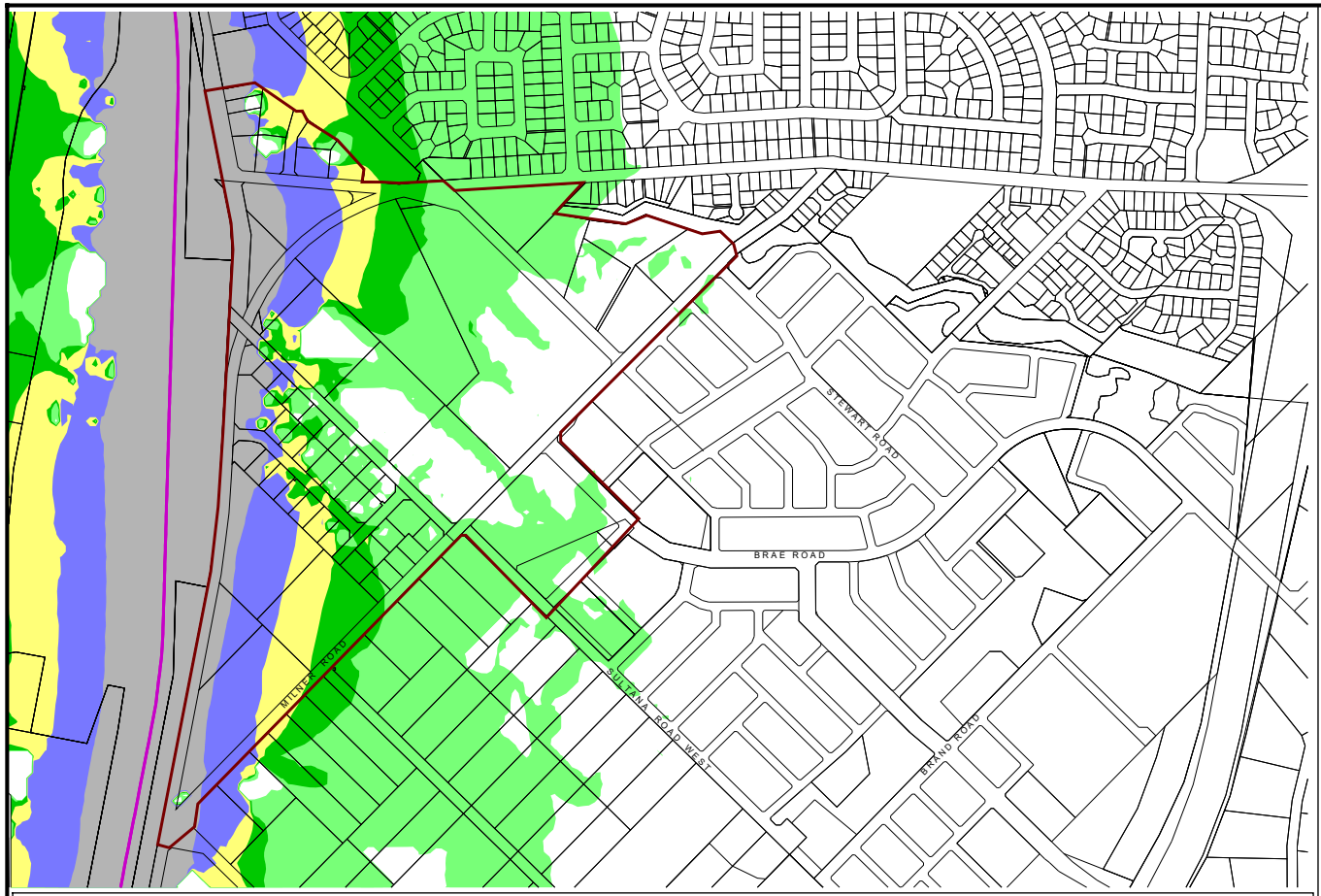

Figure 4-11

Noise levels
$L_{Aeq,Night}$ dB

<= 50	
<= 51	Exposure A
<= 52	
<= 53	
<= 54	
<= 55	
<= 56	
<= 57	Exposure A+
<= 58	
<= 59	Exposure B+
<= 60	
<= 61	Exposure C+
<= 62	
<= 63	
> 63	Exposure D

SPP 5.4 (Sep 2019)

Signs and symbols
- Railway
- Study Area

N

Length Scale 1:11000
0 50 100 200 300 400 m

Forrestfield North Local Structure Plan: TOD Precinct -
Future $L_{Aeq(Day)}$ Rail Traffic Noise Level Contours

First Floor Level
Allows for future Built Form

SoundPlan v7.4
CoRTN Algorithms

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Lloyd George Acoustics
PO Box 717
HILLARYS WA 6923
(08) 9401 7770

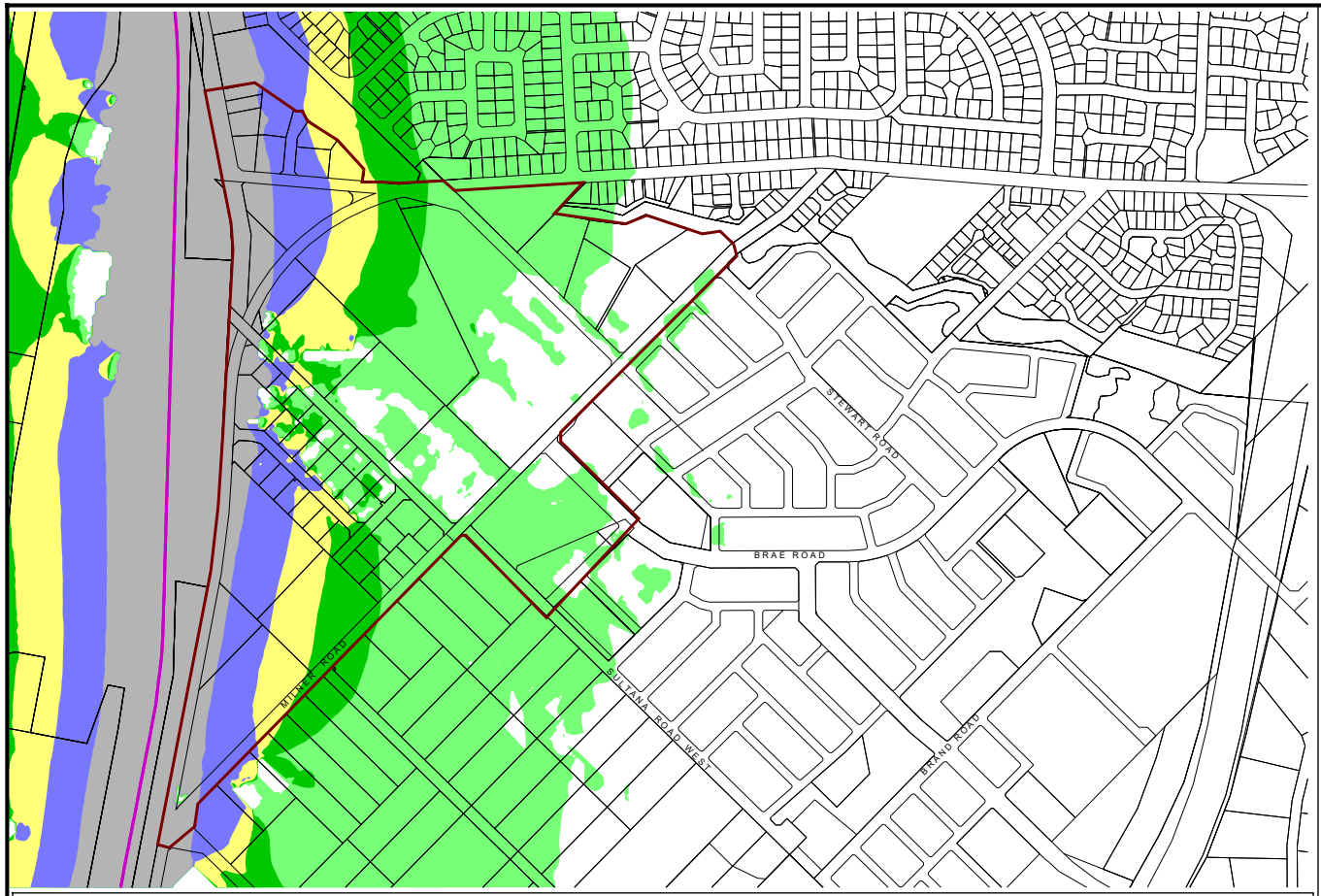


Figure 4-12

Noise levels
$L_{Aeq,Night}$ dB

<= 50	
<= 51	Exposure A
<= 52	
<= 53	
<= 54	
<= 55	
<= 56	
<= 57	Exposure A+
<= 58	
<= 59	Exposure B+
<= 60	
<= 61	Exposure C+
<= 62	
<= 63	
> 63	Exposure D

SPP 5.4 (Sep 2019)

Signs and symbols
- Railway
- Study Area

N

Length Scale 1:11000
0 50 100 200 300 400 m

Forrestfield North Local Structure Plan: TOD Precinct -
Future $L_{Aeq(Day)}$ Rail Traffic Noise Level Contours

Second Floor Level and Above
Allows for future Built Form

SoundPlan v7.4
CoRTN Algorithms

27 March 2021

Lloyd George Acoustics
PO Box 717
HILLARYS WA 6923
(08) 9401 7770

5 ASSESSMENT

5.1 Aircraft

With regards to aircraft noise, no parts of the proposed Forrestfield North structure plan result in development occurring in areas defined by AS2021 as unacceptable.

The recommendations of this report are to:

- As a minimum, all developments (residential, commercial etc) incorporate 6mm thick glass in external façades.
- Any development containing noise sensitive parts, located west of the 20 ANEF, is to have an individual assessment undertaken by a suitably qualified acoustical consultant ensuring compliance with the internal design levels specified within AS2021.
- Any noise sensitive premises shall incorporate a notification on title as follows:

  This property is situated in the vicinity of Perth Airport and is currently affected, or may be affected in the future by aircraft noise. Noise exposure levels are likely to increase in the future as a result of an increase in aircraft using the airport, changes in aircraft type or other operational changes.

The aircraft affected areas of the TOD Precinct are shown on *Figure 5-1*.

5.2 Freight Trains

With regards to vibration impacts from the freight railway, the TOD Precinct is considered outside of the affected area.

Comparing the predicted noise levels for freight trains as part of this study to those provided by SLR, the results from this study are more stringent and have therefore been adopted. The TOD Precinct will be affected by freight train noise, with parts expected to be within Exposure D.

The recommendations of this report are to:

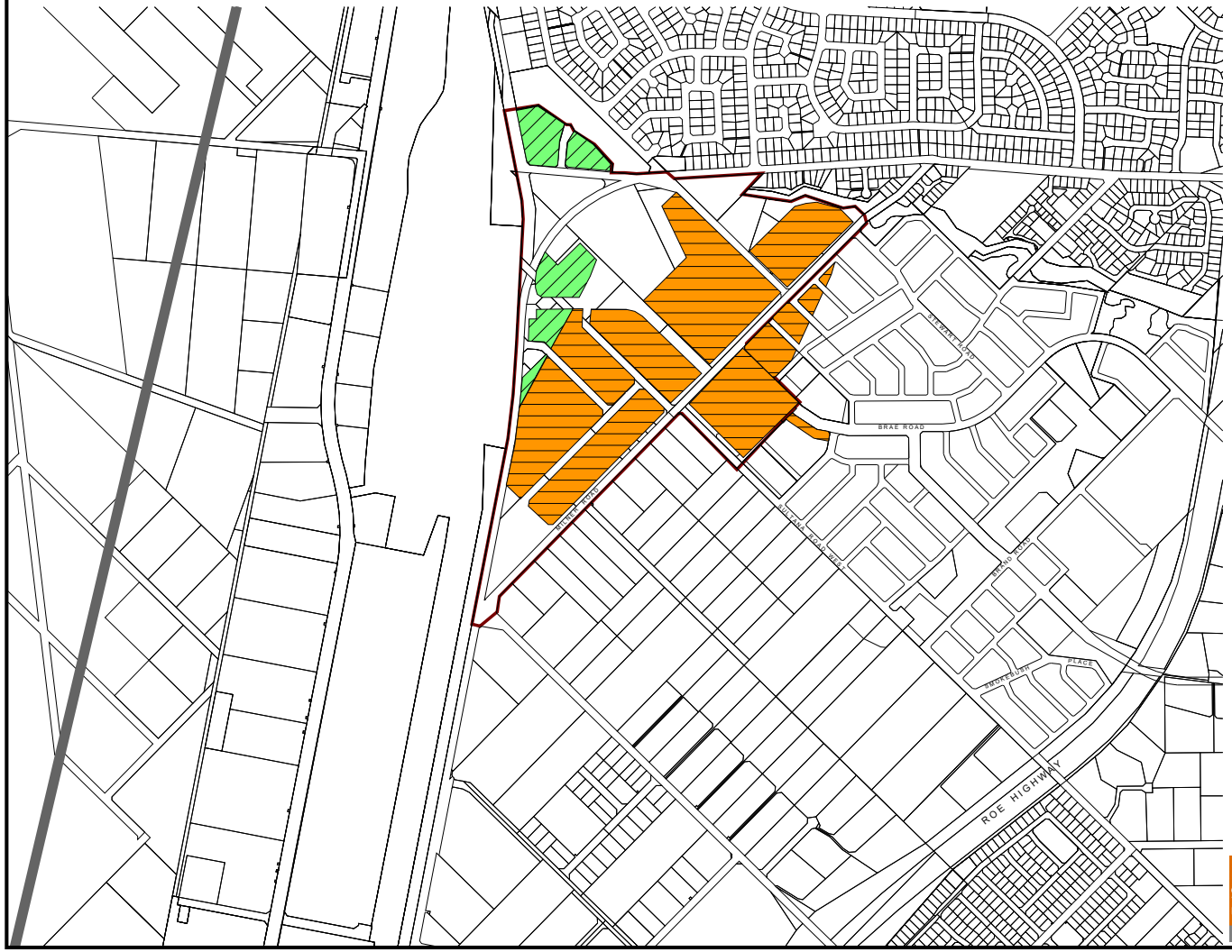
- Where a noise sensitive development is shown to be within Exposure D, these will require an individual assessment undertaken by a suitably qualified acoustical consultant ensuring compliance with SPP 5.4.
- Where a noise sensitive development is shown to be within Exposure A, A+, B+ or C+, the Quiet House Packages of SPP 5.4 Guidelines (refer Appendix A) can be adopted or alternatively, an individual assessment undertaken by a suitably qualified acoustical consultant ensuring compliance with SPP 5.4.
- Any noise sensitive premises shall incorporate a notification on title as follows:

  This lot is in the vicinity of a transport corridor and is affected, or may in the future be affected, by road and rail transport noise. Road and rail transport noise levels may rise or fall over time depending on the type and volume of traffic.

Forrestfield North Local Structure Plan: TOD Precinct

Aircraft Affected Areas

Figure 5-1



Signs and symbols

- Study Area
- Notification & 6mm glass
- Notification & Specific assessment
- Future Runway

26 March 2021

Length Scale 1:15000

0 100 200 400 600 m

Lloyd George Acoustics
by Terry George
terry@lgacoustics.com.au
(08) 9401 7770

5.3 Road Traffic

With regards to road traffic noise, SPP 5.4 sets the applicable criteria. From the DPLH PlanWA Maps, only Roe Highway falls under SPP 5.4. As such, the following is recommended:

- Where residences are located in close proximity (first row) to a road carrying reasonable volumes in 2050 (Dundas Road, Milner Road and TOD Connector), a notification on title is required. Developers of such sites may wish to obtain specialist advice from a suitably qualified acoustical consultant.

No part of the TOD precinct will be significantly affected by Roe Highway.

5.4 Other Noise Sources

In addition to the specific studies, it is further recommended that the first row of residential development incorporate notifications on title, warning of the potential for higher than normal noise levels, opposite the following locations:

- Community Purposes;
- Mixed uses.

6 CONCLUSION

This report considers various noise sources that may affect the TOD Precinct of Forrestfield North for the purposes of the structure plan. On the basis of the information available at this stage, *Figure 5-1* presents the areas affected by aircraft noise. *Figure 4-10* to *Figure 4-12* indicate the freight train noise affected areas.

It will be a requirement that as subdivision design progresses, including final layouts and finished lot levels, a more detailed assessment will be necessary to ensure compliance with the relevant Policies and criteria and to determine appropriate levels of noise mitigation (noise walls, façade packages etc.).

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Appendix A

QUIET HOUSE PACKAGES

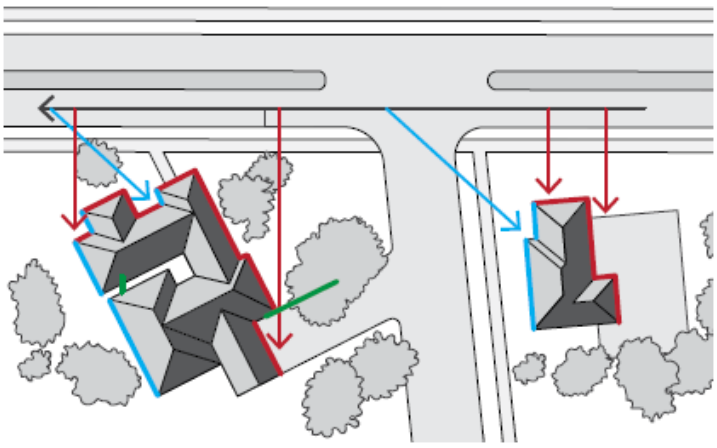
The packages and information provided on the following pages are taken from *Road and Rail Noise Guidelines* (September 2019).

Where outdoor and indoor noise levels received by a noise-sensitive land-use and/or development exceed the policy's noise target, implementation of quiet house requirements is an acceptable solution.

The quiet house packages are not the only solution to achieving acceptable internal transport noise levels. A suitably qualified acoustical engineer or consultant may also determine more tailored acoustic design requirements for buildings in a transport noise corridor by carrying out acoustic design in accordance with relevant industry standards. This includes the need to meet the relevant design targets specified in AS/NZS 2107:2016 for road traffic noise.

With regards to the packages, the following definitions are provided:

- **Facing** the transport corridor (red): Any part of a building façade is 'facing' the transport corridor if any straight line drawn perpendicular (at a 90 degree angle) to its nearest road lane or railway line intersects that part of the façade without obstruction (ignoring any fence).
- **Side-on** to transport corridor (blue): Any part of a building façade that is not 'facing' is 'side-on' to the transport corridor if any straight line, at any angle, can be drawn from it to intersect the nearest road lane or railway line without obstruction (ignoring any fence).
- **Opposite** to transport corridor (green): Neither 'side on' nor 'facing', as defined above.



Quiet House Package A

Element	Orientation	Room	
		Bedroom	Indoor Living and Work Areas
External Windows	Facing	• Up to 40% floor area ($R_w + C_{tr} \geq 28$): ○ Sliding or double hung with minimum 10mm single or 6mm-12mm-10mm double insulated glazing; ○ Sealed awning or casement windows with minimum 6mm glass. • Up to 60% floor area ($R_w + C_{tr} \geq 31$): ○ Sealed awning or casement windows with minimum 6mm glass.	• Up to 40% floor area ($R_w + C_{tr} \geq 25$): ○ Sliding or double hung with minimum 6mm single or 6mm-12mm-6mm double insulated glazing; • Up to 60% floor area ($R_w + C_{tr} \geq 28$); • Up to 80% floor area ($R_w + C_{tr} \geq 31$).
	Side On	As above, except $R_w + C_{tr}$ values may be 3 dB less or max % area increased by 20%.	
	Opposite	No specific requirements	
External Doors	Facing	• Fully glazed hinged door with certified $R_w + C_{tr} \geq 28$ rated door and frame including seals and 6mm glass.	• Doors to achieve $R_w + C_{tr} \geq 25$: ○ 35mm Solid timber core hinged door and frame system certified to R_w 28 including seals; ○ Glazed sliding door with 10mm glass and weather seals.
	Side On	As above, except $R_w + C_{tr}$ values may be 3 dB less.	
	Opposite	No specific requirements	
External Walls	All	• $R_w + C_{tr} \geq 45$: ○ Two leaves of 90mm thick clay brick masonry with minimum 20mm cavity; or ○ Single leaf of 150mm brick masonry with 13mm cement render on each face; or ○ One row of 92mm studs at 600mm centres with: ▪ Resilient steel channels fixed to the outside of the studs; and ▪ 9.5mm hardboard or fibre cement sheeting or 11mm fibre cement weatherboards fixed to the outside; ▪ 75mm thick mineral wool insulation with a density of at least 11kgkg/m^3; and ▪ 2 x 16mm fire-rated plasterboard to inside.	
Roofs and Ceilings	All	• $R_w + C_{tr} \geq 35$: ○ Concrete or terracotta tile or metal sheet roof with sarking and at least 10mm plasterboard.	
Outdoor Living Areas		At least one outdoor living area located on the opposite side of the building from the transport corridor and/or at least one ground level outdoor living area screened using a solid continuous fence or other structure of minimum 2 metres height above ground level.	

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Quiet House Package A+

Element	Orientation	Room	
		Bedroom	Indoor Living and Work Areas
External Windows	Facing	• Up to 40% floor area ($R_w + C_{tr} \geq 28$): ○ Sliding or double hung with minimum 10mm single or 6mm-12mm-10mm double insulated glazing; ○ Sealed awning or casement windows with minimum 6mm glass. • Up to 60% floor area ($R_w + C_{tr} \geq 31$): ○ Sealed awning or casement windows with minimum 6mm glass.	• Up to 40% floor area ($R_w + C_{tr} \geq 25$): ○ Sliding or double hung with minimum 6mm single or 6mm-12mm-6mm double insulated glazing; • Up to 60% floor area ($R_w + C_{tr} \geq 28$); • Up to 80% floor area ($R_w + C_{tr} \geq 31$).
	Side On	As above	
	Opposite	No specific requirements	
External Doors	Facing	• Fully glazed hinged door with certified $R_w + C_{tr} \geq 28$ rated door and frame including seals and 6mm glass.	• Doors to achieve $R_w + C_{tr} \geq 25$: ○ 35mm Solid timber core hinged door and frame system certified to R_w 28 including seals; ○ Glazed sliding door with 10mm glass and weather seals.
	Side On	As above, except $R_w + C_{tr}$ values may be 3 dB less.	
	Opposite	No specific requirements	
External Walls	All	• $R_w + C_{tr} \geq 45$: ○ Two leaves of 90mm thick clay brick masonry with minimum 20mm cavity; or ○ Single leaf of 150mm brick masonry with 13mm cement render on each face.	
Roofs and Ceilings	All	• $R_w + C_{tr} \geq 35$: ○ Concrete or terracotta tile or metal sheet roof with sarking and at least 10mm plasterboard.	
Outdoor Living Areas		At least one outdoor living area located on the opposite side of the building from the transport corridor and/or at least one ground level outdoor living area screened using a solid continuous fence or other structure of minimum 2 metres height above ground level.	

Quiet House Package B

Element	Orientation	Room	
		Bedroom	Indoor Living and Work Areas
External Windows	Facing	• Up to 40% floor area ($R_w + C_{tr} \geq 31$): ○ Fixed sash, awning or casement with minimum 6mm glass or 6mm-12mm-6mm double insulated glazing. • Up to 60% floor area ($R_w + C_{tr} \geq 34$): ○ Fixed sash, awning or casement with minimum 10mm glass or 6mm-12mm-10mm double insulated glazing.	• Up to 40% floor area ($R_w + C_{tr} \geq 28$): ○ Sliding or double hung with 6mm-12mm-10mm double insulated glazing; ○ Sealed awning or casement windows with minimum 6mm glass. • Up to 60% floor area ($R_w + C_{tr} \geq 31$); • Up to 80% floor area ($R_w + C_{tr} \geq 34$).
	Side On	As above, except $R_w + C_{tr}$ values may be 3 dB less or max % area increased by 20%.	
	Opposite	As above, except $R_w + C_{tr}$ values may be 6 dB less or max % area increased by 20%.	
External Doors	Facing	• Fully glazed hinged door with certified $R_w + C_{tr} \geq 31$ rated door and frame including seals and 10mm glass.	• Doors to achieve $R_w + C_{tr} \geq 28$: ○ 40mm Solid timber core hinged door and frame system certified to R_w 32 including seals; ○ Fully glazed hinged door with certified $R_w + C_{tr} \geq 28$ rated door and frame including seals and 6mm glass.
	Side On	As above, except $R_w + C_{tr}$ values may be 3 dB less or max % area increased by 20%.	
	Opposite	As above, except $R_w + C_{tr}$ values may be 6 dB less or max % area increased by 20%.	
External Walls	All	• $R_w + C_{tr} \geq 50$: ○ Two leaves of 90mm thick clay brick masonry with minimum 50mm cavity between leaves and 50mm glasswool or polyester insulation (R2.0+). Resilient ties used where required to connect leaves. ○ Two leaves of 110mm clay brick masonry with minimum 50mm cavity between leaves and 50mm glasswool or polyester insulation (R2.0+). ○ Single leaf of 220mm brick masonry with 13mm cement render on each face. ○ 150mm thick unlined concrete panel or 200mm thick concrete panel with one layer of 13mm plasterboard or 13mm cement render on each face. ○ Single leaf of 90mm clay brick masonry with: ▪ A row of 70mm x 35mm timber studs or 64mm steel studs at 600mm centres; ▪ A cavity of 25mm between leaves; ▪ 50mm glasswool or polyester insulation (R2.0+) between studs; and ▪ One layer of 10mm plasterboard fixed to the inside face.	
Roofs and Ceilings	All	• $R_w + C_{tr} \geq 35$: ○ Concrete or terracotta tile or metal sheet roof with sarking and at least 10mm plasterboard ceiling with R3.0+ fibrous insulation.	
Outdoor Living Areas		At least one outdoor living area located on the opposite side of the building from the transport corridor and/or at least one ground level outdoor living area screened using a solid continuous fence or other structure of minimum 2.4 metres height above ground level.	

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Quiet House Package B+

Element	Orientation	Room	
		Bedroom	Indoor Living and Work Areas
External Windows	Facing	• Up to 40% floor area ($R_w + C_{tr} \geq 31$): ○ Sealed awning or casement windows with minimum 6mm glass.	• Up to 40% floor area ($R_w + C_{tr} \geq 28$): ○ Sealed awning or casement windows with minimum 6mm glass.
	Side On	As above.	
	Opposite	As above, except $R_w + C_{tr}$ values may be 6 dB less or max % area increased by 20%.	
External Doors	Facing	• None permitted	• Doors to achieve $R_w + C_{tr} \geq 28$: ○ 40mm Solid timber core hinged door and frame system certified to R_w 32 including seals; ○ Fully glazed hinged door with certified $R_w + C_{tr} \geq 28$ rated door and frame including seals and 6mm glass.
	Side On	• None permitted	As above, except $R_w + C_{tr}$ values may be 3 dB less or max % area increased by 20%.
	Opposite	• Doors to achieve $R_w + C_{tr} \geq 25$: ○ 35mm Solid timber core hinged door and frame system certified to R_w 28 including seals; ○ Glazed sliding door with 10mm glass and weather seals.	As above, except $R_w + C_{tr}$ values may be 6 dB less or max % area increased by 20%.
External Walls	All	• $R_w + C_{tr} \geq 50$: ○ Two leaves of 90mm thick clay brick masonry with minimum 50mm cavity between leaves and 50mm glasswool or polyester insulation (R2.0+). Resilient ties used where required to connect leaves. ○ Two leaves of 110mm clay brick masonry with minimum 50mm cavity between leaves and 50mm glasswool or polyester insulation (R2.0+).	
Roofs and Ceilings	All	• $R_w + C_{tr} \geq 40$: ○ Concrete or terracotta tile roof with sarking, or metal sheet roof with foil backed R2.0+ fibrous insulation between steel sheeting and roof battens; ○ R3.0+ insulation batts above ceiling; ○ 2 x 10mm plasterboard ceiling or 1 x 13mm sound-rated plasterboard affixed using steel furring channel to ceiling rafters.	
Outdoor Living Areas		At least one outdoor living area located on the opposite side of the building from the transport corridor and/or at least one ground level outdoor living area screened using a solid continuous fence or other structure of minimum 2.4 metres height above ground level.	
Air-conditioning		Evaporative air-conditioning not recommended.	

Quiet House Package C

Element	Orientation	Room	
		Bedroom	Indoor Living and Work Areas
External Windows	Facing	• Up to 20% floor area ($R_w + C_{tr} \geq 31$): ○ Fixed sash, awning or casement with minimum 6mm glass or 6mm-12mm-6mm double insulated glazing. • Up to 40% floor area ($R_w + C_{tr} \geq 34$): ○ Fixed sash, awning or casement with minimum 10mm glass or 6mm-12mm-10mm double insulated glazing.	• Up to 40% floor area ($R_w + C_{tr} \geq 31$): ○ Fixed sash, awning or casement with minimum 6mm glass or 6mm-12mm-6mm double insulated glazing. • Up to 60% floor area ($R_w + C_{tr} \geq 34$): ○ Fixed sash, awning or casement with minimum 10mm glass or 6mm-12mm-10mm double insulated glazing.
	Side On	As above, except $R_w + C_{tr}$ values may be 3 dB less or max % area increased by 20%.	
	Opposite	As above, except $R_w + C_{tr}$ values may be 6 dB less or max % area increased by 20%.	
External Doors	Facing	• Not recommended.	• Doors to achieve $R_w + C_{tr} \geq 30$: ○ Fully glazed hinged door with certified $R_w + C_{tr} \geq 31$ rated door and frame including seals and 10mm glass; ○ 40mm Solid timber core side hinged door, frame and seal system certified to R_w 32 including seals. Any glass inserts to be minimum 6mm.
	Side On	As above, except $R_w + C_{tr}$ values may be 3 dB less or max % area increased by 20%.	
	Opposite	As above, except $R_w + C_{tr}$ values may be 6 dB less or max % area increased by 20%.	
External Walls	All	• $R_w + C_{tr} \geq 50$: ○ Two leaves of 90mm thick clay brick masonry with minimum 50mm cavity between leaves and 50mm glasswool or polyester insulation (R2.0+). Resilient ties used where required to connect leaves. ○ Two leaves of 110mm clay brick masonry with minimum 50mm cavity between leaves and 50mm glasswool or polyester insulation (R2.0+). ○ Single leaf of 220mm brick masonry with 13mm cement render on each face. ○ 150mm thick unlined concrete panel or 200mm thick concrete panel with one layer of 13mm plasterboard or 13mm cement render on each face. ○ Single leaf of 90mm clay brick masonry with: ▪ A row of 70mm x 35mm timber studs or 64mm steel studs at 600mm centres; ▪ A cavity of 25mm between leaves; ▪ 50mm glasswool or polyester insulation (R2.0+) between studs; and ▪ One layer of 10mm plasterboard fixed to the inside face.	
Roofs and Ceilings	All	• $R_w + C_{tr} \geq 40$: ○ Concrete or terracotta tile roof with sarking, or metal sheet roof with foil backed R2.0+ fibrous insulation between steel sheeting and roof battens; ○ R3.0+ insulation batts above ceiling; ○ 2 x 10mm plasterboard ceiling or 1 x 13mm sound-rated plasterboard affixed using steel furring channel to ceiling rafters.	
Outdoor Living Areas		At least one outdoor living area located on the opposite side of the building from the transport corridor and/or at least one ground level outdoor living area screened using a solid continuous fence or other structure of minimum 2.4 metres height above ground level.	

Lloyd George Acoustics

Quiet House Package C+

Element	Orientation	Room	
		Bedroom	Indoor Living and Work Areas
External Windows	Facing	• Up to 20% floor area ($R_w + C_{tr} \geq 31$): ○ Fixed sash, awning or casement with minimum 6mm glass	• Up to 40% floor area ($R_w + C_{tr} \geq 31$): ○ Fixed sash, awning or casement with minimum 6mm glass.
	Side On	As above.	
	Opposite	As above, except $R_w + C_{tr}$ values may be 6 dB less or max % area increased by 20%.	
External Doors	Facing	• None permitted	• Doors to achieve $R_w + C_{tr} \geq 30$: ○ Fully glazed hinged door with certified $R_w + C_{tr} \geq 31$ rated door and frame including seals and 10mm glass; ○ 40mm Solid timber core side hinged door, frame and seal system certified to R_w 32 including seals. Any glass inserts to be minimum 6mm.
	Side On	• None permitted	As above, except $R_w + C_{tr}$ values may be 3 dB less or max % area increased by 20%.
	Opposite	• Not recommended.	As above, except $R_w + C_{tr}$ values may be 6 dB less or max % area increased by 20%.
External Walls	All	• $R_w + C_{tr} \geq 50$: ○ Two leaves of 90mm thick clay brick masonry with minimum 50mm cavity between leaves and 50mm glasswool or polyester insulation (R2.0+). Resilient ties used where required to connect leaves. ○ Two leaves of 110mm clay brick masonry with minimum 50mm cavity between leaves and 50mm glasswool or polyester insulation (R2.0+).	
Roofs and Ceilings	All	• $R_w + C_{tr} \geq 40$: ○ Concrete or terracotta tile roof with sarking; ○ R3.0+ insulation batts above ceiling; ○ 2 x 10mm plasterboard ceiling or 1 x 13mm sound-rated plasterboard affixed using steel furring channel to ceiling rafters.	
Outdoor Living Areas		At least one outdoor living area located on the opposite side of the building from the transport corridor and/or at least one ground level outdoor living area screened using a solid continuous fence or other structure of minimum 2.4 metres height above ground level.	
Air-conditioning		Evaporative air-conditioning not recommended.	

Mechanical Ventilation requirements

In implementing the acceptable treatment packages, the following mechanical ventilation / air-conditioning considerations are required:

- Acoustically rated openings and ductwork to provide a minimum sound reduction performance of R_w 40 dB into sensitive spaces;
- Evaporative systems require attenuated ceiling air vents to allow closed windows;
- Refrigerant based systems need to be designed to achieve National Construction Code fresh air ventilation requirements;
- Openings such as eaves, vents and air inlets must be acoustically treated, closed or relocated to building sides facing away from the corridor where practicable.

Notification

Notifications on title advise prospective purchasers of the potential for noise impacts from major transport corridors and help with managing expectations.

The Notification is to state as follows:

This lot is in the vicinity of a transport corridor and is affected, or may in the future be affected, by road and rail transport noise. Road and rail transport noise levels may rise or fall over time depending on the type and volume of traffic.

Appendix B

TERMINOLOGY

The following is an explanation of the terminology used throughout this report.

Decibel (dB)
The decibel is the unit that describes the sound pressure and sound power levels of a noise source. It is a logarithmic scale referenced to the threshold of hearing.

A-Weighting
An A-weighted noise level has been filtered in such a way as to represent the way in which the human ear perceives sound. This weighting reflects the fact that the human ear is not as sensitive to lower frequencies as it is to higher frequencies. An A-weighted sound level is described as L_A dB.

L_1
An L_1 level is the noise level which is exceeded for 1 per cent of the measurement period and is considered to represent the average of the maximum noise levels measured.

L_{10}
An L_{10} level is the noise level which is exceeded for 10 per cent of the measurement period and is considered to represent the "*intrusive*" noise level.

L_{90}
An L_{90} level is the noise level which is exceeded for 90 per cent of the measurement period and is considered to represent the "*background*" noise level.

L_{eq}
The L_{eq} level represents the average noise energy during a measurement period.

$L_{A10,18hour}$
The $L_{A10,18\ hour}$ level is the arithmetic average of the hourly L_{A10} levels between 6.00 am and midnight. The *CoRTN* algorithms were developed to calculate this parameter.

$L_{Aeq,24hour}$
The $L_{Aeq,24\ hour}$ level is the logarithmic average of the hourly L_{Aeq} levels for a full day (from midnight to midnight).

$L_{Aeq,8hour}$ / $L_{Aeq\ (Night)}$
The $L_{Aeq\ (Night)}$ level is the logarithmic average of the hourly L_{Aeq} levels from 10.00 pm to 6.00 am on the same day.

$L_{Aeq,16hour}$ / $L_{Aeq\ (Day)}$
The $L_{Aeq\ (Day)}$ level is the logarithmic average of the hourly L_{Aeq} levels from 6.00 am to 10.00 pm on the same day. This value is typically 1-3 dB less than the $L_{A10,18hour}$.

R_w

This is the weighted sound reduction index and is similar to the previously used STC (Sound Transmission Class) value. It is a single number rating determined by moving a grading curve in integral steps against the laboratory measured transmission loss until the sum of the deficiencies at each one-third-octave band, between 100 Hz and 3.15 kHz, does not exceed 32 dB. The higher the R_w value, the better the acoustic performance.

C_{tr}

This is a spectrum adaptation term for airborne noise and provides a correction to the R_w value to suit source sounds with significant low frequency content such as road traffic or home theatre systems. A wall that provides a relatively high level of low frequency attenuation (i.e. masonry) may have a value in the order of –4 dB, whilst a wall with relatively poor attenuation at low frequencies (i.e. stud wall) may have a value in the order of -14 dB.

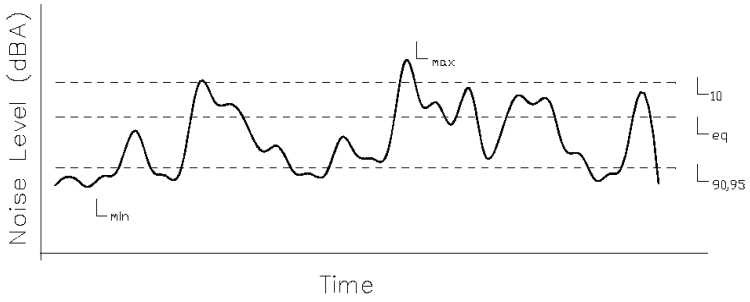
Satisfactory Design Sound Level

The level of noise that has been found to be acceptable by most people for the environment in question and also to be not intrusive.

Maximum Design Sound Level

The level of noise above which most people occupying the space start to become dissatisfied with the level of noise.

Chart of Noise Level Descriptors



Austroads Vehicle Class

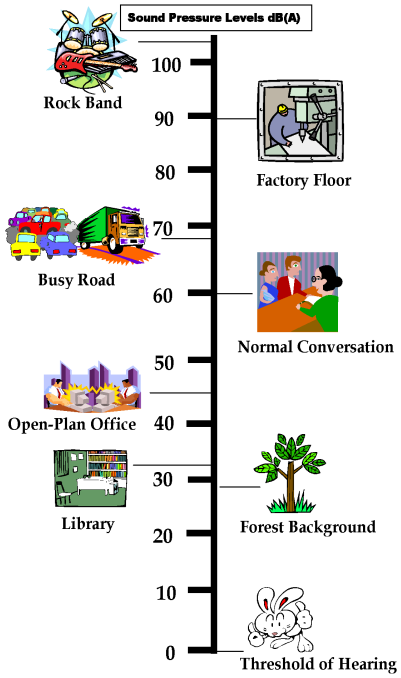
AUSTROADS Vehicle Classification System

Level 1 Length (indicative) Type	Level 2 Axles and Axle Groups Axles	Groups	Level 3 Vehicle Type Typical Description	AUSTROADS Classification Class	Parameters	Typical Configuration
			LIGHT VEHICLES			
Short up to 5.5m		1 or 2	**Short** Sedan, Wagon, 4WD, Utility, Light Van, Bicycle, Motorcycle, etc	1	d(1) ≤ 3.2m and axles = 2	
Medium 5.5m to 14.5m	3, 4 or 5	3	**Short - Towing** Trailer, Caravan, Boat, etc	2	groups = 3 d(1) > 2.1m, d(1) < 3.2m, d(2) ≥ 2.1m and axles = 3, 4 or 5	
			HEAVY VEHICLES			
	2	2	**Two Axle Truck or Bus**	3	d(1) > 3.2m and axles = 2	
	3	2	**Three Axle Truck or Bus**	4	axles = 3 and groups = 2	
	> 3	2	**Four Axle Truck**	5	axles > 3 and groups = 2	
Long 11.5m to 19.0m	3	3	**Three Axle Articulated** Three axle articulated vehicle, or Rigid vehicle and trailer	6	d(1) > 3.2m, axles = 3 and groups = 3	
	4	> 2	**Four Axle Articulated** Four axle articulated vehicle, or Rigid vehicle and trailer	7	d(2) < 2.1m or d(1) < 2.1m or d(1) > 3.2m axles = 4 and groups > 2	
	5	> 2	**Five Axle Articulated** Five axle articulated vehicle, or Rigid vehicle and trailer	8	d(2) < 2.1m or d(1) < 2.1m or d(1) > 3.2m axles = 5 and groups > 2	
	≥ 6	> 2	**Six Axle Articulated** Six axle articulated vehicle, or Rigid vehicle and trailer	9	axles = 6 and groups > 2 or axles = 6 and groups = 3	
Medium Combination 17.5m to 30.5m	> 6	4	**B Double** B Double, or Heavy truck and trailer	10	groups = 4 and axles > 6	
	> 6	5 or 6	**Double Road Train** Double road train, or Medium articulated vehicle and one dog trailer (M.A.D.)	11	groups = 5 or 6 and axles > 6	
Large Combination Over 33.0m	> 6	> 6	**Triple Road Train** Triple road train, or Heavy truck and three trailers	12	groups > 6 and axles > 6	

Definitions: Group: Axle group, where adjacent axles are less than 2.1m apart
Groups: Number of axle groups
Axles: Number of axles (maximum axle spacing of 10.0m)

d(1) Distance between first and second axle
d(2) Distance between second and third axle

Typical Noise Levels



element.

Technical Appendix D – Urbaqua Local Water Management Strategy

Transit Oriented Development, High Wycombe

Local Water Management Strategy

Prepared for Development WA

By Urbaqua

May 2021

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Disclaimer and Limitation

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1 INTRODUCTION

Urbaqua has been commissioned by Development WA to prepare this Local Water Management Strategy (LWMS) in support of the Local Structure Plan for the Transit Orientated Development precinct (herein referred to as Study Area). The Study Area is located approximately 12 km east of the Perth CBD. The site is 57 hectares in size and is bound by:

- The future above-ground High Wycombe train station to the west;
- Proposed residential precinct to the east (subject to a separate structure plan);
- Poison Gully Creek to the north; and
- Light industry to the south.

The development is aimed at providing an activity centre around the train station, including higher density residential and commercial areas owing to the improved transport connections to the Perth CBD. The LWMS is prepared to support the Local Structure Plan that is being prepared by Development WA in conjunction with the City of Kalamunda. The document will guide sustainable water management during redevelopment.

1.1 Principles and design objectives

Consistent with *State Planning Policy 2.9: Water Resources* (WAPC, 2006) and *Better urban water management* (WAPC, 2008) a local water management strategy (LWMS) is required to be submitted to support any rezoning of land in a Local Planning Scheme or adoption of a Local Structure Plan to ensure that appropriate water management strategies are identified. The position of this strategy within the state government planning framework is defined in *Better urban water management* (WAPC, 2008) and is outlined in Figure 1.

The LWMS should be prepared in accordance with the Department of Water and Environmental Regulation's (DWER) *Interim: Developing a local water management strategy* (DoW, 2008a) and should demonstrate to the satisfaction of the WAPC in accordance with any approved DWMS:

- How the key principles and strategies of this plan have been addressed;
- How the urban structure will address water use and management;
- Existing and required water management infrastructure; and,
- Detailed land requirements for water management.

The principles and strategies contained within Section 5 of this LWMS will be implemented as part of detailed land use planning and development requirements and are consistent with the framework and requirements in *Better urban water management* (WAPC, 2008) as demonstrated in Appendix 1.

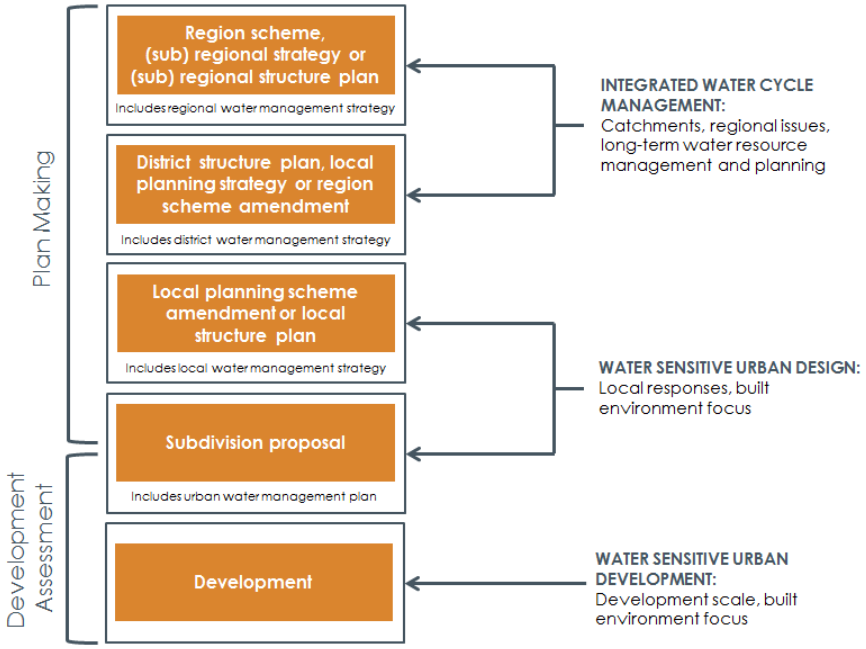


Figure 1: Integrating water planning with land planning processes (WAPC, 2008)

1.2 Planning background

The site is currently zoned as 'Urban' under the Metropolitan Regional Scheme and as 'Light Industry' (south), 'Industrial Development' (north) and 'Rural Composite' (east) under the City of Kalamunda's Local Planning Scheme No. 3.

A District Structure Plan (DSP) was prepared by the City of Kalamunda to facilitate rezoning a portion of the land from 'Rural' to the current 'Urban' zoning. The DSP addressed the Study Area and adjacent residential precinct to the east and was supported by the *Forrestfield North District Water Management Strategy* (Strategen, 2015) and subsequently the *Forrestfield North Residential Precinct LWMS* (Strategen, 2020).

1.3 Guiding documents

A number of guiding documents have been considered that are relevant to this strategy area. In addition to State Planning Policy 2.9, these documents inform the strategies and management principles contained within this Local Water Management Strategy:

- Developing a Local Water Management Strategy (DoW, 2008a);
- Stormwater Management Manual for Western Australia (DoW, 2004-09);
- Better Urban Water Management (WAPC, 2008); and,
- Decision Process for Stormwater Management in Western Australia (DWER, 2017).

2 PROPOSED DEVELOPMENT

Development of the Study Area to create an urban precinct provides opportunities to incorporate water sensitive urban design. This section outlines the key elements of the current land use and proposed re-development that influence water management.

2.1 Previous land use

The Study Area is predominantly composed of light industrial and industrial uses, with areas of remnant vegetation and rural houses. The site was largely cleared by the 1960s, with the major road layout established in the 1970s, along with the Forrestfield Marshalling Yard. Until the late 2000s, the Study Area was generally used for rural and agricultural purposes which have since been replaced by light industrial and laydown areas.

Throughout the previous development in the Study Area, pockets of vegetation have been retained, particularly along Poison Creek Gully and the wetland south of Maida Vale Road (Figure 2). The latter is mapped as a Resource Enhancement Wetland (REW) (UFI:15875) and has Threatened Ecological Communities (TEC). The hydrological function of this wetland is discussed further in Section 4.5.3, and protection is required through the proposed water management strategies.

2.2 Development

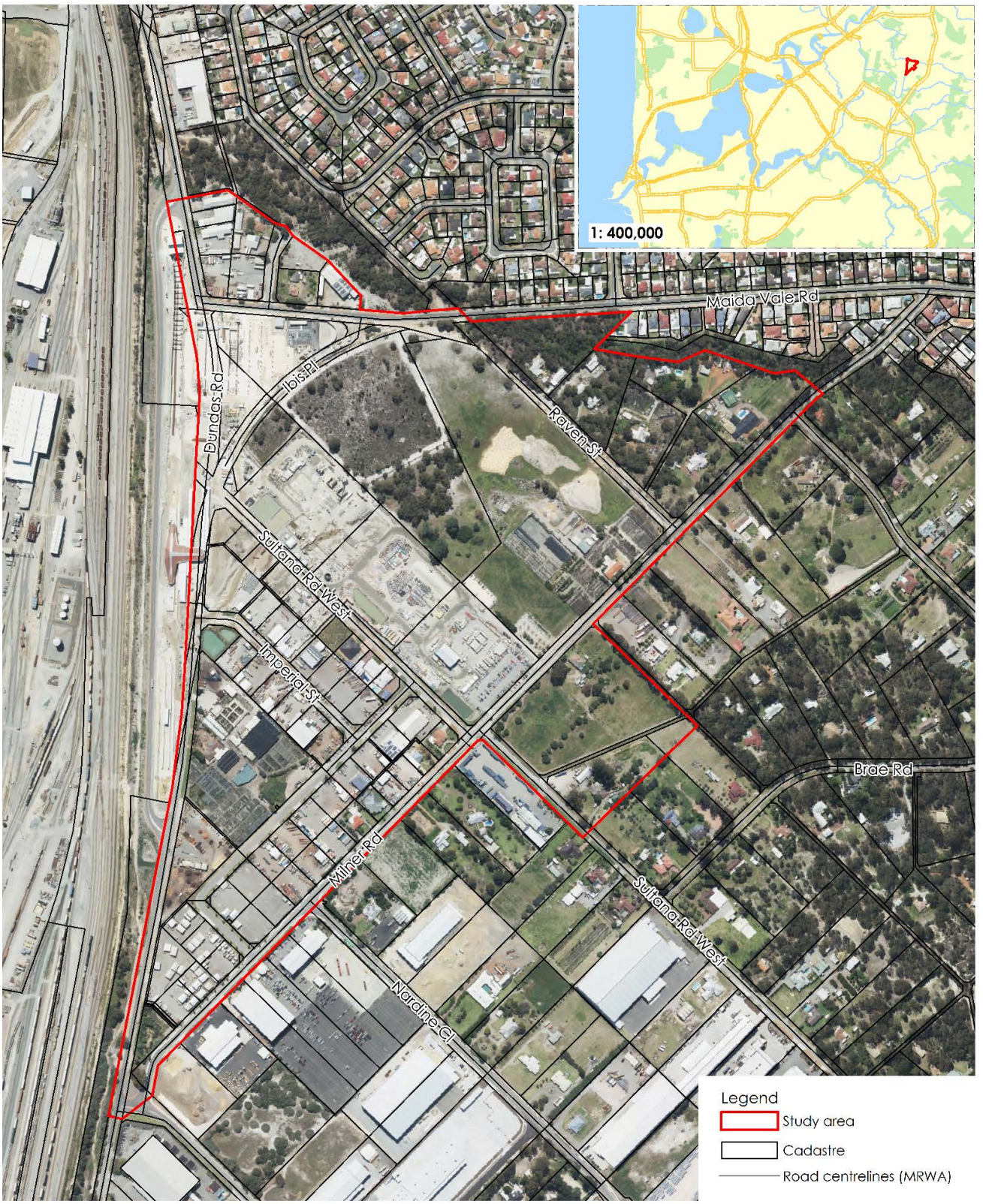
The local structure plan will guide the future urban development of the Study Area. Proposed development will feature:

- Retention and protection of the wetland and Poison Gully Creek foreshore;
- Revised major road layouts;
- Higher density residential development (in place of rural uses);
- A community hub adjacent to the railway precinct; and,
- Increased areas of public open space.

The concept design is presented in Figure 3 and included in Appendix 2.

Development will increase impervious areas and increase stormwater runoff. There is, however, an opportunity to integrate water sensitive urban design into the development and provide improved water quality and sustainability outcomes.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 2 - Study area



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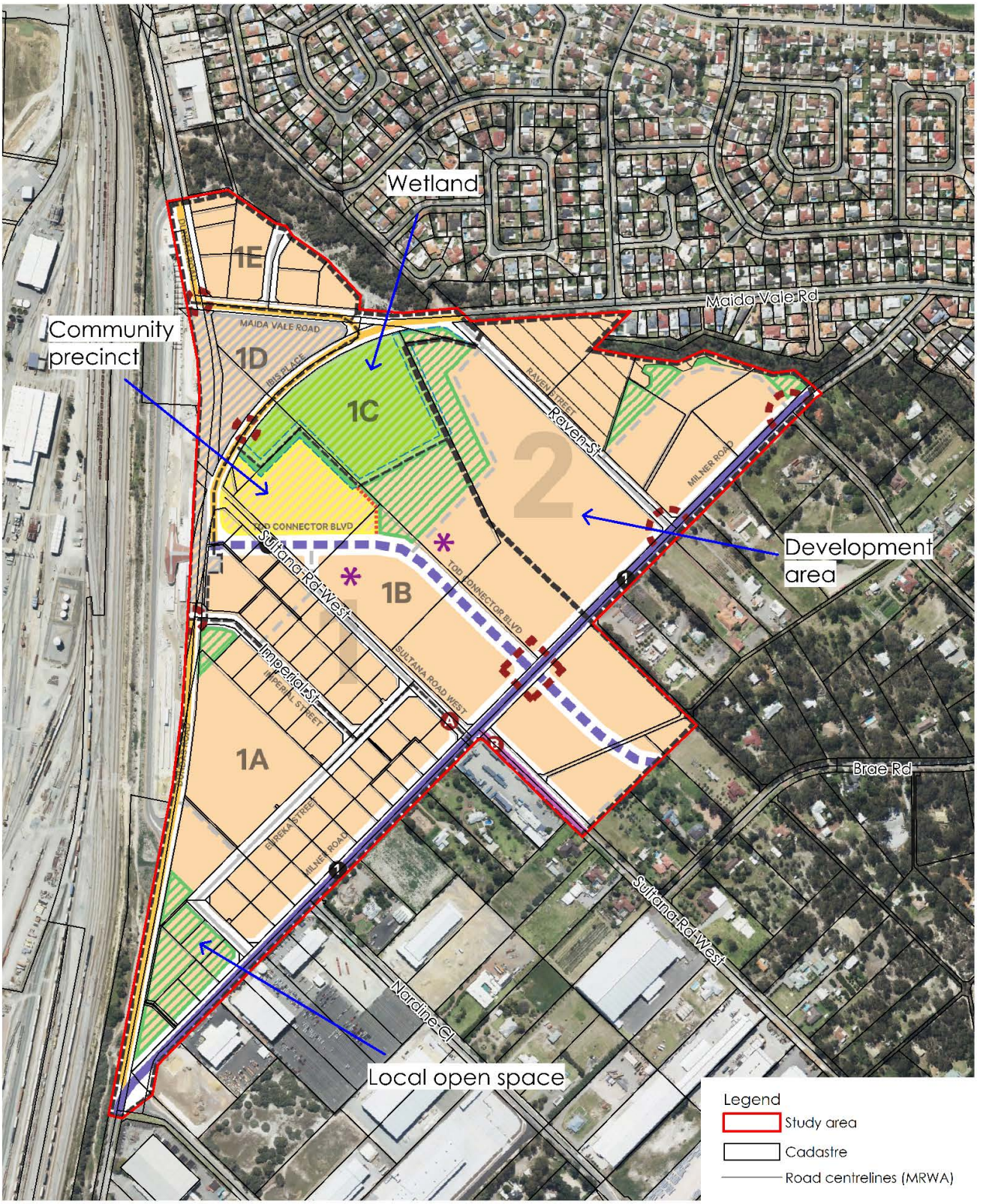
DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 3 - Concept plan



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3 DESIGN OBJECTIVES

Table 1 outlines the key design objectives to be achieved within the Study Area.

Table 1: Design objectives

Design Element	Objectives
Water sustainability	• Maximise local infiltration to replenish surface groundwater aquifers; and, • Provide alternative water sources for domestic irrigation and in-house domestic non-drinking water demands.
Protection of important environments	• Protect, and enhance where possible, Bush forever sites (Poison Gully and TEC wetland) from the potentially harmful effects of development.
Surface water management	• Maintain or restore desirable environmental flows and/or hydrological cycles; • For flood management, manage up to the 1% AEP (100-year ARI event) within the development area to pre-development flows and Water Corporation requirements (as per the Limestone Creek Capacity Review); • Ensure that the first 15 mm of rainfall receives treatment prior to discharging to a receiving environment (DWER, 2017); • Minor events should be managed to provide serviceability requirements; • Roads and public open spaces are to be designed to cater for the surface overflow for more severe storm events; and, • Habitable floors at least 300 mm above the 1% AEP flood level in the local drainage system and 500 mm above the 1% AEP flood level in basins/sumps (where there is no overflow relief) and Poison Gully Creek.
Groundwater management	• Provide appropriate separation between finished lot levels and groundwater to provide the expected level of amenity; • Ensure soakwells devices are designed at or above maximum groundwater levels; and, • Ensure that any subsoil drainage is treated prior to discharge to waterways and sensitive environments.
Management of disease vectors and nuisance insects	• Limit the creation of new sites for breeding of nuisance insects; • Prevent standing water in drainage infrastructure (infiltration within 96hrs); and, • Improve water quality throughout the development.
Implementation	• Provide a framework to implement water management strategies outlined in the LWMS; and, • Water management measures during construction to prevent damage to existing infrastructure and receiving environments.

4 EXISTING SITE CHARACTERISTICS

A summary of the existing environmental conditions in the Study Area are provided in this section, including determination of the opportunities and constraints for water management.

4.1 Climate

The climate is typical of the south western region of Western Australia and is characterised by the Koppen Climate Classification as Dry Subtropical featuring mild winters and hot to very hot summers. The dominate rainfall mechanisms are frontal systems caused by cold fronts associated with low pressure systems that extend across southern Australian between May and October. During the summer months, thunderstorms and ex-tropical cyclones can bring intense rainfall.

Average annual rainfall recorded at the nearest long-term Bureau of Meteorology (BoM) weather station (Perth Airport (no. 9021) west of the Study Area) since 1944 is 762 mm (Figure 5). Since 2000, the average annual rainfall is 667 mm, an approximately 12.5% decrease.

The significant decrease in annual rainfall is associated with a decrease in winter rainfall. Rainfall in the June and July has decreased by approximately 20%. There has been an increase in late summer rainfall associated with thunderstorms and ex-tropical cyclones, but this rainfall is often intense occurs over a short duration and does not affect the loss of winter rainfall.

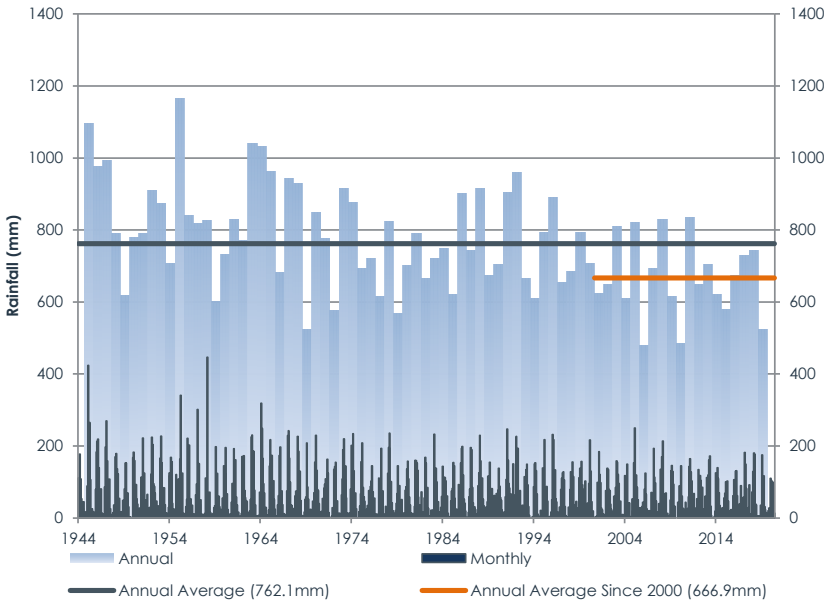

Figure 4: Rainfall summary data (Perth Airport, BoM, 2020)

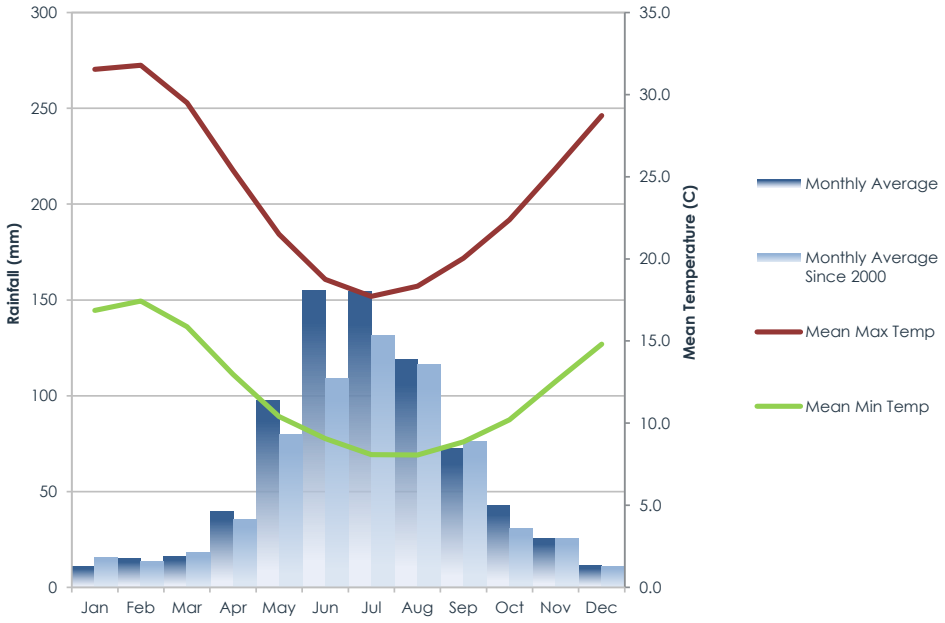

Figure 5: Climate summary data (Perth Airport, BoM, 2020)

Temperatures recorded at the Perth Metro WA (BoM station 9225) show that the average monthly maximum temperatures range between 18.0°C in July and 32.0°C in February, while average monthly minimum temperatures range between 8.0°C in July and 17.5°C in February. Prior to the year 2000, the average monthly maximum temperatures were 17.7°C in July and 31.8°C in February, while average monthly minimum temperatures range between 8.1°C in July and 17.4°C in February.

4.2 Topography

Topography at the Site varies from 37 mAHD in the west and from there it slopes downwards to the east reaching a minimum of 28.8 mAHD. The existing elevation contours are presented in Figure 6.

4.3 Geology and soils

Stormwater management is influenced by the soils of the Study Area and their ability to retain and infiltrate runoff.

4.3.1 Surface geology and soils

Regional surface geological mapping indicates the site is predominantly Sand S10, with a small portion of Mgs1in the southern corner (Figure 6).

Several bores were drilled by Strategen in 2011 and the geotechnical conditions were outlined in the DWMS (Strategen, 2015). The soils identified in these bores were described as predominantly sand with occasional gravelly sand present at surface. Low permeability

material (clayey sand) was identified in several locations, between 2.5 m and 5 m below the surface, including the wetland area. This is discussed further in the below sections.

4.3.2 Soil Permeability

4.3.2.1 Forrestfield Airport Link Geotechnical Investigation (Golder Associates, 2015)

This geotechnical investigation was conducted in 2015 as part of the Forrestfield Airport Link (FAL) Project, with test sites located within a portion of this Site Area. Infiltration pit PW1-01 is located just north of the Maida Vale Road and Dundas Road intersection. Permeability test pit locations I1-01 to I1-03 are located along Sultana Road West and Dundas Road (Figure 6).

The pumping test at PW1-01 indicated the test pit capacity was reached within 5 minutes and resulted in a negligible change in water level for 6 hours. This indicates the infiltration capacity in this location is negligible.

The permeability test results at I1-01 to I1-03 have been summarised in Table 2.

Table 2: Permeability Test Summary

Location	Test 1 K (m/day)	Test 2 K (m/day)	Test 3 K (m/day)	Average K (m/day)
I 1-01	18.6	12.9	11.7	14.40
I 1-02	5.0	3.7	3.1	3.93
I 1-03	11.4	10.5	9.8	10.57

(Golder, 2015)

The full FAL geotechnical report including test pit results has been provided in Appendix 3.

4.3.2.2 Forrestfield North Residential Precinct Geotechnical Investigations

Sands are present at surface level across the majority of this site, with clayey sand present from 2.25 to 3.5 mbGL as indicated by borehole logs and test pit logs (Appendix 3), and as discussed in Section 4.3.1. Hydraulic conductivity of sands and clayey sands were examined as part of the adjacent residential precinct LWMS (Strategen 2020). The findings indicate:

- Conductivity of fine to medium sand (the majority of the Study Area) was measured between 2.7 and >10 m/day (with a recommended design conductivity of 1.3 to 3.0 m/day).
- Clayey sand, the impermeable material at depth within the Study Area, had a measured hydraulic conductivity between 0.7 and 1.2 m/day (with a recommended design conductivity of 0.3 to 0.6 m/day).

It is noted that the City of Kalamunda have indicated that soakwells will not be permitted where the design hydraulic conductivity is less than 1m/day. This will be investigated further in the UWMP.

4.3.2.3 Depth to Low Permeability Soils

The DWMS recorded borehole logs for bores located in the northern portion of the Site Area installed in 2011. Depths to clayey sand at each borehole located within the Site Area boundaries are as follows:

- MW1: Clayey Sand from 3 mbGL to the total bore depth at 11 mbGL.
- MW7: Clayey Sand from 2.5 mbGL to the total bore depth at 13 mbGL
- MW8: Clayey Sand from 3.5 mbGL to the total bore depth at 9 mbGL
- MW9: Clayey Sand from 2.25 mbGL to the total bore depth at 5.5 mbGL

Figure 7 outlines the depth to the impermeable layers across the site, with interpolated mapping of the depths taken from the LWMS (Strategen, 2020). Further investigation will be undertaken for future UWMPs, including hydraulic conductivity.

Borehole logs have been provided in Appendix 3.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 6 - Soils and topography

Cps - peaty clay
Dark gray and black with variable sand and organic content, of lacustrine origin

Mgs1 - Pebbly silt
Strong brown silt with common, fine to occasionally coarse-grained, sub-rounded laterite quartz, heavily weathered granite pebble, some fine to medium-grained quartz sand, of alluvial origin

S10 - Sand
White to pale gray at surface, yellow at depth, fine to medium-grained, moderately well sorted, subangular to subrounded quartz, of eolian origin

S12 - Sand
Yellow, fine to medium and coarse-grained sub-angular to rounded quartz with some feldspar, heavy minerals common in some areas, minor silt and clay, mostly of colluvial origin

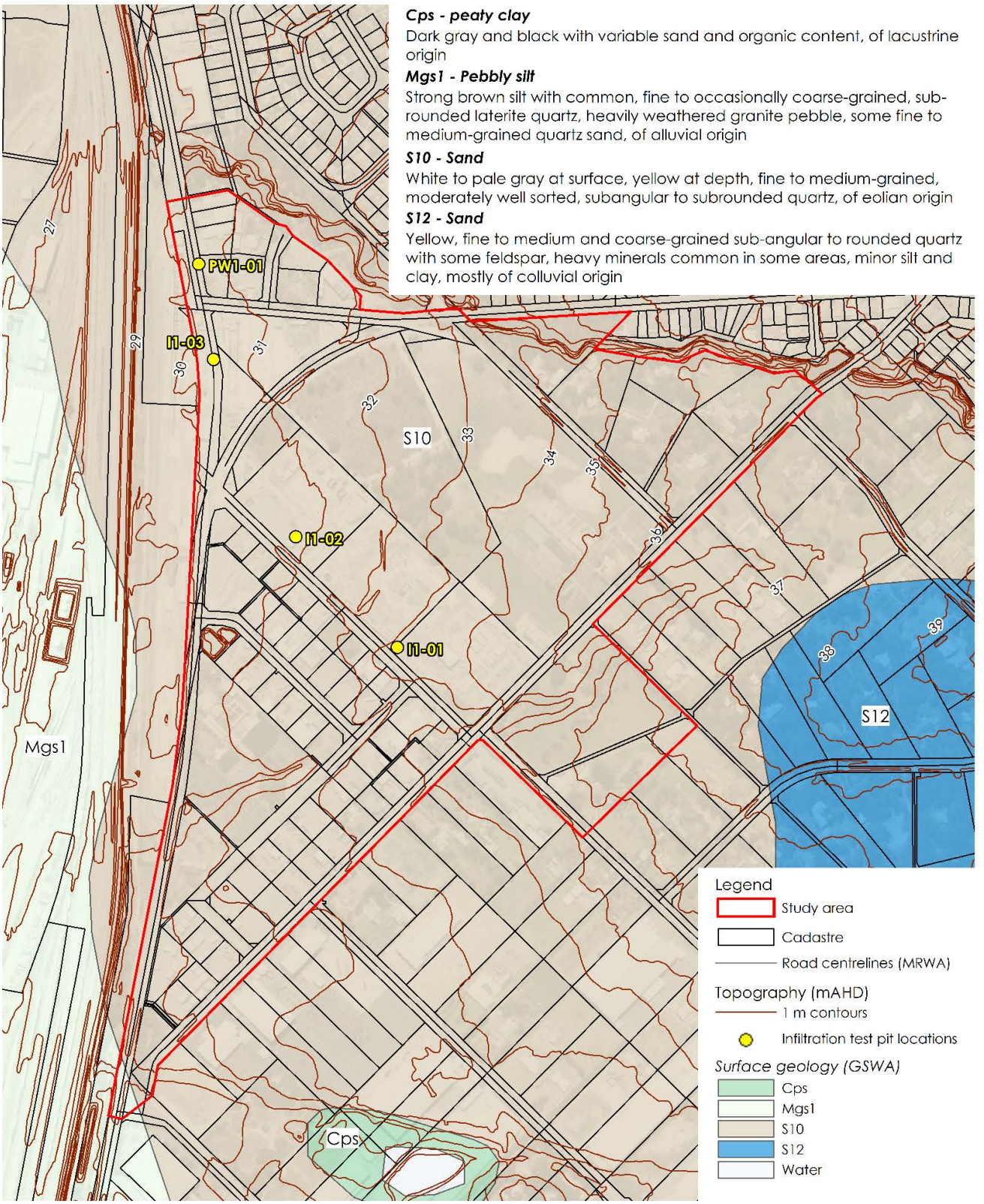

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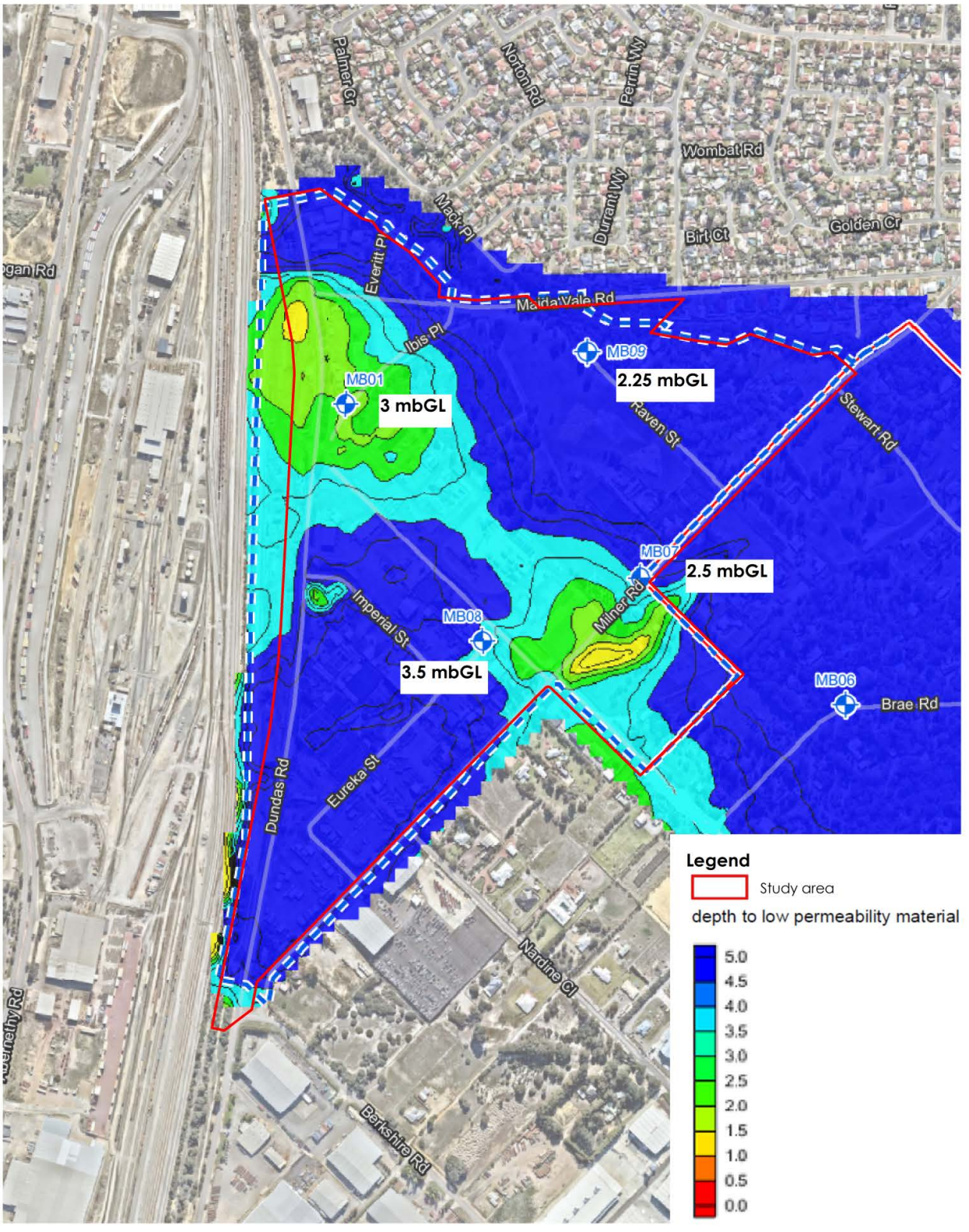
Figure 7 - Depth to low permeability material



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4.3.3 Acid sulfate soils

A review of the Department of Water and Environment Regulation Acid Sulfate Soils (ASS) risk mapping (DWER, 2020a) identifies the Study Area as having low to medium risk of ASS occurring within 3 m of the surface.

4.4 Groundwater resources

Determining the groundwater depth, quality and availability are crucial for forming total cycle water management strategies. These components are outlined below.

4.4.1 Public Drinking Water Source Area

There are no Public Drinking Water Source Areas within the Study Area.

4.4.2 Groundwater allocation

The Study Area is located within the Shire of Kalamunda groundwater management subarea. The Department of Water and Environmental Regulation's Water Register (DWER, 2020b) shows that there is limited groundwater available for allocation from the Superficial Aquifer within the Study Area. An Aquifer Allocation Report was requested from DWER in December 2020 and the details are shown in Table 3.

Table 3: Groundwater resource allocation availability (as of December 2020)

Management Area	Management Sub Area	Resource	Allocation Limit	Allocated Volume	Remaining Volume
Perth	Shire of Kalamunda	Superficial Swan	2,986,500 kL	2,891,178 kL	89,222 kL

There is currently a combination of temporary and standard licences within the Study Area. The temporary licences are associated with construction of the Airport rail line (160,000 kL/yr). The standard licences are summarised in Table 4.

Table 4: Existing groundwater licences within the Study Area

Licence Number	Licence Allocation	Groundwater Area	Groundwater Subarea	Issue Date	Expiry Date
GWL63807	89,390kL	Perth	Shire of Kalamunda	25/11/2019	25/11/2029
GWL152215	23,120kL	Perth	Shire of Kalamunda	23/10/2013	22/10/2023

4.4.3 Groundwater Levels

Site Specific Data

As outlined in the above sections, the presence of low permeability material (clayey sand) may result in perched groundwater at levels of 2.25+mbgl in parts of the site. Groundwater monitoring bores were installed by Strategen in 2011 prior to preparation of the DWMS for the

City of Kalamunda. Monitoring was undertaken for 15 months including two winters between September 2011 and December 2012, with two peak periods recorded in October 2011 and in September 2012.

Maximum Groundwater Levels (MGLs) varied between approximately 23 (MB7) and 33mAHD (MB9) (Figure 8). This equated to a depth to groundwater of approximately 0.5m to 10mbgl (Figure 9). The complete monitoring data set is provided in Appendix 4.

Perched Groundwater Assessment

A desktop review of geotechnical, groundwater and surface water data across the Site Area was conducted by Strategen in 2016. This review assessed the presence of perched groundwater within the Site Area, and specifically the Poison Gully Creek TEC and Ibis Place TEC areas. The results indicated if perched water were present, this would not be permanent.

There was no evidence of sub-surface seepage into Poison Gully from perched groundwater, even in areas where the Creek is deeply incised. The review concluded that any perched groundwater which may occur is likely to be ephemeral and limited to isolated areas, however further monitoring was proposed to assess seasonal behaviour and localised perched groundwater.

Data from the review was used to inform Strategen's calculation of perched MGLs at all DWMS bores as part of the Forrestfield North Residential Precinct LWMS (Strategen, 2020) investigations, including the bores located on this site MB1, MB7, MB8, and MB9. The desktop review has been provided in Appendix 4, with calculated MGLs discussed below and presented in Table 6. Perched groundwater was noted near MB9 as shown in Table 7.

Correlation to Historical MGLs

Rainfall in 2011 was approximately 10 % greater than the long-term average, indicating that it was a wet year and representative of higher groundwater conditions. Rainfall from September to December was considerably more than long term averages (Table 5), providing confidence that the measured levels were representative of recent maximum conditions.

Table 5: 2011 Rainfall Totals

Month	Long-term Average (mm)	2011 Rainfall Total (mm)	% Change
January	10.9	43.2	294.8%
February	15.0	0.4	-97.3%
March	16.2	0.0	-100.0%
April	39.7	26.2	-34.0%
May	97.6	58.6	-39.9%
June	155.1	143.2	-7.7%
July	154.2	164.6	6.7%
August	118.7	127.8	7.7%
September	72.3	102.4	41.6%
October	42.6	63.4	48.8%
November	25.4	38.6	51.8%

December	11.2	67.4	504.3%

Typically, maximum groundwater levels are determined through a comparison of onsite measurement with nearby long-term DWER monitoring bores. Comparisons were made to historic groundwater levels from the nearest DWER bore 61610508 and other available historical groundwater level data to provide an estimate of short-term perched MGL following high rainfall periods as part of the Forrestfield North Residential Precinct LWMS (Strategen, 2020) investigations.

The groundwater levels for 2011 and 2012 were compared to the groundwater levels at DWER bore (61610508) for an assessment of the perching that occurs where the clay layer is known to be present west of Milner Rd (and predominantly within the Site Area) (Strategen, 2020). The estimated maximum groundwater levels for each bore location have been provided in Table 6. The bores within this Site Area have been highlighted.

Figure 8 and Figure 9 display the groundwater level contours, and depth to groundwater across the site (Strategen, 2020). This indicates groundwater is shallow around MB1 and MB9, at or within 1.0m of the surface. A localised area of shallow groundwater is also located along Imperial Street.

Table 6: Groundwater levels (Strategen, 2020)

Bore	Easting	Northing	Ground level (mAHD)	Screened depth (mbgl)	2011 Peak (mbgl)	2011 Peak (mAHD)	2012 Peak (mAHD)	Difference Between 2011 and 2012	Calculated max groundwater level (mAHD)
MB01	404861	6464332	30.04	6 - 9	3.79	26.427	24.690	-1.737	30.027
MB02	405922	6464054	38.45	14 - 18	14.22	24.235	23.990	- 0.245	31.235
MB04	406211	6463643	39.73	15 - 19	15.73	23.876	23.800	- 0.076	30.876
MB05	406081	6463047	40.77	15 - 19	16.99	23.776	23.570	- 0.206	30.776
MB06	405683	6463843	36.24	13 - 16	11.92	24.319	23.970	- 0.349	31.319
MB07	405347	6464050	34.49	9 - 13	9.61	24.876	24.230	- 0.646	31.876
MB08	405086	6463948	31.84	6 - 9	3.97	27.873	25.750	- 2.123	30.873
MB09	405255	6464415	33.36	2 - 5.5	0.95	32.411	28.350	- 4.061	33.411

Table 7: Rainfall, DWER Bore (61610508) and groundwater levels data summary (Strategen, 2020)

Month	MGL 2011	MGL 2012	Difference (m)	Comment
DoW bore (61610508)	~ 14.5 mAHD	~ 14 mAHD	~ 0.5	2017 (Oct) levels ~15.2 mAHD (0.7 m higher than 2011, 1.2 m higher than 2012).
Rainfall (Jan-Sept)	666.4 mm	526 mm	0.14	Higher rainfall for Jan-Sept 2011.
Rainfall (Oct)	63.4 mm	13.8 mm	0.496	High Oct 2011 rainfall periods immediately prior to groundwater level measurement.
Oct groundwater levels East of Milner Rd	23.776 - 24.319 mAHD	23.570 - 23.990 mAHD	0.076 - 0.349 m	no significant perching is evident east of Milner Rd (absence of clay layer).
Oct groundwater levels West of Milner Rd	26.427 - 32.411 mAHD	24.690 - 28.350 mAHD	1.737 - 4.061 m	Significant perching to the northwest near MB09 due to clay layer west of Milner Rd.

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Figure 8 - Groundwater levels

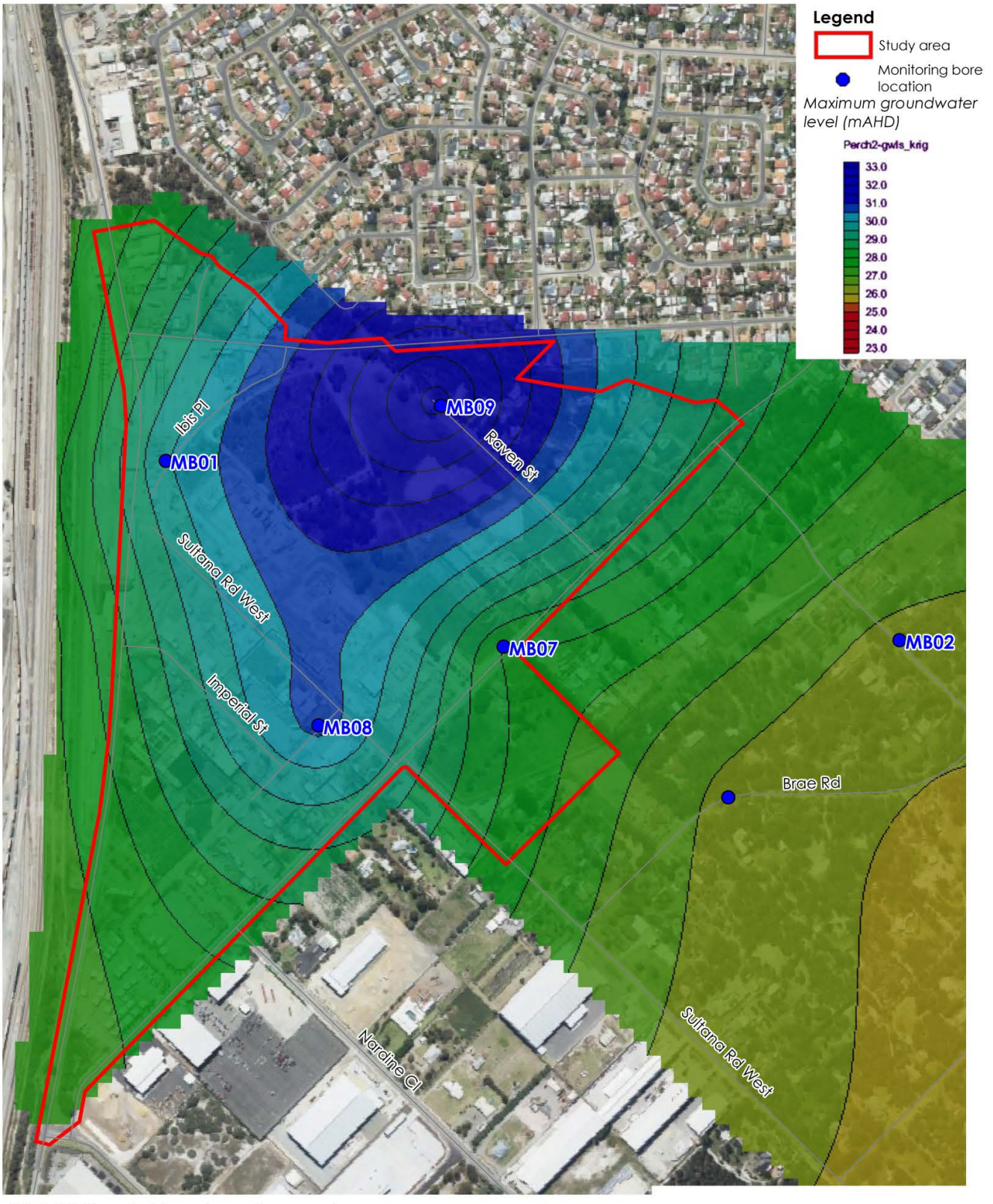

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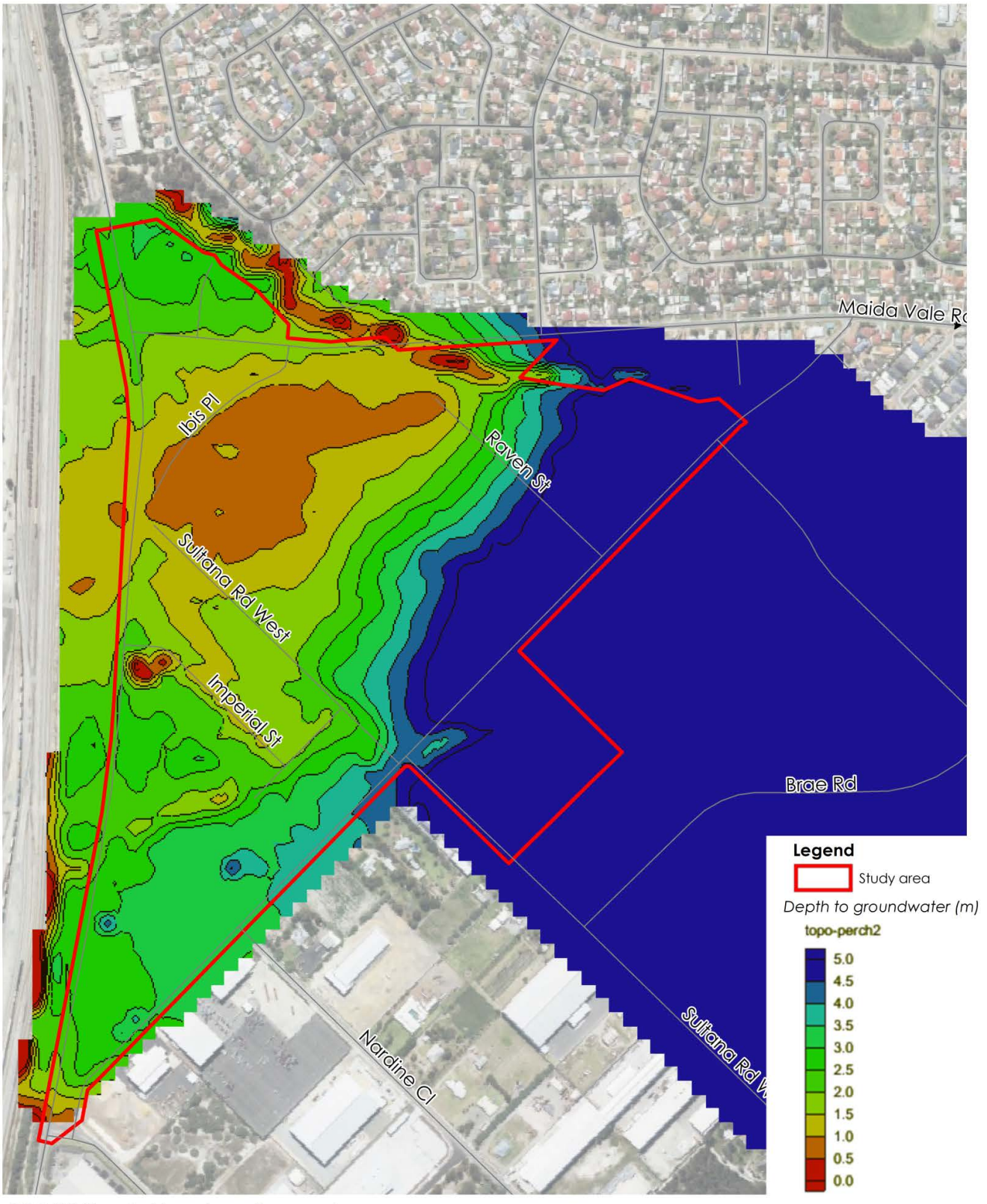
Figure 9 - Depth to groundwater



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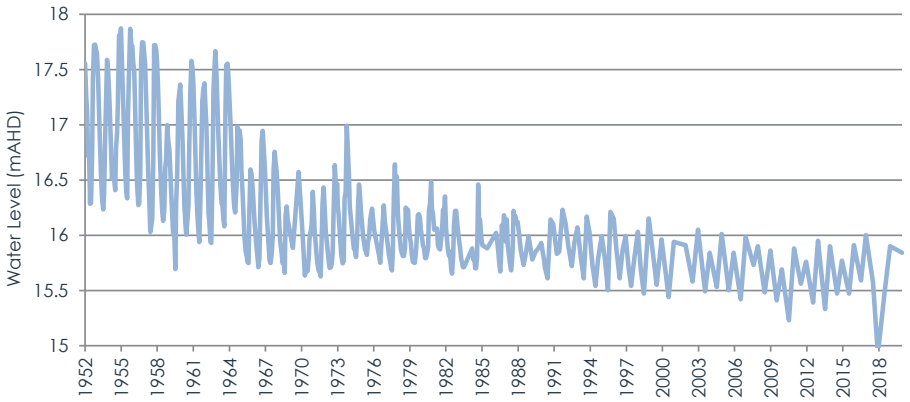


Figure 10: Groundwater levels in DWER Bore 675B

4.4.4 Groundwater Quality

Groundwater quality monitoring was undertaken by Strategen between September 2011 and December 2012. A summary of the key results is provided in Table 8, and all results are provided in Appendix 4.

Table 8: Groundwater quality monitoring results

Parameter	Results	MB01	MB07	MB08	MB09
pH	Median	5.97	6.18	6.09	6.35
	Range	5.68 – 6.62	5.84 – 6.34	5.83 – 6.58	6.02 – 6.68
Electrical Conductivity	Median	0.25	0.25	0.70	0.18
	Range	0.23 – 0.45	0.14 – 0.31	0.01 – 0.92	0.17 – 0.19
Dissolved Oxygen (%)	Median	32	69	76	53
	Range	26 – 87	49 – 134	73 – 271	44 – 62
Total Nitrogen (mg/L)	Median	1.80	8.00	21.50	1.30*
	Range	0.93 – 5.90	7.10 – 10.00	18.00 – 25.00	-
Total Phosphorus (mg/L)	Median	0.61	0.77	0.62	0.08*
	Range	0.21 – 1.70	0.20 – 1.60	0.18 – 1.20	-

** only one sample collected at MB09*

*Exceeds Swan Canning WQIP **Short term limits** or **Long-term limits** (Swan River Trust, 2009)*

*Exceeds Australian and NZ **Guidelines** for Fresh and Marine Water Quality (ANZG, 2018)*

Consistent with the former rural and agricultural land uses on the site (Section 2.1) nutrients generally exceed short- and long-term limits in the Swan Canning Water Quality Improvement Plan (Swan River Trust, 2009). Limited sampling of metals indicates concentrations of Copper, Chromium, Nickel, Lead and Zinc that exceed Australian and NZ Guidelines for Fresh and Marine Water Quality guidelines (ANZG, 2018). Results for all sampling is provided in Appendix 4 with reference to relevant guidelines. Groundwater mobilised through development (e.g. subsoil drainage) will require treatment prior to discharge to downstream waterways and wetlands.

4.5 Surface water resources

Existing surface water features within and adjacent to the Study Area will require protection from redevelopment or provide opportunities for modification to deliver the community an asset with social and ecological benefits. The surface water features are outlined below.

4.5.1 Natural water resources

Within and adjacent to the Study Area are two significant water features; Poison Gully Creek and a resource enhance wetland south of Maida Vale Road. Poison Gully Creek is an ephemeral creek that flows on the northern boundary of the Study Area and originates in Gooseberry Hill. Stormwater from the northern portion of the Study Area and residential areas in Maida Vale discharge to the creek. Downstream, Poison Gully Creek discharges into airport land via culverts under Dundas and Abernathy Roads. As shown in Figure 11, the creek eventually flows to Limestone Creek and the Swan River.

The capacity of Poison Gully Creek was assessed in the *Limestone Creek (Perth Airport Northern Main Drain) Stage 1 Capacity Review* (Water Corporation, 2010), particularly with consideration of the future development within the catchment. A hydraulic and hydrological model was established for the catchment to simulate the 10 year and 100-year ARI events. Results of the modelling indicated that there are no flooding concerns, either in the existing (2010) system or the ultimate development near the Study Area.

The hydrological model established by Water Corporation accounted for the runoff from sub-catchments in the Study Area: la_po166 and La_po016 (sub-catchments boundaries are provided in Appendix 5). Advice from Water Corporation for the Forrestfield North Residential Precinct is that post-development flows into Poison Gully Creek should not exceed pre-development flow. This is discussed further in Section 5.3. Proposed parameters for the residential precinct are provided in Table 9 and Table 10.

Table 9: Hydrological pervious and roughness parameters (Strategen, 2020)

Land Use	% Impervious	Manning's n Pervious	Manning's n Impervious
Urban roads	80%	0.04	0.025
R5-R12 lots	10%	0.05	0.025
R60/R80 lots*	90%	0.04	0.025
Rural lots	5%	0.04	0.025
Industrial	90%	0.04	0.025
POS	0%	0.04	0.025

**Initial 16 mm managed on lot, depending on soil conditions*

Table 10: Hydrological loss parameters (Strategen, 2020)

Soil Type	Initial	Continuing	Comment
Impervious	1.5 mm	0 mm/hr	
BoM – Forrestfield	26.0 mm	6 mm/hr	
Rural road impervious	16.0 mm	6 mm/hr	Orange sandy surface soils
Silty sand	16.0 mm	6 mm/hr	

Soil Type	Initial	Continuing	Comment
Guildford	10.0 mm	3 mm/hr	With some vegetation

The assumed parameters for pre-development within the Perth Airport North catchment were the Silty Sand conditions. These parameters have been adopted for the pre-development modelling outlined in Section 4.5.3, along with a description of the wetland south of Maida Vale road.

Poison Gully Creek has significant Aboriginal Heritage significance that is discussed further in Section 4.6. The foreshore area south of Maida Vale Road was previously defined in the DWMS (Strategen, 2015) which noted:

- The creek is deeply incised, with steep banks;
- Interpreted flood widths vary between 10 m and 40 m for the 100-year ARI event;
- Owing to the steep banks, typical wetland vegetation such as sedges and rushes was not abundant;
- Vegetation condition varied between completely degraded and good to very good, with the best vegetation located near Maida Vale Road; and,
- This area is recognised as a Bush Forever site (Section 4.6).

4.5.2 Existing Drainage Infrastructure

Drainage infrastructure through the site is a combination of formal and informal systems. Existing pit and pipe drainage is identified on Figure 11 and is generally restricted to Maida Vale and Dundas Roads in the north (discharging to Poison Gully Creek), and Eureka Street and Milner Road in the south, discharging to an existing drainage sump. This southern sump (between Milner Road and Dundas Road) and another sump on the corner of Imperial Street and Dundas Road are fenced-off systems with no amenity value. These systems ultimately discharge to the Perth Airport Drain (Airport South Catchment).

Minor roads feature roadside drains that are generally less than 500 mm deep and do not contain maintained vegetation. These drains convey runoff to Poison Gully Creek, the wetland or the existing sumps.

4.5.3 Wetlands

The site contains a Resource Enhancement wetland that is likely to be upgraded to a Conservation Category wetland (UFI: 15875) on Lot 12 Ibis Place, on the corner of Ibis Place and Maida Vale Road. The vegetation within the wetland has been assessed on numerous occasions. The most recent assessment by PGV Environmental confirmed the presence of a Threatened Ecological Community (TEC), which is why the wetland will likely be upgraded to and managed as a Conservation Category Wetland (CCW).

Wetland Vegetation

The vegetation on the main part of the wetland on Lot 12 is described generally as scattered Marri (*Corymbia calophylla*) trees over a low heath with common species being *Xanthorrhoea preissii, Verticordia densiflora, Stirlingia latifolia* and *Hypocalymma angustifolium*. The species composition of the vegetation, and absence of Paperbark and wetland sedge species, is consistent with the mapping of the wetland as a Dampland, which is a waterlogged basin that does not have above-ground water at any time of the year. The vegetation on the small extension of the wetland on Lot 9 is also considered the same TEC and contains Sheoak trees (*Allocasuarina fraseriana*) over a low heath understorey.

Wetland Buffers

The Environmental Protection Authority (EPA) indicate in Guidance Statement 33 *Environmental Guidance for Planning and Development* that 50m buffers should be applied to Conservation Category wetlands (EPA, 2008). Therefore, consideration has been given to include a minimum 50m buffer to the vegetation on the site (with the Local Open Space (LOS) providing a buffer greater than 50m). However, some existing and proposed land uses will make provision of a 50m buffer not possible on two of the boundaries (PGV Environmental, 2021).

Development of the High Wycombe train station and associated infrastructure, including a Western Power transformer site, is already in place to the north-west of the wetland. Accordingly, no buffer is possible to provide on most of the north-west boundary. The northern tip of the site will have a small buffer to existing Maida Vale Road (PGV Environmental, 2021).

A Community Hub is proposed to be constructed to the south-west of the wetland. The Community Hub will replace the existing industrial works that are currently on the site. No buffer is able to be provided to the south-west side unless the current concreted hardstand is dug up. This is considered impractical. The Community Hub is likely to include a library, public pool and carparking which will have a car parking component and will be sited along the boundary of the site with the buildings and pool located further away.

Urban development is proposed on the north-east side of the wetland. A 50m buffer will be provided on the north-east boundary. This area is currently cleared rural land with a few trees at the northern end. As the passive recreational needs of the community will be provided in the LOS on the south-eastern side, the north-east buffer does not need to include any additional areas for public use (PGV Environmental, 2021). It is therefore proposed to revegetate the buffer in this area. The form of the revegetation will need to consider any adverse impacts of revegetation on the bushfire hazard rating for the proposed urban development that will be constructed adjacent to the buffer (PGV Environmental, 2021).

The area to the south-east of the wetland will be 'Local Open Space' (LOS). The LOS is proposed to include grassland areas for recreational purposes as well as areas containing areas for passive recreation. In effect the buffer to the bushland on the south-east side will be well over 50m, extending up to 150m to the nearest area of urban development (PGV Environmental, 2021).

The buffer will not cater for stormwater drainage features such as infiltration and detention basins (See Section 5.3).

Wetland Hydrology

The hydrology of the wetland contrasts with the regional groundwater information, however local groundwater data (Section 4.4.3) demonstrates the presence of shallow, perched groundwater in the area around the wetland. Within the wetland and immediately to the east, the depth to groundwater is less than 1.0 m indicating that perched groundwater may result in seasonal water logging or inundation of the wetland (Figure 9).

Runoff from surrounding areas also contributes to the wetland hydrology and vegetation. A local catchment, determined from the available topographical data, is shown in Figure 12 indicating that approximately 5.5 ha drains into the wetland system. This catchment consists of cleared rural land and remnant vegetation.

Predevelopment Wetland Modelling

A hydrological model (XP-Storm) was established to determine the frequency and volume of water that discharges from this catchment into the wetland. Loss parameters for the model are based on Water Corporation suggestions for the residential precinct (Table 10). These parameters don't consider the relatively shallow depth to groundwater within the wetland catchment (Figure 9). To determine inflows, the catchment was separated into areas of deep and shallow groundwater, with different loss parameters (Type I and Type II respectively in Table 11). The estimated volumes of runoff entering the wetland for typical design storm events are provided in Table 12.

Table 11: Wetland catchment loss parameters (Strategen, 2020)

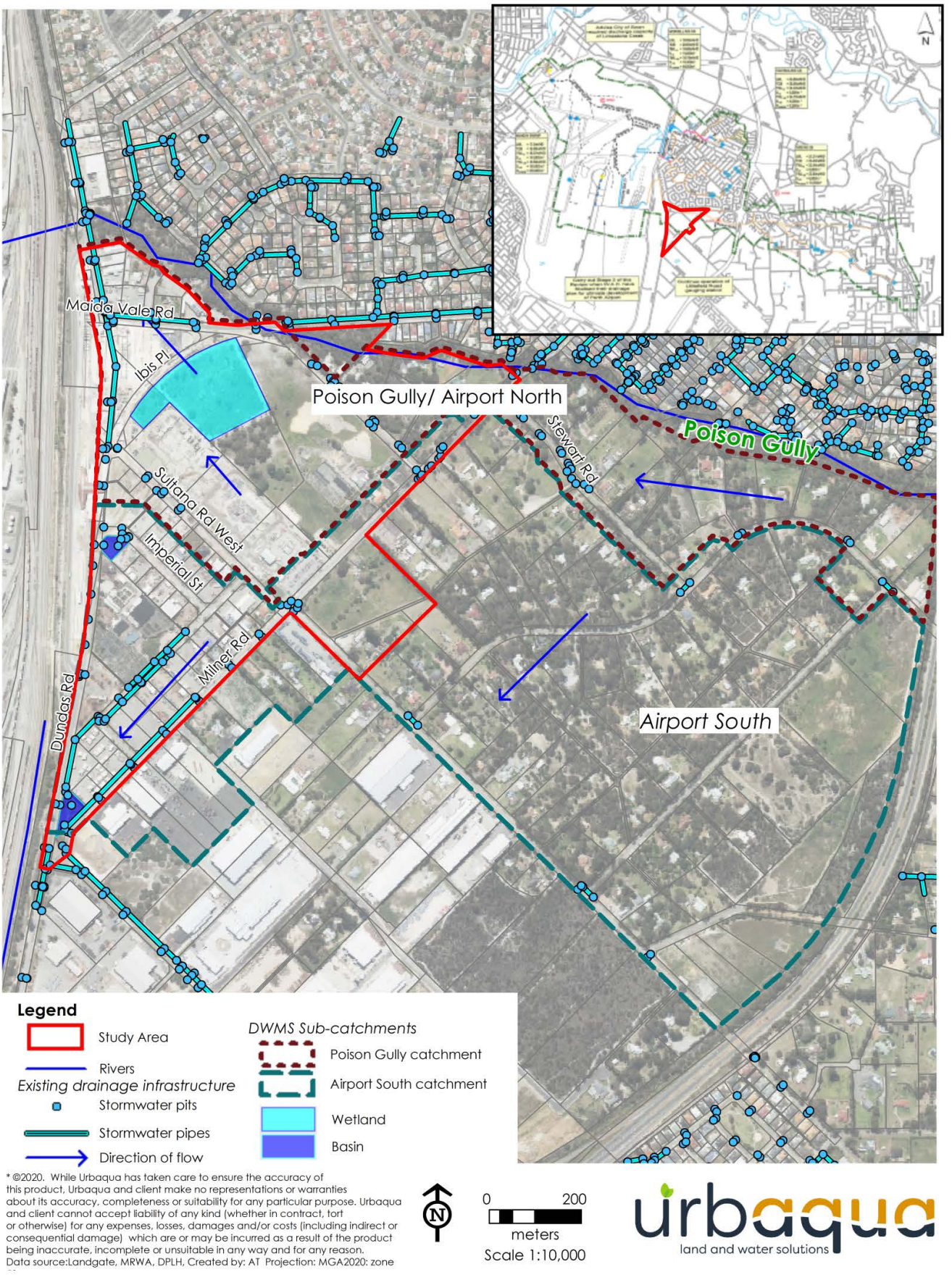
Soil Type	Groundwater Depth	Initial	Continuing	Type
Silty Sand	Deep	16 mm	6 mm/hr	I
Guildford with some vegetation	Shallow	10 mm	3 mm/hr	II

Table 12: Runoff into wetland from local catchment

Duration	63.2% AEP (1yr ARI)	20% AEP (5yr ARI)	10% AEP (10yr ARI)	1% AEP (100yr ARI)
1 hr	374 m^3	489 m^3	519 m^3	752 m^3
3 hr	456 m^3	638 m^3	745 m^3	1,250 m^3
6 hr	389 m^3	711 m^3	868 m^3	1,513 m^3
9 hr	466 m^3	666 m^3	915 m^3	1,651 m^3
12 hr	485 m^3	816 m^3	869 m^3	1,731 m^3
18 hr	469 m^3	738 m^3	944 m^3	1,858 m^3
24 hr	355 m^3	742 m^3	841 m^3	1,790 m^3
48 hr	205 m^3	223 m^3	460 m^3	908 m^3
72 hr	36 m^3	147 m^3	305 m^3	490 m^3

The modelling results suggest that regular runoff occurs into the wetland for all storm events. The shallow groundwater and impermeable material favours runoff from the local catchment and runoff occur for higher intensity, shorter duration events and lower intensity, higher duration. The peak runoff into the catchment occurs in the 1% AEP 18-hour event (1,858 m^3). Directing a portion of the stormwater runoff from the development into the wetland is therefore considered appropriate to maintain the hydrology and associated ecology.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 11 - Surface water features



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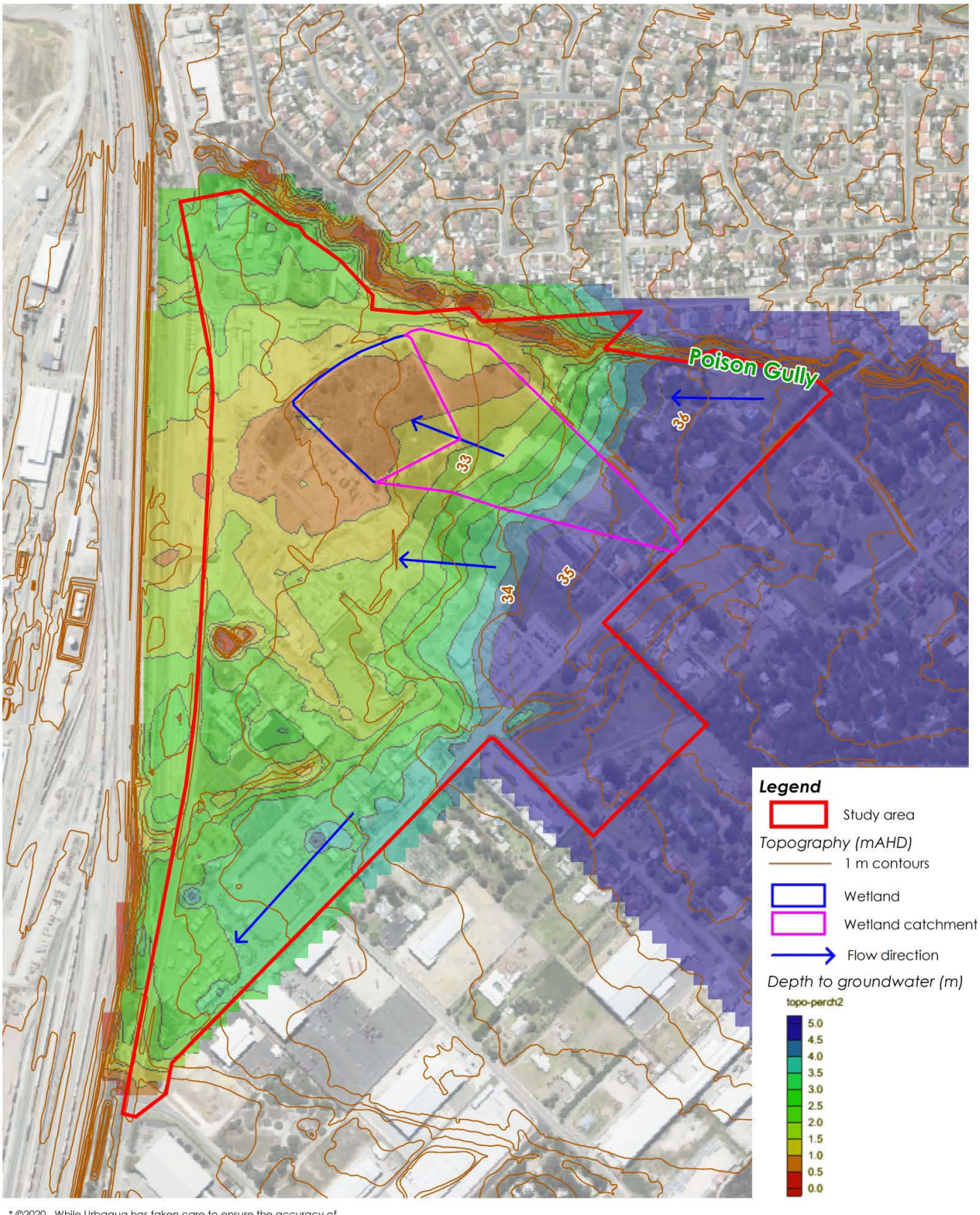
DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 12 - Wetland catchment



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4.6 Environmental and cultural

A search of the DWER Contaminated Sites Database (DWER, 2020d) indicates that there are no contaminated sites located within the Study Area.

The Poison Gully Creek foreshore area and a portion of private land north of Raven Street are mapped as Bush Forever (Site No. 45) as shown in Figure 13. This Bush Forever Site is inferred to contain TEC 3a: *Corymbia calophylla* (marri) – *Kingia australis* woodlands on heavy soils, Swan Coastal Plain (Government of Western Australia, 2000). The Bush Forever site will be maintained as part of the planning process. There are also areas of scattered remnant vegetation that have been included in future open space. The future open space adjacent to the wetland is shown in Figure 3 and the scattered vegetation in that open space can be seen in Figure 2.

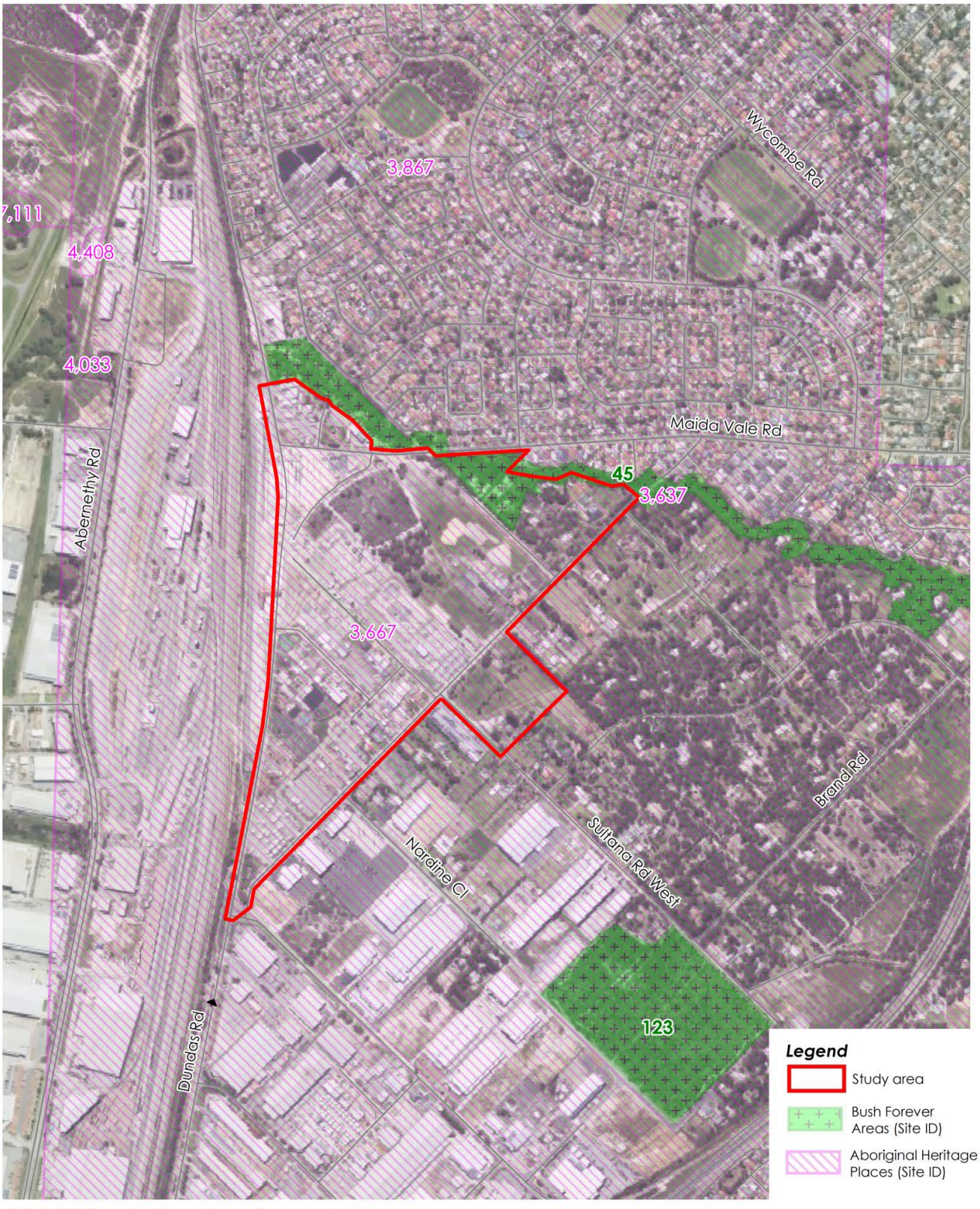
The Aboriginal Heritage Sites database (Department of Aboriginal Affairs, 2020) indicates that the Study Area is part of the Poison Gully Creek registered site (No. 25023) (Figure 13).

4.7 Summary

Based on the review of geological, hydrological and environmental information for the Study Area, the key considerations for water management are as follows:

- There is an existing wetland (REW) within the northern portion of the Study Area that features a TEC;
- Variable depth to groundwater across the site, with shallow perched groundwater identified surrounding the existing wetland;
- Poison Gully Creek flows on the northern boundary of the site and ultimately discharges to Limestone Creek and the Swan River;
- The southern portion of the site drains to fenced sumps that discharge to the Perth Airport Drain (Airport South Catchment);
- Drainage infrastructure through the site contains a combination of formal (sumps, pipes and pits) and informal (roadside drains);
- The existing wetland receives runoff from the local catchment;
- There is a Bush Forever site along Poison Gully Creek, inclusive of the foreshore area and pockets of remnant vegetation near the wetland; and,
- Groundwater monitoring onsite did not indicate any significant issues to prevent development.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 13 - Environmental and cultural features



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5 WATER MANAGEMENT STRATEGY

Water management strategies for the Study Area have been prepared in accordance with the guiding documents, policies and strategies (Section1), the intended development concept (Section 2) and the site considerations (Section 4). Strategies for water sustainability, stormwater, groundwater, and water quality improvement are outlined in this Section.

5.1 Water sustainability initiatives

The key objectives for water sustainability are:

- Maximise local infiltration to replenish surface groundwater aquifers; and,
- Provide alternative water sources for domestic irrigation and in-house domestic non-drinking water demands.

5.1.1 Water supply

The Study Area is located in an area served by the Water Corporation's integrated water supply scheme. All buildings will be connected reticulated drinking water distribution network.

There are approximately 4.97 ha of Public Open Space (POS) in the proposed design concept, including 2.03 ha of space that will not require ongoing irrigation (remnant vegetation). Based on the DWER standard irrigation rate of 7,500 kL/ha/yr approximately 22,065 kL/yr of allocation is required.

There is allocation available within the groundwater management sub-area, though current licences within the Study Area will become available following development. Transfer of the current licences would require negotiation with respective private licence holders. The City current holds a licence (GWL No. 158077) for irrigation of POS through the wider groundwater management sub-area for 936,725 kL/yr. The City has indicated a portion of this licence is available for irrigation within the Study Area and is the favoured source.

Confirmation of the final irrigation arrangement will be provided in the Urban Water Management Plan.

5.1.2 Wastewater treatment and disposal

The Study Area is located in an area served by the Water Corporation's integrated sewerage scheme and will be connected to a reticulated sewerage network.

5.1.3 Water conservation and efficiency measures

To reduce the consumption of scheme water newly constructed houses will be recommended to meet the Water Corporation's Waterwise homes and gardens criteria. That is:

- All showerheads installed will be better than the minimum WELS 3 Star rating;
- All taps installed will be better than the minimum WELS 4 Star rating;
- All toilets will be duel flush and exceed the minimum WELS 4 Star rating; and
- All water using appliances installed are rated WELS 4 Star or above.

Considerable savings can be achieved in potable water demand by using alternative sources for non-potable demands such as toilets, washing machines and irrigation. The construction of new commercial and residential buildings provides the opportunity to integrate these systems during design and maximise the potential for efficient use of potable water.

Proposed commercial and residential buildings should be required to harvest roof runoff with plumbing to toilets and washing machines. The harvesting of roof runoff can contribute to a reduction in stormwater runoff downstream. The design and performance of these systems would vary with each building and the City should provide guidance on volume and water saving benefits to encourage uptake. However, it is not possible to estimate the level of uptake for stormwater harvesting at this stage of planning and design, therefore, stormwater modelling for this LWMS does not assume that harvesting of roof runoff will occur and is therefore conservative in this regard.

The majority of the saving in potable water is the use of non-potable sources for irrigation of gardens and open space. At the single lot/dwelling scale these systems can be unfeasible, however with higher density and larger open space areas these approaches become more effective.

Greywater is wastewater from showers, basins and taps from bathrooms and laundry that can be treated and used as a non-potable source either for irrigation of gardens and open space or toilet flushing in commercial or residential buildings. In the proposed built form in the Study Area, greywater collected from residential, commercial and mixed used buildings can be treated and stored within basements and used for irrigation of open space. Designing and implementing these systems during the initial building design prevents retrofitting which is typically unfeasible.

Greywater should also be considered for the irrigation of green roofs and green walls to assist in reducing urban heat. These systems reduce heat by providing shade and evapotranspiration, and green roofs can also reduce stormwater runoff.

To further reduce irrigation requirements, frequent runoff from building carparks and hardstand areas should be directed towards gardens, tree pits or open space, or collected and used for this irrigation. This approach can also be used in carparks for passive watering of trees and to prevent ongoing watering. The Activity Hub presents an opportunity to demonstrate efficient non-potable water use to the community.

5.2 Groundwater management

The key objectives for groundwater management are:

- Provide appropriate separation between finished lot levels and groundwater to provide the expected level of amenity;
- Ensure soakwells devices are designed above maximum groundwater levels; and,
- Ensure that any subsoil drainage is treated prior to discharge to waterways and sensitive environments.

The following planning measures are adopted to achieve the above objectives:

- Subsoil drainage and imported fill to ensure adequate separation to groundwater;
- Ensure infiltration of stormwater runoff, consistent with existing conditions; and,
- Use of bio-retention areas within raingardens, tree pits and swales to improve groundwater quality compared with the existing conditions.

5.2.1 Subsoil Drainage

As discussed in Section 4.4.3, groundwater near the wetland portions of the Study Area is shallow and generally assumed to be at or within 1 m of the surface for the design of the groundwater management system (Figure 9). Given the depth to groundwater, subsoil drainage (slotted PVC wrapped in geo-fabric) will be installed at the maximum groundwater levels (MGLs) to control groundwater. This drainage prevents groundwater from rising uncontrolled through imported fill material and avoids the potential for water logging on residential lots or within road reserves.

Extensive use of subsoil drainage is not anticipated across the site and will be limited to areas of shallow groundwater. Subsoils will be located as follows;

- In areas where subsoil is required, the subsoil drainage will be located within the road reserve.
- Subsoil drainage will be excluded from wetland and buffer areas and should be located on opposite verges from wetland buffers where possible.
- Elsewhere, localised subsoil drainage for retaining walls and roads will be considered during the construction phase to alleviate any problematic soil moisture.
    - Where these conditions are identified short length subsoil drains will be installed.

To be cost effective and free draining, subsoil drains will be connected to the nearest road drainage system, with water quality treatment provided downstream. The minimum separation criteria to stormwater infrastructure are provided in Table 13.

Table 13: Groundwater separation

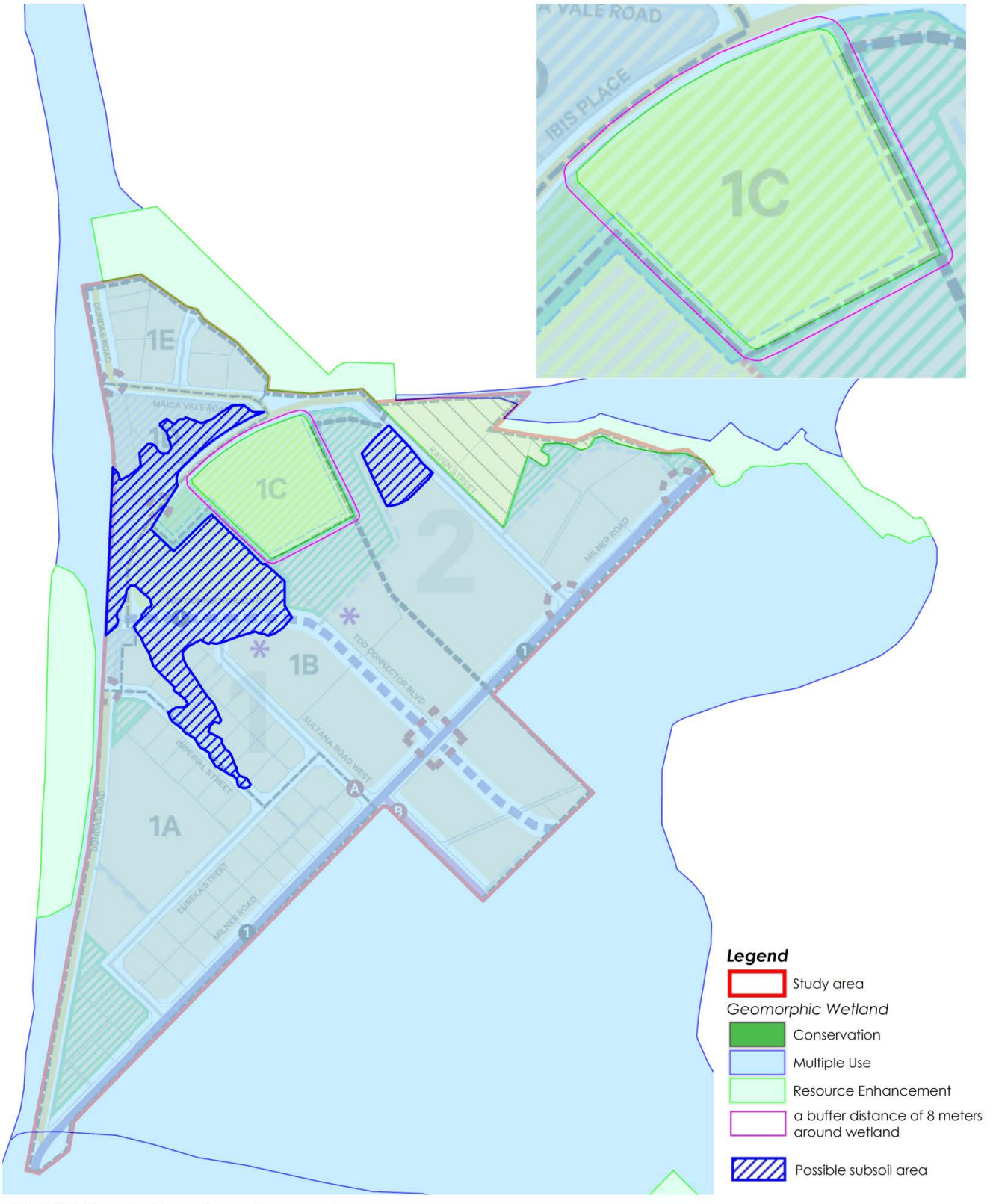
Location	Specification	Comment
Soakwells	500 mm	From base of soakwell to MGL
Stormwater infiltration	300 mm	From invert of vegetated systems such as raingardens and swales to MGL

Further modelling of subsoil spacing will be undertaken following confirmation of internal road layouts in the proposed development areas. That level of detail is not required for the LWMS however further information and a map showing the possible/ likely areas that may require subsoil (ie. Shallower depths to groundwater) are presented in Figure 14.

BUWM guidelines (WAPC, 2008) and the UWMP guidelines (DoW, 2012) require the following information to be supplied at UWMP stage with regards to subsoil drainage. This information will be provided in the UWMP;

- Subsoil criteria (locations, grade, outfalls, spacing).
- Outline subsoil drainage strategy, including avoidance and management of impacts on water dependent ecosystems and treatment of subsoil drainage water, prior to discharge to the surface water system. The subsoil drainage strategy should also address areas with nutrient-rich groundwater.
- Include subsoil drainage plan with invert levels at control points with likely lot levels.
- Permeability and compaction requirements for imported fill will be provided in the UWMP.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 14 - Geomorphic Wetland



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5.3 Stormwater management

5.3.1 *Stormwater Management Objectives*

The key objectives for surface water management on the Site are:

- Maintain or restore desirable environmental flows and/or hydrological cycles.
- For flood management, manage up to the 1% AEP (100-year ARI event) within the development area to pre-development flows and Water Corporation requirements (as per the Limestone Creek Capacity Review);
- Ensure that the first 15 mm of rainfall receives treatment prior to discharging to a receiving environment (DWER, 2017).
- Minor events should be managed to provide serviceability requirements which have been specified by the City of Kalamunda as:
    - 20%AEP for residential roads, and
    - 10%AEP for Maida Vale Road which is a district distributer road.
- Roads and public open spaces are to be designed to cater for the surface overflow for more severe storm events; and,
- Habitable floors at least 300 mm above the 1% AEP flood level in the local drainage system and 500 mm above the 1% AEP flood level in basins/sumps (where there is no overflow relief) and Poison Gully Creek.

Post development catchments are shown in Figure 15, with a breakdown of respective land uses in Table 14 below.

Table 14: Catchment area breakdown

Land use (ha)	Catchment 1 (North West)	Catchment 2 (North Central)	Catchment 3 (North East)	Catchment 4 (West)	Catchment 5 (South)
High Density Residential	0	3.48	3.56	0	0.51
Mixed Use / Activity Centre	0	2.23	0	0	0
Light Industrial	2.24	0.23	0	2.77	29
Road Reserve	1.21	2.48	0.75	0.8	7.67
POS / Wetland	0	6	0	0.25	1.89
Total Area (ha)	**3.45**	**14.43**	**4.31**	**3.82**	**39.07**

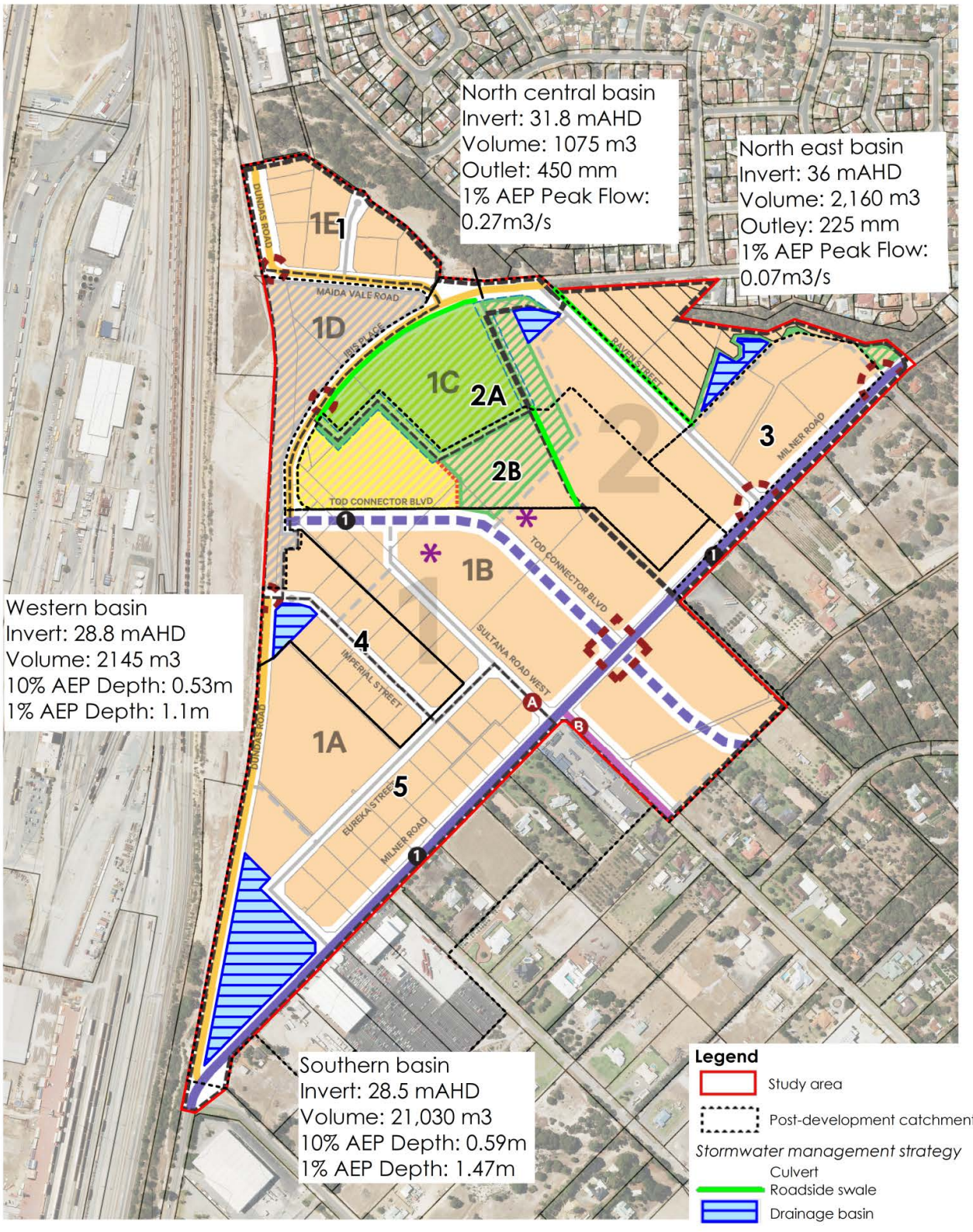
5.3.2 *Small event management*

Development within the Study Area will retain at least the first 15 mm of rainfall within lots and road reserves in accordance with the *Decision Process for Stormwater Management in WA* (DWER, 2017). Frequent events will be managed for the different lot typologies (indicated in Table 14 above) as per the following:

- Residential lots and strata properties to manage the 15 mm event through a combination of soakwells, rainwater tanks and raingardens. Residential development is proposed in Catchments 2, 3, and 5.
- Light Industry, mixed use and activity centres to retain at least the first 15 mm of rainfall within property boundaries. These developments should be encouraged to retain up to the 10% AEP 6 hr event (54.8 mm), using above ground or below ground storage for harvesting of roof runoff for internal re-use (toilets, washing machines) and irrigation of gardens. Light industrial development is proposed within catchments 1, 2, 4 and 5.
- Minor roads will manage the first 15 mm event through a combination of raingardens and swales; and,
- Major roads will manage the first 15 mm event using roadside and median swales. These are proposed for Dundas Road, Ibis Place, Raven Street, and the proposed along local open space within Catchment 2.

The first 15mm of rainfall will receive water quality treatment via the above drainage infrastructure mechanisms and is further discussed in Section 5.4.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 15 - Stormwater management system



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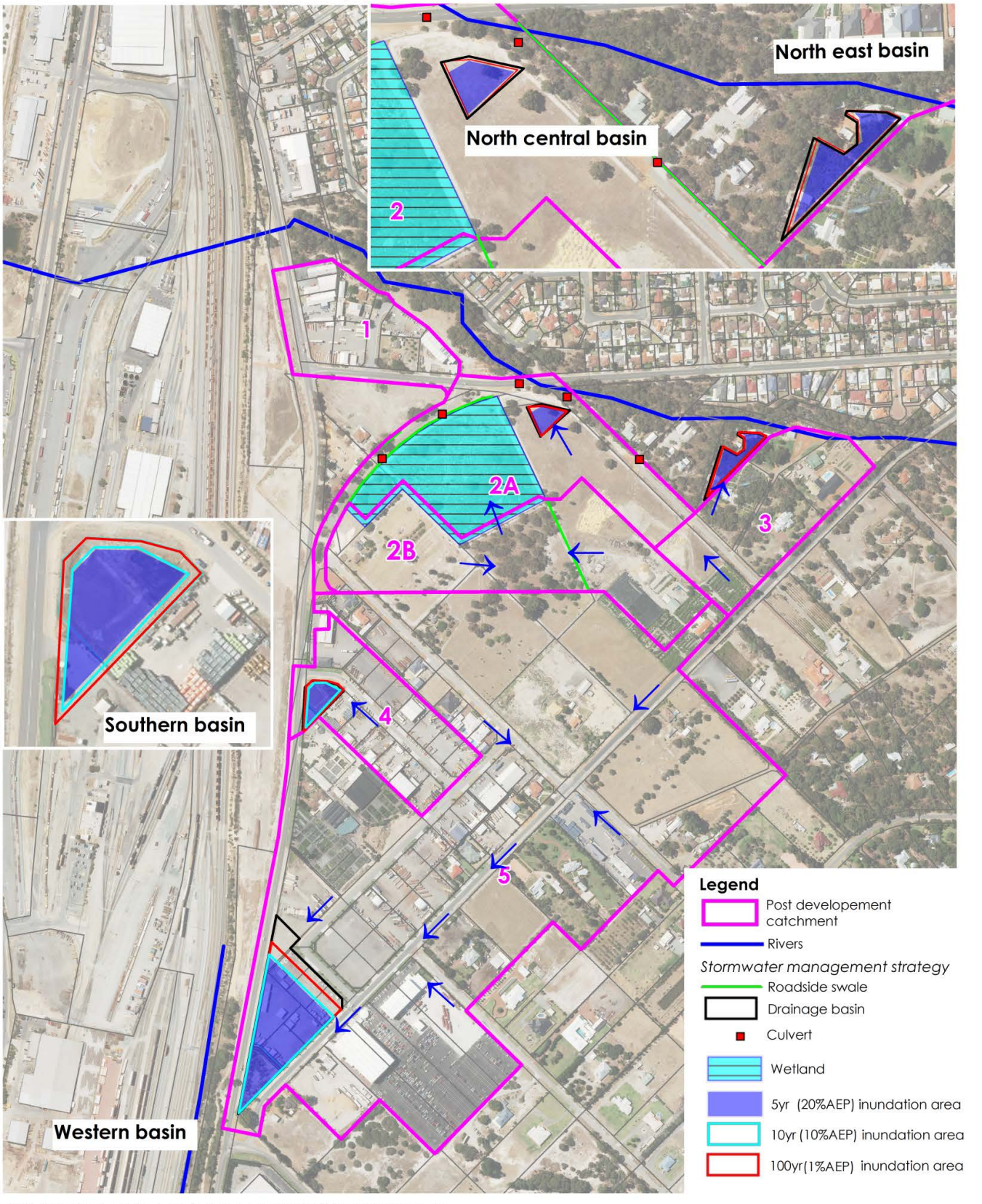
DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 16 - Stormwater event plan



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5.3.3 Minor event management

As detailed in Section 5.3.2, residential and industrial lots will infiltrate the small events onsite in soakwells. Runoff from residential and industrial lots above these events will enter the road drainage system. This runoff has been included in the hydrological modelling (Section 5.3.4) and basins have been sized accordingly to accommodate that runoff.

Conveyance System

Runoff from up to the 20% AEP (~5yr ARI) event which is in excess of the capacity of on-site lot retention systems (soakwells) will be conveyed through a combination of piped drainage and roadside swales into POS and drainage areas.

Surface conveyance (roadside swales) will be used wherever possible on major roads, with piped drains used only where swales cannot be reasonably accommodated in minor roads.

The use of soakage pits through any piped drainage system is recommended to further improve at-source infiltration through the site, particularly in the southern catchments (Catchments 4 and 5).

5.3.4 Major event management

Major flood runoff (1% AEP) will be conveyed via overland flow within the Study Area prior to discharge to drainage areas throughout the development. To size the various components of the system, XP-Storm modelling was completed and confirm discharge rates with respect to pre-development constraints. The requirement to maintain the wetland hydrology was also considered throughout modelling.

Basins and swales have been modelled with side slopes to allow for planting to provide water quality treatment functionality, as discussed further in Section 5.4

DWERs UNDO modelling has been undertaken to assess the water quality treatment effectiveness of the strategy. This is discussed further in Section 5.4.

XP Storm Modelling Method

The XP-Storm model has been developed to assess catchment flows and storages for the 63.2%, 20%, 10% and 1% AEP (approximately 1, 5, 10 and 100 years ARI) storm events based on temporal patterns and rainfall depths provided for the different storm events (BoM, 2019). The storm durations assessed for 63.2%, 20%, 10% and 1% AEPs was 1, 3, 6, 9, 12, 18, 24, 48 and 72 hours, including 10 temporal patterns for each storm event. Assumed losses for each land use are provided in Table 15. A description of each catchment is provided below.

Table 15: Post-development land use losses

Land use	% Impervious	Impervious IL	Impervious CL	Pervious IL	Pervious CL
Catchment 1 (North West)	87%	8	0	16	31.8
Catchment 2 (North Central)	52%	10	0	16	18.9
Catchment 3 (North East)	84%	10	0	16	26.7

Land use	% Impervious	Impervious IL	Impervious CL	Pervious IL	Pervious CL
Catchment 4 (West)	82%	10	0	16	34.8
Catchment 5 (South)	84%	10	0	16	35.8

IL: Initial Loss
CL: Continuing Loss

Catchment 1 – North West (3.45 ha)

Catchment 1 is currently developed (light industry) and is largely impervious. Minimal increases in the post-development runoff are anticipated. The proposed water management, both for minor/major roads upgrades and onsite retention within new light industry will reduce peak runoff and provide improved water quality protection. No additional drainage infrastructure is proposed for this area.

Catchment 2 – North Central (14.43 ha)

The proposed development in this catchment is anticipated to include high density residential areas, a community hub, public open space, and minor and major road reserves. Earthworks associated with the rail precinct and civil engineering design in the Study Area will alter the pre-development catchments.

Catchment 2 discharges to Poison Gully and provides runoff to the wetland. Post-development stormwater management therefore is required to meet the dual criteria of limiting discharge to the creek to pre-development rates and providing sufficient flows to the wetland to maintain the ecological function. The flows into the wetland have been established previously in Section 4.5.3, and is discussed in more detail in the section below.

Poison Gully

The pre-development discharge for Catchment 2 to Poison Gully has been modelled in XP-Storm. The total catchment is 19.1 ha, with approximately 3.1 ha of road reserve. Applying the recommended losses from Water Corporation (Strategen, 2018), a peak discharge rate of 0.27 m^3/s was adopted for the 1% AEP event to the Creek. This equates to 14 L/s/ha, which for comparison is lower than for catchments in the adjacent residential areas to the east (25 L/s/ha and 43 L/s/ha respectively). Erosion control measures are to be put in place at outflow points to prevent adverse impacts to Poison Gully during discharge.

The proposed development in this catchment is anticipated to include high density residential areas, a community hub, public open space, and minor and major road reserves. Earthworks associated with the rail precinct, and civil engineering design in the Study Area, will alter the pre-development catchments. The revised catchment discharging to the wetland is shown in Figure 16.

The opportunities for drainage within the development concept include POS and road reserves, as anticipated buffers for the wetland protection are not suitable for drainage infrastructure, particularly untreated runoff. Planted swales along major and minor roads provide suitable stormwater conveyance and attenuation of stormwater runoff. As shown in Figure 15, swales can be incorporated within the road reserves on non-active frontages for water quality treatment prior to runoff entering the wetland (generally located adjacent to open space, Bush Forever areas and wetlands).

Swales are assumed to be up to 5 m wide (1 m base width, 1:4 side slopes and 0.5 m deep), though a smaller footprint can be determined through detailed design and further site information. The swales will collect runoff from adjacent roads and development areas, providing water quality treatment (planting and amended soil media in some locations, discussed in Section 5.4) for at least the first 15 mm event. Small 375 mm culverts (as shown in Figure 15, allow for the detention of runoff in the swales prior to discharge to Poison Gully.

The other component of the stormwater management system in this catchment is a small detention basin in the local open space on Maida Vale Road. The proposed open space is already cleared and is in a low point in the catchment. Modelling identified that 1,075m^3 of storage is required, with a 450 mm outlet to Poison Gully. The post-development peak outflow from this basin was determined to be 0.27 m^3/s for the 1% AEP event, which is equal to the pre-development discharge rate (0.27 m^3/s). Details of the basin are provided in Table 16.

Table 16: North central basin concept

Parameter		Parameter	
Type	Basin	20% Pre-Dev Outflow	0.12 m^3/s
Side Slopes	1:6	20% AEP Depth	0.82 m
Base Elevation	31.8 mAHD	20% AEP Area	1,078 m^2
Base Area	405 m^2	20% AEP Volume	595 m^3
Top Area	1,456 m^2	20% AEP Outflow	0.12 m^3/s
Total Volume	1075 m^3	1% Pre-Dev Outflow	0.27 m^3/s
Total Depth	1.2 m	1% AEP Depth	1.05 m
Outlet size	450 mm	1% AEP Area	1,302m^2
Outlet U/S invert	32.4 mAHD	1% AEP Volume	868 m^3
Outlet D/S invert	31.9 mAHD	1% AEP Outflow	0.27 m^3/s
Infiltration Rate	1 m/day	Critical Duration	1 hr
Notes	On lot retention and swales within road reserved to convey 1% AEP		

Wetland Runoff

Post-development runoff into the wetland is required to maintain the ecological function and mitigate impacts of reduced rainfall (Section 4.1). The pre-development catchments identified for the wetland include areas of shallow, perched groundwater that contribute to runoff when the catchment is saturated. The proposed post-development catchment for the wetland based on predicted earthworks, includes the Community Hub, POS and light industrial residential area west of Raven Street (Figure 15). Roadside swales will treat the first 15mm from these areas (see Section 5.4 for water quality management) to reduce pollutants entering the wetland, with larger runoff events entering the wetland. The estimated runoff from various events is provided in Table 17, with a comparison to predevelopment runoff volume estimates for various design storm events.

Table 17: Pre- and Post-development runoff into wetland from local catchment

Duration	Scenario	63.2% AEP (1yr ARI)	20% AEP (5yr ARI)	10% AEP (10yr ARI)	1% AEP (100yr ARI)
1 hr	Post-dev	222 m^3	617 m^3	907 m^3	1,919 m^3
	Pre-dev	(374 m^3)	(489 m^3)	(519 m^3)	(752 m^3)
3 hr	Post-dev	216 m3	631 m3	966 m3	2,431 m3
	Pre-dev	(456 m^3)	(638 m^3)	(745 m^3)	(1,250 m^3)
6 hr	Post-dev	161 m3	447 m3	652 m3	2,491 m3
	Pre-dev	(389 m^3)	(711 m^3)	(868 m^3)	(1,513 m^3)
9 hr	Post-dev	92 m^3	347 m^3	518 m^3	2,190 m^3
	Pre-dev	(466 m^3)	(666 m^3)	(915 m^3)	(1,651 m^3)
12 hr	Post-dev	55 m^3	269 m^3	301 m^3	1,715 m^3
	Pre-dev	(485 m^3)	(816 m^3)	(869 m^3)	(1,731 m^3)
18 hr	Post-dev	87 m^3	306 m^3	357 m^3	1,710 m^3
	Pre-dev	(469 m^3)	(738 m^3)	(944 m^3)	(1,858 m^3)
24 hr	Post-dev	86 m^3	247 m^3	307 m^3	1,115 m^3
	Pre-dev	(355 m^3)	(742 m^3)	(841 m^3)	(1,790 m^3)
48 hr	Post-dev	98 m^3	64 m^3	102 m^3	417 m^3
	Pre-dev	(205 m^3)	(223 m^3)	(460 m^3)	(908 m^3)
72 hr	Post-dev	3.73 m^3	24 m^3	73 m^3	174 m^3
	Pre-dev	(36 m^3)	(147 m^3)	(305 m^3)	(490 m^3)

Highlighted are events predicted to contribute a higher volume of runoff to the wetland than predevelopment.

The results in Table 17 indicate there is a significant reduction in runoff volume for frequent events (63.2% AEP, 1 hour) as water is managed within the catchment to provide water quality improvement for this event. Although runoff inflows to the wetland have been reduced, it should be noted that the direct rainfall into the wetland (3.2 ha) remains unchanged following development. The wetland catchment features approximately 5 ha of impervious area with respective losses (Table 15), whereas the predevelopment catchment contributed runoff to the wetland only while soils were saturated. Increased runoff to the wetland is predicted to occur during less frequent events up to 9 hours (1% AEP).

Additional runoff in the wetland could be achieved by reducing the size of roadside swales along Ibis Place (northern boundary of the wetland) or permitting overflow from the north central basin into the wetland (and reconfiguration of the discharge to Poison Gully Creek). The existing swale and basin are designed to contain and convey the 1% AEP event but could be modified to manage smaller events with the larger events discharging into the wetland.

Catchment 3 – North East (4.31 ha)

It is anticipated that the predominant land use in this catchment will be residential. A new drainage basin will be installed adjacent to the river to restrict flows to pre-development discharge rates. Pre-development runoff rates determined using the loss parameters in Table

10, based on a catchment area of 3.1 ha. The post-development catchment is larger at 4.3 ha and includes additional roads and high-density residential areas. The proposed basin configuration is outlined in Table 18, including outflow rates with comparison to pre-development rates. The basin concept is also provided in Figure 15.

Table 18: North east basin concept

Parameter		Parameter	
Type	Basin	20% Pre-Dev Outflow	0.01 m^3/s
Side Slopes	1:6	20% AEP Depth	0.43 m
Base Elevation	36.0 mAHD	20% AEP Area	1,613 m^2
Base Area	1,215 m^2	20% AEP Volume	606 m^3
Top Area	2,460 m^2	20% AEP Outflow	N/A
Total Volume	2,160 m^3	1% Pre-Dev Outflow	0.07 m^3/s
Total Depth	1.2 m	1% AEP Depth	0.98 m
Outlet size	225 mm	1% AEP Area	2,200 m^2
Outlet U/S invert	36.6 mAHD	1% AEP Volume	1,650 m^3
Outlet D/S invert	36.2 mAHD	1% AEP Outflow	0.07 m^3/s
Infiltration Rate	1 m/day	Critical Duration	9 hrs
Notes	Harvesting of stormwater in residential areas may reduce storage requirements and should be confirmed in detailed design.		

Catchment 4 – West (3.82 ha)

The western catchment currently includes a small drainage sump that will be modified with development and be integrated into the proposed POS. The design will include fence removal and minimum 1:6 side slopes to allow access and use of the space by the community. The system currently discharges under Dundas Road to the downstream drainage system. The proposed basin concept is provided in Table 19.

Table 19: Western basin concept

Parameter		Parameter	
Type	Basin	20% AEP Depth	0.39 m
Side Slopes	1:6	20% AEP Area	1,544 m^2
Base Elevation	28.8 mAHD	20% AEP Volume	525 m^3
Base Area	1,158 m^2	10% AEP Depth	0.53 m
Top Area	2,487 m^2	10% AEP Area	1,694 m^2
Total Volume	2,145 m^3	10% AEP Volume	752 m^3
Total Depth	1.2 m	1% AEP Depth	1.1 m
Infiltration Rate	1 m/day	1% AEP Area	2,360 m^2
		1% AEP Volume	1,903 m^3

Parameter		Parameter	
		Critical Duration	9hrs
Notes	On lot retention and swales within road reserved to convey 1% AEP		

Catchment 5 – South (39.07ha)

The southern catchment is the largest within the Study Area and will feature a 1.89 ha POS area between Dundas Road and Milner Road (Figure 3). The current and proposed land use within the catchment is predominantly light industry. As with the western catchment, the existing sump will be modified to create a higher amenity landscape outcome.

The sump will be replaced with a basin that features varied side slopes that will allow some planting. On the western side (adjacent to the rail reserve) side slopes will be 1:3. On the eastern side (adjacent to development) the side slopes will be 1:6 at the bottom of the basin (the bottom metre) and 1:3 for the top 0.5m of the basin.

Road runoff from up to the 10% AEP event will be conveyed through a piped drainage system into the drainage basin, with light industry lots to contain to the 10% AEP onsite where possible. Major flood runoff (1% AEP) will be conveyed via overland flow prior to discharge to POS and drainage areas throughout the development. The basin will be vegetated with locally native, drought tolerant species but will be provided with irrigation to ensure healthy plant growth. Trees will be integrated with the planting to provide shade and to assist in maintaining good infiltration rates. The concept size for this basin is provided in Table 20.

Table 20: Southern basin concept

Parameter		Parameter	
Type	Basin	20% AEP Depth	0.42 m
Side Slopes	Varied (1:3 to 1:6)	20% AEP Area	13,109 m^2
Base Elevation	28.5 mAHD	20% AEP Volume	5,239 m^3
Base Area	11,850 m^2	10% AEP Depth	0.59 m
Top Area	15,700 m^2	10% AEP Area	13,636 m^2
Total Volume	21,030m^3	10% AEP Volume	7,512 m^3
Total Depth	1.5 m	1% AEP Depth	1.47 m
Infiltration Rate	1 m/day	1% AEP Area	15,637 m^2
		1% AEP Volume	20,405 m^3
		Critical Duration	9 hrs
Notes	- On lot retention and swales within road reserves to convey 1% AEP		

5.4 Water quality management

5.4.1 Biofiltration for Quality Treatment

Retention/infiltration of the first 15mm of runoff will limit the mobilisation of pollutants to downstream environments, providing improved water quality outcomes. The retention and

infiltration of runoff close to source through the various systems is supported by the existence of sandy soil and clearance to groundwater within the majority of the Study Area. Separation to shallow groundwater will be created through the importation of fill material in other areas and subsoil drainage may be required to prevent perched groundwater under the system. As discussed in Section 5.2, outflow from the subsoil drainage will be treated prior to discharge to existing waterways. It is also noted that the hydraulic conductivity of less than 1 m/day shall not be considered for soakwell design.

Roadside raingardens, swales and/or bio-retention areas are proposed for managing road runoff. The basins will be planted with nutrient stripping vegetation for water quality treatment where roadside swales are not available for treatment. These systems will be installed with at least 0.5 m of amended soil in accordance with *Adoption Guidelines for Stormwater Bio-filtration* (CRC, 2015). These systems are expected to provide for treatment and infiltration of the first 15 mm of rainfall and provide the opportunity to plant trees and mitigate urban heat impacts.

5.4.2 UNDO Water Quality Modelling

Site specific targets have been proposed for estuarine catchments of the Swan Coastal Plain as a part of background work undertaken by the Department of Water and Environmental Regulation during the development of the Urban Nutrient Decision Outcomes (UNDO) water quality modelling tool. The targets were developed based on consideration of the sensitivity of the receiving water body and the proximity of the development site. Applying this approach, the Study Area would lie outside the suggested 'proximity zones' of the estuarine portion (>1000 m). This would indicate that the following targets could be applied:

- 3.6 kg/ hectare / year of Nitrogen
- 0.2 kg/ hectare / year of Phosphorus

UNDO is a simple empirical decision support model with a flexible framework that evaluates nutrient reduction decisions for new urban developments on the Swan Coastal Plain. It has been developed by the Department of Water to provide urban development proponents with an easy-to-use tool for assessment by local and state government authorities.

An UNDO model has been developed for the Study Area and was run for existing conditions and the proposed redevelopment scenarios. For both scenarios, the Study Area is assumed to be located within Pinjarra soil type with at least 2 m clearance to groundwater.

Results of UNDO modelling, in the form of a report which is generated by the software containing details of all assumptions and inputs, are provided in Appendix 6 (existing conditions) and Appendix 7 (proposed development).

The outcomes of UNDO modelling indicate that on-site retention of the frequent rainfall event within raingardens and soakwells on lots and roadside raingardens will provide discharge loads of:

- 1.15 kg/ hectare / year of total nitrogen
- 0.06 kg/ hectare / year of total phosphorus

These loads are well within the recommended targets for developments within the Swan Coastal Plain that have been developed for a discussion paper as a part of supporting information for the UNDO modelling tool by the Department of Water and Environmental Regulation.

5.5 Management of disease vectors and nuisance insects

The key objectives for management of disease vectors and nuisance insects are:

- Limit the creation of new sites for breeding of nuisance insects.
- Prevent standing water in drainage infrastructure.
- Improve water quality throughout the development.

The construction of above ground water quality treatment systems (raingardens and roadside swales) is proposed within the Study Area. These systems will drain by infiltration through sandy soil with relatively high infiltration rate which minimise standing water times. Subsoil drainage will be installed underneath drainage basins in the northern portion of the Study Area to prevent standing water.

Physical, chemical and biological control methods can be used to manage mosquito populations. Methods which may to be employed (and their order of priority) include:

- Improving water quality, minimising nutrient loads and thereby reducing potential for algal blooms and fish kills; and,
- Should Mosquitos and Chironomid Midges become a nuisance, pesticides (larvicides and/or adulticides) will be used as required to kill mosquito larvae in breeding sites.

Table 21 below displays the approximate calculated emptying time for each basin, following the calculation method outlined by the *Stormwater Management Manual for Western Australia* (based on Engineers Australia (2009) and Argue (2004)). These are all below the maximum 3.5 day (84 hour) emptying time recommended for a 100-year ARI event (1%AEP). A conservative estimate on the infiltration rate has been used to demonstrate a worst-case scenario. For the Northern basins, there is a discharge to Poison Gully at predevelopment flow rates however the calculations below just assume loss through infiltration, so the emptying time is also conservative in that regard as well.

Table 21: Approximate Emptying Times

Basin	Area (m^2) [1]	1% AEP depth	Infiltration rate (m/day) [2]	Emptying time (hours)
North Central	1,456	1.00	1	48
North East	1,450	0.89	1	43
Southern Basin	17,900	1.46	1	70
Western Basin	2,487	1.10	1	53

1 Areas provided are the 1% AEP TWLs.

2 These infiltration rates are the same infiltration rates used to size each basin.

6 IMPLEMENTATION

The success of the water management strategies outlined in this document depends on their implementation through further planning, detailed design, construction and maintenance.

6.1 Urban Water Management Plans

Urban Water Management Plans (UWMPs) are the final water management documents within the state government planning framework outlined in Section 1.1. These documents are prepared as a condition of the subdivision (in support of local development plans) to demonstrate that designs achieve the objectives, strategies and design criteria outlined in this LWMS.

UWMPs should be prepared for consolidated development areas or sub-precincts (Appendix 2) in consultation with the City of Kalamunda and be based on local site investigations appropriate to the proposal and level of risk to water resources. The UWMP should be consistent with the requirements of the DWER's *Urban water management plans: Guidelines for preparing plans and for complying with subdivision conditions* (DoW, 2008b).

Specifically, the UWMP should include detailed engineering and landscaping designs and design of bio-retention systems and non-structural controls measures to manage impacts from construction.

6.1 Monitoring

Pre- and post-development monitoring is discussed below.

6.1.1 Pre-development monitoring

It is considered that there is no value in additional groundwater level monitoring prior to engineering design and further planning. Monitoring data has indicated the maximum groundwater is considered near surface surrounding the wetland, and water quality is reflective of the previous rural land uses. Further groundwater data collected during geotechnical investigation should be considered against monitoring data in this document during engineering design.

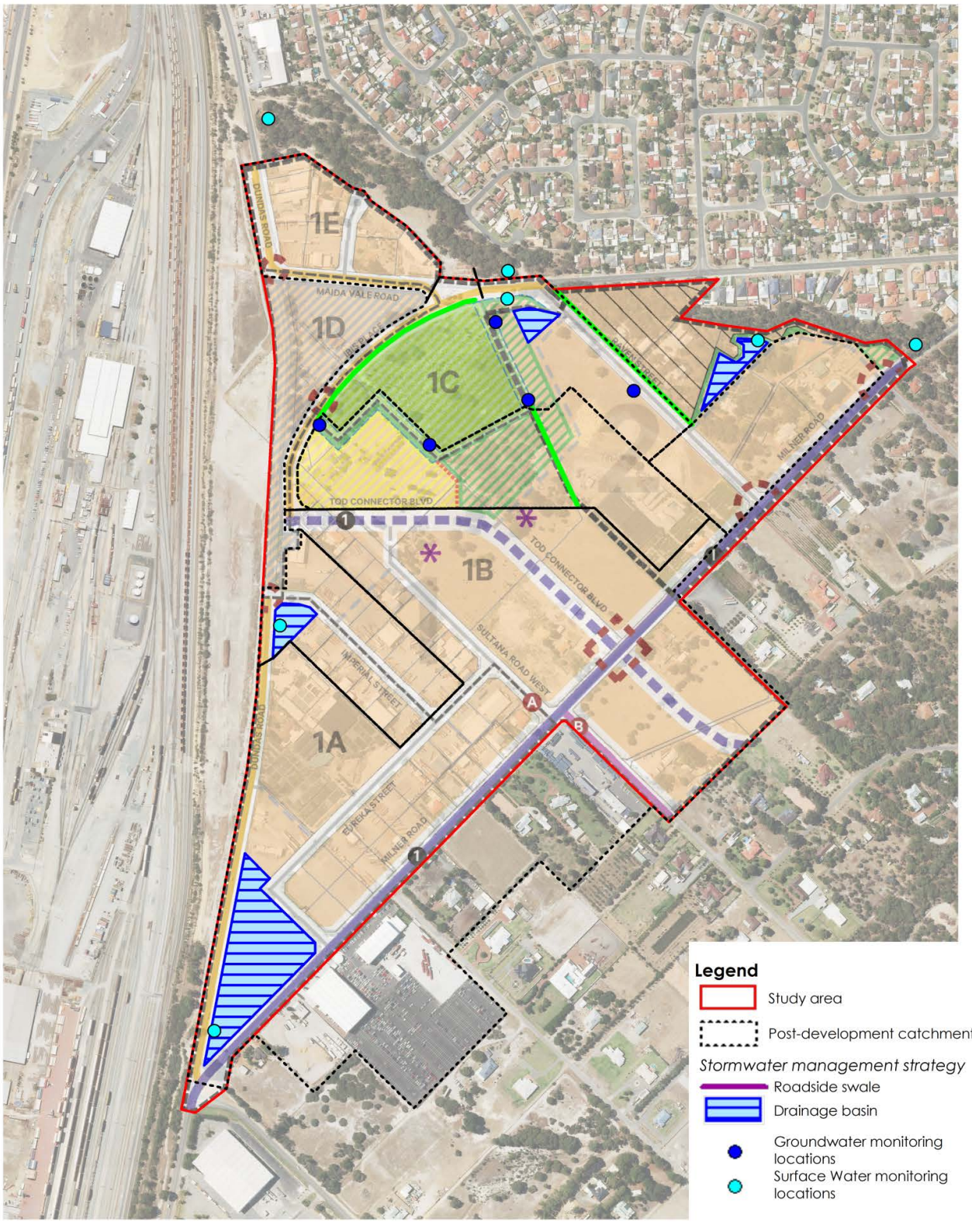
6.1.2 Post-development monitoring

A post-development monitoring program will be implemented and include the following:

Surface water

Surface water monitoring sites at the outlets of the drainage infrastructure and Poison Creek Gully at Milner Road, Maida Vale Road and Dundas Road are recommended to address the key objectives of site-specific post-development monitoring (Figure 17). Surface water sampling should be undertaken during August to October to capture peak winter flows, and in March to capture the first flush events post-summer. Surface water samples should be submitted to a NATA-accredited laboratory in accordance with Australian Standards and analysed for the following parameters:

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 17 - Water monitoring sites



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- In situ pH, electrical conductivity (EC), dissolved oxygen, temperature;
- Total suspended solids (TSS);
- Total nitrogen (TN) and total Kjeldahl nitrogen (TKN);
- Ammonium (NH4);
- Nitrate and nitrite (NOx-N);
- Total phosphorous (TP); and
- Filterable reactive phosphorus (FRP).

Groundwater

Post development groundwater monitoring should include both levels and quality. Monitoring of groundwater levels and the collection of groundwater samples should be undertaken on a monthly and quarterly basis respectively. Groundwater samples should be submitted to a NATA-accredited laboratory in accordance with Australian Standards and analysed for the following parameters:

- In situ pH, electrical conductivity (EC), dissolved oxygen, temperature;
- Total suspended solids (TSS);
- Total nitrogen (TN) and total Kjeldahl nitrogen (TKN);
- Ammonium (NH4);
- Nitrate and nitrite (NOx-N);
- Total phosphorous (TP); and
- Filterable reactive phosphorous (FRP).

The following additional parameters should be included in the laboratory analysis on an annual basis:

- Major anions (chloride, bromide and sulphate);
- Major cations (calcium, magnesium, sodium and potassium); and
- Iron (Fe) and aluminium (Al).

Post-development monitoring will be undertaken for a period of three (3) years. Commencement will be scheduled to include at least one (1) year following completion of the majority (80%) of development. The results of post-development monitoring will be reported to the City of Kalamunda on an annual basis. Indicative monitoring locations are provided in Figure 17, with the focus on shallow groundwater areas surrounding the wetland and in the northern portion of the Study Area.

Trigger values for the site, based on the pre-development monitoring are provided in Table 22. The trigger values are based on the 80th percentile of the pre-development monitoring results, which are considered representative of baseline conditions. Exceedance of these trigger values will require immediate resampling to confirm the results. The response to confirmed and regular exceedances will be to review maintenance practices to identify any shortcomings, followed by inspection and improvement to nearby water quality improvement structures.

Table 22: Trigger values for post-development monitoring

Parameter	Trigger Value	Parameter	Trigger Value
Total Nitrogen	8.00 mg/L	Ammonia	0.10 mg/L
Nitrite and Nitrate	7.50 mg/L	Total Phosphorus	1.12 mg/L
Total Kjeldahl Nitrogen	4.10 mg/L	Filterable Reactive Phosphorus	0.004 mg/L

Note that trigger values have excluded pre-development Nitrogen results from MB08. The median TN value was 21.50 mg/L and skews the trigger values for the remainder of the site. Monitoring in this area should consider the pre-development results in local analysis.

6.1 Construction

Construction activities have the potential to directly and indirectly impact local water resources and water management measures are required.

6.1.1 Licencing

Water will be required for construction activities such as dust suppression. Water for construction purposes could be sourced from groundwater allocation sought in Section 5.1.1

6.1.2 Construction Management

To ensure downstream waterways are protected, developers, builders and landscapers must implement best management practices to control erosion and sedimentation. Contractors and staff should be notified of specific construction management requirements including appropriate disposal of waste material, erosion control and dust suppression.

6.2 Roles and responsibilities

Key tasks, roles and responsibilities relating to delivery of local water management strategies are outlined in Table 23.

Table 23: Summary of roles and responsibilities

Task	Responsibility	Planning stage
Preparation of the UWMP	Landowner/ developer	Subdivision (UWMP)
Assessment / Approval of the UWMP	City of Kalamunda / DWER	Subdivision (UWMP)
Potable water supply planning and connection to main distribution network	Water Corporation	Subdivision (UWMP)
Design of water distribution networks	Landowner/ developer	Subdivision (UWMP)
Water and wastewater planning and connection to main distribution networks	Water Corporation	Subdivision (UWMP)
Design of wastewater reticulation networks	Landowner/ developer	Subdivision (UWMP)
Design of drainage networks including design of water quality treatment areas	Landowner/ developer	Subdivision (UWMP)
Development of detailed Landscaping plan incorporating stormwater management strategies	Landowner/ developer	Subdivision (UWMP)
Confirmation of ongoing management and maintenance requirements and agreement with the City for handover of responsibilities	Landowner/ developer/ City of Kalamunda	Subdivision (UWMP)
Wetland buffer planting/landscaping design	Landowner/ developer/ City of Kalamunda	Subdivision (UWMP)

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APPENDIX 1 LWMS CHECKLIST

Local water management strategy Item	Deliverable	☑	Comments
Executive summary			
Summary of the development design strategy, outlining how the design objectives are proposed to be met	Table 1: Design elements & requirements for BMPs and critical control points	☑	
Introduction			
Total water cycle management – principles & objectives Planning background Previous studies		☑	
Proposed development			
Structure plan, zoning and land use. Key landscape features Previous land use	Site context plan Structure plan	☑ ☑	
Landscape – proposed POS areas, POS credits, water source, bore(s), lake details (if applicable)	Landscape Plan	☑	
Design criteria			
Agreed design objectives		☑	
Pre-development environment			
Existing information and more detailed assessments (monitoring). How do the site characteristics affect the design?		☑	
Site Conditions – existing topography/ contours, aerial photo underlay, major physical features	Site condition plan	☑	
Geotechnical – topography, soils including acid sulphate soils and infiltration capacity, test pit locations	Geotechnical plan	☑	
Environmental – areas of significant vegetation, wetlands and buffers, waterways and buffers, contaminated sites	Environmental Plan plus supporting data where appropriate	☑	
Surface Water – topography, 100-year floodways and flood fringe areas, water quality of flows entering and leaving (if applicable)	Surface Water Plan	☑	
Groundwater – topography, predevelopment groundwater levels and water quality, test bore locations	Groundwater Plan plus details of groundwater monitoring and testing	☑	
Water sustainability initiatives			
Water supply & efficiency measures – private and public open spaces		☑	
Fit-for-purpose strategy and agreed actions. If non-potable supply, support with water balance		☑	
Wastewater management		☑	
Stormwater management strategy			
Flood protection – peak flow rates, volumes and top water levels at control points,100-year flow paths and100 year detentions storage areas	major event Plan Long section of critical points	☑	
Manage serviceability – storage and retention required for the critical 5-year ARI storm events Minor roads should be passable in the 5-year ARI event	minor event Plan	☑	

Local water management strategy Item	Deliverable	☑	Comments
Protect ecology – detention areas for the 1 yr 1 hr ARI event, areas for water quality treatment and types of (including indicative locations for) agreed structural and non-structural best management practices and treatment trains. Protection of waterways, wetlands (and their buffers), remnant vegetation and ecological linkages	small event Plan Typical cross sections	☑	
Groundwater management strategy			
Post development groundwater levels and fill requirements (including existing and likely final surface levels), outlet controls, and any subsoils	Groundwater/subsoil Plan	☑	
Actions to address acid sulfate soils or contamination		☑	
The next stage – subdivision and urban water management plans			
Content and coverage of future urban water management plans to be completed at subdivision. Include areas where further investigations are required prior to detailed design.		☑	
Monitoring			
Recommended future monitoring plan including timing, frequency, locations and parameters, together with arrangements for ongoing actions		☑	
Implementation			
Developer commitments		☑	
Roles, responsibilities, funding for implementation		☑	
Review		☑	

APPENDIX 2 CONCEPT PLAN

LEGEND

Activity Centre Plan Boundary

EXISTING PLANNING FRAMEWORK

Metronet East Redevelopment Area

Project Area: Forrestfield North (RAC-0)

Ecological Conservation (Conservation Category Wetland)

Bush Forever

PROPOSED PLANNING FRAMEWORK

Region Scheme Reserves

Railway

Local Scheme Reserves and Environmental Conservation

Local Open Space

Public Purposes: Community Purpose

Environmental Conservation

Other Categories

District Integrator A (Existing / Proposed)

District Integrator B (Existing / Proposed)

Neighbourhood Connector (Existing / Proposed)

Local Street (Existing / Proposed)

Proposed Roundabout

Proposed Signalised Intersection

A Left-in, Left-out Intersection

B No Right Out Intersection

Strategic Site

Hazard seperation required at subsequent planning stages to ensure that no residential land is affected by BAL-40 or BAL-FZ

Indicative Drainage Corridor

Notes

1 Final road widths and indicative cross sections to be determined following a traffic assessment

Precincts

1 Forrestfield Station Precinct

2 Residential Precinct

Sub-Precincts

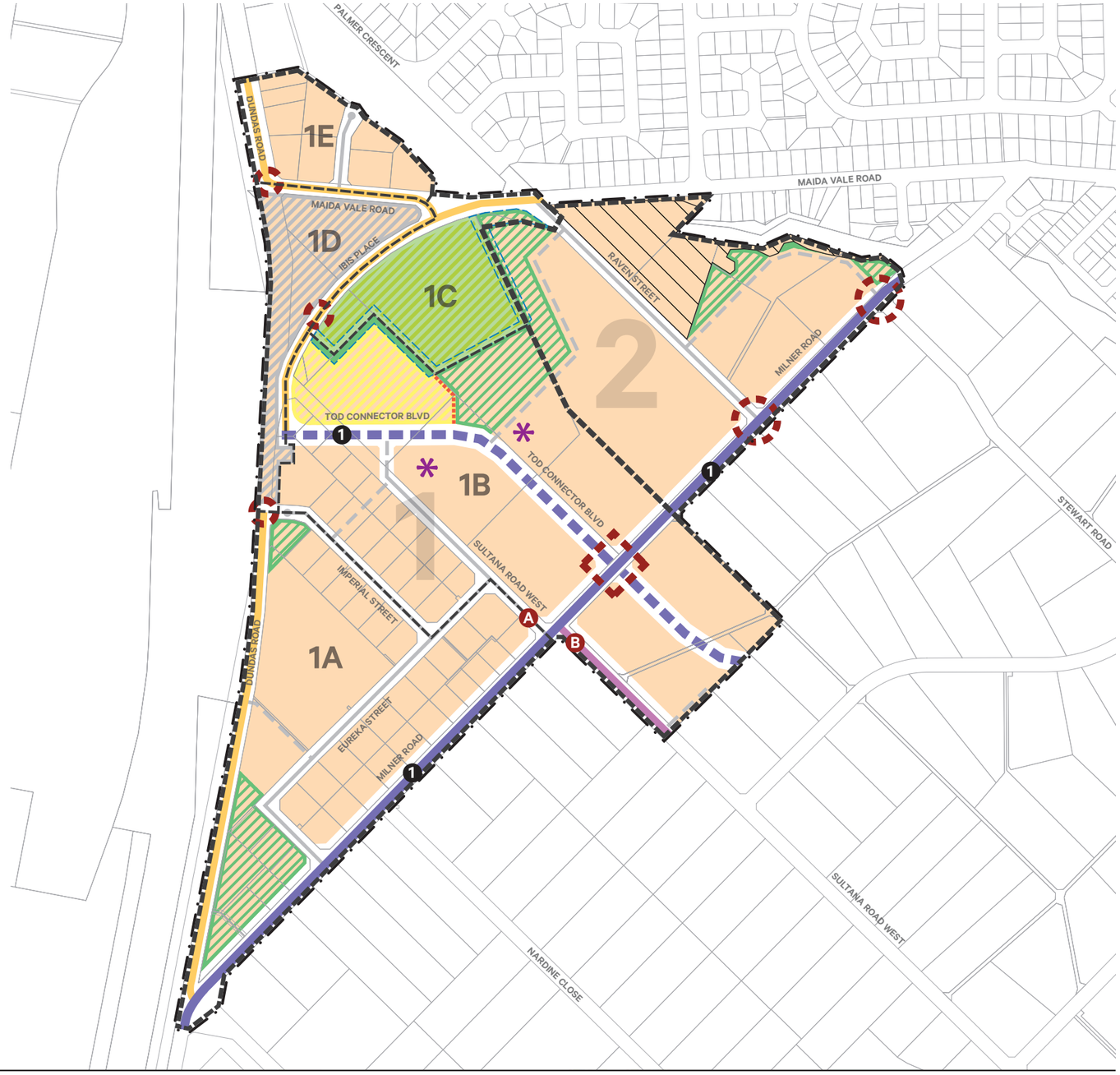
1A Eureka Sub-Precinct

1B Core Sub-Precinct

1C Conservation Sub-Precinct

1D Station Sub-Precinct

1E Maida Vale Sub-Precinct



Activity Centre Plan

Forrestfield North TOD Precinct

DRAFT

Date: 23 Mar 2020 Scale: 1:6000@ A3 1:3000 @ A1 File: **17-527 ST-1 A** Staff: MC GW Checked: MC

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element.

Level 18, 191 St Georges Terrace, Perth Western Australia 6000.
PO Box 7375 Cloisters Square, Perth Western Australia 6850.
T. +61 8 9289 8300 | **E.** hello@elementwa.com.au **elementwa.com.au**

APPENDIX 3 GEOTECHNICAL INFORMATION

Government of **Western Australia**
Public Transport Authority

Forrestfield Airport Link Project

Stage 2 Geotechnical Investigation

Groundwater Conditions Report

Prepared by: Golder Associates Pty Ltd

FAL Document Number: FAL-PTAWA-GE-RPT-00009

May 2015

CONFIDENTIAL

Version control

Version No.	Date	Document Status	Prepared By	Endorsed By
A	17/04/2015	Draft	Golder Associates Pty Ltd	Eric Hudson-Smith
0	26/05/2015	Final	Golder Associates Pty Ltd	Eric Hudson-Smith

Approvals

The signatures below certify that this report has been reviewed, endorsed and approved, and that the signatories are aware of all the requirements contained herein and are committed to ensuring their provision.

Reviewed
……*Signature* *Date* Dave Thomas Delivery Manager

Approved
……*Signature* *Date* Ray Seman Project Director

CONFIDENTIAL

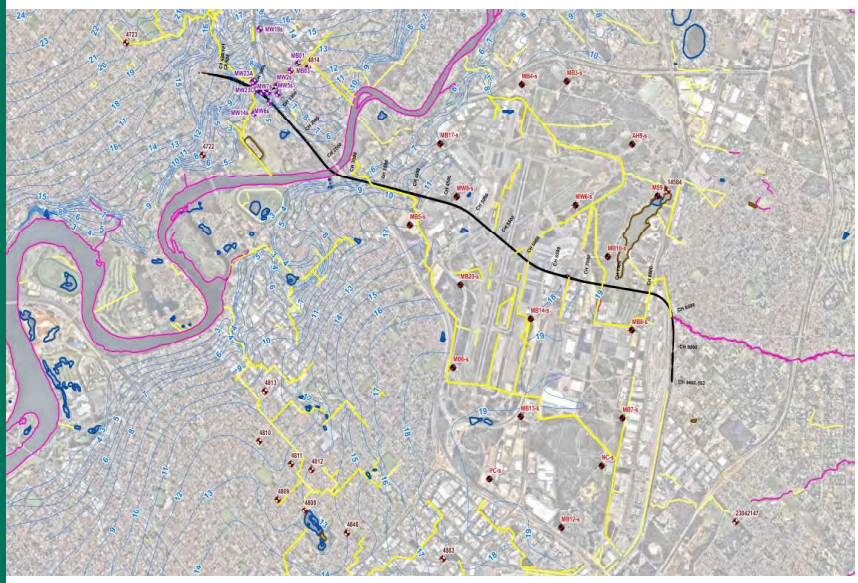
May 2015

FORRESTFIELD-AIRPORT LINK PROJECT, STAGE 2 GEOTECHNICAL INVESTIGATION

Groundwater Conditions Report

FOR INFORMATION

Submitted to:
Public Transport Authority
Public Transport Centre
West Parade
PERTH WA 6000



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REPORT

Golder Associates

Record of Revision

Company	Client Contact	Revision	Description	Date Issued
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List of Abbreviations, Acronyms, Definitions and Units

This list contains abbreviations, acronyms, definitions and units used on this project and in the complete Golder Associates Pty Ltd (Golder) Stage 2 geotechnical investigation report series.

AASS	Actual Acid Sulfate Soil
AF	Ascot Formation
AFa	Ascot Formation – Ascot Beds
AFj	Ascot Formation – Jandakot Beds
a_h	Peak horizontal ground acceleration
AHD	Australian Height Datum
AS	Australian Standard
ASS	Acid Sulfate Soil
ASTM	American Society for Testing and Materials
ATV	Acoustic Televiewer
AWS	Airport West Station
bgl	below ground level
BH	Borehole
BS	Bassendean Sand
$CaCO_3$	Calcium carbonate
CATS	Consolidated Airport Terminal Station
CBD	Central Business District
CBR	California Bearing Ratio
C_c	Compression index
CERCHAR	Laboratoire du Center d' Études et Recherches des Charbonnages de France, Abrasiveness Index
CH	Chainage
CIU	Consolidated, Isotropic and Undrained
CP	Cross Passage
CPT	Cone Penetration Test
CPTU	Piezocone Penetration Test
C_r	Recompression index
CRR	Cyclic Resistance Ratio
CRS	Chromium Reducible Sulfur
C_s	Swelling index
CSR	Cyclic Stress Ratio
c_u	Undrained shear strength (as shown on flat plate dilatometer test reports) = s_u
DBYD	Dial Before You Dig
DCP	Dynamic Cone Penetrometer
DDR	Dry Density Ratio
DIRD	Department of Infrastructure and Regional Development
DMT, D	Flat Plate Dilatometer
DPR	Drilling Parameter Recording

E'	Drained Young's modulus
EA	axial stiffness of tunnel lining
EE	Emergency Egress shaft
EI	bending stiffness of tunnel lining
EPB	Earth Pressure Balance
E_r	Equivalent rock Young's modulus
ERI	Electrical Resistivity Imaging
FAL	Forrestfield-Airport Link
FFS	Forrestfield Station
F_L	Liquefaction triggering factor
FR	Friction Ratio (cone penetration testing)
f_s	Sleeve friction (cone penetration testing)
FWS	Full Waveform Sonic
G_0	Small strain shear modulus
G	Working-strain shear modulus
GF	Guildford Formation
GPS	Global Positioning System
GS	Gnangara Sand
HA	Hand Augered borehole
HCl	Hydrochloric acid
IBC	Intermediate Bulk Containers
I_D	Material index (flat plate dilatometer testing)
I_r	Rigidity index = G/s_u
I_{s50}	Point Load Strength Index (PLI) corrected to 50 mm diameter
ISSMGE	International Society of Soil Mechanics and Geotechnical Engineering
k	hydraulic conductivity (in situ permeability)
k_h	horizontal hydraulic conductivity (permeability)
k_v	vertical hydraulic conductivity (permeability)
k_x	hydraulic conductivity (permeability) in the x direction in the model (= k_h)
k_y	hydraulic conductivity (permeability) in the y direction in the model (= k_v)
K_0	Coefficient of at-rest earth pressure
K_D	Horizontal stress index (flat plate dilatometer testing)
LCPC	Laboratoire des Ponts et Chaussées
LG	Gas monitoring well
M	Constrained modulus (flat plate dilatometer testing)
MASW	Multi-channel Analysis of Surface Waves
MG	Made Ground (Fill)
MIS	Marine Isotope Stage (palaeochannel reference)
MMDD	Maximum dry density/moisture content relationship using modified compactive effort
MRWA	Main Roads Western Australia
MW	Groundwater Monitoring Well
NATA	National Association of Testing Authorities, Australia
NQ	Drilling rod size with rod outside diameter = 69.9 mm
OF	Osborne Formation
OFf	Osborne Formation – Kardinya Shale Member fines dominated

OFm	Osborne Formation – Mirrabooka Member (including Molecap Greensand)
OFs	Osborne Formation – Kardinya Shale Member sand dominated
OLS	Obstacle Limitation Surfaces
P60	Piston sample (inner diameter 60 mm)
PAPL	Perth Airport Pty Ltd
PASS	Potential Acid Sulfate Soil
PCG94	Perth Coastal Grid 1994
PF	Perth Formation
PFc	Perth Formation clay
PFs	Perth Formation sand
PGA	Peak Ground Acceleration
Phi	Friction angle (as shown on flat plate dilatometer test reports) = ϕ'
PLI	Point Load Index = I_{s50}
PP	Pocket Penetrometer
PQ	Drilling core size with core diameter = 85 mm
PSD	Particle Size Distribution
PSP	Perth Sand Penetrometer
PTA	Public Transport Authority
PW	Pumping test Well
q_c	Cone resistance (cone penetration testing)
q_t	Cone resistance corrected for pore pressure at cone shoulder
R	Cone penetrometer radius
R	Return period factor
R_{int}	Interface strength parameter in Plaxis
RL	Reduced Level
RTK	Real Time Kinematic Global Positioning System
SAT	Soil Abrasion Test
SCPT	Seismic Cone Penetration Test (includes measurement of shear wave velocity)
SEM	Scanning Electron Microscopy
SF	Swan River Formation
SMDD	Maximum dry density/moisture content relationship using standard compactive effort
SPOCAS	Suspended Peroxide Oxidation Combined Acidity and Sulfur
SPT	Standard Penetration Test
SRA	Swan River Alluvium
SRB	Sulfate Reducing Bacteria
s_u	Undrained Shear Strength
SWTC	Scope of Works and Technical Criteria
T	Houlsby and Teh (1988) Modified Time Factor (for interpretation of piezocone dissipation tests)
T	Transmissivity
t_{50}	time to dissipate 50% of excess pore pressure (for interpretation of piezocone dissipation tests)
TBM	Tunnel Boring Machine
TDS	Total Dissolved Solids

TP	Test Pit
u	Pore pressure (piezocone penetration testing)
U63	Thin walled tube (inner diameter 63 mm)
UCS	Uniaxial Compressive Strength
Ud	Pore pressure index (flat plate dilatometer testing)
VS	Vane Shear
V_s	Measured shear wave velocity
VWP	Vibrating Wire Piezometer
XRD	X-Ray Diffraction
Z	seismic hazard factor
ν	Poisson's ratio
γ_{sat}	saturated unit weight
γ_{unsat}	unsaturated unit weight
ϕ'	effective friction angle
ϕ_g	geotechnical strength reduction factor
ρ	density
σ_{v0}	total vertical stress
σ'_{v0}	effective vertical stress
σ_{atm}	atmospheric pressure

Units:

°	degree
GPa	gigapascal
K	degrees Kelvin
ka	thousands of years ago
kg/m^3	kilograms per cubic metre
kg/t	kilograms per tonne
km	kilometre
kN/m^3	kilonewtons per cubic metre
kPa	kilopascal
L/s	litres per second
m	metre
m/d	metres per day
m^2/d	metres squared per day
m^2/yr	metres squared per year
Ma	millions of years ago
mg/L	milligrams per litre
mg/kg	milligrams per kilogram
ml/kg	millilitres per kilogram
mm	millimetre
µm	micrometre/micron
MPa	megapascal
ms	millisecond

ppm	parts per million
s	second
t/m^3	tonnes per cubic metre

Summary of soil particle size and soil description in accordance with Australian Standard AS1726-1993:

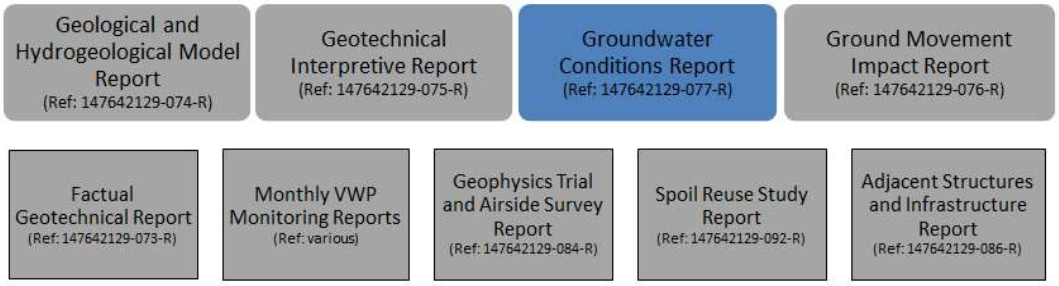
Major Division	Sub-Division	Particle Size	Generalised Material Description
Boulders		> 200 mm	Coarse grained soils with less than 50% smaller than 0.075 mm.
Cobbles		63 to 200 mm	
Gravel	Coarse	20 to 63 mm	
	Medium	6.0 to 20 mm	
	Fine	2.0 to 6.0 mm	
Sand	Coarse	0.6 to 2.0 mm	
	Medium	0.2 to 0.6 mm	
	Fine	0.075 to 0.2 mm	
Silt		0.002 to 0.075 mm	Fine grained soil with greater than 50% of material smaller than 0.075 mm. Material plots below the "A-Line" on plasticity charts.
Clay		< 0.002 mm	Fine grained soil with greater than 50% of material smaller than 0.075 mm. Material plots above the "A-Line" on plasticity charts.

1.0 INTRODUCTION

This report presents the results of hydrogeological interpretive studies carried out as part of Stage 2 Geotechnical Investigation for the proposed Forrestfield-Airport Link (FAL) project. The work was carried out for the Public Transport Authority (PTA) under contract number PTA2014009.

1.1 Related Reports

This report is one of a set of reports that was produced during the Stage 2 geotechnical investigation as shown in Sketch A.



Sketch A: Forrestfield-Airport Link Project - Golder Report Series Breakdown. This report is shown coloured and other reports in the series are shown in grey.

The reports are arranged in the following manner, with a brief overview of the report content provided below as an aid to the reader, although each report should be referred to for full details of the work presented:

1) Primarily interpretive reports:

- The **Geological and Hydrogeological Model Report** presents the geological and hydrogeological conceptual models, descriptions of the geological units and hydrogeological features and interpretation of historic and current investigation data in the form of geological sections.
- The **Geotechnical Interpretive Report** includes interpretation of the subsurface conditions, geotechnical design parameters and key geotechnical risks and mitigation measures.
- The **Groundwater Conditions Report** includes groundwater level data, analysis of hydraulic testing (pumping tests, slug testing and infiltration testing), groundwater modelling results and discussion regarding groundwater control during construction, stormwater disposal and drainage control.
- The **Ground Movement Impact Report** includes analysis and discussion of the effects of construction on adjacent infrastructure, including ground movements from construction activities including any dewatering-related ground movements.

2) Reports containing both factual and interpretive components:

- The **Factual Geotechnical Report** contains the data from the field and laboratory investigations and includes the current monthly Vibrating Wire Piezometer (VWP) Monitoring Report at the date of the factual report.
- The monthly **VWP Monitoring Reports** contain the progressive results of VWP monitoring along the alignment.

- The **Geophysics Trial and Airside Survey Report** contains the results and interpretation of a geophysical investigation trial and airside geophysical survey along a selected part of the alignment.
- The **Spoil Reuse Study Report** contains the results and interpretation of a laboratory testing program aimed at assessing the suitability of construction spoil for reuse as engineered fill.
- The **Adjacent Structures and Infrastructure Report** contains data on existing adjacent structures and major infrastructure.

2.0 PROJECT BACKGROUND

Forrestfield-Airport Link is a State Government project to deliver a rail service to the eastern suburbs. The proposed project alignment is about 8.5 km long, as presented on Figure 1, General Site Location Plan. At the time of reporting the project was planned to include two dive and portal structures, approximately 7.5 km of twin underground rail tunnels between the dive and portal structures, 13 cross passages between the two tunnels, with a permanent egress shaft to the surface at three of the cross passages, and three new railway stations at Airport West, Consolidated Airport Terminal and Forrestfield. Airport West and Consolidated Airport Terminal Stations are planned to be constructed underground. Forrestfield Station is planned to be constructed at-ground together with an associated at-grade parking area.

In preparing this report, reference was made to FAL Project chainages and cross passage and emergency egress shaft locations as shown on PTA Drawing No. 16-C-24-0044, Rev F, Reference Design Plan and Profile, dated 9 April 2015.

The proposed rail line spurs from near the existing at-ground Bayswater Station on the Midland Line, dives underground on the west side of Guildford Road, then generally follows adjacent to and on the south side of Tonkin Highway, passes under the Swan River, deviates under Tonkin Highway towards Perth Airport, runs beneath the airport estate, including both runways, and surfaces and terminates east of the freight rail main lines at Forrestfield.

The current proposed design tunnel configuration involves twin bored tunnels, each with internal diameter of 6.15 m. The separation between the two tunnel centrelines is planned to vary in the range 10 m to 15.5 m, but is typically about 13.5 m. The tunnels will be constructed using a Tunnel Boring Machine (TBM).

The Rev F Reference Design Plan and Profile includes ten cross passages between the twin bored tunnels (CP1, CP2, CP3, CP4, CP4a, CP5, CP5a, CP6, CP6a and CP6b), and a further three cross passages that are incorporated with emergency egress shafts (Emergency Egress Shaft Wright Crescent, Emergency Egress Shaft Domestic Airport Terminal and Emergency Egress Shaft Abernethy Road).

3.0 OBJECTIVES

The objectives of the groundwater conditions studies were to evaluate existing conditions and provide interpretive advice and numerical modelling of construction dewatering for various assumed construction methodologies and design assumptions, based on advice from PTA for possible scenarios, as follows:

- Report the results and interpretation of hydraulic testing carried out during the Stage 2 Geotechnical investigation.
- Provide recommended hydrogeological design parameters.
- Provide recommendations on the design parameters for disposal of stormwater by soakage at Bayswater dive structure, Airport West Station, Consolidated Airport Terminal Station, Forrestfield Station and associated car park.

- Provide advice on possible risks from seasonal and periodic variations and trends to groundwater conditions. Evaluate the maximum design groundwater level for the next 50 years and 120 years at each of the Bayswater dive structure, Airport West Station, Consolidated Airport Terminal Station, Forrestfield Station and Forrestfield car park based on review of historical records and new monitoring data.
- Provide advice on methods of groundwater control during construction, likely dewatering discharge rates, methods of pumped groundwater disposal and constraints or limitations on dewatering activities at each of the Bayswater dive structure, Airport West Station, Consolidated Terminal Station and Forrestfield dive.
- Conduct preliminary 3D groundwater drawdown and recharge modelling at each of the 4 main sites above and show steady state drawdown contours and cross-sections at each structure. Where settlement due to groundwater drawdown is predicted to cause damage that requires repair, suggest means of reducing effects of drawdown.
- Assess the requirements for drainage control measures to protect the integrity of excavations and rail formation and present design concepts, if appropriate. Any requirements for long-term drainage shall be included with appropriate advice on maintenance requirements to satisfy long term effective operation of the drainage system.

4.0 SITE DESCRIPTION

The following sections provide a brief overview of the site conditions including a brief discussion on the geology and hydrogeology. Full details and discussion on the geological model and hydrogeological model developed for the project is provided in the Geological and Hydrogeological Model report (Golder, 2015b).

4.1 Topography and Surface Features

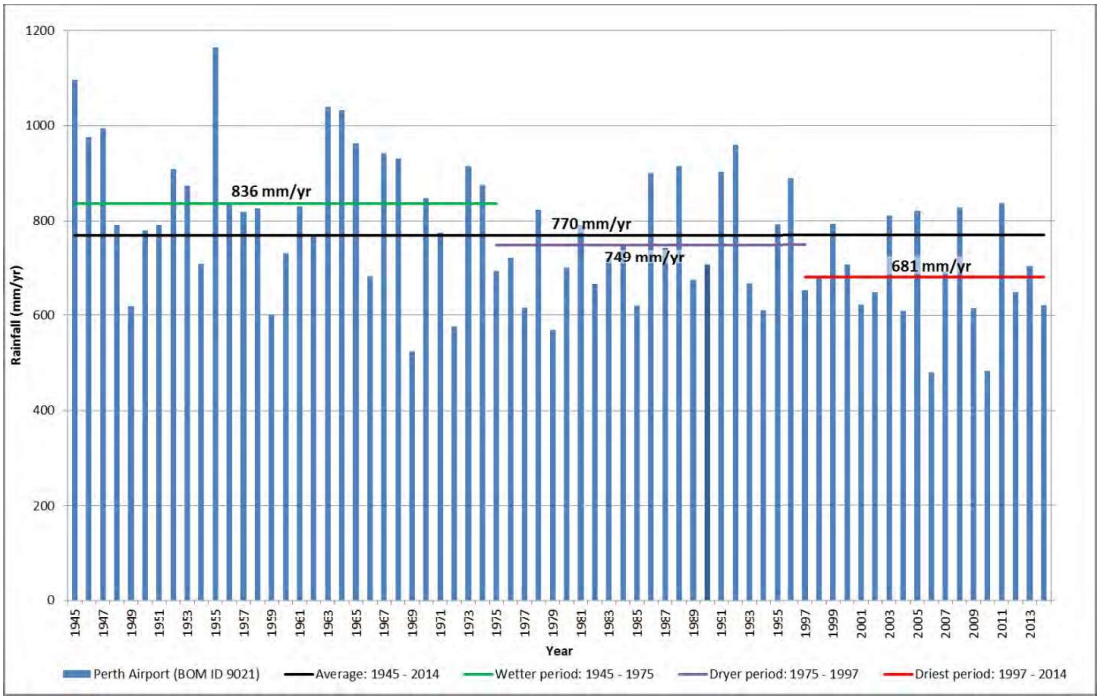
Local topography and surface features are discussed in further detail in (Golder, 2015b). In summary there are three main topographic areas across the proposed alignment as shown on Figure 2. From west to east these include:

- Bassendean Dune System and Swan River Terraces (CH428 to CH3200) – The western end of the proposed alignment has deflated sand dunes of the Bassendean Dune System set alongside alluvial terraces of small creeks and the ancient Swan River. The resulting topography is gently rolling between the crests and troughs of the deflated dune system and eroded river terraces. Overall the elevations are between RL 20 m AHD and RL 25 m AHD at CH428 and gently decreasing to RL 0 m AHD at the Swan River (CH2700). On the eastern side of the Swan River, the elevation quickly rises to between RL 5 m AHD and RL 11 m AHD.
- Bassendean Dune System and Sand Plain (CH3200 to CH7700) – The middle portion of the alignment is largely comprised of sand of the Bassendean Dune System. Over much of this area interdunal depressions previously containing swamps, wetlands and damplands have been infilled during development of the area. In some instances infilling has been completed by excavating local sand for fill. Anecdotal evidence suggests infilling will have modified the current surface level by less than 5 m. Elevation gently increases from approximately RL 8 m AHD at CH3200 to approximately RL 20 m AHD at CH7700.
- Pinjarra Plain (CH7700 to CH9492) – The eastern end of the alignment is located on the Pinjarra Plain which generally comprises alluvial fan deposits extending out from the Perth Hills. Across the alignment a thin layer (1 m to 2 m thickness) of Bassendean Sand covers most the alluvial fan deposits. The terrain begins to rise from approximately RL 20 m AHD at CH 7700 up to RL 30 m AHD by CH8600. As with many geomorphological features in the Perth area, the Pinjarra Plain tends to have an overall north to south alignment. This means the portion of the proposed alignment that runs approximately north to south between CH8600 and CH9492 has a relatively constant elevation of approximately RL 30 m AHD.

4.2 Climate

Sketch B shows the annual rainfall from the Bureau of Meteorology rainfall station (BOM ID 9021) located at the Perth Airport for the period from 1945 to 2014 and indicates the following:

- The average annual rainfall from 1945 to 2014 is 770 mm/yr.
- Perth experienced a wetter period between 1945 and 1975 with an average annual rainfall of 836 mm/yr.
- Since 1975 it has experienced a drying climate with an average from 1975 to 1997 of 749 mm/yr.
- The average annual from 1997 to present is only 681 mm/yr.



Sketch B: Annual Rainfall from 1945-2014 (BOM ID 9021 – Perth Airport)

4.3 Geology

Full details of the geological model and detailed description of geological units relevant to the project are provided in the Geological and Hydrogeological Model Report (Golder, 2015b). A summary of the identified relevant geological units is provided in Table 1. A conceptual geological model along the alignment (in long section) is provided on Figure 2.

Table 1: Summary of Project Specific Geological Units

Geological Unit		Typical Description	Depositional Environment	Approximate Age	Diagenesis
Name	**Unit Colour**				
Fill (MG)		Primarily fine to medium grained yellow to brown sand. Also road base and other types of fill	Controlled and uncontrolled sand fill in recent history.	<200 years	Some cementation of limestone road base may have occurred.
Swan River Formation (SF)		Silty clay, clayey silt, silty sand and sand, dark grey, dark brown to black with shells and organic material, low to high plasticity fines, very soft to firm, or very loose to loose.	Palaeochannel deposits in marine, estuarine and fresh water conditions.	0 to 30 ka	Natural consolidation of this unit may have been enhanced by construction of the Redcliffe Bridge approach embankments
Perth Formation (PF)	PF / PFs	PFs: Sand to silty sand, yellow, brown and grey, fine to coarse grained, medium dense to dense. Some fine to coarse gravel, generally rounded to sub-rounded quartz, may be present.	Palaeochannel deposits in marine, estuarine and fresh water conditions.	80 to 150 ka	Some local pockets (up to approximately 2 m thick) of very weakly cemented (non-carbonate) sand may be present at the contact with the overlying Bassendean Sand.
	PF / PFc	PFc: Clay, some silt and sand, blue-grey, mottled red, yellow, brown, generally low to medium plasticity and generally stiff to hard some soft to firm materials close to the Swan River.			
Bassendean Sand (BS)		Sand, light grey, yellow, dark brown, fine to medium grained, loose to dense, fining upwards where fluvial in origin, with thin (up to 1 to 2 m) localised iron cemented layers. May contain peaty sand, silty and clay associated with wetland or dampland interdunal deposits.	Mixed fluvial and aeolian origin.	80 to 750 ka	Localised iron-cementation has occurred in this unit creating pockets and layers that can be up to approximately 2 m thick of very weakly to moderately well cemented sandstone ("coffee rock").
		transitional boundary / facies change			
Guildford Formation (GF)		Clayey sand, silty sand, sand and clay, brown, pale grey, orange, fine grained layers stiff to hard and low plasticity, coarse grained layers are medium dense to very dense. Includes potential sand deposits of the Yoganup Formation east of the RAC driving centre (~ CH7600).	Fluvial origin generally deposited as part of alluvial fan system. Yoganup formation is a littoral (coastal) deposit.	170 ka to 2 Ma	Aside from natural consolidation no other extensive diagenetic processes have been noted.
		transitional boundary / facies change			
Gnangara Sand (GS)		Sand and silty sand, blue-green, dark green, fine grained, loose to dense.	Freshwater and marine deposit in nearshore coastal environment.	170 ka to 2 Ma	Alteration of materials at contact with the underlying unit may have led to reduction in strength at the contact (e.g. decomposition of organics).
Ascot Formation (AF)		Ascot Beds (AFa): Carbonate sandy gravel, gravelly sand and sand, fine to medium grained sand, grey, yellow, medium dense to dense, some siliceous calcarenite layers.	Inner shelf, nearshore marine environment, deposited during multiple marine transgressions.	750 ka to 3 Ma	This unit appears to have undergone carbonate cementation since deposition of the original sediments. Subsequent leaching may have created small voids and cavities that have now been infilled with sand. Recementation of previously leached sand and gravel layers may still be occurring.
		Jandakot Beds (AFj): Carbonate sand, sandy gravel and gravelly sand, fine to coarse grained, grey, dark grey and blue-grey, medium dense to very dense, some siliceous calcarenite. Polished rounded black phosphatic gravel often found at the base. High strength conglomerate boulders may also be present at the base of this unit but have only been noted in the vicinity of the Perth Airport air traffic control tower to date.			
		major unconformity			
Osborne Formation (OF)	OFm	Mirrabooka Member (OFm): Sand, silty sand and clayey sand, dark green to dark grey, medium to coarse grained, dense to very dense, siliceous and glauconitic. Includes the Molecap Greensand.	Shallow-marine origin.	100 to 120 Ma	Potential secondary formation of pyrite may have occurred.
		facies change			
		Kardinya Shale Member sand dominated (OFs): Silty sandstone, fine-grained, layered, dark green and black, moderately weathered to fresh, extremely low to medium strength.			
		facies change			
		Kardinya Shale Member fines dominated (OFf): Sandy mudstone and sandy siltstone, black and dark green, moderately weathered to fresh, extremely low to medium strength.			

Notes:

- This table is intended to represent a basis for broad geological subdivision of the geology and is not the basis of engineering characterisation. Indications of consistency and relative density in the table are generally typical for the units and do not necessarily represent the full range of values encountered. Not every permutation of material grading is provided. Rock materials may exist in units that are principally soils and vice versa and not all diagenesis processes may have been identified.
- The carbonate and non-carbonate soil and rock classification systems used for this study are defined in Appendix B of (Golder, 2015a).

City of Kalamunda
899
Ordinary Council Meeting 12 October 2021 Attachments
Attachment 10.5.2.3

5.0 HYDROGEOLOGICAL CONDITIONS

5.1 Definitions

The following definitions have been adopted in this report:

- Groundwater level means the level at which the pressure in the pore water in the subsurface material is atmospheric. The terms groundwater level, phreatic surface, groundwater table and water table may be used interchangeably. If perched groundwater conditions exist there may be more than one groundwater level at any particular plan position.
- Piezometric pressure means pore water pressure measured at a point below ground level.
- Piezometric level means piezometric pressure referenced to the level in metres AHD that water would rise to in a standpipe installed with a porous tip registering the pressure at a point below ground.
- Perched groundwater means a saturated lens of water supported by a low permeability unit that occurs above the regional groundwater table (refer to Figure 3). The extent of the perched water table depends on the extent and continuity of the low permeability unit.

5.2 Assignment of Hydrogeological Units

A conceptual hydrogeological model showing the main aquifers, regional groundwater and piezometric levels and flow directions is shown on Figure 3. A summary of the hydrogeological units in the model is provided in Table 2. The hydrogeological units have been split into two zones; East and West, based on the regional geology. Figure 3 shows that the West Zone extends to east of the Swan River where the Perth Formation terminates and the East Zone extends east from this point.

Table 2: Summary of Project Specific Hydrogeological Units

Zone	Regional Unit	Hydrogeological Unit	Local Aquifer Type	Geological Units Included
West (CH428-CH3200)	Superficial Aquifer	Upper Aquifer	Unconfined	MG, BS, PFs
		Bayswater Aquitard	Aquitard	PFc
		Lower Aquifer	Confined (west of Swan River) Semi-confined (east of Swan River)	PFs
	Osborne Aquitard		Aquitard	OF (OFs)
East (CH3200-CH9493)	Superficial Aquifer	Bassendean Sand	Unconfined	MG, BS
		Guildford-Gnangara	Semi-confined	GF, GS
		Ascot Formation		AF
	Mirrabooka Aquifer		Semi-confined Aquifer	OF (OFm)
	Osborne Aquitard		Aquitard	OF (OFf)

Notes: This table is intended to represent a basis for broad hydrogeological subdivision of the geology and is not the basis of engineering characterisation.

The two main regional aquifers of relevance beneath the proposed alignment are the Superficial Aquifer (present in both the east and west zones) and the Mirrabooka Aquifer (present in the east zone only). Along the proposed alignment the Superficial Aquifer comprises all the materials overlying the Osborne Formation. In the west zone, the Superficial Aquifer comprises an Upper and Lower Aquifer separated by an aquitard (PFc). The Superficial Aquifer includes seven of the main geological units listed previously, which all have unique hydrogeological properties. The Superficial Aquifer and the Mirrabooka Aquifer is in direct hydraulic connection.

5.2.1 West Zone

Within the West Zone, groundwater of relevance to the project is largely contained within the sandy dominated materials of the Perth Formation and the overlying Bassendean Sand. However, the occurrence of fines dominated sediments within the Perth Formation (termed the Bayswater Aquitard in Table 2) separates the water bearing materials into two aquifers. Differences in groundwater levels measured within these two aquifers substantiate the presence of these two aquifers and this is discussed in more detail in Section 5.3.5. For the purpose of this report the two aquifers have been named the Upper Aquifer and Lower Aquifer. The Lower Aquifer is underlain by the Osborne Aquitard (KSc).

5.2.2 East Zone

Within the east hydrogeological zone all materials overlying the Osborne Formation are part of the regional Superficial Aquifer. However, the materials have been separated into distinct hydrogeological units (BS, GF, GS and AF) due to variations in their hydrogeological characteristics. Clayey/silty soils within the Guildford Formation and Gnangara Sands are expected to be responsible for creating semi-confined conditions. Underlying the Ascot Formation over part of the east zone is the semi-confined Mirrabooka Member (Mirrabooka Aquifer), which is in direct hydraulic connection with the Ascot Formation. All materials are underlain by the Osborne Aquitard (KSc).

5.3 Groundwater Levels

5.3.1 Published Information

The Perth Groundwater Atlas (WRC, 1997), which presents the inferred maximum historical groundwater level contours, covers the alignment between CH428 and CH4250 and at CATS (refer to Figure 4). Approximate groundwater levels from the 1997 Groundwater Atlas for each main structure are summarised below:

- Bayswater Dive Structure (at portal) – between RL 5 m AHD and RL 6 m AHD (contours suggest there has historically been a surface drain at the portal.
- Airport West Station – between RL 10 m AHD and RL 11 m AHD.
- Consolidated Airport Terminal Station – RL 19 m AHD.
- Forrestfield Dive Structure – No coverage.

It should be noted that the 1997 Groundwater Atlas in some areas presents the groundwater level prior to installation of some of the drains that are used to control groundwater levels in built-up areas. Therefore, the maximum historical groundwater levels shown in the 1997 Groundwater Atlas may not represent the possible maximum future groundwater levels.

5.3.2 Historical Trends

Historical groundwater level trends for the Superficial Aquifer were evaluated using data obtained from the Department of Water (DoW) WIN database, Perth Airport Pty Ltd (PAPL) and CSBP investigation (Parsons Brinckerhoff, 2013). Figure 4 shows the location of selected monitoring wells with available hydrographs while Appendix A presents the hydrographs. The historical groundwater level data indicates:

- A step-decline in groundwater levels occurred in the Belmont area in the late 1950's (e.g. WIN ID 4809) and 1960's (e.g. WIN ID 4883) which is due to installation of drains to lower the groundwater level to allow for land development.
- Steady groundwater levels between the mid 1960's and the present day.
- The Superficial Aquifer along the alignment is full and surface drains generally constrain the maximum groundwater level in the area.

- Although annual rainfall totals indicate a drying climate (Section 4.2), rainfall (and subsequent recharge) experienced in recent years is still sufficient to fill the aquifer resulting in flow into the surface drains that constrain groundwater levels to their maximum local height. A decline in groundwater levels due to change in climate is therefore not observed in this area yet.

5.3.3 Seasonal Fluctuations

Seasonal groundwater level fluctuations along the alignment were assessed by reviewing the following hydrographs (refer to Figure 4 for location of wells):

- DoW hydrographs (data from the 1950's to present).
- PAPL hydrographs (data from 1999 to present).
- CSBP hydrographs (Parsons Brinckerhoff, 2013) (data from 2003 to 2011)
- Vibrating wire piezometer (VWP) data (Golder, 2015i) from instruments installed by GHD (GHD, 2014c) during the Stage 1 geotechnical investigation (data from one season only 2014-2015).

A summary of the seasonal fluctuation data is provided below:

- West Zone:
  - Upper Aquifer – 0.7 m to 1.1 m
  - Lower Aquifer – 1.2 m to 1.6 m (based on 2014/2015 hydrographs from BH 0-02, BH 0-03 and BH 0-08 only).
- East Zone:
  - Superficial Aquifer – General range between 0.6 and 1.6 m with an average around 1.2 m, but at BH 0-07 in Forrestfield the seasonal variation is observed to be 3.1 m (Golder, 2015i). This larger seasonal fluctuation is expected to be due to the predominance of clayey and lower permeability soils close to the ground surface that make up the Guildford Formation in this area, which is also referenced in published information (Davidson, 1995). VWP readings from BH1-02, BH1-04 and BH1-07 show similar seasonal declines as BH0-07, suggesting that the 3 m seasonal variation could exist over the whole Forrestfield Area.
  - Mirrabooka Aquifer – no seasonal data is currently available, but based on the available readings from VWPs installed in the Mirrabooka Aquifer (BH1-22 and BH1-24), the piezometric levels show similar trends to piezometers installed in the Superficial Aquifer at CATS. The seasonal variation would therefore be expected to be similar to the seasonal variations in the Superficial Aquifer (the maximum seasonal variation observed in MB14s is 1.2 m).

5.3.4 March 2015 Levels

Groundwater levels were measured in 95 monitoring wells (installed as part of the geotechnical and environmental investigations for the FAL project) on 10 March 2015. Appendix B provides the groundwater and piezometric level measurements for the monitoring wells and the VWPs while Figures 5 shows the monitoring locations and the values measured during the March 2015 groundwater level survey. The inferred approximate groundwater and piezometric levels for each dive and station location are summarised below:

- Bayswater Dive Structure, extending from the shallowest to deepest section of the proposed dive structure, respectively:
  - Upper Aquifer – RL 10 m AHD to RL 5 m AHD
  - Lower Aquifer – RL 4 m AHD to RL 1.5 m AHD (east of the Bayswater Main Drain).
- Airport West Station – RL 8.2 m AHD to RL 8.8 m AHD

- Consolidated Airport Terminal Station – RL 16.8 m AHD to RL 17.0 m AHD
- Forrestfield Dive – RL 23 m AHD.

5.3.5 Multi Aquifer System (West Zone)

Groundwater levels measured in the Bayswater Dive area indicate a significant downward vertical hydraulic gradient between the Upper and Lower Aquifers (head difference of 3.7 m at the Bayswater Dive Portal). This difference in levels indicates that the Upper Aquifer in this area is "perched" above the Bayswater Aquitard which is therefore likely to have a very low hydraulic conductivity. It is possible that the head difference between the two aquifers could be further exacerbated as a result of potential pumping from the Lower Aquifer.

Based on the current monitoring locations the difference in groundwater level between the two aquifers is most distinct along the alignment between the Bayswater Dive Structure east toward CH1800. There is a change in groundwater level of 3.8 m between CH1800 and CH2200, after which there is insignificant difference in groundwater/piezometric levels between monitoring wells screened in the shallow and deep portions of the aquifer, suggesting that there is no longer a significant multi-layered aquifer system east of this position. The reason for an apparent merging of the multi-layer aquifer system is likely associated with both a thinning of the Bassendean Sand and intersection of the groundwater table with the Perth Formation east of approximately CH1800.

5.3.6 Perched Groundwater Levels

5.3.6.1 West Zone

Perched groundwater is unlikely to be present in this zone between CH428 and CH1800 because the groundwater level in the Upper Aquifer is already close to the ground surface in many places and the unsaturated soil mainly consists of Bassendean Sand. However, although not encountered during this investigation, the presence of localised coffee rock that may cause locally perched groundwater conditions cannot be ruled out.

Between CH1800 and CH2330 (west of Swan River) and between CH2700 and CH3200 (east of Swan River) the groundwater table is located in the Perth Formation, with overlying unsaturated Bassendean Sand. It is possible that perched groundwater conditions could form during the wet season above the contact of the Bassendean Sand with the underlying clay dominated Perth Formation (PFc).

5.3.6.2 East Zone

Groundwater levels near the proposed Forrestfield Dive structure and station indicate the presence of perched groundwater conditions in the Superficial Aquifer (Guildford Formation). Insufficient groundwater level data is currently available to identify whether these perched conditions are seasonal or persistent throughout the year. A summary of groundwater levels from March 2015, which indicated perched conditions, is provided below:

- MW 3-044 (s) (RL 23.2 m AHD) and MW 3-044 (d) (22.9 m AHD) located together adjacent to the dive structure – head difference of approximately 0.3 m.
- MW 3-060 (s) (RL 23.9 m AHD) and MW 3-060 (d) (RL 22.4 m AHD) located together adjacent to Abernethy Road at CH8100 – head difference of approximately 1.5 m.
- MW3-019 (RL 26.2 m AHD) located approximately 125 m east of MW3-026 (RL 22.8 m AHD) and MW3-012 (22.9 m AHD) – head difference of approximately 3.3 m.
- MW3-004 (RL 24.3 m AHD) and MW3-009 (RL 24.5 m AHD) located in between MW3-001 (RL 23.3 m AHD) and MW1-05 (RL 23.4 m AHD) – head difference of between approximately 1.0 m and 1.2 m.

Given that the level of perching is locally highly variable, it is likely that these perched conditions are caused by localised and discontinuous bands/layers of low permeability clay or coffee rock within the Guildford Formation soils.

5.4 Groundwater Recharge

Rainfall and stormwater runoff, via soakage basins and stormwater drains, are considered to be the major contributors to groundwater recharge for all water bearing hydrogeological units of relevance to the proposed alignment. This relationship is somewhat indirect for the Ascot Formation and Mirrabooka Member as these units receive recharge via downward leakage from the overlying Bassendean Sand and Guildford Formation where direct rainfall recharge occurs.

5.5 Groundwater Discharge

5.5.1 Surface Water

The Swan River crosses the alignment at around chainage CH2700 and ultimately acts as the main surficial discharge point for the majority of groundwater of relevance to the construction of the project. Discharge to the river is evident by the observed upward gradient in monitoring wells at the Swan River as well as the observed hydraulic gradient towards the river (Figure 4).

The following summarises other surface water bodies in the vicinity of the alignment:

- Gobba Lake is located on the western side of the Swan River approximately 130 m south-west of the alignment at about chainage CH1750. This lake has existed as a swamp/wetland since before development in the area (Golder, 2015b) and exists due to a local topographic low point (< RL 4 m AHD). This surface water body is understood to be continually inundated and therefore is expected to act as a groundwater discharge point with evaporative losses occurring predominantly during the summer months and potentially recharging the aquifer locally during the winter months.
- Munday Swamp is located approximately 150 m north of the alignment at approximate chainage CH7550. The area comprises a local topographic low point (< RL18 m AHD) explaining the presence of Munday Swamp and the seasonally flooded terrain north of the Manheim vehicle auction yard and the RAC Driving Track. This area is expected to act as a discharge point for groundwater, particularly through evaporation directly from the lake surface during summer months and where the groundwater level is high enough to reach the surface and cause inundation in the surrounding damp lands. The likely presence of low permeability soils below Munday Swamp is likely to limit any recharge to the aquifer from the swamp.
- Poison Gully Creek crosses the proposed alignment at CH8550 and was realigned in the late 1960's or early 1970's during construction of the Forrestfield Rail Yard. This creek is ephemeral and discharges into a constructed drain (part of the Perth Airport drain system) after crossing beneath Dundas Road.

5.5.2 Drains

Groundwater levels across the proposed alignment are affected by the drainage network constructed for various residential and commercial developments. The two main drains along the alignment include the Bayswater Main Drain and Airport Main Drain/Belmont Main Drain, both of which empty into the Swan River. However, there is a wide network of surface and subsurface drains present that feed into both of these drains which help control groundwater levels, particularly in former wetland areas present beneath the airport estate and over the eastern zone of the proposed alignment:

- The alignment crosses the Bayswater Main Drain at approximately CH1000. Review of historical aerial photography (Golder, 2015c) shows this drain was once a small creek and a topographical low for the area that drained wetlands located to the north, west and east. Although the drain still completes this function many of the creeks and wetlands have been infilled or piped underground to facilitate development.
- The Belmont Main Drain runs parallel (approximate offset of 25 m) from about CH3375 to CH4000
- The Perth Airport drain system crosses the alignment at about CH4700, CH5860, CH6080, CH7280, CH8030 and CH8400.

5.5.3 Local Groundwater Users

Groundwater is abstracted along the alignment from all aquifers identified in this report, which is likely to be mainly for reticulation (household and business) and industrial supply. The effect of pumped abstraction on groundwater can be seen in VWP data (Golder, 2015i) as summarised below:

- The majority of the VWPs show daily to twice daily groundwater level drawdown of up to 0.5 m along most of the alignment except in the Forrestfield area (up to CH8100 only). These drawdowns only last for a few hours, suggesting that these responses are due to reticulation bores being turned on.
- The greatest drawdown from pumping is observed between CH5000 and CH6000 at Perth Airport (BH0-01, BH0-05, BH0-09 and BH0-10) where drawdown ranges between 0.5 m and 1.5 m with greatest drawdown occurring in the Ascot Formation. The drawdown pattern is more irregular and can last for up to 2 weeks followed by no pumping for weeks or months. The drawdown pattern is more regular (weekly) during the dry season and less frequent during the wet season.
- None of the monitoring locations in the Upper Aquifer at Bayswater have to date shown an effect from pumping.

The observed effect from local groundwater abstraction (other than the pumping tests carried out as part of this investigation) in the vicinity of the four main structures is as follows:

- Bayswater Dive – VWP data (BH0-02, BH2-22 and BH2-19) indicates daily groundwater level drawdown responses between 0.1 m and 0.5 m in the Lower Aquifer with a maximum response of up to 1 m in BH2-22 at the end of December 2014.
- Airport West Station – VWP data (BH0-04, BH2-05 and BH2-11) indicates daily and in some cases twice-daily drawdown response of up to approximately 0.3 m in the Guildford, Ascot and Osborne Formations.
- Consolidated Airport Terminal Station – VWP data (BH0-06, BH1-22 and BH1-24) indicates daily drawdown responses of around 0.1 m in the Guildford Formation, 0.25 m in the Ascot Formation and less than 0.05 m in the Osborne Formation.
- Forrestfield Dive – VWP data (BH0-07, BH1-04 and BH1-02) show negligible effect in response to nearby abstraction of groundwater.

Given that groundwater level monitoring along the alignment is showing temporary (daily) effects of groundwater level drawdown from reticulation wells, any groundwater abstraction for the project (e.g. construction dewatering) would also affect nearby existing wells. It is not uncommon in Perth that reticulation bores used for individual household reticulation consist of large diameter shallow liners, which would only have up to a few metres of water column in the bores during summer. Therefore there is a potential risk that some of these reticulation bores could become dry during construction dewatering, if drawdown is not carefully managed.

5.6 Groundwater Flow

The groundwater levels measured in March 2015 are consistent with the groundwater flow direction shown in the Perth Groundwater Atlas (WRC, 1997) which indicates two main regional groundwater flow components relevant to the project.

To the north of the Swan River, the regional groundwater flow direction is south to south-east, discharging into the Swan River. Groundwater flowing through this part of the alignment originates from the southern part of the Gnangara Mound which is the dominant regional groundwater head (Davidson, 1995).

South of the Swan River, the regional groundwater flow direction is north-west, discharging into the Swan River. Groundwater flowing through this part of the alignment originates from the Darling Scarp area to the east and from recharge along the alignment (Davidson, 1995).

An assessment of groundwater flow velocities across the site was not carried out within the scope of this study. We note that for ground improvement processes such as ground freezing, the groundwater flow velocity in the treatment area is an important factor. Further evaluation of groundwater flow velocity may be required in areas where ground improvement is required, depending on the level of risk at each location.

5.7 Hydraulic Properties

The following methods and tests were used and undertaken to assess the hydraulic properties of the different hydrogeological units and aquifers:

- Analysis of particle size distribution (PSD) data from 207 samples (refer to Appendix C for results) to derive hydraulic conductivity through empirical relationships.
- Single well hydraulic (slug) testing in 31 monitoring wells along the alignment (refer to Appendix D for method description and test results).
- Test pumping in four pumping test wells, one at each of the dive structures and stations (refer to Appendix E for description of tests and results).

In addition, a desktop study was undertaken to summarise hydraulic properties from previous available investigations, (Parsons Brinckerhoff, 2013), (HydroSolutions, 2004).

It is important to note that the different tests reflect different scales of the aquifer being tested. The PSD is only carried out on small and discrete soil samples and therefore the results do not represent the entire aquifer, but only the tested sample. Similarly, a slug test only affects the immediate area around the well and does not necessarily represent the wider extent of the aquifer. The results from PSD samples and slug testing should therefore be used with caution and can generally only be used to provide an indication of the order of magnitude of the hydraulic conductivity and provide a better understanding of the potential variability within the soils and the overall homogeneity of the aquifer.

The results from properly conducted pumping tests are the most important tool in hydrogeological investigations and far superior to other methods of assessing hydraulic properties and aquifer performance as the results are representative of a much larger extent of the aquifer that will require dewatering. Therefore more weight is put on the pumping test results than results from the slug testing or by derivation from the PSD data.

5.7.1 Transmissivity

The analysis of pumping test data provides an estimate of the transmissivity of the aquifer being tested, which is equal to the average hydraulic conductivity times the saturated thickness of the aquifer.

Hydrosolutions carried out a hydrogeological investigation at Perth Airport in 2004 (HydroSolutions, 2004), which included undertaking five pumping tests, of which four were completed in the Guildford Formation and one in the Mirrabooka Aquifer. The reported results indicated transmissivity ranging from 30 m^2/d to 300 m^2/d in the Guildford Formation and 80 m^2/d in the Mirrabooka Aquifer.

Mackie Martin & Associates carried out two pumping tests in the Upper Aquifer in August 1990 at the CSBP site located 200 m northeast of the Bayswater Dive Structure. The reported results indicated a transmissivity of 100 m^2/d in the Upper Aquifer. With a reported saturated thickness of 4 m (Bassendean Sand), this would correspond to a hydraulic conductivity of 25 m/d.

Table 3 presents the results from the four pumping tests carried for the Stage 2 Geotechnical Investigation, which are based on different analytical solutions, distance drawdown curves and replication of each pumping test in a numerical groundwater model (see Appendix E). The results from the groundwater modelling analysis of the pumping test data is considered to be more representative than other approaches as it includes the effects from all the different hydrogeological units and boundary conditions and has therefore been given more weight compared to analytical solutions. The distance drawdown curves were generally found to give similar results as to the numerical modelling of the pumping test.

Table 3: Transmissivity Values from the Pumping Test Analysis

Location	Aquifer	Range (m^2/d) **	Chosen Value (m^2/d)
Bayswater Dive Structure *	Lower	20–300	113
Airport West Station	Superficial	40–140	76
Consolidated Airport Terminal Station	Superficial and Mirrabooka	130–700	274
Forrestfield Dive Structure	Superficial (GF)	100–320	180

Notes: *No pumping test was carried out in the Upper Aquifer during the Stage 2 Geotechnical Investigation
** The values have been rounded to nearest 10.

5.7.2 Hydraulic Conductivity

Table 4a and Table 4b present a summary of the interpreted and adopted values of the horizontal hydraulic conductivity (in m/d and m/s, respectively) for each of the hydrogeological units and presents the range and median from the different methods used to estimate the hydraulic conductivity. The Tunnel Proximity Study (Golder, 2014c) values were collated based on a review of both published and unpublished reports. The last two columns provide range of values adopted for this study for each of the geological units.

Given the sedimentary nature of the geological units, which have been deposited in fluvial environments, it would be expected that the ratio between the vertical and horizontal hydraulic conductivity (k_v/k_h) would be less than 1 (i.e. the vertical hydraulic conductivity is smaller than the horizontal hydraulic conductivity). The general literature suggests that k_v could be several orders of magnitude smaller than k_h and it is not uncommon to assume a k_v/k_h ratio of 0.1.

The ratio between the vertical and horizontal hydraulic conductivity (k_v/k_h) was assessed during the groundwater modelling of the pumping tests and was found to generally range between 0.1 and 1. The analysis at Bayswater Dive Structure, Airport West Station and Consolidated Airport Terminal Station indicated that the ratio in the units could be around 0.1 while at Forrestfield the results did not provide a definitive conclusion regarding the ratio (use of 0.1 to 1 gave almost similar results – best fit was 0.25). Based on the composition, heterogeneity and distinct layering observed in the materials encountered in the boreholes (e.g. in OF) we would expect that the k_v/k_h ratio is smallest in the PF, GS, GF and OF. The greatest ratio is likely to be observed in the BS, but also the sandy GF encountered in the Forrestfield Area could have a higher ratio as indicated by the pumping test analysis. There was no distinct layering observed in the AF and this unit has a low percentage of fines in most areas, which would suggest that the ratio in AF could approximately be somewhere in between the ratio for PF and BS.

Based on the above, Golder judge that it is reasonable to use a k_v/k_h ratio of 0.1. However, since the installation of D-walls would result in mainly vertical flow upwards into the excavations once dewatering begins, the vertical hydraulic conductivity of the hydrogeological units would significantly influence the dewatering rates and drawdown. A sensitivity analysis was therefore carried out on the k_v/k_h ratio to assess the effect it could have on dewatering rates and drawdown (refer to Section 6.0).

5.7.3 Storage Coefficient

The estimated storage coefficients from the analytical solutions for the four pumping tests were found to range between 1×10^{-5} and 5×10^{-2} suggesting confined to semi confined conditions. The highest storage coefficient was estimated for the pumping test at the Consolidated Airport Terminal Station while the lowest was estimated for the confined Lower Aquifer at Bayswater Dive Structure.

The storage coefficients were also estimated through the numerical modelling of the pumping tests and the results indicated that the best matched shape between the modelled drawdown and observed drawdown for the tests was obtained by using confined conditions with storage coefficients of 1.5×10^{-5} (Bayswater Dive Structure) and 2.5×10^{-3} (Forrestfield Dive Structure).

Table 4a: Summary of Horizontal Hydraulic Conductivities by Hydrogeological Unit (m/d)

Geological Unit – Stage 1 (2014)	Geological Unit – Stage 2 (this study)		Tunnel Proximity Study		Stage 2							
			Range	Suggested Value	Particle Size Distribution^		Slug Testing		Test Pumping		Adopted Values	
					Range	Median	Range	Median	Range	Median	Range	Value
			m/d									
Swan River Alluvium & Alluvial Sand	Swan River Formation		0.1-10	1	3-9	6	0.2	0.2	-	-	0.001-1.0	0.001
Bassendean Sand	Bassendean Sand		10-40	15	1-25	9	***	***	-	-	10-40	15
Guildford Formation	Perth Formation	Sand	0.1-70	12	1-23	5	0.5-28	11	1-7.5	4	1-30	7.5
		Clay			-	-	**	**	-	-	0.0001-0.1	0.0001**
	Guildford Formation				1-36	2	0.2-8	2.5	6-11	8	1-10	7
	Gnangara Sand		-	-	0.4-8	2	-	-			0.5-5	1
Ascot Formation	Ascot Formation		5-100	10	3-20	4	3-50	7.5	5-30#	11	5-50	8-14##
Osborne Formation	Osborne Formation	Kardinya Shale	0.01-1	0.01	-	-	0.0002 to 0.0006	0.0005	0.0005*	-	0.0002 to 0.001	0.0005
		Mirrabooka Member			0.3-20	9	3-6	4.5	1-3*	2	1-10	1-3^^

Notes: * Values have been estimated through numerical analysis in the groundwater model.

** No monitoring wells are installed in the Perth Formation Clay (Bayswater Aquitard) - Value is based on the unit acting as an aquitard.

*** No monitoring wells are screened entirely in Bassendean Sand.

Lower part of range is from the Airport West pumping test while the upper range is from the Consolidated Airport Terminal Station pumping test.

Lower part of range is for the Airport West area while the upper range is from the Consolidated Airport Terminal Station area.

^ Estimations exclude PSD samples that have fines content greater than 20% and the values therefore represent the more permeable samples.

^^ The higher range is for the top of the Mirrabooka Aquifer with hydraulic conductivity reducing with depth.

Parameters are intended to be used for groundwater seepage analysis. Consolidation analysis may require refined values for Swan River Formation and Perth Formation (clay), refer to the GIR (Golder, 2015c).

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Table 4b: Summary of Horizontal Hydraulic Conductivities by Hydrogeological Unit (x10^{-5} m/s)

Geological Unit – Stage 1 (2014)	Geological Unit – Stage 2 (this study)		Tunnel Proximity Study		Stage 2							
			Range	Suggested Value	Particle Size Distribution^		Slug Testing		Test Pumping		Adopted Values	
					Range	Median	Range	Median	Range	Median	Range	Value
			x10^{-5} m/s									
Swan River Alluvium & Alluvial Sand	Swan River Formation		0.12-12	1.2	3.5-10.4	6.9	0.23	0.23	-	-	0.0012-1.2	0.0012
Bassendean Sand	Bassendean Sand		11.6-46.3	17.4	1.2-28.9	10.4	***	***	-	-	11.6-46.3	17.4
Guildford Formation	Perth Formation	Sand	0.12-81	13.9	1.2-26.6	5.8	0.58-32.4	12.7	1.2-8.7	4.6	1.2-34.7	8.7
		Clay			-	-	**	**	-	-	0.00012-0.12	0.00012 **
	Guildford Formation				1.2-41.7	2.3	0.23-9.3	2.9	6.9-12.7	9.3	1.2-11.6	7
	Gnangara Sand		-	-	0.5-9.3	2.3	-	-			0.6-5.8	1
Ascot Formation	Ascot Formation		5.8-115.7	11.6	3.5-23.1	4.6	3.5-57.9	8.7	5.8-34.7$^{\#}$	12.7	5.8-57.9	9.3-16.2$^{\#\#}$
Osborne Formation	Osborne Formation	Kardinya Shale	0.012-1.2	0.012	-	-	0.0002 - 0.0007	0.0006	0.0006*	-	0.0002 to 0.0012	0.0006
		Mirrabooka Member			0.3-23.1	10.4	3.5-6.9	5.2	1.2-3.5*	2.3	1-10	1.2-3.5^^

Notes: * Values have been estimated through numerical analysis in the groundwater model.

** No monitoring wells are installed in the Perth Formation Clay (Bayswater Aquitard) - Value is based on the unit acting as an aquitard.

*** No monitoring wells are screened entirely in Bassendean Sand.

$^{\#}$ Lower part of range is from the Airport West pumping test while the upper range is from the Consolidated Airport Terminal Station pumping test.

$^{\#\#}$ Lower part of range is for the Airport West area while the upper range is from the Consolidated Airport Terminal Station area.

^ Estimations exclude PSD samples that have fines content greater than 20% and the values therefore represent the more permeable samples.

^^ The higher range is for the top of the Mirrabooka Aquifer with hydraulic conductivity reducing with depth.

Parameters are intended to be used for groundwater seepage analysis. Consolidation analysis may require refined values for Swan River Formation and Perth Formation (clay), refer to the GIR (Golder, 2015c).

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5.8 Groundwater Quality

This section only provides a general overview and high level discussion on groundwater quality to assist with the hydrogeological conceptualisation and general discussion on dewatering discharge disposal options. The overview relates to groundwater samples taken at the beginning and end of the pumping tests (Appendix E) as well groundwater quality results from the Environmental Site Investigation (Golder, 2015h). Given that the results from the pumping tests represent a more average value of the aquifer tested, more emphasis has been put on the pumping test results in this report. For specific and detailed groundwater quality information along the alignment, the Environmental Site Investigation (Golder, 2015h) should be referred to.

The following summarises the general groundwater quality data along the alignment:

- pH generally ranged between 5.5 and 8.1 indicating between slightly acidic and alkaline conditions. For the pumping tests the higher pH (alkaline) was observed at the AWS and CATS, which is not surprising given that the pumping wells were pumping for the carbonate rich Ascot Formation. The lowest pH (acidic) was observed in Forrestfield where the field average pH was 5.7 while the average pH in the Lower Aquifer during the pumping test was 7.1.
- Groundwater range between fresh and brackish with highest salinity observed in the Swan River Formation in the vicinity of the Swan River. The Total Dissolved Solids (TDS) values ranged between approximately 250 mg/L (Lower Aquifer at BDS) and 750 mg/L (AWS) during the pumping tests, indicating fresh water. The salinity was generally found to remain stable during the pumping tests.
- Some metal concentrations were generally identified above detection limits at all locations. Total iron, which is one of the key parameters to dewatering discharge disposal and effectiveness of recharge wells, was found to range between 0.1 mg/L and 22 mg/L. The highest iron concentration was measured in the Lower Aquifer at the end of the Bayswater pumping test and the concentrations was found to increase by almost an order of magnitude between the start and end of the pumping test. The total iron concentration during the three other pumping tests ranged between 0.03 mg/L (Forrestfield) and 2.8 mg/L (AWS) with similar concentrations at the beginning and end of the tests.
- Nutrient (as total nitrogen), another key parameter for dewatering discharge disposal options, was found to range between 0.1 mg/L and 66 mg/L, with the highest concentrations observed in the Guildford Formation at Emergency Egress Shaft Abernethy Road. Similarly, the highest nutrient concentrations during the pumping tests were observed in Forrestfield (33 mg/L to 44 mg/L), while at the remaining pumping test locations total nitrogen ranged between 0.1 mg/L (CATS) and 0.78 mg/L (AWS).
- Total alkalinity, which indicates the buffering capacity of the water, ranged between 20 mg/L and 390 mg/L, lowest in Forrestfield and highest at AWS. The higher alkalinity concentrations were not surprisingly found in areas where Ascot Formation is present, while the lower total alkalinity was found in the Guildford Formation. The results indicate that the water in the Guildford Formation could have insufficient buffering capacity to maintain a stable pH (DEC, 2011).
- The following Contaminants of Potential Concern (CoPC) were encountered during the environmental groundwater quality sampling (Golder, 2015h):
  - Hydrocarbons (at 26 sample locations), chloroform (at 3 sample locations) and cyanide (Bayswater only) were found to be above detection levels in some groundwater samples.
  - PFOS (a fire fighting chemical) was found in the groundwater (at 5 sample locations) between AWS and Emergency Egress Shaft Domestic Airport Terminal.

The above CoPC (chloroform was not analysed) were not detected in any of the samples collected during the pumping tests except for Cyanide, which was above the detection limit in the sample taken at the beginning of the pumping test, but was subsequent below the detection limit in the sample taken at the end of the pumping test.

The groundwater quality will have an effect on the dewatering discharge treatment requirements and disposal options as discussed in Section 6.6.

The contamination monitoring undertaken as part of the CBSP investigation indicate that the groundwater quality in the Upper and Lower Aquifers at Bayswater are different, with nutrient and metal concentrations in the Upper Aquifer being up to an order of magnitude higher than in the Lower Aquifer (Australian Environmental Auditors, 2013). This large difference in groundwater quality between the aquifers suggests that the Upper Aquifer and Lower Aquifer are disconnected, which is supported by the high head difference in groundwater level (Section 5.3.5).

6.0 DEWATERING

This section discusses the construction dewatering requirements for the four main structures; Bayswater Dive Structure, Airport West Station, Consolidated Airport Terminal Station and Forrestfield Dive Structure and outlines preliminary dewatering rate estimates, dewatering methodologies and groundwater disposal options and constraints. The section is based on PTA's Reference design Rev F, a document titled Reference Design Summary Table RevB provided by PTA and assumed construction methodologies and cut-off wall elevations as outlined in this section.

6.1 Dewatering Requirements

Table 5 presents the construction details and dewatering requirements for the structures considered in this study based on the Rev F alignment and plan and profile drawings.

Table 5: Construction Details and Dewatering Requirements

Structure	Approximate Excavation Dimensions (m)			Required GWL Drawdown** (m)	Dewatering Period (weeks) ***	Geological Unit in which D-wall Terminates
	Length	Width	Depth*			
Bayswater Dive West of BMD	270	10	0.0–7.8	0.0–4.0	16	BS and PF
Bayswater Dive East of BMD	230	11–20	7.8–12	2.9–7.1 #		BS and PF
Bayswater Retrieval Box	20	20	12	7.1 #		PF
Airport West Station	156	27	12	12.1	52	OF (OFs)
Consolidated Terminal Station	160	28–46	17	16.2	52	OF (OFm)
Forrestfield Retrieval Box	20	20	10	5.5	16	GF
Forrestfield Dive Structure	300	10–24	0-10	0.0–5.5		

Notes: * Depth is below current ground surface, GWL = Groundwater Level, BMD = Bayswater Main Drain
** Estimated drawdown from groundwater level to 1 m below the base of excavation based on an estimated average seasonal groundwater level.
*** Total period during which dewatering is required.
# Required drawdown in the Lower Aquifer

The Reference Design Revision F indicates that D-walls are planned to form the four main structures. This approach significantly reduces the dewatering requirements because horizontal groundwater inflow to the excavation (except for any leakage through the D-wall) is essentially cut off. Accordingly, the groundwater inflow into the excavation becomes predominantly vertical where groundwater flows under the D-walls and up into the floor of the excavation.

The dewatering requirements and effect of dewatering (rates and extent of groundwater level drawdown) is dependent upon the wall depth and the geological unit in which the D-wall terminates. The analysis reported here is based on PTA's Reference Design Summary Table RevB (which is based on the Rev F Reference design):

- The D-walls on all sides will terminate 3 m into the Osborne Formation at the Airport West Station and Consolidated Airport Terminal Station (including the concourse level box).
- At the Bayswater and Forrestfield Dive Structures the D-Walls will extend to a depth that is between about 1.9 and 2.3 times the depth of excavation, respectively.

Bayswater Dive Structure

At Bayswater Dive Structure dewatering will be required from both the Upper Aquifer and Lower Aquifer as follows:

- West of the Bayswater Main Drain excavation and dewatering will only be required from the Upper Aquifer.
- East of the Bayswater Main Drain dewatering will only be required from the Lower Aquifer, because the D-walls will extend into the PFc and Lower Aquifer and thereby cut-off any horizontal groundwater inflow into the Upper Aquifer (apart from any leakage) and any flow down the open decline between the walls.
- At the deepest part of the dive and tunnel portal the D-walls will extend into the Osborne Formation, which will significantly reduce the groundwater inflow from the Lower Aquifer into the excavation. Nevertheless, the greatest dewatering is expected to be required from the Lower Aquifer at the Bayswater Retrieval Box where the piezometric level would need to be lowered by up to 7.1 m.

Airport West Station

The D-Walls will extend into the Kardinya Shale (KS) of the Osborne Formation and thereby cut off direct groundwater inflow from the Superficial Aquifer. Given the low hydraulic conductivity (0.0005 m/d) of the KS, dewatering would be required to initially remove stored water within the soils inside the excavation and thereafter remove any upward seepage through the Kardinya Shale and leakage through the D-walls. The dewatering requirements for the Airport West Station should therefore be relatively low.

Consolidated Airport Terminal Station

The same concept as for Airport West Station applies to the Consolidated Airport Terminal Station where the Superficial Aquifer is cut-off and seepage into the excavation can only occur through upward seepage from the Osborne Formation and leakage through the D-walls. However, given that the Osborne Formation below the Consolidated Airport Terminal Station consists of the Mirrabooka Member which has a hydraulic conductivity that is 4 orders of magnitude higher than the Kardinya Shale, the groundwater inflow into the excavation will be significantly higher at this station.

Forrestfield Dive Structure

The aquifer at the Forrestfield Dive Structure is comprised of the Guildford Formation within which the D-walls will terminate. Groundwater inflow into the excavation will occur from the open end of the dive unless slurry wall cut-offs are deployed and vertically upward from below the excavation between the D-walls. The most dewatering for this structure will be required at the Forrestfield Launch Box where the groundwater would need to be lowered by up to 5.5 m.

Other Structures

Construction dewatering will be required for other structures such as the emergency egress shafts. These structures have not been assessed within the scope of this current study, but will need to be addressed in the detailed design.

6.2 Dewatering Methodology

The dewatering methodology for this project will depend on the specific site and dewatering requirements, but is likely to consist of a combination of:

- **Dewatering Wells** – drilled wells with screens and associated gravel packing placed around the internal perimeter of the excavation between the D-walls.
- **Sump Pumping –** strategically placed dewatering sumps or trenches (drains) within the excavation where groundwater is removed using sump pumps.
- **Dewatering Spears (Well Points) –** shallow spears could be used to lower the groundwater level locally or where the saturated thickness of the aquifer is limited.

The use of dewatering wells and sump pumping is considered the preferred option for all four structures. However, the use of dewatering spears could be advantageous for dewatering of the Upper Aquifer at the Bayswater Dive Structure where the saturated aquifer is of limited thickness and dewatering wells will be less effective. However, dewatering wells will still be required to dewater the Lower Aquifer.

The dewatering wells must be placed inside the excavations to minimise the dewatering rates and drawdown in the aquifers. The number and spacing of the dewatering wells have not been evaluated as part of this work; this will need to be done as part of a detailed dewatering design once the methodology and sequence for the overall construction of each structure is known.

Sump pumping from inside the D-wall supported excavations is likely to be required to remove residual shallow groundwater not drained by the dewatering wells or to control surface water from rainfall events. Some local shallow dewatering may be required in the Forrestfield car park area for site preparation and/or installation of services, etc. Due to the expected limited groundwater level drawdown requirements, the dewatering methodology for this area would likely consist of a combination of sump pumps and dewatering spears.

6.3 Groundwater Modelling

6.3.1 Groundwater Modelling

The 3D groundwater modelling software Visual MODFLOW 2011.1 Premium (numerical finite difference groundwater flow modelling software used extensively throughout the world) was used to simulate existing groundwater conditions along the alignment and evaluate preliminary dewatering rates and the effect the construction dewatering could have on the groundwater level and piezometric levels. Appendix F provides details on the construction and assumptions used in the 3D numerical groundwater model.

6.3.2 Model Scenario Runs

Table 6 presents a summary of the scenarios modelled.

Table 6: Model Scenarios.

Structure	Recharge of Dewatering Discharge	Scenario	Comment
Bayswater Dive Structure	No*	1A	k_v/k_h ratio of 0.1 in Upper and Lower Aquifer
		1B	k_v/k_h ratio of 1 in the Upper and Lower Aquifer
Airport West Station	No	2A	D-walls extend into Osborne Aquitard resulting in limited dewatering (i.e. recharge wells not likely to be required or preferred disposal option)
Consolidated Airport Terminal Station	Yes	3A	k_v/k_h ratio of 0.1 in all layers
		3B	k_v/k_h ratio of 1 in Mirrabooka Aquifer
Forrestfield Dive Structure	Yes	4A	k_v/k_h ratio of 0.1 in all layers
		4B	k_v/k_h ratio to 1 in in all layers

* Estimated average dewatering rates are low and there is minimal drawdown effect in the Upper Aquifer, suggesting that recharge wells may not be required.

Given that installation of D-walls would result in mainly vertical flow upwards into the excavations once dewatering begins, the vertical hydraulic conductivity of the hydrogeological units in which the D-walls terminate will have a significant influence on the dewatering rates and extent of groundwater level drawdown outside the excavation. Sensitivity runs (scenarios with the suffix "B") were therefore carried out where the k_v/k_h ratio has been increased 10 times (i.e. vertical hydraulic conductivity is the same as horizontal hydraulic conductivity) in the hydrogeological units into which the D-walls are installed. At Airport West Station the hydraulic conductivity of the KS is so low that a change k_v/k_h ratio or a change of one order of magnitude in hydraulic conductivity would not noticeably change the dewatering rates or the groundwater level drawdown. A sensitivity run was therefore not performed for Airport West Station.

A preliminary dewatering assessment carried out by Golder in 2014 (Golder 2014) indicated that on-site recharge of the abstracted groundwater would be required to minimise and manage the groundwater level caused by the groundwater abstraction. Recharge wells were therefore installed around the excavations in the models to conceptually show the effect of the wells. However, the actual number and location of the recharge wells and distance from the excavation area will need to be assessed during the detailed design phase. The recharge wells in the models were generally screened across the whole Superficial Aquifer to a similar depth as the D-walls (wells were not modelled deeper than the D-walls as this could further promote groundwater flow back into the aquifer, resulting in increased dewatering rates).

Based on recent experience from projects where recharge wells have been used in the Perth Formation for dewatering disposal, the following assumptions were included in the model:

- The long-term recharge rate of a recharge well would not exceed 1 L/s
- Approximately 85% of the abstracted groundwater can be introduced back into the aquifer, which is considered a more realistic scenario than assuming a perfect system with 100% recharge of the abstracted water. With 85% recharge the model takes into account maintenance of recharge wells, short-term temporary shutdown, etc. However, the amount of abstracted water that is returned back into the aquifer could have an effect on the extent of the groundwater level drawdown around each excavation and it is therefore recommended that the dewatering system design and the aim during operation should be to achieve as high a recharge percentage as possible.

At Airport West Station the removal of storage inside the excavation (main dewatering requirement) would not result in a lowering of the groundwater level outside of the excavation and recharging this water back into the aquifer would therefore not be required. Given the expected low seepage rates at Airport West Station after the initial pumping of stored water, recharge wells are not likely to be the most efficient or preferred disposal option and therefore the abstracted groundwater was not recharged back into the aquifer in the Airport West Station scenario run (Scenario 2A).

At Bayswater Dive Structure the D-walls will cut-off groundwater inflow into the Upper Aquifer except for limited inflow down the open decline between the walls. The D-walls will extend into the Osborne Formation at the tunnel portal where the deepest excavation is required and therefore dewatering from the Lower Aquifer will be limited. Given that the groundwater model results indicate that the average dewatering rates will be less than 3 L/s, the abstracted groundwater was not recharged back into the aquifer in the Bayswater Dive Structure scenario runs.

6.4 Dewatering Rates

Table 7 presents the estimated dewatering rates and volume for the four main structures for the different scenarios and indicates:

- The dewatering rate at Airport West Station will be limited if the D-walls extend into the Kardinya Shale (Osborne Formation). The model run indicated that of the estimated 3 L/s, less than 1 L/s was attributed to seepage through the Kardinya Shale while the rest was due to estimated leakage through the D-walls.
- The modelled low dewatering rate for the Bayswater Dive Structure is expected, given that the pumping test (at 12 L/s) was able to lower the piezometric pressure in the Lower Aquifer by up to 9 m in the vicinity of the well (only 7 m is required during construction) and by up to 1.5 m at a distance of 500 m from the well. Given that D-walls will be used to significantly limit groundwater inflow into the excavation, the construction dewatering rates would be expected to be significantly lower than during the pumping test.
- The modelled dewatering rates at the Consolidated Airport Terminal Station are based on the D-walls on all sides terminating 3 m into the Osborne Formation, including the concourse level box. Should the D-walls surrounding the concourse level box terminate in the Ascot Formation, the dewatering rates would increase.
- The vertical hydraulic conductivity has a significant effect on the dewatering rates with an increase of the average dewatering rate of 1.5 times to 2.6 times for the Consolidated Airport Terminal Station and Forrestfield Dive Structure, respectively when k_v was increased.

Table 7: Modelled Dewatering Rates and Volume

Location	Scenarios	Dewatering Rate (L/s)		Dewatering Volume (kL)
		Initial*	Average	
Bayswater Dive Structure	1A	8	3	39,000
	1B (sensitivity analysis)	8	4	42,000
Airport West Station	2A	9	3	91,000
Consolidated Airport Terminal Station	3A	40	30	948,000
	3B (sensitivity analysis)	54	44	1,399,000
Forrestfield Dive Structure	4A	18	14	138,000
	4B (sensitivity analysis)	46	35	354,000

Notes: *Initial rate for the first 2 weeks to lower the groundwater level to required dewatering level

The depth and distance of the recharge wells would also have an effect on the dewatering rates due to the "recycling" effect of water back into the excavation from the recharge wells. The closer and deeper the recharge wells are to the excavation and bottom of the D-wall the greater the "recycling" effect and thereby dewatering rates would be greater.

6.5 Effects of Dewatering

The abstraction of groundwater during the construction dewatering will result in a lowering of the groundwater table and the piezometric pressure in the hydrogeological units. The effect will depend on the groundwater management practices and methods adopted during the dewatering operation and on the chosen disposal option.

Figures 6 to 11 show a plan and cross-sectional view of each main structure with the modelled groundwater drawdown contours at the end of the dewatering period for the different scenarios while Table 8 presents the distance to the contours of groundwater table drawdown and change in piezometric level.

Table 8: Modelled Extent of Groundwater Level Drawdown (m)

Scenarios	Groundwater Table			Piezometric Level			
	0.1 m	0.5 m	1.0 m	0.1 m	0.5 m	1 m	Geological Unit
1A *	70	-	-	2,000	380	60	PFs
1B *							
2A *	300	40	-	300	50	-	AF/OFs
3A	430	-	-	450	150	100	OFm
3B	590	-	-	590	170	110	AF/OFm
4A	210	-	-	210	60	25	GF
4B	500	170	65	490	170	70	GF/BS

Notes: *No recharge wells.
The reported distances (in metres) are from the external wall face to the groundwater level/piezometric contour

The results in Table 8 and Figures 6 to 11 indicate:

- **Groundwater table:**
  - Groundwater table drawdown of greater than 1 m does not occur in any of the modelled scenarios indicating the recharge wells are capable of reducing the groundwater table drawdown to within the SWTC requirements, except for Scenario 4B in Forrestfield where the groundwater table drawdown of 1 m extends approximately 65 m from the D-walls.
  - The groundwater table drawdown of 0.5 m is predicted in Scenario 2A (AWS) and 4B (Forrestfield Dive Structure) at a distance of 40 m and 170 m, respectively.
  - The difference in drawdown between Scenarios A and B for all structures indicates that the drawdown will extend further if the k_v/k_h ratio is 1 (compared to 0.1).
  - Groundwater mounding of up to 0.2 m is predicted around the recharge wells.
  - At Airport West Station the difference in drawdown shape between north and south (Figure 7B) is due to the presence of the Belmont Main Drain which reduces the extent of the groundwater level drawdown toward the south (i.e. Belmont Drain acts a positive flow boundary at the water table).
- **Piezometric level:**
  - The drawdown in piezometric levels extends further than the drawdown in the groundwater table. The drawdown is expectedly greatest in the hydrogeological unit where the D-walls terminate as this is the unit where the greatest groundwater inflow into the excavation occurs.

- For the Consolidated Airport Terminal Station the modelled drawdown in piezometric levels in the Mirrabooka Aquifer do not propagate to the ground surface (Figures 8B and 9B), which is due to a combination of the effect of the recharge wells and the presence of the more permeable Ascot Formation above the Mirrabooka Aquifer. The effect that Ascot Formation alone has on the shape of the piezometric level contours can be seen for the Airport West Station scenario (Figure 7) where no recharge wells are modelled.
- The difference in drawdown between Scenarios A and B for all structures except Bayswater Dive Structure indicates that the drawdown will extend further if the k_v/k_h ratio is 1 (compared to 0.1). The reason this effect is not noticeable at the Bayswater Dive Structure is that the groundwater inflow into the excavation is mainly horizontal due to the D-walls extending into the Osborne Formation along three sides of the excavation.

The results from the preliminary groundwater modelling indicate that it should be possible to design a dewatering system incorporating recharge wells that would limit the drawdown in the groundwater table outside each excavation to less than 1 m in accordance with SWTC requirements, particularly at the two Airport Stations where the D-walls extend below the Ascot Formation. At Forrestfield, the modelling indicates that greater than 85% of pumped groundwater would need to be recharged to meet the SWTC requirements.

6.5.1 Groundwater Quality

Normally the greatest concerns of the effects that construction dewatering could have on groundwater quality is related to the presence of potential ASS within the groundwater table drawdown cone of depression and mobilisation of contaminated water through the change in groundwater flow direction.

Drawdown of the groundwater table surrounding a site can lead to oxidation of ASS resulting in sulfuric acid being formed which would result in release of metals, nutrients and acidity into the soil and groundwater system (DEC, 2011). This risk can be minimised by reducing the groundwater table drawdown. However, increasing oxygen concentrations through aeration of abstracted groundwater during dewatering could potentially also cause oxidation of ASS when the groundwater is recharged back into the aquifer. This would need to be considered during the design of the dewatering and recharge system.

The results from the preliminary groundwater model indicate that it would be possible to design a dewatering system that would limit changes in groundwater flow direction and thereby limit the risk of mobilisation of contaminated water.

6.5.2 Surface Water

The results from the preliminary groundwater model indicate that it would be possible to design a dewatering system that would not affect the surface water bodies discussed in Section 5.5.1.

The model results indicate that the Belmont Main Drain will act as a positive boundary (i.e. water will flow from the drain into the water table aquifer to keep the groundwater table steady at the drain). Given that this man-made drain was constructed to lower the groundwater table, this flow is currently not considered to be an issue. It is possible that some of the Perth Airport drains could also be affected during the dewatering of the Consolidated Airport Terminal Station.

6.5.3 Means of Reducing Effects

If the piezometric level drawdown would need to be further reduced below the drawdown modelled in the current scenarios, recharge wells could be installed to a greater depth (i.e. below the D-walls), which should reduce the piezometric level drawdown outside of the D-walls. However, this deepening of recharge wells is likely to result in an increase in dewatering rates as the "recycling" effect would become greater.

If groundwater monitoring during the dewatering operation shows a greater than expected effect on the groundwater level and piezometric level, additional recharge wells could be installed between the excavation and the area of concern (e.g. a row of recharge wells could be installed adjacent to the airport control tower, if this was required).

6.6 Dewatering Disposal Options

Dewatering discharge should be disposed of so as to not cause harm to the environment or allow flooding.

The SWTC specifies that the groundwater levels in all aquifers within 500 m of the dewatering activity, unless otherwise specified, must not be elevated/lowered by more than 1 m during the project works when compared to locally recorded seasonal highs/lows in that aquifer. Golder considers that to achieve these criteria the use of onsite recharge wells is likely to be the most feasible dewatering disposal option, where dewatering is required.

Any pumped groundwater that is in excess of that required to be recharged to meet the above SWTC criteria could be disposed of by alternative methods. The following dewatering discharge disposal options (in addition to recharge wells) have been identified:

- Reuse on site
- Infiltration
- Stormwater system
- Sewer.

These options would also serve as contingency disposal options to the recharge well scheme.

6.6.1 Recharge Wells

The advantage of using recharge wells is that the abstracted water will be reintroduced back into the aquifer within the vicinity of where it is abstracted, which will result in a much lower net withdrawal of groundwater and thereby minimise the effect the dewatering will have on changes to the groundwater level and flow direction. Recharge wells can also be placed strategically between the dewatering area and sensitive structures and environments to control the groundwater levels and thereby groundwater flow direction.

The performance of the recharge wells would depend on the geological units they are screened in and the water quality of the discharged water. From experience in the Perth Metropolitan Area it is not uncommon to assume that the long-term recharge rate of a recharge well installed in the PF and GF would not exceed 1 L/s. However, the recharge well capacity would depend on the aquifer transmissivity and it is expected that higher recharge rates may be achievable for wells installed in the AF and the sandy GF in the Forrestfield area.

The dewatering discharge quality, particularly high iron concentrations, could significantly limit the long-term performance of the recharge wells and therefore water treatment and ongoing maintenance (cleaning) of the dewatering discharge system is likely to be required to minimise the risk of iron clogging of screens.

The SWTC specifies specific recharge system requirements.

6.6.2 Reuse on Site

Reuse of abstracted groundwater as a means for disposal will be dependent on the demand of the various contractors on site and the dewatering discharge quality and may include use for dust suppression or water supply to the TBM. The groundwater quality would need to be assessed at each specific site location. Site specific groundwater quality is detailed in the Environmental Site Investigation (Golder, 2015h).

6.6.3 Infiltration

The infiltration capacity of the soils at the dive and station locations is discussed in detail in Section 8.0. The viability of shallow infiltration (as opposed to deeper recharge) as a disposal option depends on the following factors:

- The area available close to the excavation where an infiltration basin can be constructed. The greater the area, the higher the achievable infiltration rate.

- The depth to groundwater table. Infiltration capacity reduces significantly where the groundwater table rises to the base of the basin in response to infiltration.
- The permeability of the subsurface profile at the infiltration basin location. The presence of layers of clay or fines can significantly reduce the infiltration capacity.

The shallow depth to groundwater at all four major structure locations is considered a significant restriction on infiltration capacity and therefore on infiltration as a disposal option. Based on infiltration testing (refer to Section 8.0), infiltration may not be a feasible option at Forrestfield and limited infiltration capacity would also be expected at the Bayswater Dive structure, Airport West Station and Consolidated Station.

6.6.4 Stormwater System

Disposal of dewatering discharge into the local stormwater drainage system will ultimately result in the dewatering effluent discharging into either the Bayswater or Belmont Main Drains and ultimately into the Swan River. The groundwater quality results indicates that some treatment of the dewatering discharge would be required prior to disposal to the stormwater system, which could include adjustment of pH and removal of some metals and nutrients to meet the Swan River Trust (SRT) guideline criteria for disposal to the Swan River.

Prior to commencement of dewatering, approval to discharge the abstracted groundwater into the stormwater system would need to be obtained from the local council (various) and asset owners (Water Corporation operates the Belmont and Bayswater Main Drains while PAPL operates the Perth Airport drain system). The asset owner is unlikely to provide access to their system before necessary regulatory approval has been obtained for discharge of water into the end environment (likely to be Department of Environment Regulation (DER) and SRT). This option is likely to require rigorous water treatment and monitoring requirements.

During the pumping tests at Forrestfield, Consolidated Airport Terminal Station and Airport West Station the abstracted groundwater was discharged into the stormwater drains at rates of up to 12 L/s after receiving approvals from Shire of Kalamunda (for Forrestfield), PAPL (Consolidated Airport Terminal Station) and City of Belmont, the Water Corporation and SRT (for Airport West Station). Given the short-term disposal duration approval was obtained without having to the treat the water for metals and nutrients. However, additional water monitoring was undertaken from the Belmont Main Drain during the pumping test at the Airport West Station as requested by the SRT. This data could be utilised for future applications if regulatory approval to discharge into the stormwater system at the Airport West Station will be sought.

It is important to note that the regulators will assess the disposal options based on a site specific risk assessment, which would require site specific water quality data as well as information on the estimated flow rates, duration of dewatering and the time of year that the dewatering is to be carried out.

6.6.5 Sewer

For discharge to sewer the following should be considered:

- Approval would be required from the Water Corporation by lodging a “one-off discharge of industrial waste”.
- Proximity to current sewer locations suitable for discharge will need to be considered and discussed with the Water Corporation.
- Disposal is normally restricted by the sewer capacity, which would need to be discussed with the Water Corporation and would therefore depend on the expected discharge rates and volumes. This could, therefore, be a limitation as dewatering rates may be higher than the available capacity.
- This option is unlikely to require on-site treatment, though it may require pH adjustment (criteria is pH 6 to pH 10) and removal of Total Suspended Solids (TSS) by installing a settling trap (mandatory requirement). To prevent escape of sewer gases to the environment, a water trap would also need to be installed between the settling trap and the sewer connection point.

There is a unit cost (per kL) for disposal to sewer, which would depend on the water quality.

7.0 DESIGN GROUNDWATER LEVELS (50 AND 120 YEAR)

This section presents the estimated maximum design groundwater level along the alignment for 50 and 120 year design periods with particular emphasis on Bayswater Dive Structure, Airport West Station, Consolidated Airport Terminal Station and Forrestfield Station.

At Bayswater where there is a dual aquifer system, the estimated design groundwater level refers to the maximum groundwater level in the Upper Aquifer.

At Forrestfield the presence of low permeability material above the regional groundwater level has been found to cause perched groundwater conditions in some areas, with the perched groundwater levels locally found to be several metres higher than the regional groundwater level following rainfall. These perched conditions may be seasonal or may occur year-round (Section 5.3.6). For Forrestfield both regional and perched design groundwater levels have been discussed.

7.1 Methodology

We have taken a pragmatic approach to the estimation of the design groundwater levels, where the present and historic groundwater levels, hydrogeological features and key parameters that would affect the groundwater level are closely considered.

The installed groundwater monitoring network along the alignment provides a comprehensive record of present-day groundwater levels for developing current groundwater level contours and allowing direct comparison with historical records.

7.1.1 Key Parameters

The identified key parameters that could have an effect on the maximum groundwater levels are:

- Change in Climate
- Geology
- Drains
- Groundwater Abstraction.

Change in Climate

As outlined in Section 4.2, Perth has experienced a drying climate over the last 40 years, which has become even more severe over the last 20 years. The reduction in rainfall is generally known to have resulted in a decline in groundwater levels over large parts of the Perth Metropolitan Area. However, the currently available groundwater level hydrographs for various locations along the alignment do not indicate any significant decreasing trend in the groundwater level, even with the drying climate, which suggests that the current lower annual rainfall is still sufficient to fill the aquifer in the study area (Section 5.2.2).

A flood modelling study was carried for the FAL project (BG&E, 2015), which included an extensive review of available information on climate change scenarios. The main overarching conclusions from the reviewed information suggests that there will be an increase in sea level (between 0.5 m and 1.1 m) and that the rainfall intensities will increase even though the annual rainfall trend in the western region of Australia could decrease (i.e. the climate could become drier, but storm events could become more severe). The rise in sea level would result in a rise in the water level in the Swan River, which would result in an increase in groundwater level in the vicinity of the Swan River. The more severe storm events could potentially increase short-term rises in groundwater levels during the wet season, particularly in areas with thicker unsaturated zones.

The climate changes are currently not expected to have a major effect on the future groundwater levels along most of the alignment except for an increase in groundwater level at the Swan River due to global sea level rise. A lower future annual rainfall could result in decreasing groundwater levels, while an increase in the rainfall to pre-1960's rainfall volumes could result in increasing groundwater levels.

Geology

Over the majority of the alignment the top soil consists of sandy soils promoting rainfall recharge. However, the geotechnical and environmental investigation results indicate that clay or low permeability material is present within less than 1 m from the ground surface in the Forrestfield area (along the tunnel alignment and in the associated car park). This low permeability material has been found to cause perched groundwater conditions in some areas with groundwater level being close to the surface. The groundwater level could possibly even reach the ground surface or cause local inundation in some local areas after large rainfall events (which could be monitored in the future during the wet season). Therefore, in the Forrestfield area the geology is causing locally perched aquifer conditions.

If climate change would result in more severe storm events, this could result in more frequent and severe inundation in the Forrestfield area.

Drains

A large network of man-made drains exists over the study area and along the alignment. The main drains are described in Section 5.5.2. The main purpose of the drainage networks is to lower the groundwater level in the full aquifer to allow for land development in the area. The drains were installed progressively from the 1950's to the 1970's (see the aerial photography review presented in (Golder, 2015c) resulting in lowering of the groundwater level.

The existing drainage networks are considered to drain the groundwater table, particularly during the wet season and thereby control the maximum groundwater level in the area. The drains are therefore currently considered to be the main parameter that determines the maximum groundwater level in the area.

Groundwater Abstraction

The main groundwater abstraction along the tunnel alignment is considered to be for reticulation (residential, public parks and Perth Airport) and for industrial purposes. No large scale groundwater abstraction has been identified in the area. The VWPs are showing temporary (daily) effects of groundwater level drawdown from reticulation wells, but no long-term decreasing trends are observed in any wells with historical groundwater level data that have been reviewed during this study (Section 5.2.2). In addition, groundwater abstraction for reticulation purposes would be highest during the dry season which is when the groundwater levels are lowest.

Groundwater abstraction is therefore currently not expected to have a major effect on the design groundwater levels.

7.1.2 Design Groundwater Level Estimation

7.1.2.1 50 Year Design Groundwater Levels

The following data was utilised to generate the 50 year design groundwater levels:

- Groundwater levels from monitoring wells and VWP's installed along the alignment during the geotechnical and environmental investigation.
- Historical hydrographs available from DoW, PAPL and the former CSBP Bayswater site (Parsons Brinckerhoff, 2013).
- Drain and wetland elevations estimated from topographic contours (1 m interval) from Landgate.
- Information on global sea level rise.

Based on the analysis of the key parameters it is considered that the existing man-made drainage network will control the maximum groundwater level over a large part of the alignment in the future. It is therefore important that the drainage network is incorporated into the estimation of the design groundwater level.

The design groundwater level contours for the study area have been estimated by contouring groundwater level interpolation points created as follows:

- **Drainage system** – The invert levels for the drainage system were obtained from 1 m topographic contours. It was assumed that the groundwater level will be up to 1.0 m above the invert levels during the wet season (i.e. 1 m was added to invert levels) based on observations at Belmont Main Drain. It has been assumed that all existing drains will remain in operation throughout the design period.
- **Wetlands and lakes –** Aerial photography was utilised together with surface contours to estimate the water levels.
- **Swan River Level Rise –** A rise in average Swan River water level due to global sea level rise of 0.4 m (from RL 0.2 m AHD to RL 0.6 m AHD).
- **Wells with historical hydrographs** – The maximum historical groundwater level was estimated for each well based on the groundwater levels measured over the period of monitoring (generally up to 15 years for PAPL wells, 8 years for CSBP wells and up to 60 years in the DoW wells).
- **Wells with no historical hydrographs (new wells installed along the alignment) –** The March 2015 measured groundwater level was used as a starting point. The groundwater level difference between the estimated maximum groundwater level and the March groundwater levels in nearby wells with historical hydrographs was then added to the March 2015 groundwater levels. This process assumes that similar historical trends would be exhibited by nearby wells. The difference between the maximum and March 2015 levels was found to vary along the alignment between +0.8 m and +4.0 m.

The interpolation points resulting from the above approach were then manually contoured. The contours are based on currently available data. If additional information (e.g. if sub-soil drains exist beneath car parks or roads, or additional monitoring wells) becomes available or if existing drains are infilled, the design groundwater level would need to be reassessed and adjusted accordingly.

7.1.2.2 120 Year Design Groundwater Levels

The derived 50 year design groundwater level was utilised as the base for generation of the 120 year design groundwater level. The following assumptions were made to develop the 120 year design groundwater level:

- The Swan River average level will rise by another 0.4 m to RL 1 m AHD
- The groundwater level would rise by 0.5 m over the whole alignment except for:
  - Where the Bayswater and Perth Airport drains cross the alignment, the level was only increased by 0.25 m
  - Along CH3400 to CH4000 the level was only increased by 0.25 m because of the presence of the Belmont Main Drain
  - In the Forrestfield area where an increase of 1 m was adopted due to the available unsaturated thickness and high seasonal variation due to the presence of lower permeability soils.

7.2 Design Groundwater Level

Table 9 summarises the design groundwater level at the stations, dive structure and Forrestfield car park while Figure 12 shows the estimated design groundwater levels over a long-section along the tunnel alignment together with the current ground surface. The results indicate:

- The depth to the design groundwater levels from the current ground surface ranges between 0 m and 5 m with the design groundwater level being within 2 m of the ground surface over the majority of the alignment.
- The groundwater level reaches the surface at several locations, which typically corresponds to where drains were previously installed.

Table 9: 50 and 120 Year Design Groundwater Levels

Location	50 Year Design Groundwater Level			120 Year Design Groundwater Level		
	Range	Average	Average Depth to Groundwater Level	Range	Average	Average Depth to Groundwater Level
	m AHD		m	m AHD		m
Bayswater Dive Portal	5.9–6.2	6.1	<1	6.4–6.7	6.6	<0.5
Airport West Station	9.3–9.9	9.6	1.3	9.5–10.2	9.9	1.0
Consolidated Airport Terminal Station	18.2–18.4	18.3	<1	18.7–18.9	18.8	<0.5
Forrestfield Dive Portal	27*	27*	3	28*	28*	2
Forrestfield Station	27*	27*	2	28*	28*	1
Forrestfield Car Park	27–34 *	NA	2-3	28–35*	NA	1–2

Notes: *Regional Aquifer – Perched groundwater level conditions exist in this area (see Section 7.2.1).
**Range refers to change in groundwater level along the length of the structure.
NA = average is not applicable due to the large area of the car park.

It should be noted that the design groundwater levels over parts of the alignment are lower than the maximum inferred groundwater level contours in the 1997 Perth Groundwater Atlas (WRC, 1997). The main reason for this difference is that the inferred maximum groundwater level contours in the 1997 Atlas include data that dates back to before some of the drains were installed.

7.2.1 Perched Aquifer at Forrestfield

Figure 5 shows the area in Forrestfield where shallow monitoring wells indicate that perched groundwater is either present or is considered likely to occur during the wet season. As described in Section 5.3.6 the differences in groundwater levels measured in shallow and deep wells have been found to vary between 0.3 m and 3.3 m, suggesting that the level of perching is locally highly variable, which is likely because of localised and discontinuous bands/layers of low permeability clay within the Guildford Formation soils. Given that the differences are based on groundwater levels measured toward the end of the dry season, these differences are likely to further increase during the wet season.

With the currently available groundwater level information it is not possible to accurately establish a 50 or 120 year design groundwater level surface for the perched groundwater. It is considered possible that perched groundwater in some areas could reach the ground surface or cause local inundation after large rainfall events. Further groundwater level monitoring would be required to establish the extent of the perched groundwater level areas and the connectivity/separation of the perched groundwater levels and the regional aquifer.

Some of the shallow environmental monitoring wells were reported to be dry during installation. It is recommended that these wells should still be part of the monitoring program in the wet season and following rainfall events to assess the possible perched groundwater level conditions in the area.

7.3 Risks Associated with Design Groundwater Levels

The following risks have been identified for the estimated maximum design groundwater levels:

- Underground structures will need to be designed to resist forces due to buoyancy (uplift) and hydrostatic pressures as well as to resist the ingress of water through walls of the structure.
- The river flood level may cause local short-term changes in groundwater level within and immediately adjacent to the Swan River flood plain. The design of any structures within this zone must consider the river flood level.
- Due to the groundwater level being within a few metres from the ground surface, the infiltration capacity for disposal of stormwater is considered to be low over majority of the alignment and at Bayswater Dive, Airport West Station and Consolidated Airport Terminal Station. Though the depth to the regional groundwater level in the Forrestfield area ranges between 3 m and 6 m, the presence of perched groundwater level conditions could also result in reduced infiltration capacity in this area.
- The presence of shallow perched groundwater and potential for groundwater inundation in the Forrestfield area could affect pavements and rail track subgrades if not managed appropriately during design and construction.
- Any facilities that will be constructed on or under the ground surface may require to be designed for moist or wet conditions (due to capillary rise).

7.4 Drainage Control

It is understood that PTA will require that all underground structures be 'tanked' which should eliminate any requirements for under-drainage systems below the base slabs.

To facilitate excavation and construction of the dive structures and stations, retaining walls will be required to be constructed around the perimeter of the structures. Reference Design Revision F indicates that the retaining structures will comprise D-walls. Depending on the depth, location and orientation in relation to groundwater flow, the installation of the D-walls could act as groundwater flow barriers, which could result in groundwater mounding behind the D-walls and may result in an increase in hydrostatic pressure against the D-walls. Therefore, permanent groundwater control in areas where groundwater mounding would occur may be required:

- ***Bayswater Dive Structure -*** The D-walls will extend through the Upper Aquifer into the Lower Aquifer, resulting in a permanent groundwater flow barrier in the Upper Aquifer where the walls are aligned obliquely or perpendicular to the groundwater flow direction:
  - East of Bayswater Main Drain – The groundwater flow is in a westerly to south-westerly direction toward the Bayswater Main drain, which could result in some groundwater mounding behind the northern D-walls.
  - West of Bayswater Main Drain – The groundwater flow is mainly in a north-easterly to easterly direction toward the Bayswater Main Drain and limited groundwater mounding could occur behind the southern D-walls.

  At the Bayswater Main Drain, the D-walls may need to be extended to below the invert level of the Bayswater Main Drain, which would require diversion of the Bayswater Main Drain. This diversion may require additional groundwater control (dewatering) during construction.
- ***Airport West Station and Consolidated Airport Terminal Station -*** The station boxes are generally located parallel to the groundwater flow direction from east to west and are therefore unlikely to significantly impede groundwater flow. However, some groundwater mounding could occur behind the eastern D-walls.

- ***Forrestfield Area –*** The D-walls associated with the Forrestfield Dive Structure will be constructed perpendicular to the groundwater flow direction. However, according to Reference Design Summary Table RevB (for Reference design Rev F) the D-walls will not extend through the full thickness of the Superficial Aquifer and it is therefore considered unlikely that the D-walls will significantly impede groundwater flow and result in significant groundwater mounding behind the up-gradient D-wall.

  The presence of perched groundwater in Forrestfield could result in some groundwater drainage control requirements at the Forrestfield Station and car park area. The required groundwater control could consist of the installation of sub-soil drains or granular drainage blankets.

It is not uncommon to have issues in the Perth Metropolitan Area with clogging of sub-soil drains due to high iron concentrations in the groundwater which precipitates out in the drain pipes and thereby reduces the efficiency of the system over time. The groundwater quality results from along the alignment indicate a large range in iron concentrations ranging between 0.1 mg/L and 22 mg/L, which is lower than compared to groundwater in other parts of Perth (e.g. up to 100 mg/L in Perth CBD). The risk of clogging would therefore depend on the site specific groundwater quality. Other reductions in efficiency of the pipes could be caused by scaling of the pipes due to calcium carbonate or gypsum, but this is normally more easily managed than iron precipitation. The design of a drainage system must consider that access is provided to the drain system to allow for periodic cleaning of the pipes and maintenance/replacement of pumps.

8.0 INFILTRATION CAPACITY

This section presents the estimated infiltration capacity based on infiltration tests carried out at Bayswater Dive Structure, Airport West Station, Consolidated Airport Terminal Station and the Forrestfield area. The development of the pumping test wells at the dive structures and two airport stations gave the opportunity for larger scale infiltration tests (infiltration in pits) and at Bayswater Dive Structure the water was also infiltrated on site using an infiltration basin during the pumping test.

The following infiltration tests were therefore carried out:

- **Inverse auger hole method –** A 75 mm diameter hole was drilled using a hand auger to install a 72 mm diameter slotted PVC casing which was filled with water and then the decline in water level was measured over time.
- **Infiltration pits** – Infiltration pits with base areas of 3 m^2 to 30 m^2 were excavated to a depth of 1 m and then filled with water from the development of the pumping wells; the declining water level was then measured over time.
- **Infiltration basin** – At Bayswater an infiltration basin with an area of approximately 250 m^2 was excavated to allow for infiltration of the abstracted water during the pumping test; the rate of pumping into the basin was used to estimate the infiltration rate.

Given that the infiltration pits and basins have significantly larger footprints and that significantly larger volumes of water are used, these methods are considered to be more representative of likely infiltration rates and more weight should therefore be given to the results from these tests. The inverse auger hole method was therefore only undertaken in the Forrestfield area (three tests) and at Consolidated Airport Terminal Station (three tests) to obtain additional information.

The results from the Inverse Auger Hole method are presented in Appendix G while the results from the infiltration pits and basin are presented in Appendix H.

A summary of the infiltration test results is provided in Table 10 and indicates that infiltration rates range from negligible to 12 m/d.

Table 10: Infiltration Test Results

Location	Geological Unit	Approximate Depth to Groundwater (m bgl)	Test Type	Infiltration Rates (m/d)
Bayswater Dive Structure	Bassendean Sand	2.0	Inverse Auger	NT
			Infiltration Pit	6* - 11
			Infiltration Basin	~2
Airport West Station	Bassendean Sand	2.2	Inverse Auger	NT
			Infiltration Pit	3
Consolidated Airport Terminal Station	Bassendean Sand	2.0	Inverse Auger	4 – 10
			Infiltration Pit	1**
Forrestfield area	Bassendean Sand	Unknown	Inverse Auger	3 – 12
		6.2	Infiltration Pit	Negligible

Notes: *The lower range is after consecutive tests in the same infiltration pit.
**Some Coffee rock encountered in the infiltration pit,
NT = No Tests.

The infiltration capacity of the ground below an infiltration area such as a drainage swale or soak well will depend on a number of factors including:

- the soil profile below the base of the infiltration area, both within the unsaturated (above the groundwater level) and the saturated zone below the groundwater level
- the depth to groundwater level below the base of the infiltration area
- a reduction in permeability caused by the build-up of any fines or other contaminants at the base of the infiltration area carried in by the water that is discharged to the infiltration area
- any compaction of the soil below the base of the infiltration area undertaken during construction.

The inverse auger hole test indicates a high permeability of the tested Bassendean Sand and Fill, which is expected given that the sand is fine to coarse grained and that the tests were short-term and carried out at a shallow depth that would generally not be influenced by the proximity to groundwater level. However, the larger area and longer duration infiltration pit and basin tests indicate lower infiltration rates. These lower infiltration rates are considered to be more representative of conditions applicable to a drainage swale since these tests would have resulted in saturation of the zone between the base of the pit/basin and the groundwater level. Once the saturation front reaches the groundwater level, the hydraulic gradient decreases significantly, which leads to a proportional decrease in infiltration rate.

Given the shallow depth to the groundwater level it is therefore estimated that short-term (less than 1 hour) infiltration rates could be about 5 to 10 m/d in the Bassendean Sand while longer term (several hours to days) infiltration rates are likely to be around 1 to 5 m/d.

Based on the infiltration test results and the presence of low permeability Guildford Formation soils at shallow depth (which are found to cause locally perched groundwater conditions), the Forrestfield area is considered to have the poorest infiltration capacity of the four areas assessed along the alignment. Therefore even though the results from the inverse auger test indicate relatively high infiltration rates, the presence of clayey material within a few metres from the ground surface would significantly reduce the infiltration capacity in this area.

The shallow depth to groundwater level over the whole alignment and the lower permeability soils in the Forrestfield area must be considered in the design of stormwater infiltration/soakage.

9.0 KEY HYDROGEOLOGICAL CHARACTERISTICS AND CONSIDERATIONS

The following sections summarise the key hydrogeological characteristics observed during the Stage 2 Geotechnical Investigation, which will need to be considered during design and construction.

9.1 Bayswater Dive Structure

For the Bayswater Dive Structure the following key points have been identified:

- A two aquifer system exists in the area with a difference in groundwater level and piezometric level in March 2015 between the Upper and Lower Aquifers of about 3.7 m and with a downward vertical gradient (i.e. groundwater level is highest in the Upper Aquifer).
- The results from the pumping test, which was carried out in the Lower Aquifer, indicated that the two aquifers are isolated from each other. This isolation is clearly illustrated by the response in two monitoring wells that were installed 5 m from the pumping well, which during the test pumping indicated no drawdown in the Upper Aquifer and 9 m of drawdown in the Lower Aquifer.
- Pumping from the Lower Aquifer resulted in a large cone of depression with a reduction in piezometric level in the Lower Aquifer observed to be approximately 8.5 m at a distance of 40 m from the pumping well and up to 1.5 m approximately 500 m away from the pumping well towards the Swan River.
- The pumping test analysis indicates that heterogeneous conditions are present in the Lower Aquifer.
- Previous investigations indicate that the groundwater quality is significantly different in the Upper and Lower Aquifers and that the Upper Aquifer is impacted from groundwater contamination from a site located to the north-east of the Bayswater Dive Structure. Careful management will therefore be required to avoid cross-contamination between the two aquifers during and post-construction and the dewatering system for the Upper and Lower Aquifers will need to be designed so that each aquifer is dewatered separately.
- The installation of the D-walls to facilitate excavation and construction of the dive structure would create a permanent groundwater flow barrier in the Upper Aquifer where the walls are aligned obliquely or perpendicular to the groundwater flow direction, which could result in groundwater mounding against the up-gradient side of the D-walls. The groundwater mounding could potentially require some groundwater control in areas where the groundwater level is close to the ground surface and may result in an increase in hydrostatic pressure against the D-walls.
- Reference Design Summary Table RevB (for Reference design Rev F) indicates that at the deeper part of the dive structure and tunnel portal the D-walls will extend into the Osborne Formation. This would also significantly reduce the groundwater inflow from the Lower Aquifer into the excavation. The estimated low dewatering rate is therefore expected given that the pumping test well was able to dewater the Lower Aquifer to below the required dewatering level (without retaining walls) during the pumping test.
- At the Bayswater Main Drain, the D-walls may need to be extended to below the invert level of the Bayswater Main Drain, which would require diversion of the Bayswater Main Drain. This diversion could require additional groundwater control (dewatering) during construction.

9.2 Airport West Station

For the Airport West Station the following key points have been identified:

- Hydraulic testing at Airport West indicates that that the hydraulic conductivity of the Kardinya Shale of the Osborne Formation is very low. According to the Reference Design Revision F the D-walls will extend into the Kardinya Shale (OFs) of the Osborne Formation and thereby cut off direct inflow from the Superficial Aquifer. Dewatering requirements will therefore only consist of emptying out the storage

contained by the D-walls and any groundwater seepage through the Kardinya Shale as well as through the D-walls. Minimising seepage through the D-walls would reduce the inflow rates. Minimisation of seepage through the walls would require that water stops between panels are installed to the bottom of the D-walls (or at close as practicable).

- Given the expected low dewatering rates, a recharge well system for dewatering discharge disposal may not be required. Alternative options such as disposal to the stormwater system may be a suitable option.
- The pumping test indicates that the Belmont Main Drain will act as a positive boundary to drawdown of the groundwater table. However, a reduction in piezometric pressure was also observed in the monitoring wells on the southern side of the Belmont Drain, indicating that the cone of depression in the Ascot Formation will extend beyond the Belmont Drain to the south, should it be decided that groundwater abstraction from the Ascot Formation would be required.
- The groundwater and piezometric levels in the monitoring wells were found to be affected by local council and individual household groundwater bore reticulation systems, suggesting that any abstraction from the Superficial Aquifer could affect nearby existing wells if drawdown is not carefully managed.
- The tunnel alignment in this area and the Airport West Station are generally located parallel to the groundwater flow direction from east to west toward the Swan River. As such, construction of the tunnel and installation of D-walls for the Airport West Station is unlikely to significantly impede groundwater flow. Nevertheless, given the shallow depth to groundwater level, it is recommended that the effect of construction of the tunnel and station infrastructure be assessed during detailed design.

9.3 Consolidated Airport Terminal Station

For the Consolidated Airport Terminal Station the following key points have been identified:

- According to the Reference Design Revision F the D-walls will extend 3 m into the Mirrabooka Member of the Osborne Formation (OFm) and thereby cut off direct groundwater inflow from the Superficial Aquifer. The main groundwater inflow into the excavation will be from the Mirrabooka Aquifer. Understanding the depth, extent and hydrogeological properties of the Mirrabooka Aquifer therefore becomes the most important factor in estimating the dewatering requirements for the Consolidated Airport Terminal Station.
- The available borehole logs in the Mirrabooka Aquifer indicate that the hydraulic conductivity will reduce with depth due to the increasing influence of interbedded layers of sandy mudstone which are expected to have a significantly lower hydraulic conductivity. None of the boreholes drilled into the Mirrabooka Aquifer penetrated through it and the thickness of the Mirrabooka Aquifer is therefore not well defined.
- The analysis of the pumping test from the Consolidated Airport Terminal Station showed that the best fit when replicating the pumping test data in the groundwater model was obtained using a ratio of vertical to horizontal hydraulic conductivity of 0.1 for all the hydrogeological units. The dewatering scenarios indicate that the ratio would have a significant influence on the final dewatering rates.
- Based on the importance of the Mirrabooka Aquifer it is recommended that additional investigation is undertaken to determine the thickness and aquifer properties of the Mirrabooka Aquifer. The pumping test should include a monitoring well configuration that allows for determination of the vertical and horizontal hydraulic conductivity and to assess whether the hydraulic conductivity reduces with depth.
- The tunnel alignment in this area and the Consolidated Airport Terminal Station are generally located parallel to the groundwater flow direction from east to west toward the Swan River. As such, the construction of the tunnel and installation of D-walls for the Consolidated Airport Terminal Station is therefore unlikely to significantly impede groundwater flow. Nevertheless, given the shallow depth to

groundwater level, it is recommended that the effect of construction of the tunnel and station infrastructure be assessed during detailed design.

9.4 Forrestfield Area

For the Forrestfield area the following key points have been identified:

- The pumping test analysis at the Forrestfield Dive Structure does not provide definitive conclusion to the ratio between the vertical and horizontal hydraulic conductivity in this area (the use of 0.1 to 1 gave almost similar results – best fit was 0.25). The groundwater model scenario runs indicate that this ratio will influence the dewatering rates.
- The groundwater level measurements in the installed monitoring wells and VWPs indicate a large (approximately 3 m) seasonal variation in groundwater level as well as the presence of perched groundwater conditions in this area. Given that the Stage 2 investigation was carried out in the dry season and perched groundwater may dry out during summer in some areas, the extent and connectivity with the regional aquifer is currently unknown and can only be established through groundwater level monitoring over the wet season. It is therefore recommended that additional monitoring wells are installed and groundwater level monitoring undertaken during the wet season. It is also recommended that following a large rainfall event during the wet season a site walkover be undertaken to gain a better understanding of the potential extent of inundated areas.
- The groundwater quality during the pumping test was found to be acidic (average pH of 5.7) with limited buffering capacity, which would need to be considered during the dewatering design. The high nutrient concentrations may restrict construction dewatering and long-term drainage disposal to the stormwater system.
- Based on the infiltration test results and the presence of low permeability soils at shallow depth the Forrestfield area is considered to have the poorest infiltration capacity of the four areas assessed along the alignment.
- The D-walls associated with the Forrestfield Dive Structure will be constructed perpendicular to the groundwater flow direction. However, according to Reference Design Summary Table RevB (for Reference design Rev F) the D-walls do not extend through the full thickness of the Superficial Aquifer and it is therefore considered unlikely that the D-walls will significantly impede groundwater flow and result in significant groundwater mounding behind the up-gradient D-wall. Nevertheless, given the presence of the seasonally perched groundwater conditions, it is recommended that the effect the installation of D-walls could have on the groundwater level at the dive structure be assessed during the detailed design.

10.0 RECOMMENDATIONS FOR FURTHER WORK

Further hydrogeological investigation is recommended to allow detailed design of specific areas or to provide additional information to reduce uncertainty. In summary the following additional investigation work is currently recommended:

1) General:
   a) Establish a groundwater monitoring program along the alignment.
   b) Carry out single well hydraulic testing in future installed wells.
2) At Airport West Station:
   a) Install additional monitoring wells which are screened in the Osborne Formation at the new proposed Airport West Station location.
   b) Carry out single well hydraulic testing in the new wells to provide confirmation of the hydraulic conductivity of the Kardinya Shale Member.

3) At Consolidated Airport Terminal Station:

   a) Drill an additional geotechnical borehole to determine the thickness of the Mirrabooka Aquifer.

   b) Install a new pumping test well in the Mirrabooka Aquifer and carry out a pumping test. .

4) At Forrestfield Area:

   a) Undertake further investigation to assess the extent of the perched groundwater conditions.

   b) Undertake further assessment to enable a better understanding of the connectivity between the regional aquifer and the perched groundwater conditions.

   c) Carry out additional infiltration testing in areas where stormwater soakage may be considered.

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12.0 LIMITATIONS

Your attention is drawn to the document – "Limitations", which is included as Appendix I to this report. This document is intended to assist you in ensuring that your expectations of this report are realistic, and that you understand the inherent limitations of a report of this nature. If you are uncertain as to whether this report is appropriate for any particular purpose please discuss this issue with us.

Report Signature Page

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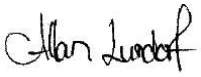

Michael Bartlett
Hydrogeologist

Allan Lundorf
Senior Water Resources Engineer

MJB,AL/AL,DPS/asu

A.B.N. 64 006 107 857

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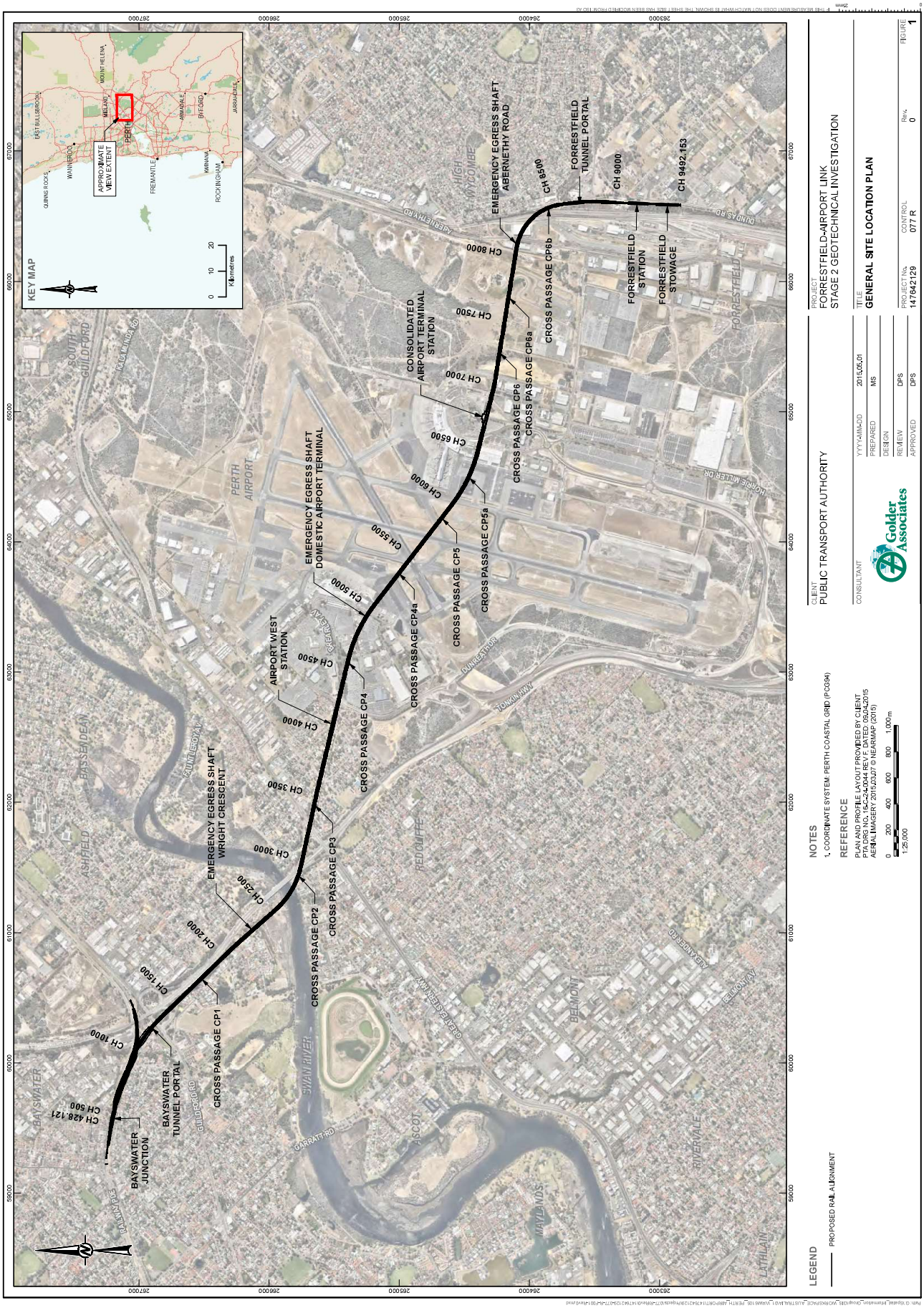
May 2015
Report No. 147642129-077-R-Rev0


KEY MAP
APPROXIMATE VIEW EXTENT
PERTH
MIDLAND
MOUNT HELENA
ARMADALE
BYFORD
FREMANTLE
KWINANA
ROCKINGHAM
WANNEROO
EAST BULLSBROOK
QUINNS ROCKS
0 10 20 Kilometres
LEGEND
PROPOSED RAIL ALIGNMENT
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PERTH AIRPORT 2.5M CONTOURS DATED 25/02/2015
AERIAL IMAGERY 2015,2017 & NEARMAP (2019)
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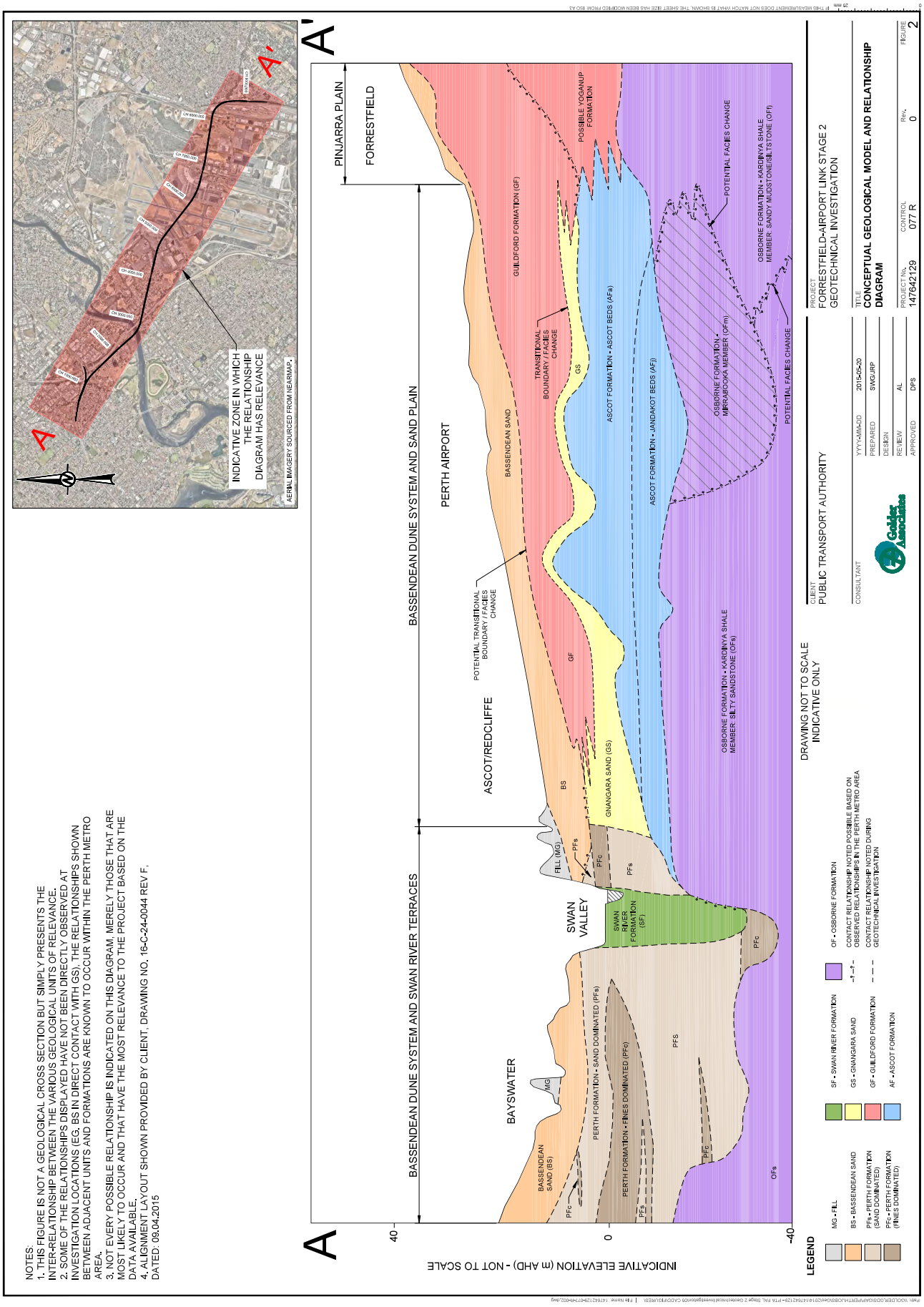
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.01
PREPARED MS
DESIGN
REVIEW DPS
APPROVED DPS
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GENERAL SITE LOCATION PLAN
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 1
BAYSWATER JUNCTION
BAYSWATER TUNNEL PORTAL
CROSS PASSAGE CP1
EMERGENCY EGRESS SHAFT WRIGHT CRESCENT
CROSS PASSAGE CP2
CROSS PASSAGE CP3
AIRPORT WEST STATION
CROSS PASSAGE CP4
CROSS PASSAGE CP4a
EMERGENCY EGRESS SHAFT DOMESTIC AIRPORT TERMINAL
CROSS PASSAGE CP5
CROSS PASSAGE CP5a
CONSOLIDATED AIRPORT TERMINAL STATION
CROSS PASSAGE CP6
CROSS PASSAGE CP6a
CROSS PASSAGE CP6b
EMERGENCY EGRESS SHAFT ABERNETHY ROAD
FORRESTFIELD TUNNEL PORTAL
FORRESTFIELD STATION
FORRESTFIELD STOWAGE
CH 428.121
CH 500
CH 1000
CH 1500
CH 2000
CH 2500
CH 3000
CH 3500
CH 4000
CH 4500
CH 5000
CH 5500
CH 6000
CH 6500
CH 7000
CH 7500
CH 8000
CH 8500
CH 9000
CH 9492.153
BAYSWATER
GUILDFORD
BASSENDEAN
ASHFIELD
SOUTH GUILDFORD
PERTH AIRPORT
HIGH WYCOMBE
FORRESTFIELD
REDCLIFFE
BELMONT
ASCOT
RIVERVALE
MAYLANDS
LATHLAIN
SWAN RIVER
GUILDFORD RD
GREAT EASTERN HWY
TONKIN HWY
ABERNETHY RD
DUNDAS RD
HORRIE MILLER DR
KALAMUNDA RD
GARRATT RD
GOSNELLS STATION HWY
BELMONT AV
KALAMUNDA RD



NOTES:
1. THIS FIGURE IS NOT A GEOLOGICAL CROSS SECTION BUT SIMPLY PRESENTS THE INTER-RELATIONSHIP BETWEEN THE VARIOUS GEOLOGICAL UNITS OF RELEVANCE.
2. SOME OF THE RELATIONSHIPS DISPLAYED HAVE NOT BEEN DIRECTLY OBSERVED AT INVESTIGATION LOCATIONS (EG, BS IN DIRECT CONTACT WITH GS), THE RELATIONSHIPS SHOWN BETWEEN ADJACENT UNITS AND FORMATIONS ARE KNOWN TO OCCUR WITHIN THE PERTH METRO AREA.
3. NOT EVERY POSSIBLE RELATIONSHIP IS INDICATED ON THIS DIAGRAM, MERELY THOSE THAT ARE MOST LIKELY TO OCCUR AND THAT HAVE THE MOST RELEVANCE TO THE PROJECT BASED ON THE DATA AVAILABLE.
4. ALIGNMENT LAYOUT SHOWN PROVIDED BY CLIENT, DRAWING NO. 16-C-24-0044 REV F, DATED: 09.04.2016

NOTES:
1. THIS FIGURE IS NOT A HYDROGEOLOGICAL CROSS SECTION BUT SIMPLY PRESENTS THE INTER-RELATIONSHIP BETWEEN THE VARIOUS HYDROGEOLOGICAL UNITS OF RELEVANCE.
2. SOME OF THE RELATIONSHIPS DISPLAYED HAVE NOT BEEN DIRECTLY OBSERVED AT INVESTIGATION LOCATIONS (EG. BS IN DIRECT CONTACT WITH GS), THE RELATIONSHIPS SHOWN BETWEEN ADJACENT UNITS AND FORMATIONS ARE KNOWN TO OCCUR WITHIN THE PERTH METRO AREA.
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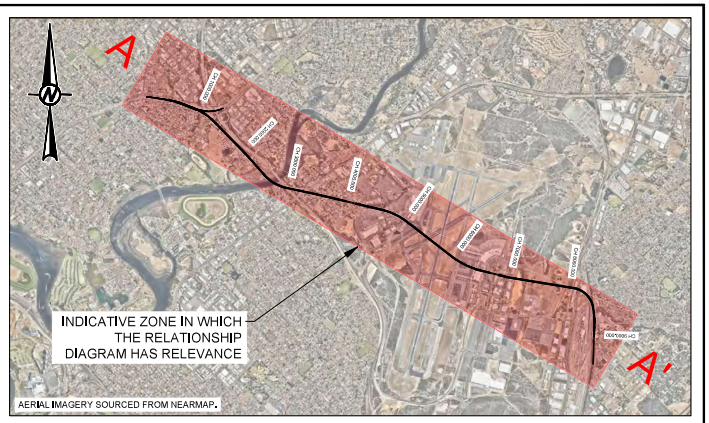
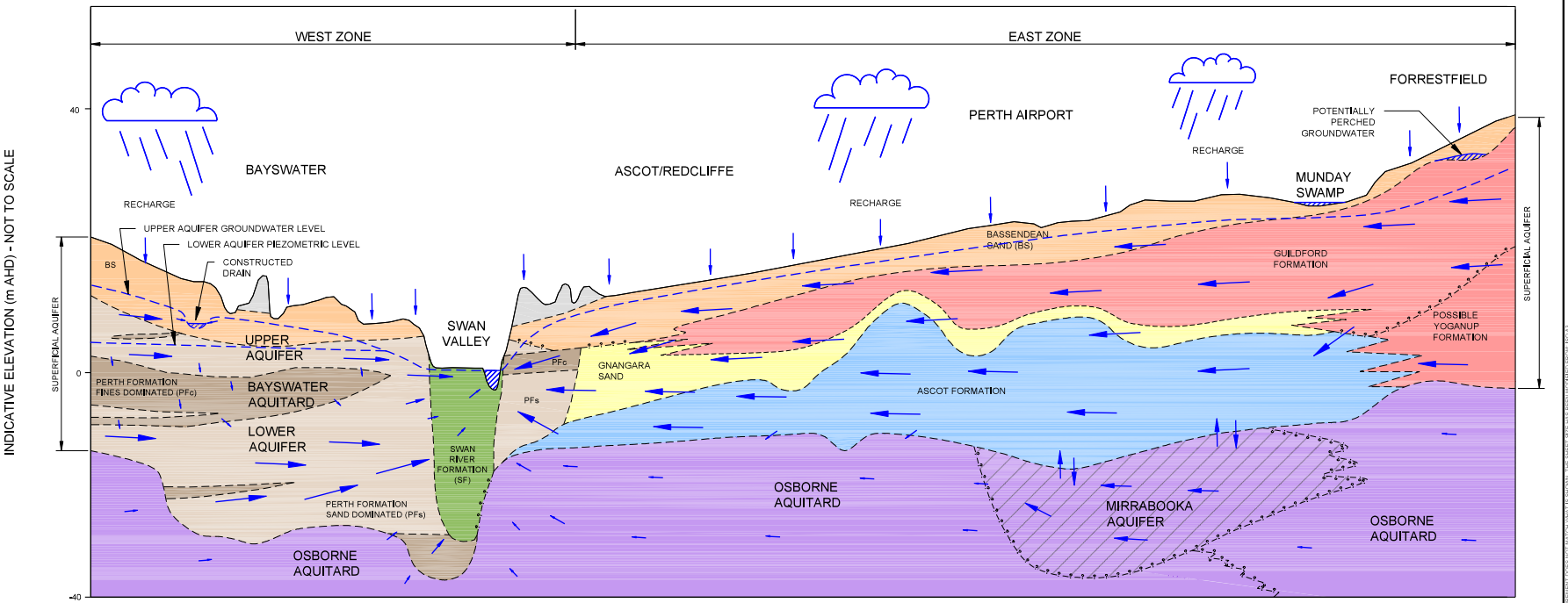
4. ALIGNMENT LAYOUT SHOWN PROVIDED BY CLIENT, DRAWING NO. 16-C-24-0044 REV F, DATED: 09.04.2015





LEGEND

- MG - FILL
- BS - BASSENDEAN SAND
- PFs - PERTH FORMATION (SAND DOMINATED)
- PFc - PERTH FORMATION (FINES DOMINATED)
- SF - SWAN RIVER FORMATION
- GS - GNANGARA SAND
- GF - GUILDFORD FORMATION
- AF - ASCOT FORMATION
- OF - OSBORNE FORMATION
- CONTACT RELATIONSHIP NOTED POSSIBLE BASED ON OBSERVED RELATIONSHIPS IN THE PERTH METRO AREA
- CONTACT RELATIONSHIP NOTED DURING GEOTECHNICAL INVESTIGATION
- INDICATIVE GROUNDWATER FLOW LINE
- APPROX. INTERPRETED GROUNDWATER LEVEL

DRAWING NOT TO SCALE
INDICATIVE ONLY

CLIENT: PUBLIC TRANSPORT AUTHORITY

CONSULTANT: Golder Associates

YYYY-MM-DD	2015-05-20
PREPARED	SWG/JRP
DESIGN	SWG
REVIEW	AL
APPROVED	DPS

PROJECT: FORRESTFIELD-AIRPORT LINK STAGE 2 GEOTECHNICAL INVESTIGATION

TITLE: **CONCEPTUAL HYDROGEOLOGICAL MODEL**

PROJECT No.	CONTROL	Rev.	FIGURE
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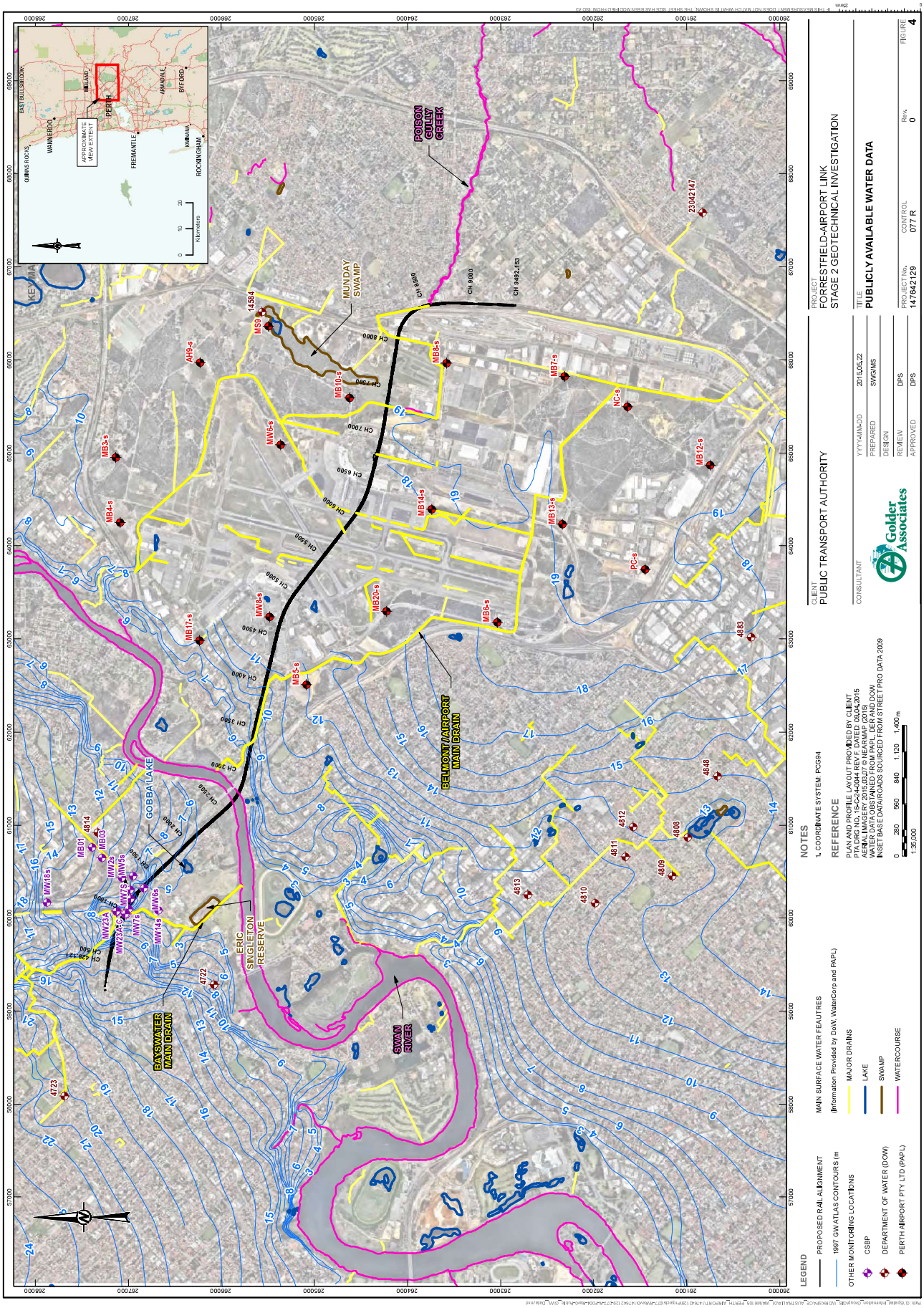
LEGEND
PROPOSED RAIL ALIGNMENT
1997 GWATLAS CONTOURS (m
OTHER MONITORING LOCATIONS
CSBP
DEPARTMENT OF WATER (DOW)
PERTH AIRPORT PTY LTD (PAPL)
MAIN SURFACE WATER FEAUTRES
(Information Provided by DoW, WaterCorp and PAPL)
MAJOR DRAINS
LAKE
SWAMP
WATERCOURSE
NOTES
1. COORDINATE SYSTEM: PCG94
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO.: 12-24-0044 REV F, DATED 03.04.2015
AERIAL IMAGERY: 2015 SOURCED FROM NEARMAP
WATER DATA OBTAINED FROM PAPL, DER AND DOW
INSET BASE DATA/ROADS SOURCED FROM STREET PRO DATA 2009
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
PUBLICLY AVAILABLE WATER DATA
PROJECT No.
147642129
CONTROL
077 R
Rev.
0
FIGURE
4
YYYY-MM-DD
2015.05.22
PREPARED
SWG/MS
DESIGN
REVIEW
DPS
APPROVED
DPS
APPROXIMATE VIEW EXTENT
EAST TULLAMORE
QUINNS ROCKS
WANNEROO
PERTH
MIDLAND
ARMADALE
BYFORD
FREMANTLE
KWINANA
ROCKINGHAM
Kilometers
POISON GULLY CREEK
MUNDAY SWAMP
BELMONT AIRPORT MAIN DRAIN
BAYSWATER MAIN DRAIN
GOBBA LAKE
ERIC SINGLETON RESERVE
SWAN RIVER
KEYMA
CH 9500
CH 9000
CH 9452.153
CH 8000
CH 7500
CH 7000
CH 6500
CH 6000
CH 5500
CH 5000
CH 4500
CH 4000
CH 3500
CH 3000
CH 2500
CH 2000
CH 1500
CH 1000
CH 500
CH 0
14584
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4848
4812
4811
4809
4808
4813
4810
4722
4723
AH9-s
MB3-s
MB4-s
MB17-s
MB01
MB02
MB5-s
MB6-s
MB7-s
MB8-s
MB10-s
MB12-s
MB13-s
MB14-s
MB20-s
NC-s
PC-s
MW9-s
MW18s
MW23A
MW23s
MW17s
MW16s
MW7s
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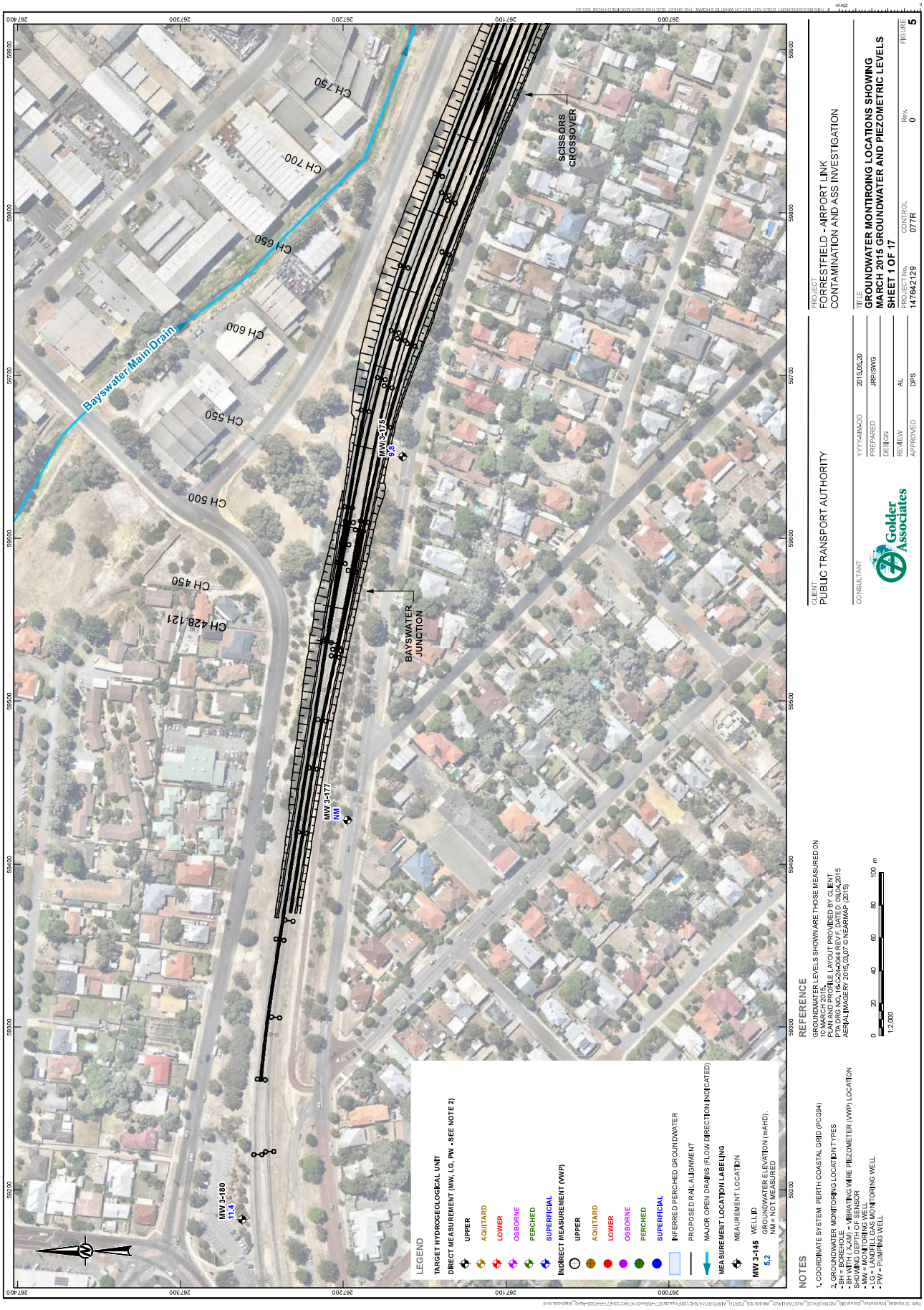
Bayswater Main Drain
CH 750
CH 700
CH 650
CH 600
CH 550
CH 500
CH 450
CH 428.121
SCISSORS CROSSOVER
BAYSWATER JUNCTION
MW 3-175
9.8
MW 3-177
NM
MW 3-180
11.4
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELING
MEAUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD), NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (X.XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-24-0044 REV F, DATED: 09.04.2015
AERIAL IMAGERY 2015.03.07 © NEARMAP (2015)
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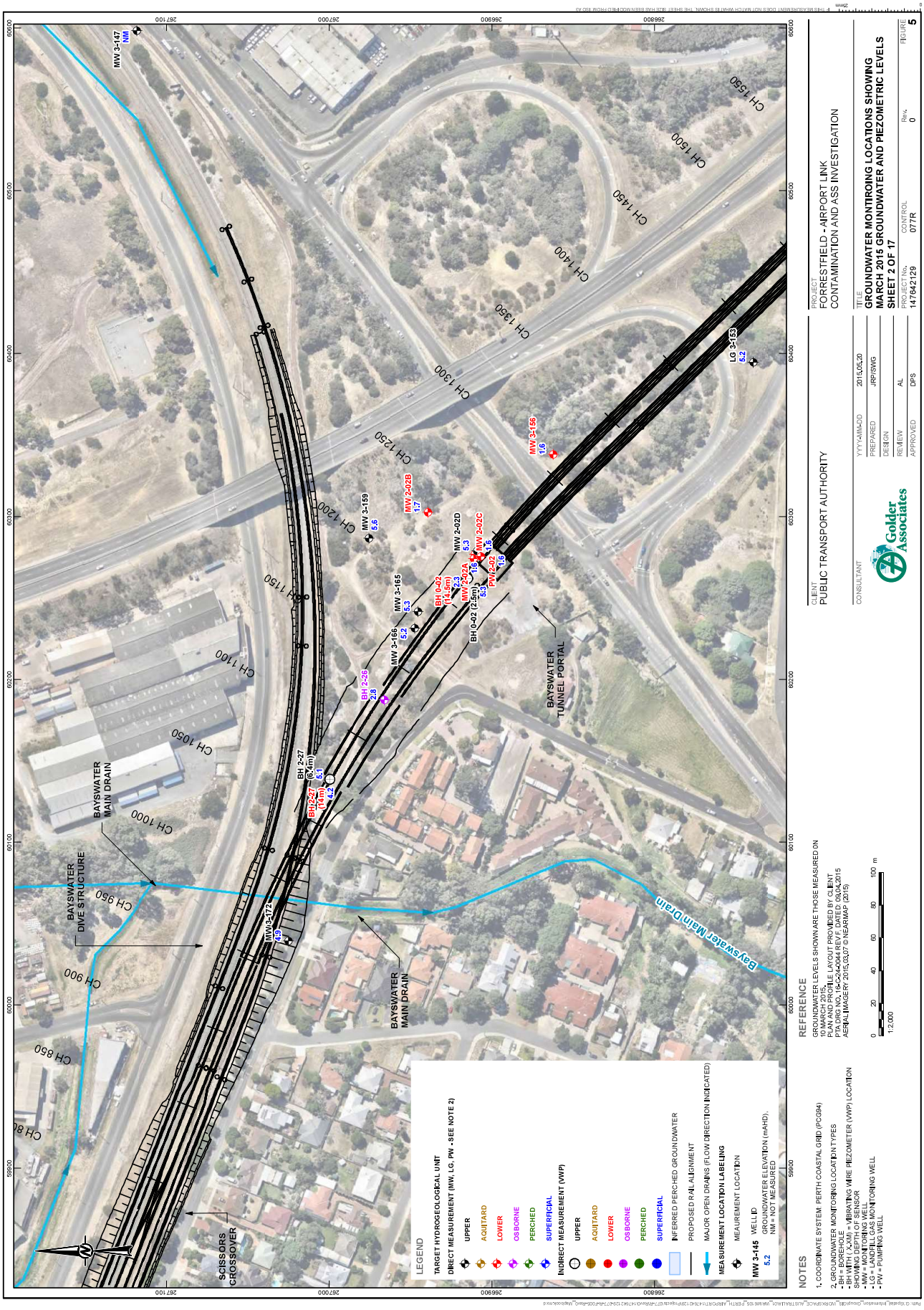
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONTIROING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 1 OF 17
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELING
MEAUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD), NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (X.XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-24-0044 REV F, DATED: 09.04.2015
AERIAL IMAGERY 2015.03.07 © NEARMAP (2015)
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CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONTIROING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 2 OF 17
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5
BAYSWATER MAIN DRAIN
BAYSWATER DIVE STRUCTURE
BAYSWATER TUNNEL PORTAL
SCISSORS CROSSOVER
Bayswater Main Drain
MW 3-147 NM
MW 3-156 1.6
LG 3-153 5.2
MW 2-02B 1.7
MW 3-159 5.6
MW 2-02D 5.3
MW 2-02C 1.6
MW 2-02A 2.3
PW 2-02 1.6
BH 0-02 (14.5m) 2.3
BH 0-02 (2.5m) 5.3
MW 3-165 5.3
MW 3-166 5.2
BH 2-26 2.8
BH 2-27 (6.4m) 5.1
BH 2-27 (14m) 4.2
MW 3-172 4.9
CH 1550
CH 1500
CH 1450
CH 1400
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CH 1300
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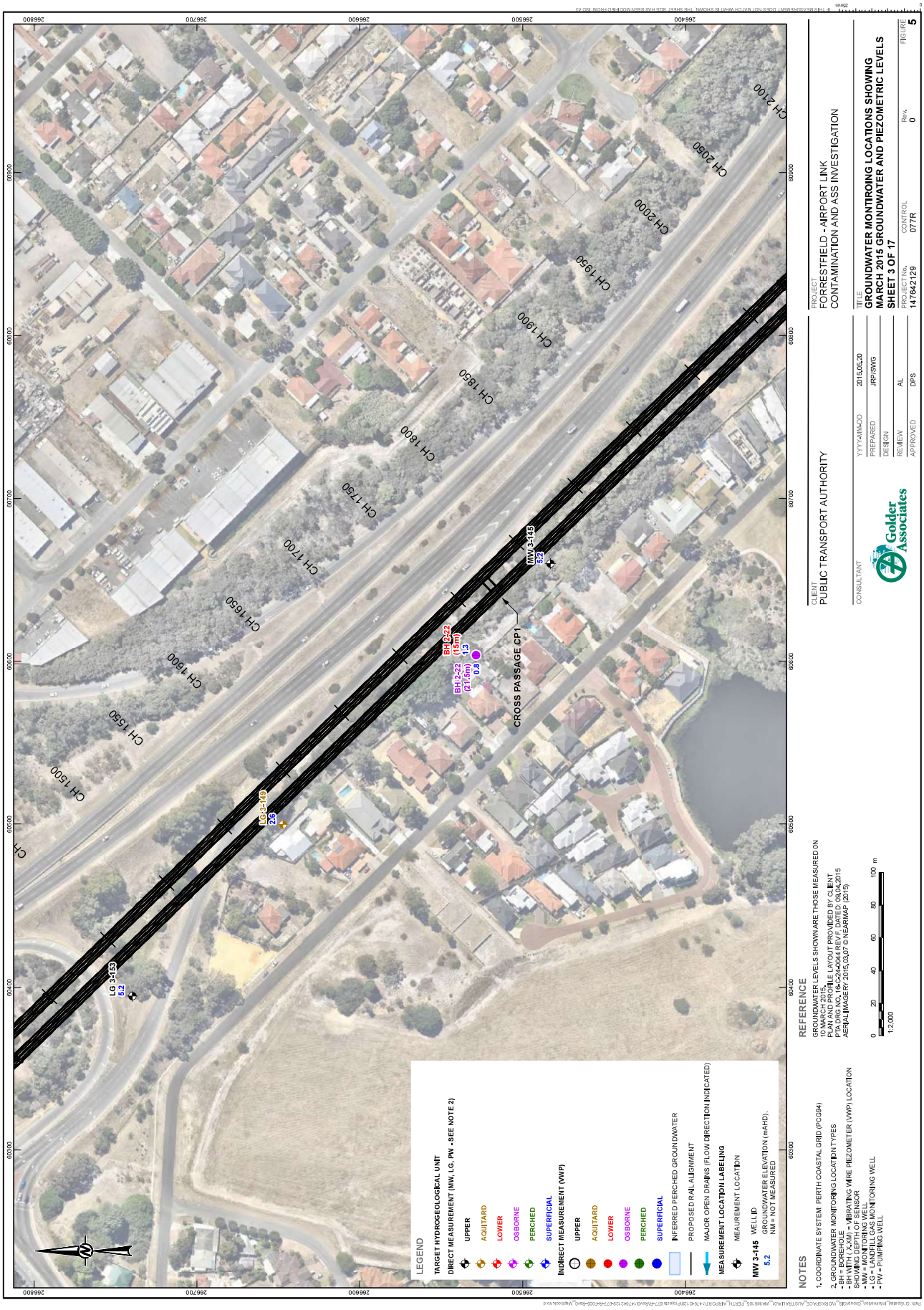
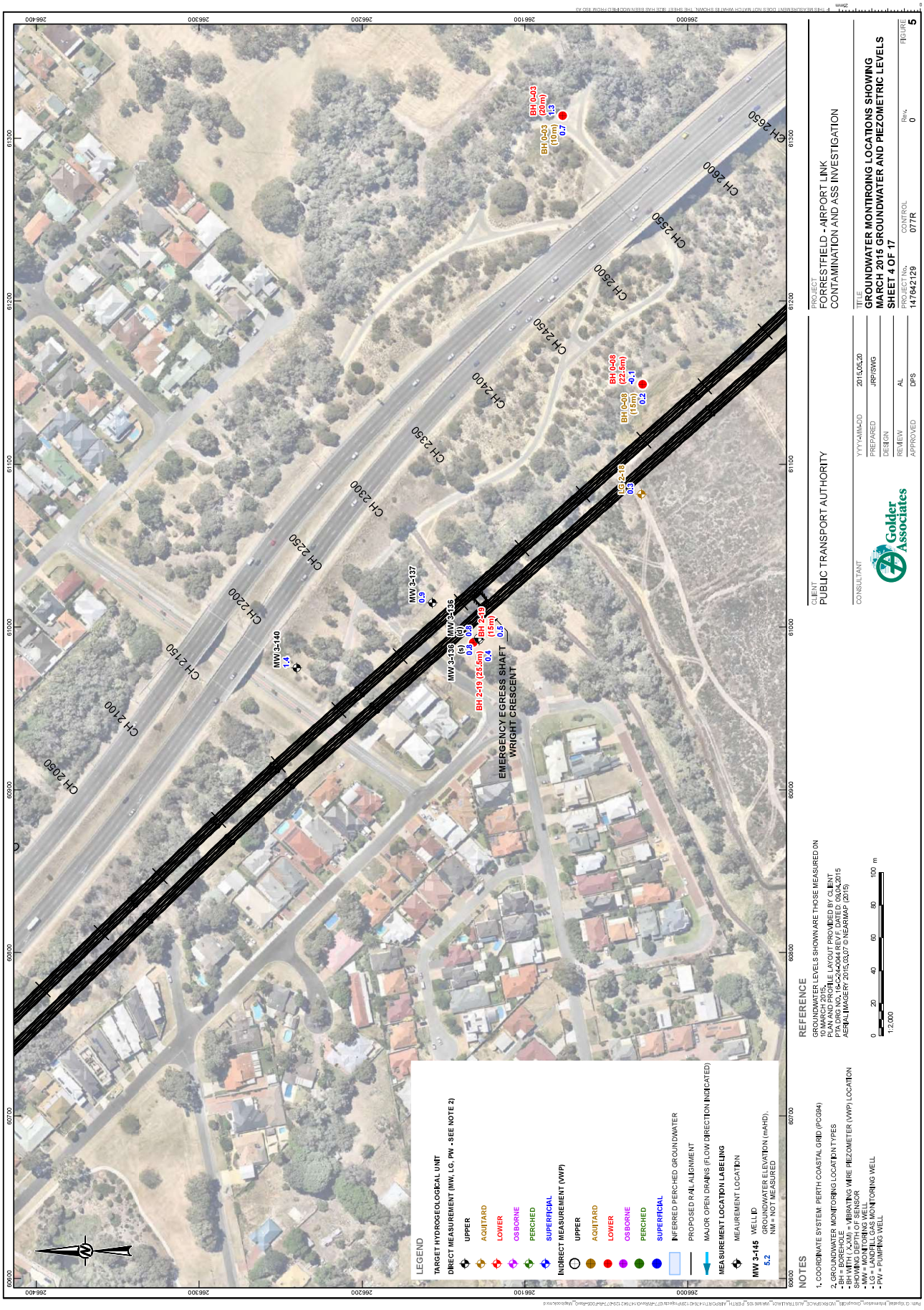
CH 1000
CH 950
CH 900
CH 850
CH 80


LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015, 03.07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CROSS PASSAGE CP1
BH 2-22 (16m) 1.3
BH 2-22 (21.5m) 0.8
MW 3-145 5.2
LG 3-153 5.2
LG 3-149 2.6
CH 1500
CH 1550
CH 1600
CH 1650
CH 1700
CH 1750
CH 1800
CH 1850
CH 1900
CH 1950
CH 2000
CH 2050
CH 2100
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 3 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5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TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = GROUNDWATER MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
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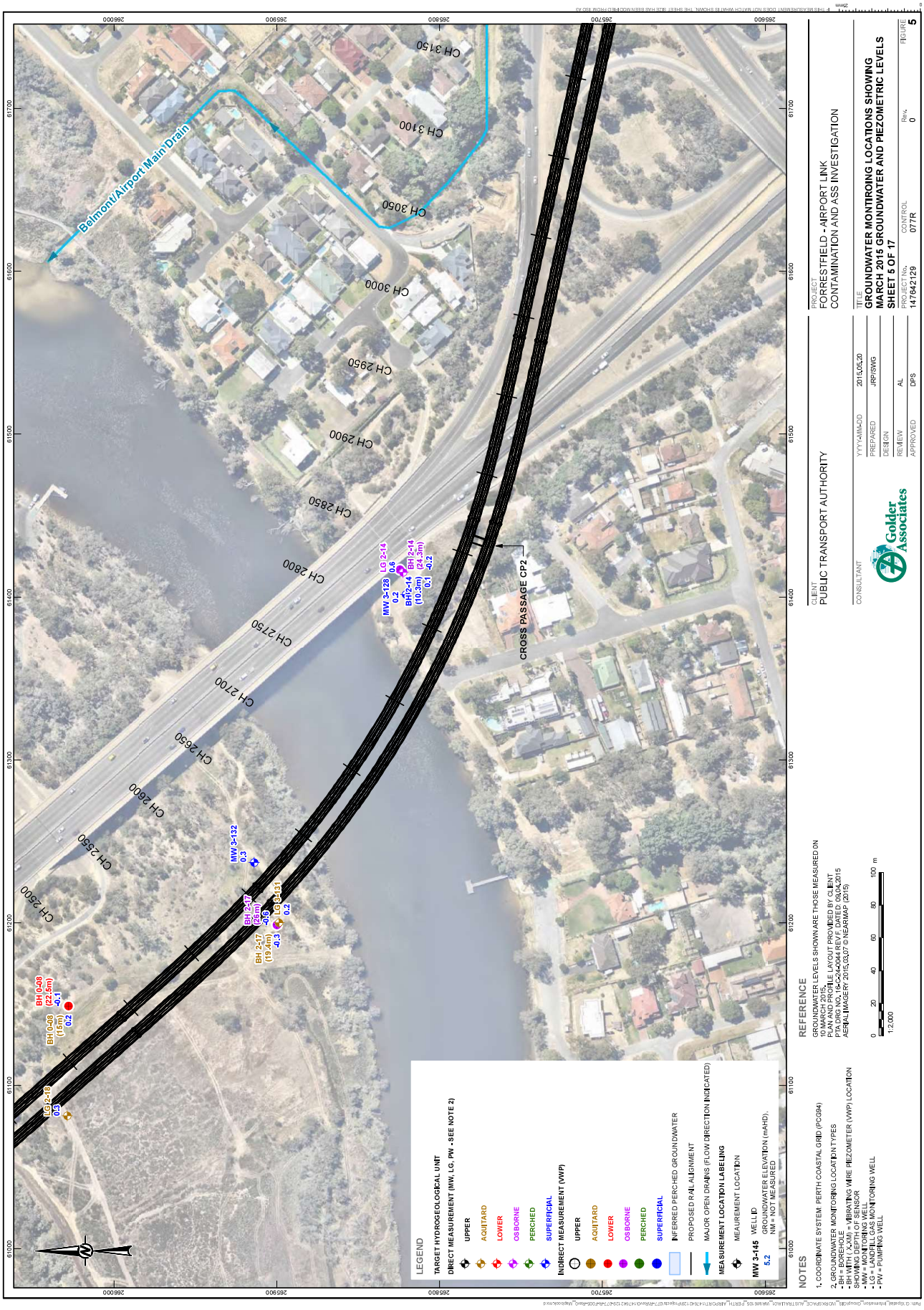
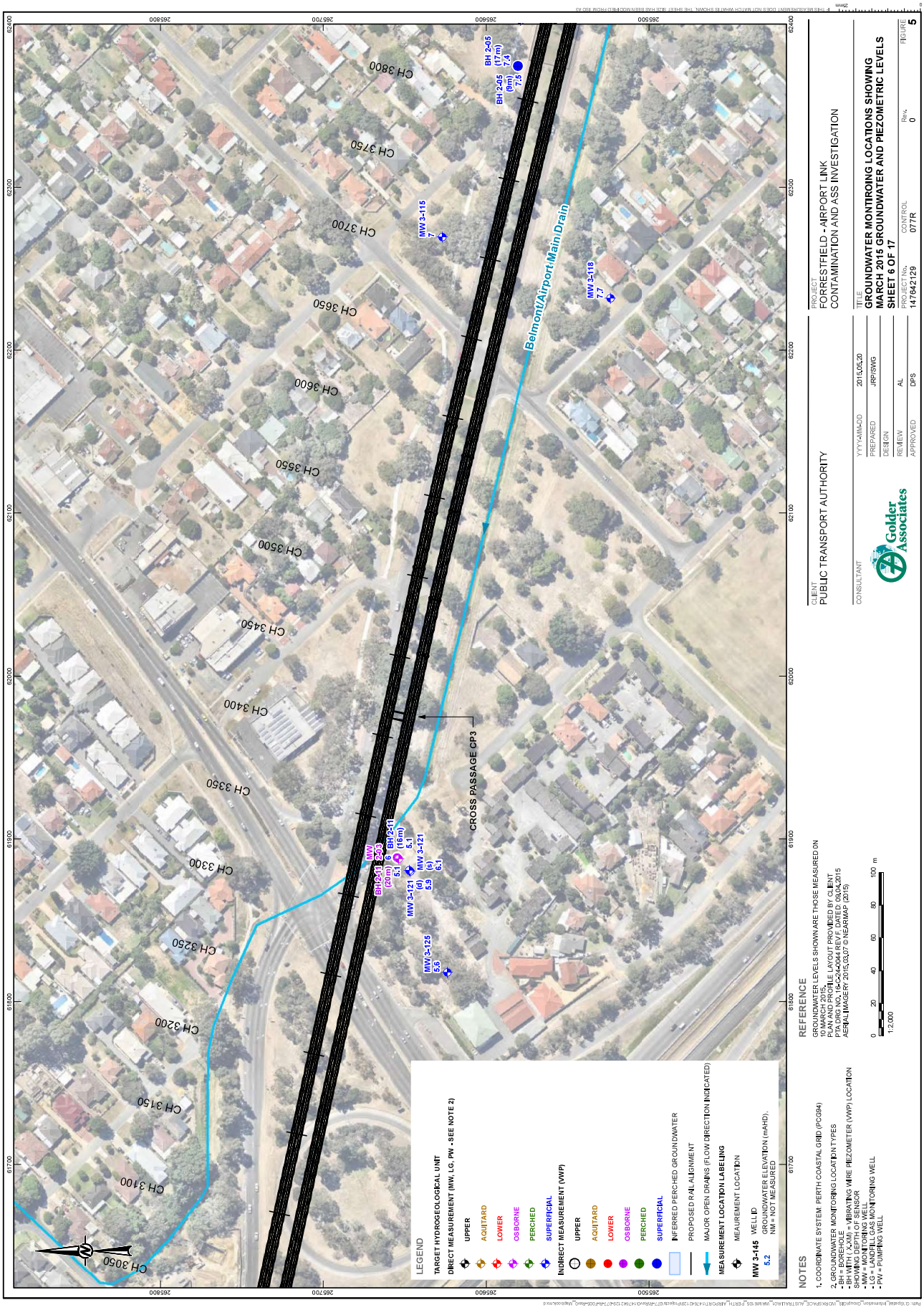
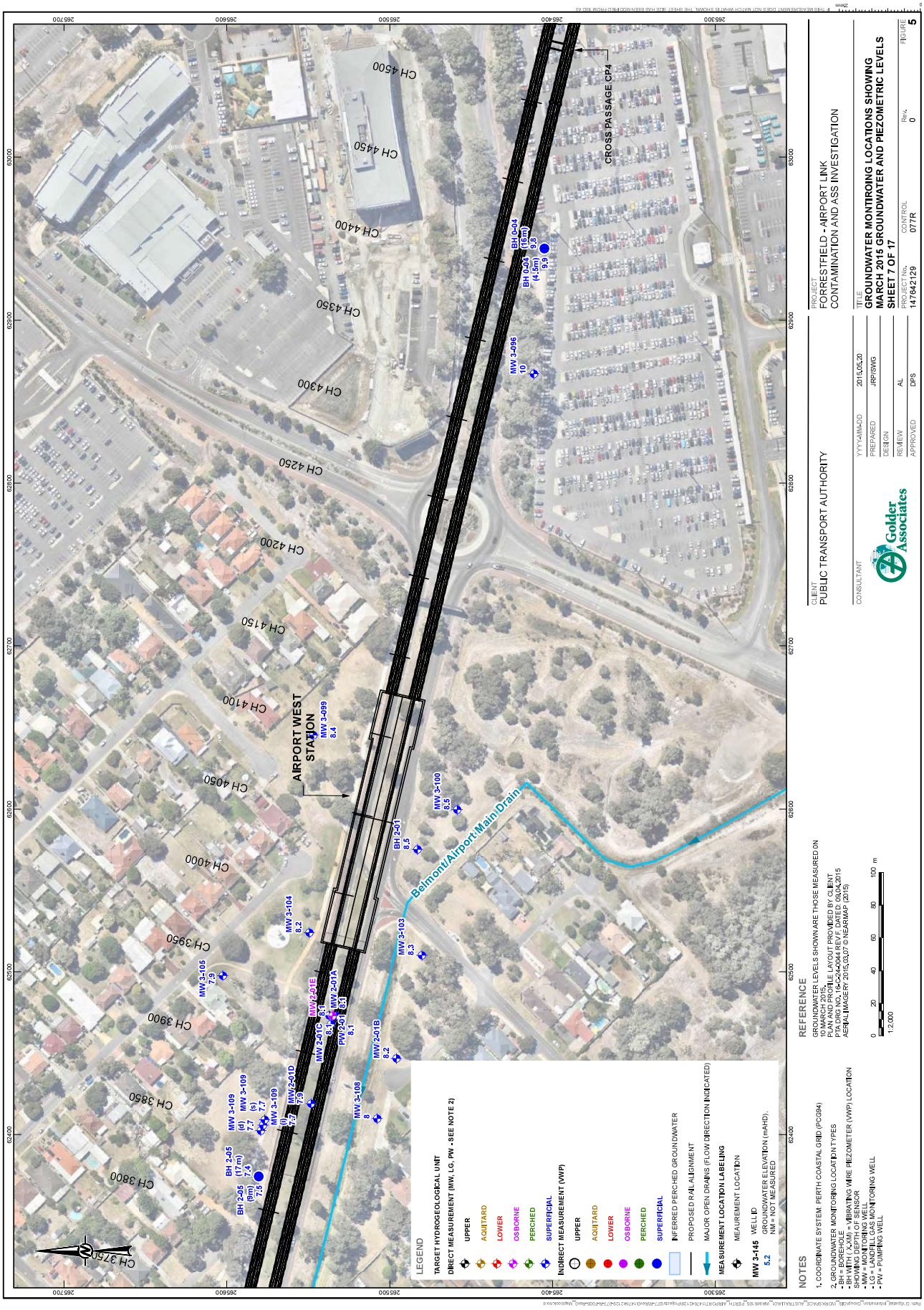
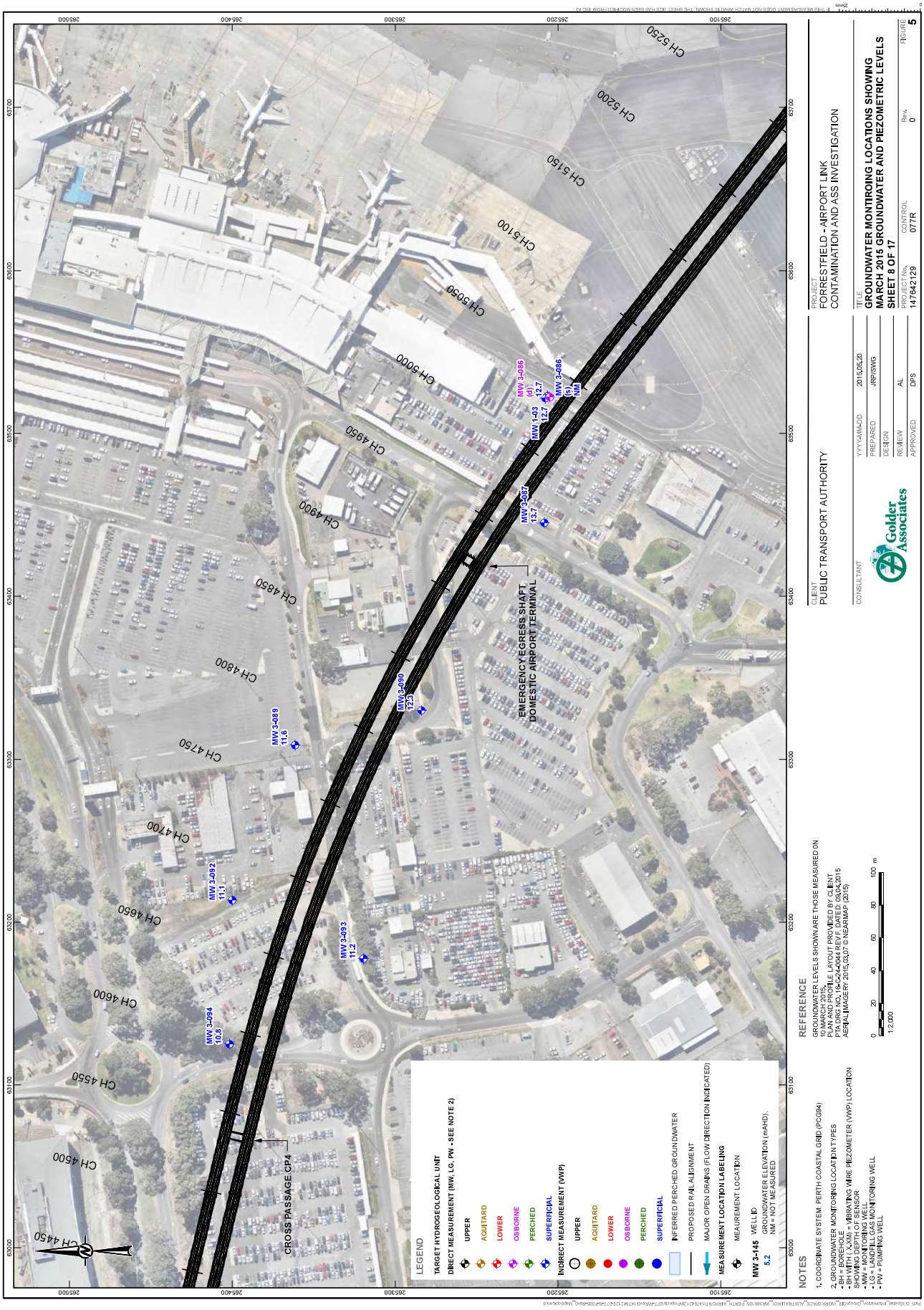
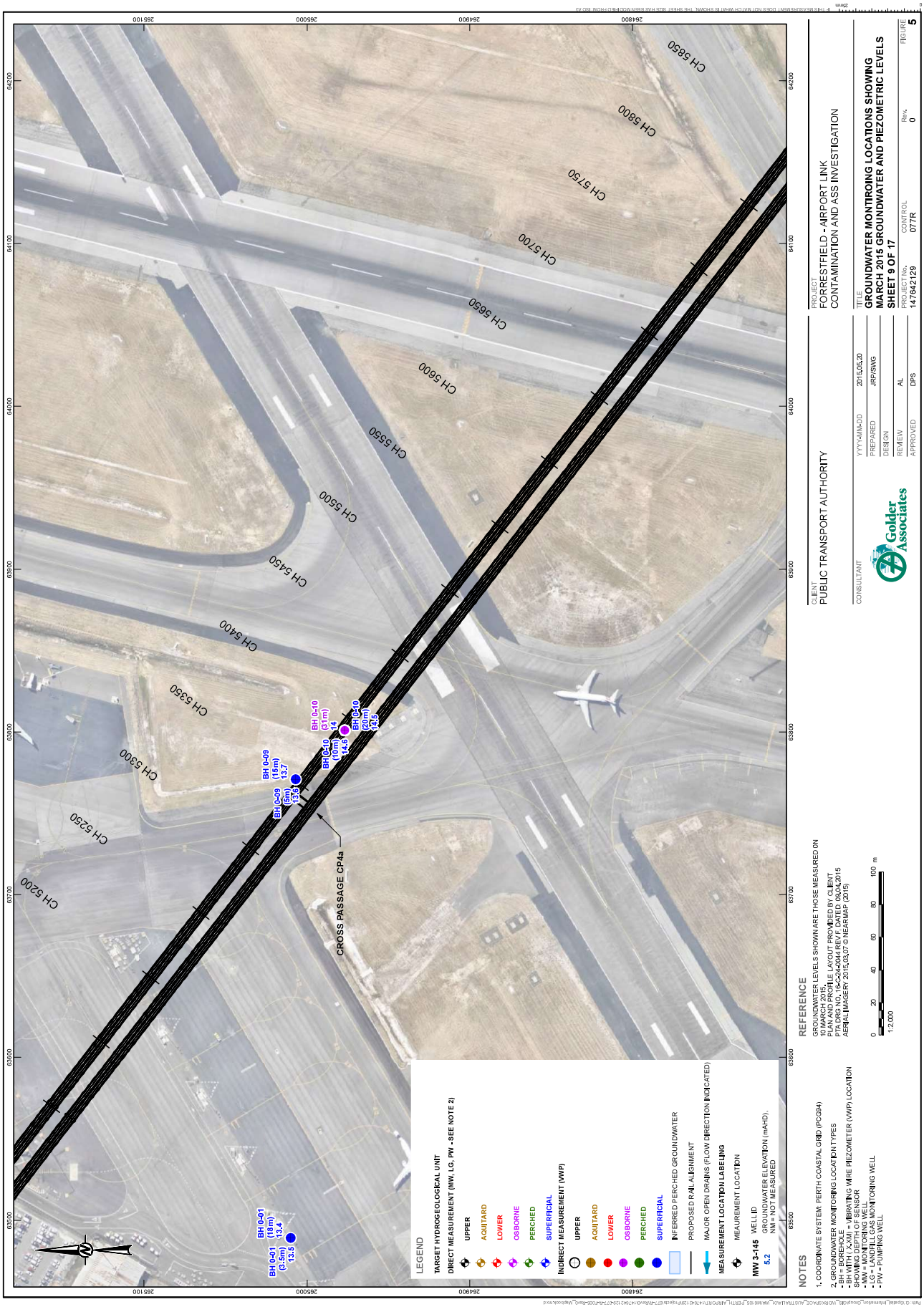
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 4 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5
EMERGENCY EGRESS SHAFT
WRIGHT CRESCENT
MW 3-140 1.4
MW 3-137 0.9
MW 3-138 (d) 0.8
MW 3-138 (s) 0.8
BH 2-19 (25.5m) 0.4
BH 2-19 (15m) 0.5
LG 2-19 0.3
BH 0-08 (22.5m) -0.1
BH 0-08 (15m) 0.2
BH 0-03 (20m) 1.3
BH 0-03 (10m) 0.7
CH 2050
CH 2100
CH 2150
CH 2200
CH 2250
CH 2300
CH 2350
CH 2400
CH 2450
CH 2500
CH 2550
CH 2600
CH 2650

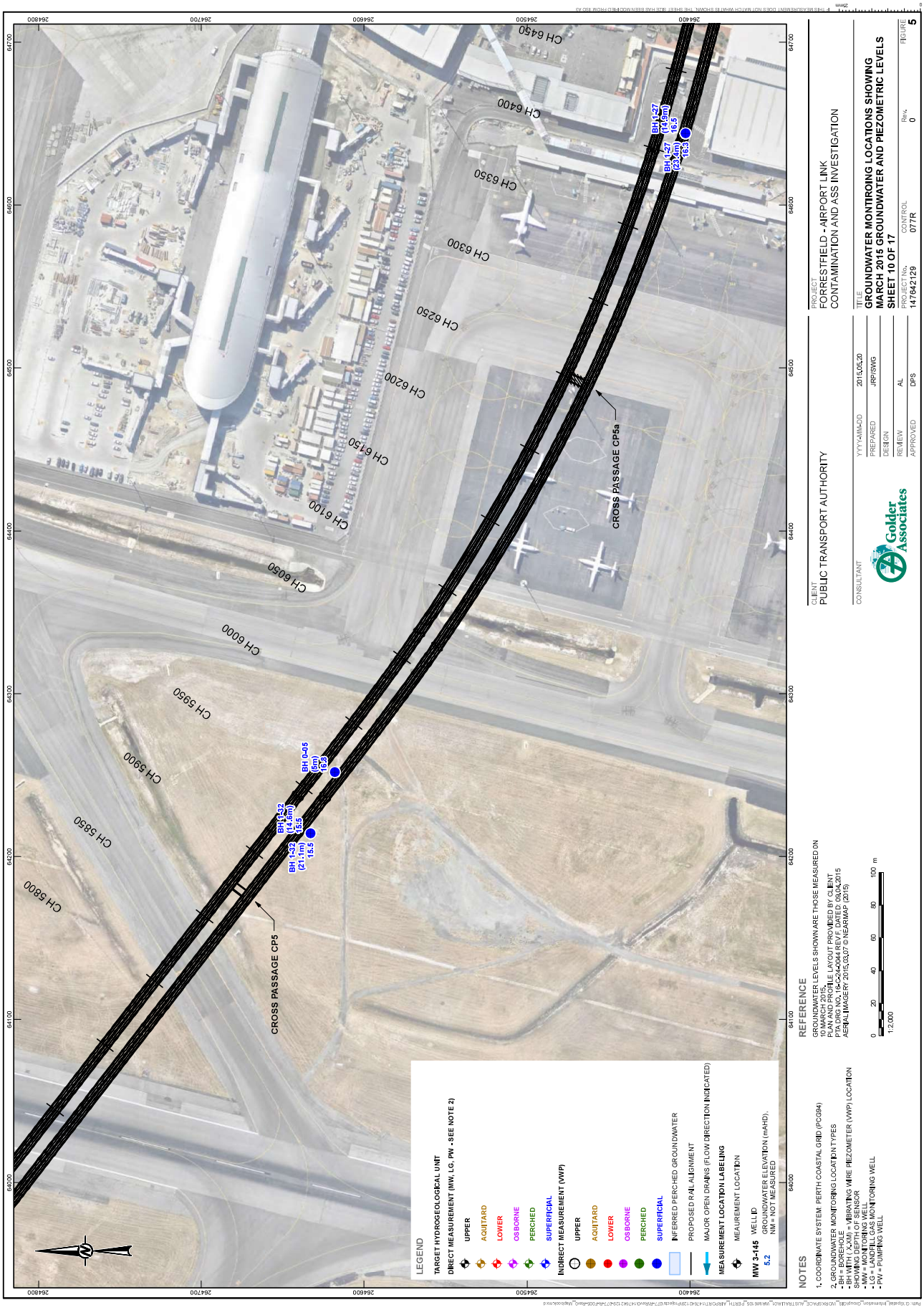

Belmont/Airport Main Drain
CH 3150
CH 3100
CH 3050
CH 3000
CH 2950
CH 2900
CH 2850
CH 2800
CH 2750
CH 2700
CH 2650
CH 2600
CH 2550
CH 2500
CROSS PASSAGE CP2
MW 3-129 0.2
LG 2-14 0.6
BH 2-14 (10.3m) 0.1
BH 2-14 (24.3m) -0.2
MW 3-132 0.3
BH 2-17 (26m) -0.5
BH 2-17 (19.4m) -0.3
LG 2-11 0.2
BH 0-08 (22.6m) -0.1
BH 0-08 (15m) 0.2
LG 2-10 0.3
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X.XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-2044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 5 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5

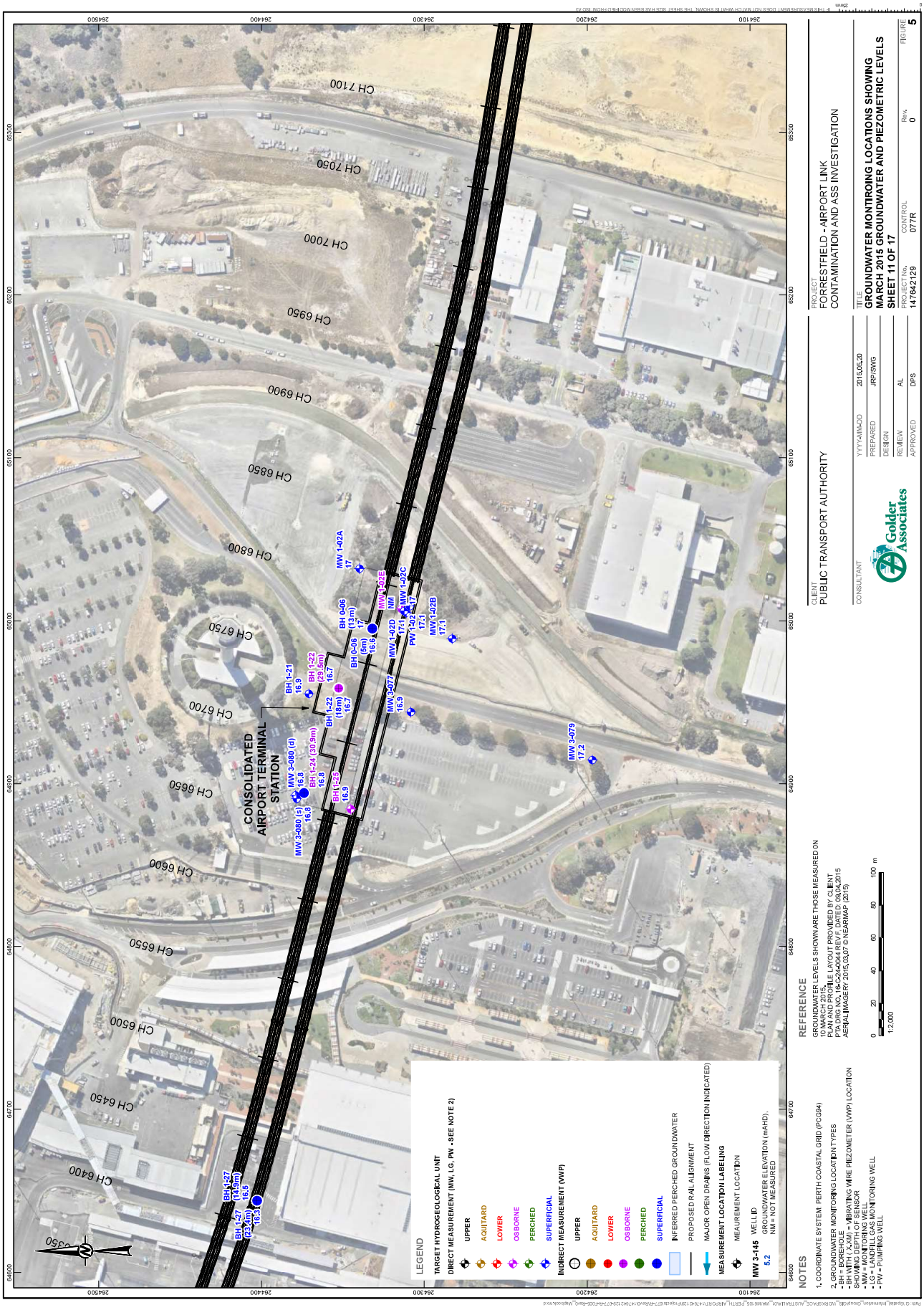

LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
2. GROUNDWATER MONITORING LOCATION TYPES
• BH = BOREHOLE
• BH WITH (-X-M) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
• MW = MONITORING WELL
• LG = LANDFILL GAS MONITORING WELL
• PW = PUMPING WELL
0 20 40 60 80 100 m
1:2,000
CROSS PASSAGE CP3
Belmont Airport Main Drain
BH 2-05 (17m) 7.4
BH 2-05 (9m) 7.5
MW 3-115 7
MW 3-118 7.7
BH 2-11 (16m) 5.1
MW 3-121 (s) 6.1
MW 3-121 (d) 5.8
MW 3-125 6.6
CH 3050
CH 3100
CH 3150
CH 3200
CH 3250
CH 3300
CH 3350
CH 3400
CH 3450
CH 3500
CH 3550
CH 3600
CH 3650
CH 3700
CH 3750
CH 3800
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 6 OF 17
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


AIRPORT WEST STATION
CROSS PASSAGE CP4
Belmont/Airport Main Drain
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-M) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-24-2044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 7 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELING
MEAUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD), NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (X,XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-24-0044 REV F, DATED: 09.04.2015
AERIAL IMAGERY 2015.03.07 © NEARMAP (2015)
1:2,000
0 20 40 60 80 100 m
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONTIROING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 8 OF 17
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5
EMERGENCY EGRESS SHAFT
DOMESTIC AIRPORT TERMINAL
CROSS PASSAGE CP4
MW 3-086 (d) 12.7
MW 3-086 (s) NM
MW 1-03 12.7
MW 3-087 13.7
MW 3-090 12.3
MW 3-089 11.6
MW 3-092 11.1
MW 3-093 11.2
MW 3-094 10.8
CH 4450
CH 4500
CH 4550
CH 4600
CH 4650
CH 4700
CH 4750
CH 4800
CH 4850
CH 4900
CH 4950
CH 5000
CH 5050
CH 5100
CH 5150
CH 5200
CH 5250


LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-M) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
1:2,000
CROSS PASSAGE CP4a
BH 0-01 (18m) 13.4
BH 0-01 (3.5m) 13.5
BH 0-09 (15m) 13.7
BH 0-09 (5m) 13.9
BH 0-10 (17m) 14.4
BH 0-10 (10m) 14.6
BH 0-10 (2m) 14.8
CH 5200
CH 5250
CH 5300
CH 5350
CH 5400
CH 5450
CH 5500
CH 5550
CH 5600
CH 5650
CH 5700
CH 5750
CH 5800
CH 5850
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 9 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X.XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = GROUNDWATER MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244-0044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CROSS PASSAGE CP5
CROSS PASSAGE CP5a
BH 0-05 (5m) 16.5
BH 1-32 (14.5m) 16.5
BH 1-32 (21.1m) 15.5
BH 1-27 (17.5m) 16.5
BH 1-27 (23.2m) 16.5
CH 6450
CH 6400
CH 6350
CH 6300
CH 6250
CH 6200
CH 6150
CH 6100
CH 6050
CH 6000
CH 5950
CH 5900
CH 5850
CH 5800
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 10 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


CONSOLIDATED AIRPORT TERMINAL STATION
CH 7100
CH 7050
CH 7000
CH 6950
CH 6900
CH 6850
CH 6800
CH 6750
CH 6700
CH 6650
CH 6600
CH 6550
CH 6500
CH 6450
CH 6400
MW 1-02A 17
MW 1-02E NM
MW 1-02C 17
MW 1-02D 17.1
PW 1-02 17.1
MW 1-02B 17.1
BH 0-06 (13m) 17
BH 0-06 (6m) 16.9
BH 1-21 16.9
BH 1-22 (29.5m) 16.7
BH 1-22 (18m) 16.7
MW 3-077 16.9
BH 1-24 (30.9m) 16.8
BH 1-25 16.9
MW 3-080 (d) 16.8
MW 3-080 (s) 16.8
MW 3-079 17.2
BH 1-27 (14.5m) 16.5
BH 1-27 (23.4m) 16.3
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
• BH = BOREHOLE
• BH WITH (-XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
• MW = MONITORING WELL
• LG = LANDFILL GAS MONITORING WELL
• PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015, 03.07 © NEARMAP (2015)
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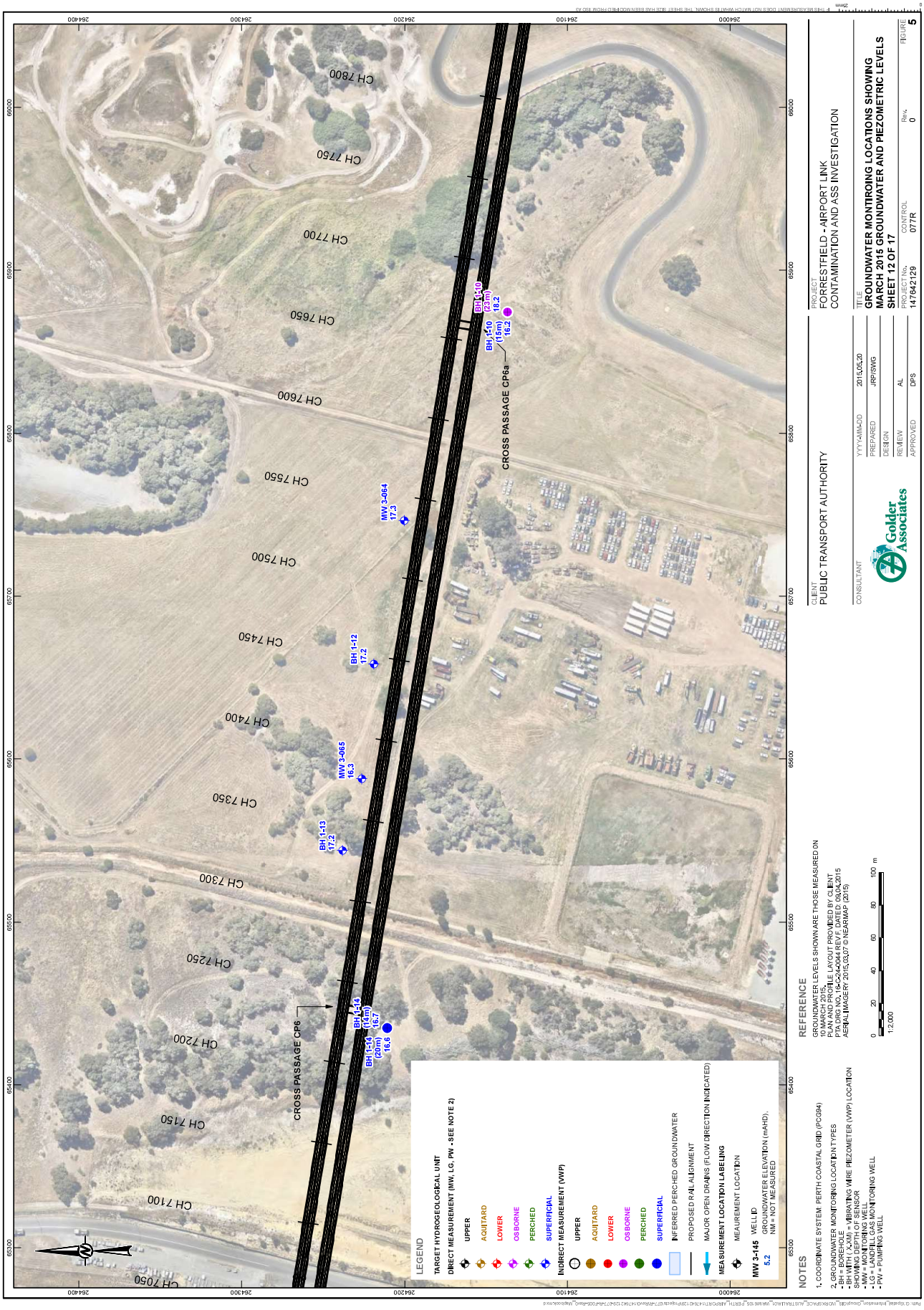
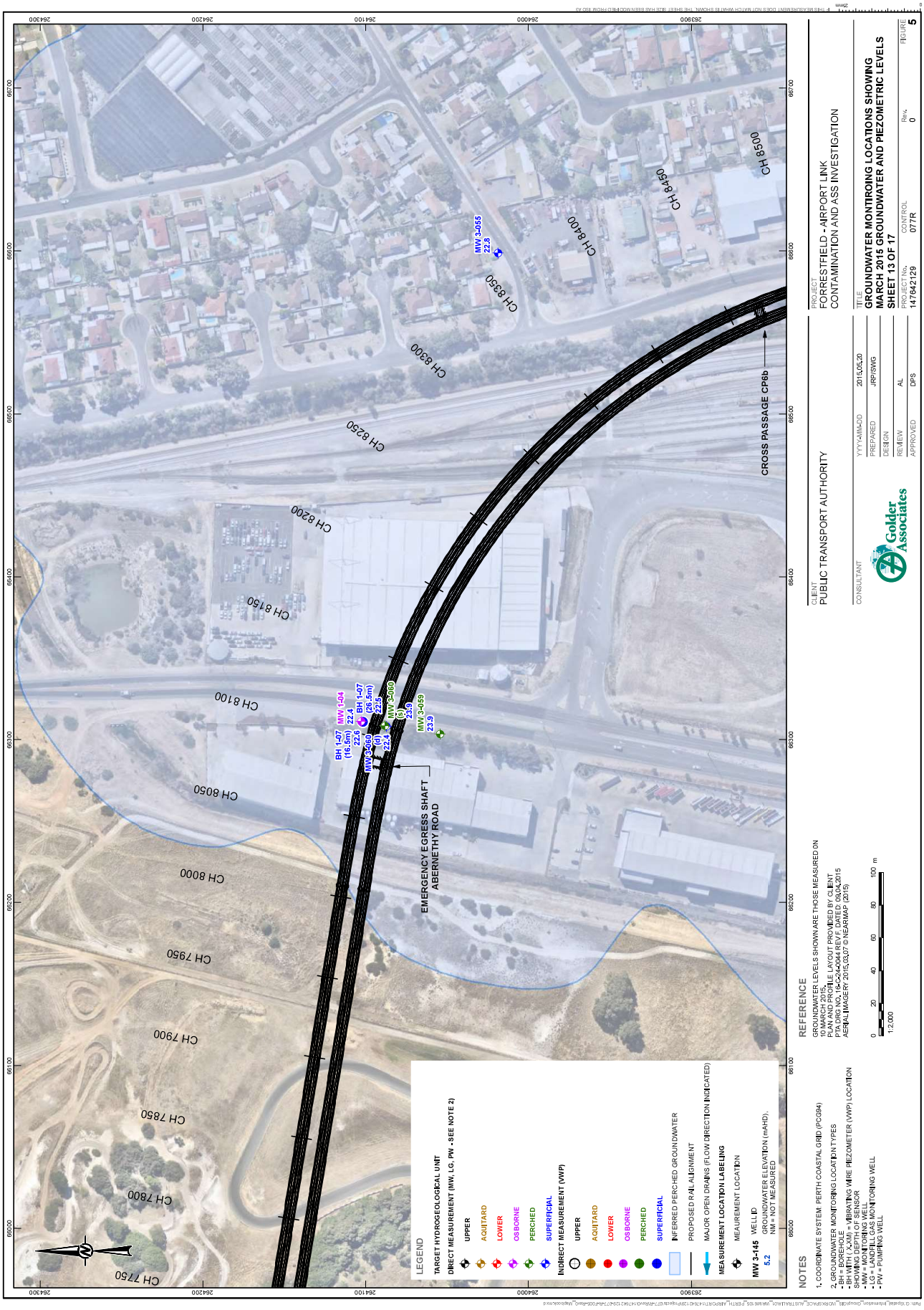
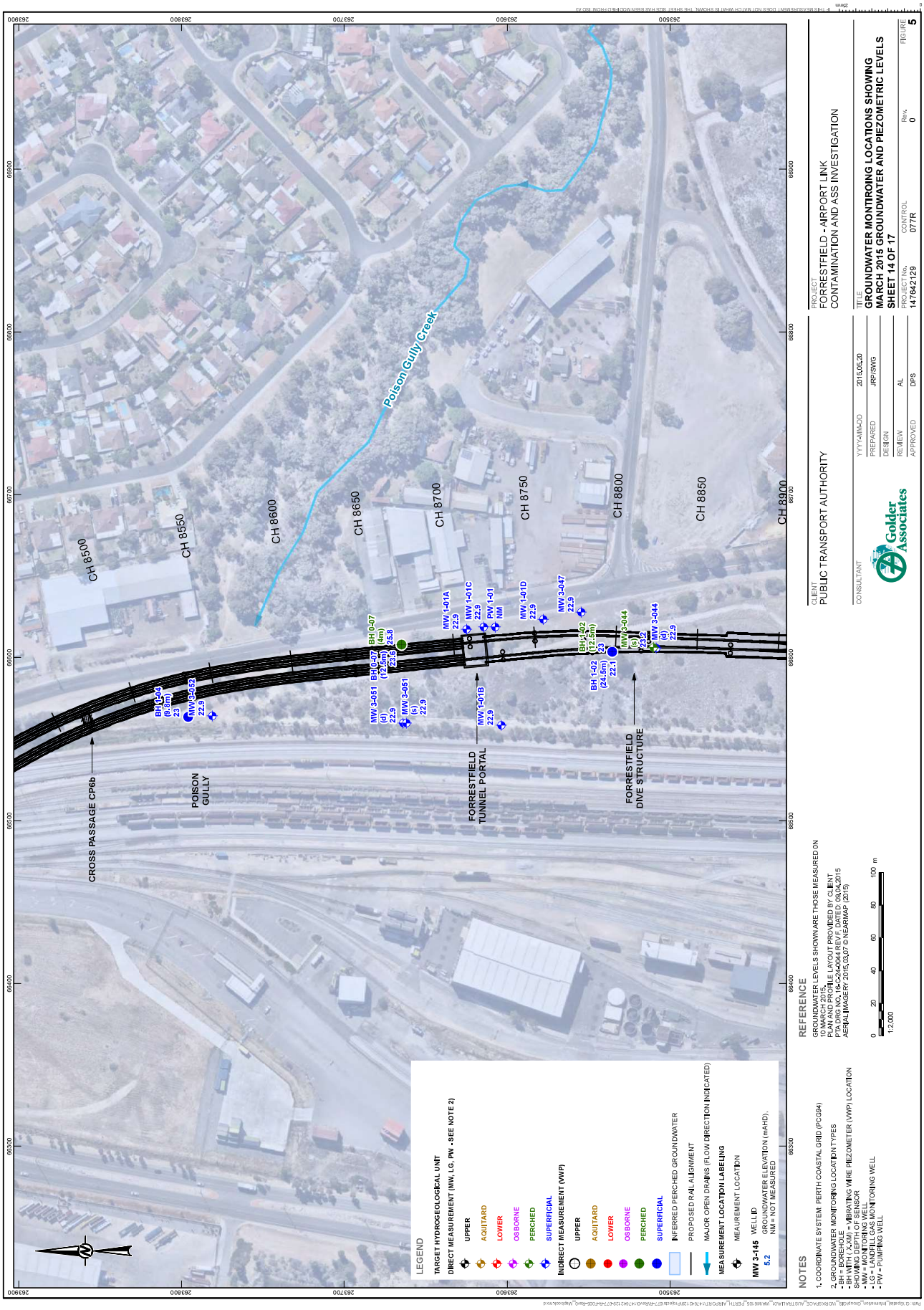
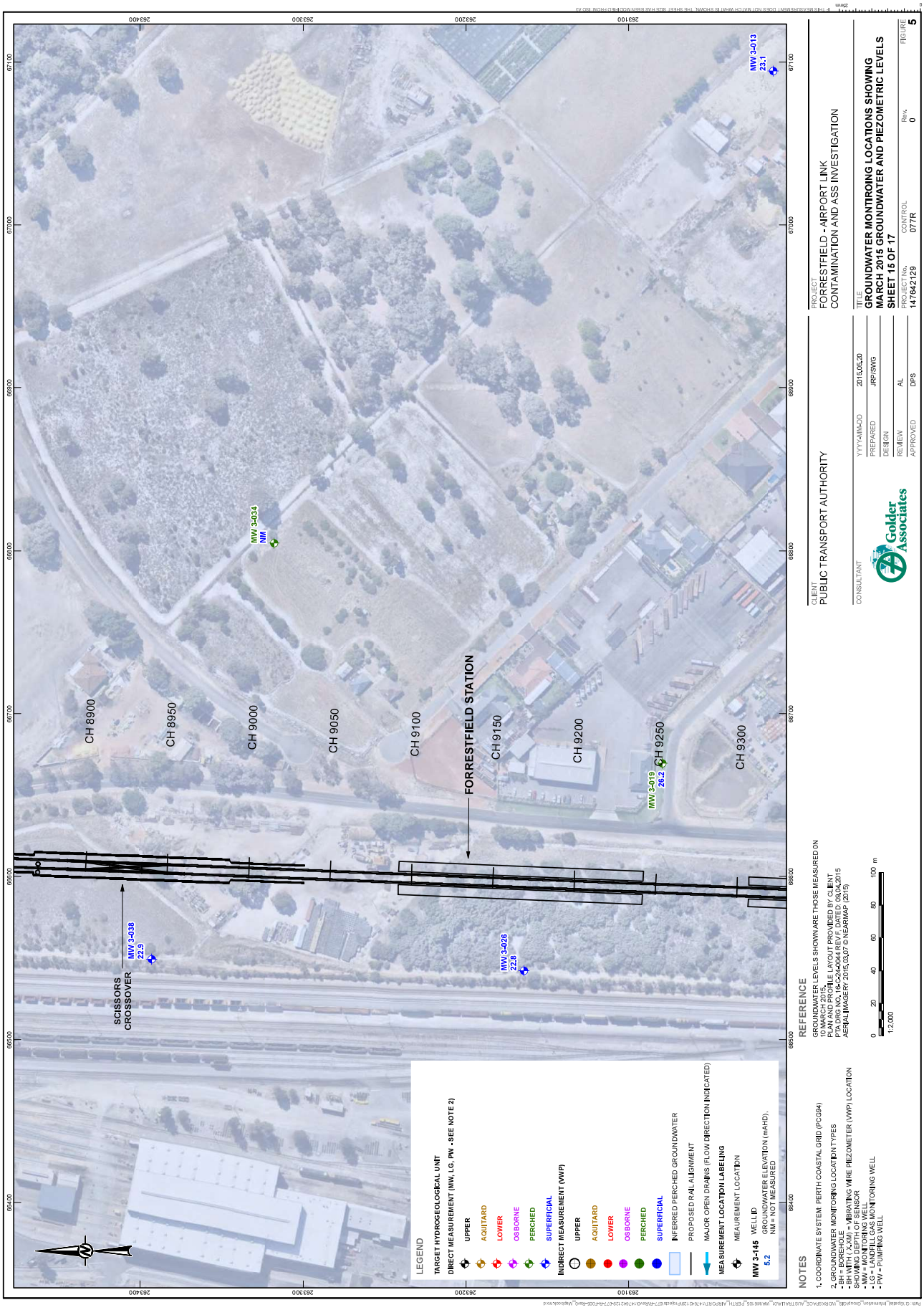
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 11 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


CH 7800
CH 7750
CH 7700
CH 7650
CH 7600
CH 7550
CH 7500
CH 7450
CH 7400
CH 7350
CH 7300
CH 7250
CH 7200
CH 7150
CH 7100
CH 7050
CROSS PASSAGE CP6
CROSS PASSAGE CP6a
BH 1-14 (14m) 16.7
BH 1-14 (20m) 16.6
BH 1-13 (17.2)
MW 3-065 16.3
BH 1-12 17.2
MW 3-064 17.3
BH 1-10 (23m) 18.2
BH 1-10 (15m) 16.2
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEAUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-XXM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = GROUNDWATER MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT PTA DRG NO. 16-C-244-0044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015, ©SJP © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 12 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = GROUNDWATER MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015, 03.07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
EMERGENCY EGRESS SHAFT
ABERNETHY ROAD
CROSS PASSAGE CP6b
CH 7750
CH 7800
CH 7850
CH 7900
CH 7950
CH 8000
CH 8050
CH 8100
CH 8150
CH 8200
CH 8250
CH 8300
CH 8350
CH 8400
CH 8450
CH 8500
BH 1-07 (16.5m) 22.6
MW 1-04 22.4
BH 1-07 (26.5m) 22.5
MW 3-060 (d) 22.4
MW 3-060 (s) 23.9
MW 3-059 23.9
MW 3-055 22.8
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 13 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


CH 8500
CH 8550
CH 8600
CH 8650
CH 8700
CH 8750
CH 8800
CH 8850
CH 8900
Poison Gully Creek
CROSS PASSAGE CP6b
POISON GULLY
FORRESTFIELD TUNNEL PORTAL
FORRESTFIELD DIVE STRUCTURE
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-M) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = GROUNDWATER MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244-0044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015, 03.07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 14 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


FORRESTFIELD STATION
CH 8900
CH 8950
CH 9000
CH 9050
CH 9100
CH 9150
CH 9200
CH 9250
CH 9300
SCISSORS CROSSOVER
MW 3-038 22.9
MW 3-026 22.8
MW 3-019 26.2
MW 3-034 NM
MW 3-013 23.1
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-M) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = GROUNDWATER MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244-0044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015, 03.07.15 NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 15 OF 17
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


FORRESTFIELD STOWAGE
CH 9300
CH 9350
CH 9400
CH 9450
CH 9492.153
MW 3-013
23.1
MW 3-012
22.9
MW1-105
23.4
MW 3-009
24.5
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-M) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING
MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 16 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


MW 1-05
23.4
MW 3-009
24.5
MW 3-004
24.3
MW 3-001
22.3
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
WELL ID
MW 3-145
GROUNDWATER ELEVATION (mAHD).
5.2
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-M) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = GROUNDWATER MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244-0044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015, ©SJP © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 17 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5

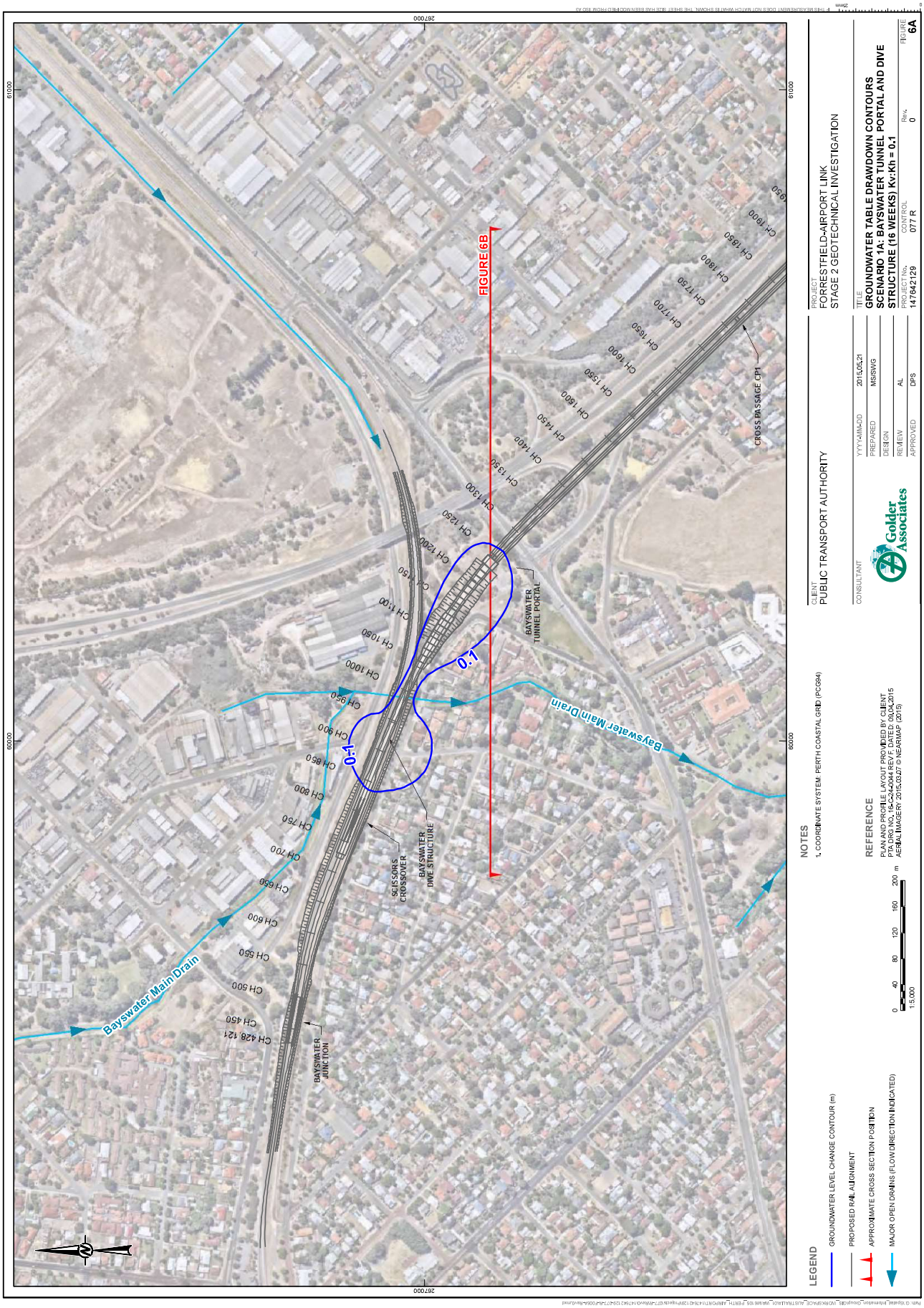
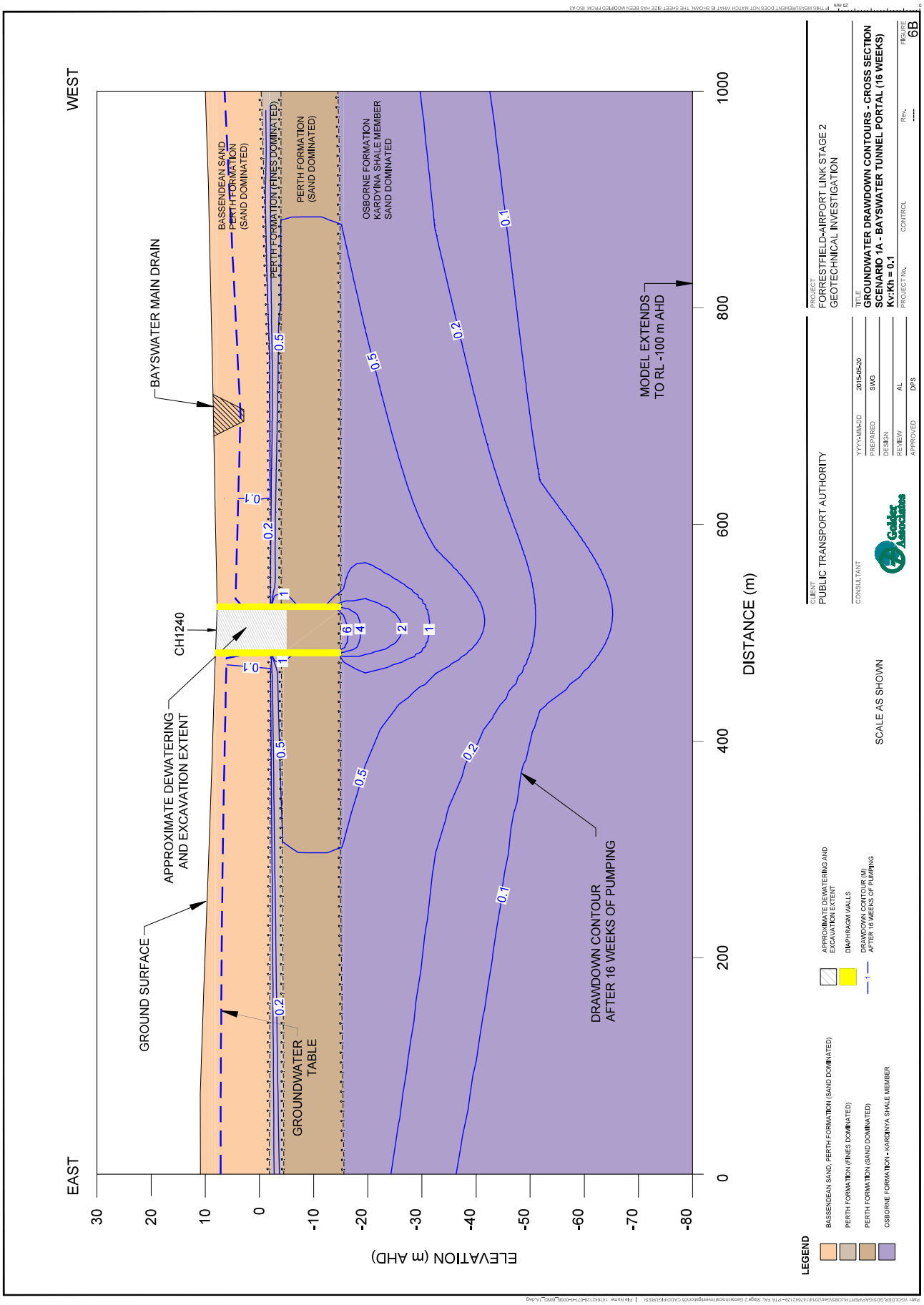
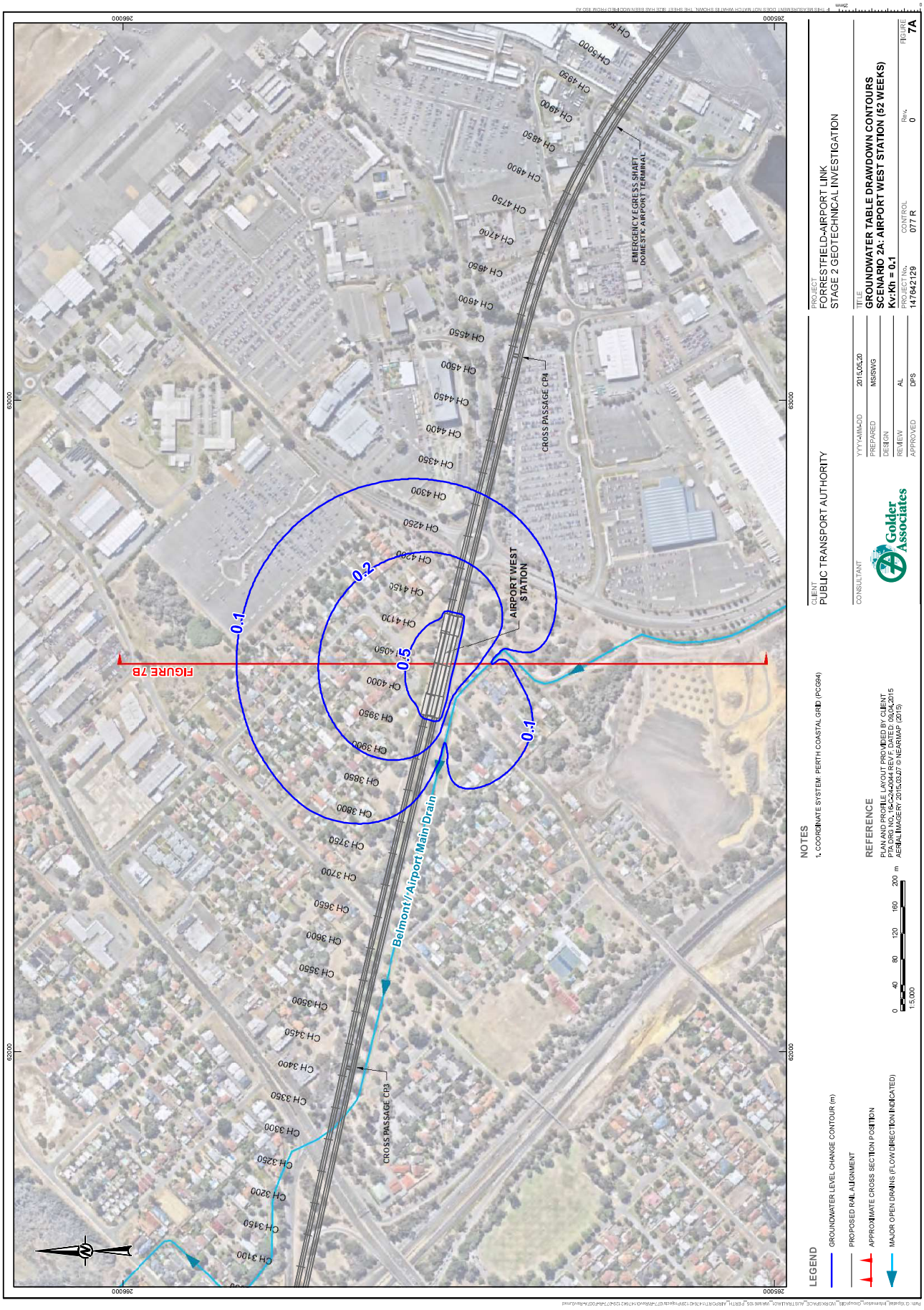
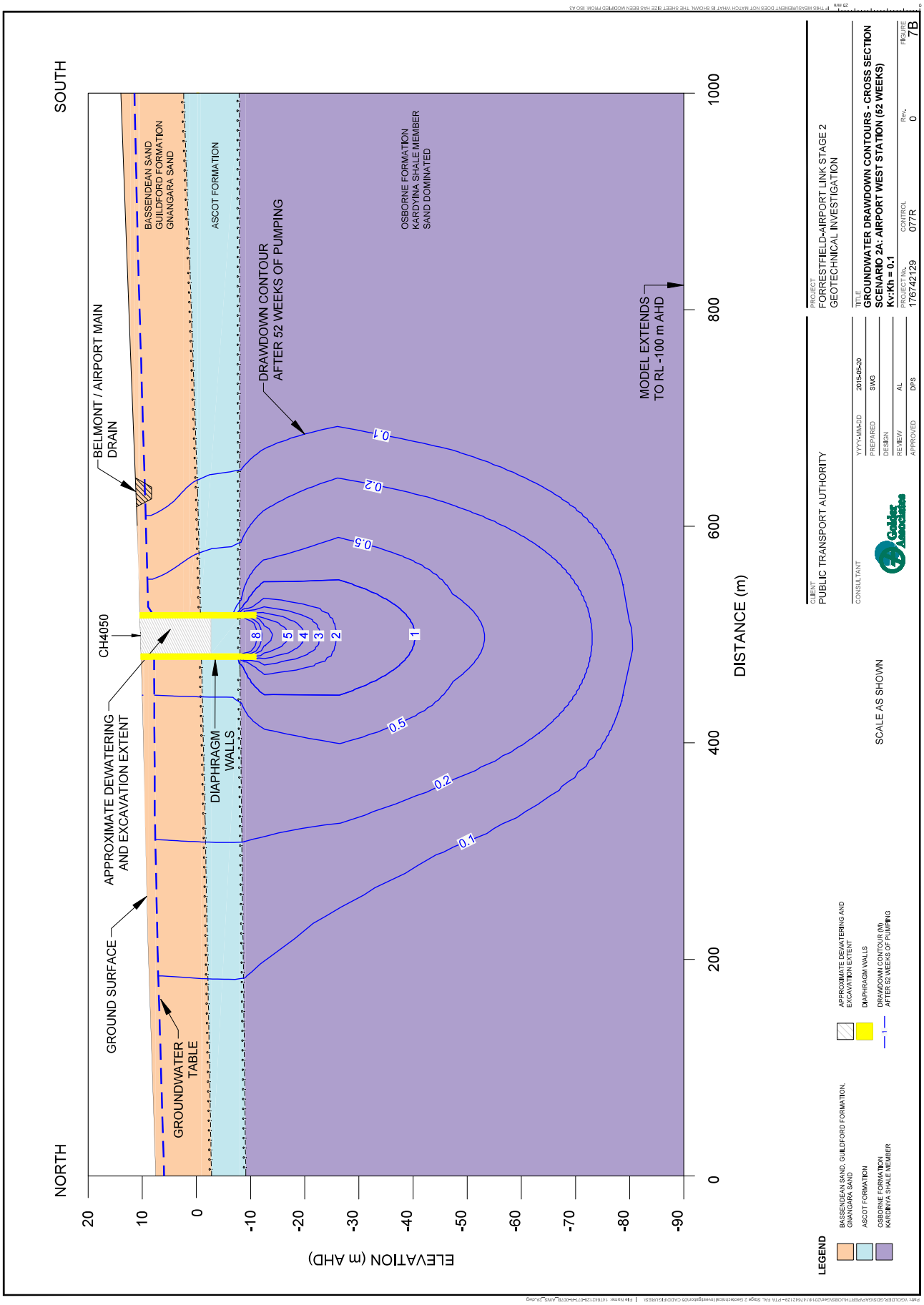
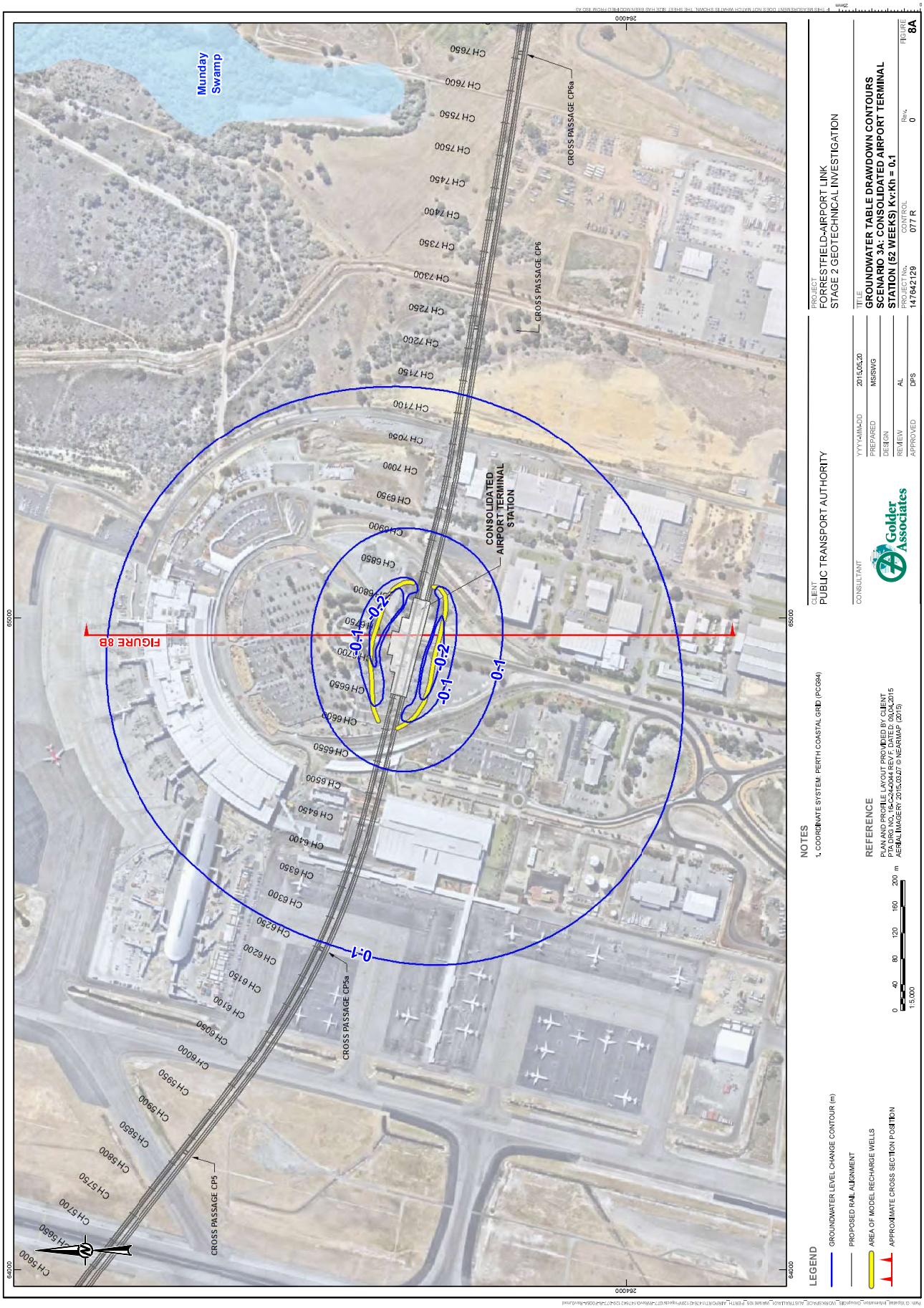


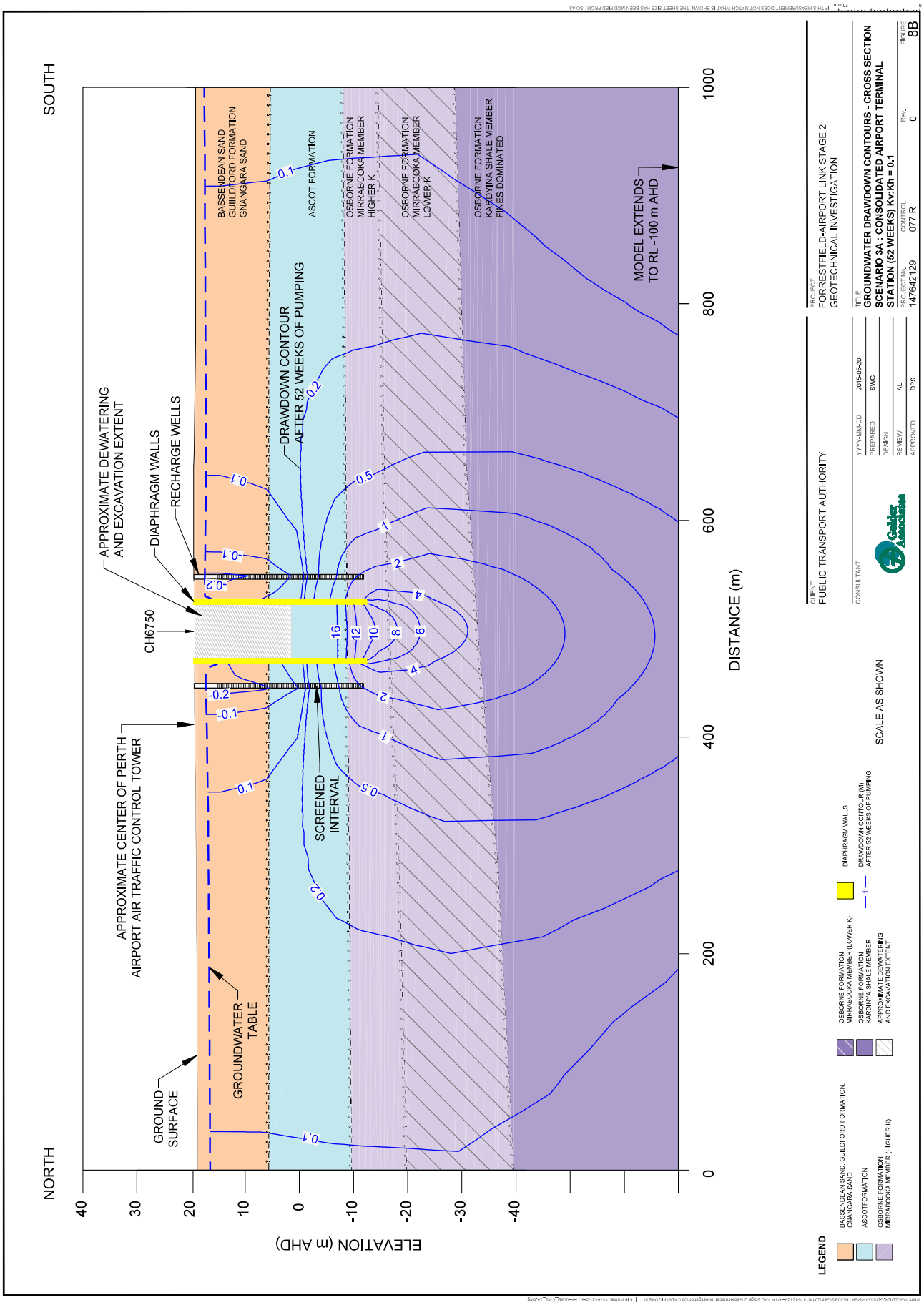
FIGURE 6B
0.1
0.1
BAYSWATER TUNNEL PORTAL
BAYSWATER DIVE STRUCTURE
SCISSORS CROSSOVER
BAYSWATER JUNCTION
CROSS PASSAGE CP1
Bayswater Main Drain
Bayswater Main Drain
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY: CLIENT
PTA DRG NO.: 15-C-24-0044 REV: F, DATED: 09.04.2015
AERIAL IMAGERY: 2015.03.07 © NEARMAP (2015)
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
APPROXIMATE CROSS SECTION POSITION
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
1:5,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 1A: BAYSWATER TUNNEL PORTAL AND DIVE
STRUCTURE (16 WEEKS) Kv:Kh = 0.1
YYYY-MM-DD 2015.05.21
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 6A

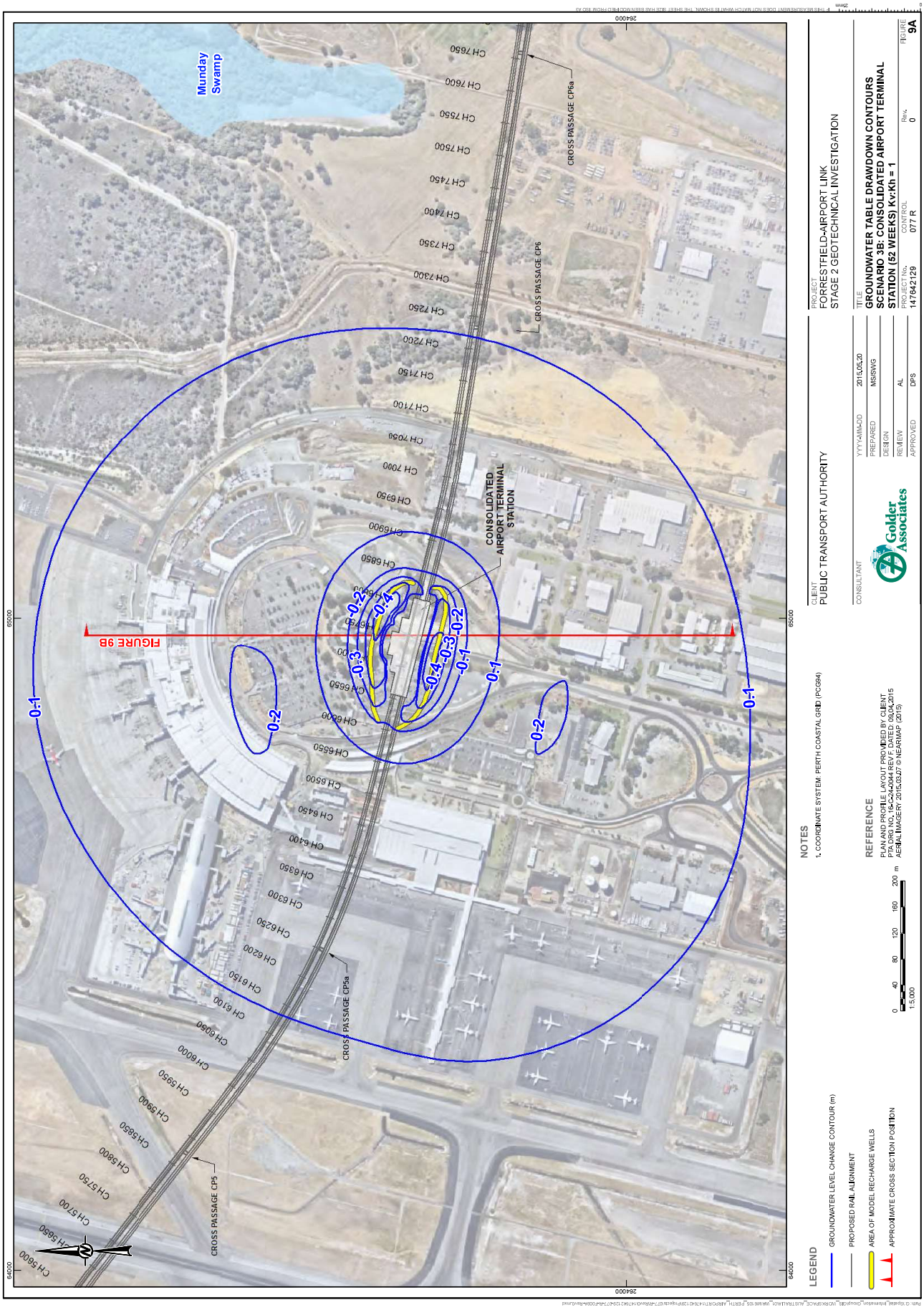

WEST
EAST
BAYSWATER MAIN DRAIN
BASSENDEAN SAND
PERTH FORMATION
(SAND DOMINATED)
PERTH FORMATION (FINES DOMINATED)
PERTH FORMATION
(SAND DOMINATED)
OSBORNE FORMATION
KARDINYA SHALE MEMBER
SAND DOMINATED
MODEL EXTENDS
TO RL -100 m AHD
CH1240
APPROXIMATE DEWATERING
AND EXCAVATION EXTENT
GROUND SURFACE
GROUNDWATER
TABLE
DRAWDOWN CONTOUR
AFTER 16 WEEKS OF PUMPING
DISTANCE (m)
ELEVATION (m AHD)
0
200
400
600
800
1000
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
0.1
0.2
0.5
1
2
4
6
LEGEND
BASSENDEAN SAND, PERTH FORMATION (SAND DOMINATED)
PERTH FORMATION (FINES DOMINATED)
PERTH FORMATION (SAND DOMINATED)
OSBORNE FORMATION - KARDINYA SHALE MEMBER
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M) AFTER 16 WEEKS OF PUMPING
SCALE AS SHOWN
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 1A - BAYSWATER TUNNEL PORTAL (16 WEEKS)
Kv:Kh = 0.1
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No.
CONTROL
REV.
FIGURE
6B

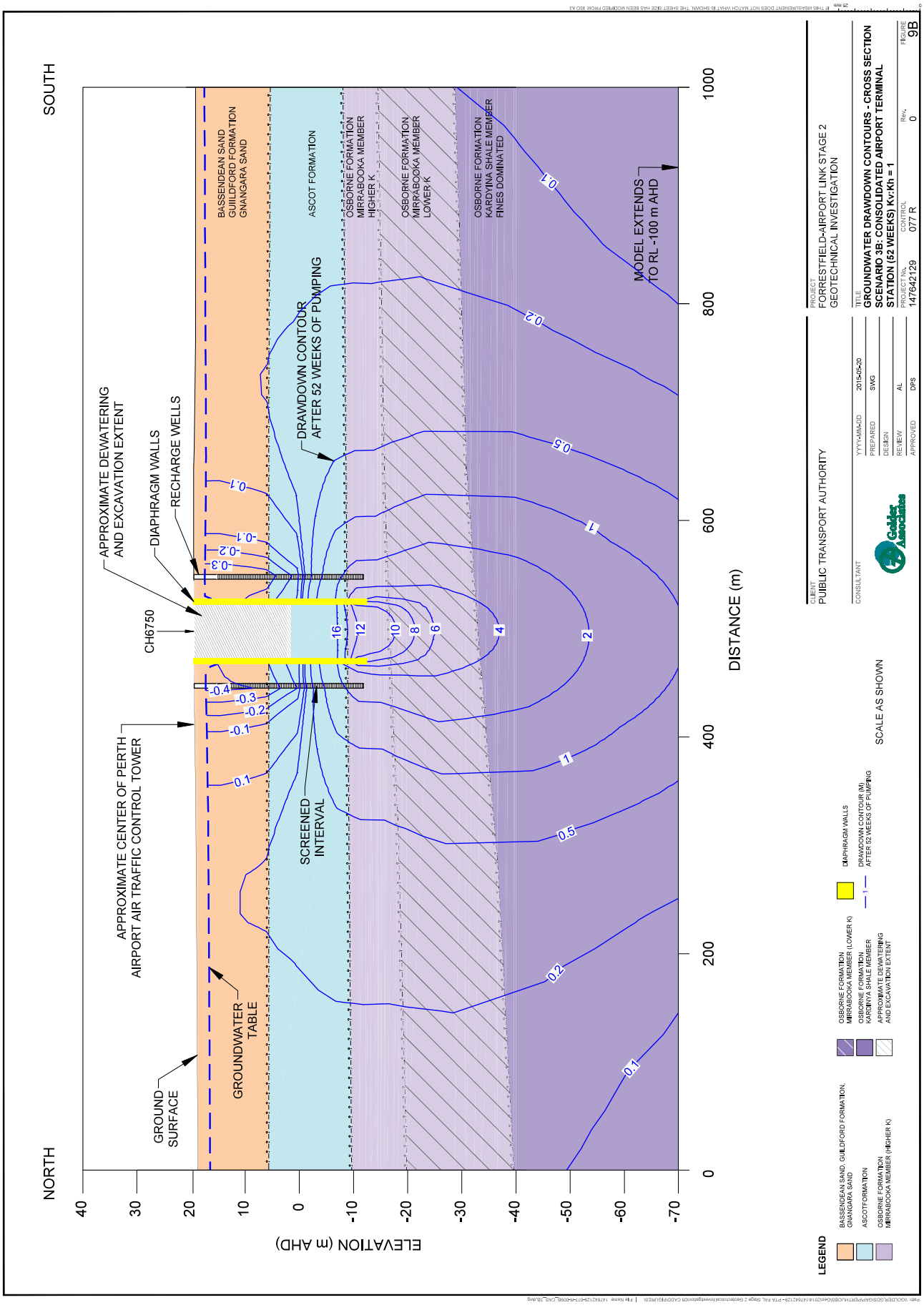

AIRPORT WEST STATION
EMERGENCY EGRESS SHAFT DOMESTIC AIRPORT TERMINAL
CROSS PASSAGE CP4
CROSS PASSAGE CP3
Belmont//Airport Main Drain
0.1
0.2
0.5
FIGURE 7B
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY: CLIENT
PTA DRG NO.: 15-C-24-0044 REV: F, DATED: 09.04.2015
AERIAL IMAGERY: 2015.03.07 © NEARMAP (2015)
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
APPROXIMATE CROSS SECTION POSITION
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
0 40 80 120 160 200 m
1:5,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 2A: AIRPORT WEST STATION (52 WEEKS)
Kv:Kh = 0.1
YYYY-MM-DD 2015.05.20
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 7A


NORTH
SOUTH
BELMONT / AIRPORT MAIN DRAIN
CH4050
GROUND SURFACE
GROUNDWATER TABLE
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
BASSENDEAN SAND
GUILDFORD FORMATION
GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION
KARDINYA SHALE MEMBER
SAND DOMINATED
DRAWDOWN CONTOUR
AFTER 52 WEEKS OF PUMPING
MODEL EXTENDS
TO RL -100 m AHD
0.1
0.2
0.5
1
2
3
4
5
8
ELEVATION (m AHD)
DISTANCE (m)
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
0
200
400
600
800
1000
LEGEND
BASSENDEAN SAND, GUILDFORD FORMATION, GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION KARDINYA SHALE MEMBER
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M) AFTER 52 WEEKS OF PUMPING
SCALE AS SHOWN
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 2A: AIRPORT WEST STATION (52 WEEKS)
Kv:Kh = 0.1
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 176742129
CONTROL 077R
REV. 0
FIGURE 7B


Munday Swamp
CONSOLIDATED AIRPORT TERMINAL STATION
CROSS PASSAGE CP6a
CROSS PASSAGE CP6
CROSS PASSAGE CP5a
CROSS PASSAGE CP5
FIGURE 8B
-0.1
-0.2
-0.1
-0.2
-0.1
-0.1
-0.1
CH 7650
CH 7600
CH 7550
CH 7500
CH 7450
CH 7400
CH 7350
CH 7300
CH 7250
CH 7200
CH 7150
CH 7100
CH 7050
CH 7000
CH 6950
CH 6900
CH 6850
CH 6800
CH 6750
CH 6700
CH 6650
CH 6600
CH 6550
CH 6500
CH 6450
CH 6400
CH 6350
CH 6300
CH 6250
CH 6200
CH 6150
CH 6100
CH 6050
CH 6000
CH 5950
CH 5900
CH 5850
CH 5800
CH 5750
CH 5700
CH 5650
CH 5600
64000
65000
264000
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY: CLIENT
PTA DRG NO.: 15-C-24-0044 REV: F, DATED: 09.04.2015
AERIAL IMAGERY: 2015.03.07 © NEARMAP (2015)
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
AREA OF MODEL RECHARGE WELLS
APPROXIMATE CROSS SECTION POSITION
0 40 80 120 160 200 m
1:5,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 3A: CONSOLIDATED AIRPORT TERMINAL
STATION (52 WEEKS) Kv:Kh = 0.1
YYYY-MM-DD 2015.05.20
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 8A


SOUTH
NORTH
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
RECHARGE WELLS
CH6750
APPROXIMATE CENTER OF PERTH AIRPORT AIR TRAFFIC CONTROL TOWER
GROUND SURFACE
GROUNDWATER TABLE
SCREENED INTERVAL
DRAWDOWN CONTOUR AFTER 52 WEEKS OF PUMPING
BASSENDEAN SAND
GUILDFORD FORMATION
GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION MIRRABOOKA MEMBER HIGHER K
OSBORNE FORMATION MIRRABOOKA MEMBER LOWER K
OSBORNE FORMATION KARDINYA SHALE MEMBER FINES DOMINATED
MODEL EXTENDS TO RL -100 m AHD
ELEVATION (m AHD)
DISTANCE (m)
LEGEND
BASSENDEAN SAND, GUILDFORD FORMATION, GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION MIRRABOOKA MEMBER (HIGHER K)
OSBORNE FORMATION MIRRABOOKA MEMBER (LOWER K)
OSBORNE FORMATION KARDINYA SHALE MEMBER
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M) AFTER 52 WEEKS OF PUMPING
SCALE AS SHOWN
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 3A : CONSOLIDATED AIRPORT TERMINAL
STATION (52 WEEKS) Kv:Kh = 0.1
PROJECT No. 147642129
CONTROL 077 R
REV. 0
FIGURE 8B
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS


Munday Swamp
CONSOLIDATED AIRPORT TERMINAL STATION
CROSS PASSAGE CP6a
CROSS PASSAGE CP6
CROSS PASSAGE CP5a
CROSS PASSAGE CP5
FIGURE 9B
-0.1
-0.2
-0.3
-0.4
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
AREA OF MODEL RECHARGE WELLS
APPROXIMATE CROSS SECTION POSITION
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY: CLIENT
PTA DRG NO.: 15-C-24-0044 REV: F, DATED: 09.04.2015
AERIAL IMAGERY: 2015.03.07 © NEARMAP (2015)
1:5,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 3B: CONSOLIDATED AIRPORT TERMINAL
STATION (52 WEEKS) Kv:Kh = 1
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 9A
YYYY-MM-DD 2015.05.20
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS


NORTH
SOUTH
ELEVATION (m AHD)
DISTANCE (m)
GROUND SURFACE
GROUNDWATER TABLE
APPROXIMATE CENTER OF PERTH AIRPORT AIR TRAFFIC CONTROL TOWER
SCREENED INTERVAL
CH6750
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
RECHARGE WELLS
DRAWDOWN CONTOUR AFTER 52 WEEKS OF PUMPING
MODEL EXTENDS TO RL -100 m AHD
BASSENDEAN SAND
GUILDFORD FORMATION
GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION MIRRABOOKA MEMBER HIGHER K
OSBORNE FORMATION MIRRABOOKA MEMBER LOWER K
OSBORNE FORMATION KARDINYA SHALE MEMBER FINES DOMINATED
LEGEND
BASSENDEAN SAND, GUILDFORD FORMATION, GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION MIRRABOOKA MEMBER (HIGHER K)
OSBORNE FORMATION MIRRABOOKA MEMBER (LOWER K)
OSBORNE FORMATION KARDINYA SHALE MEMBER
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M) AFTER 52 WEEKS OF PUMPING
SCALE AS SHOWN
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 3B: CONSOLIDATED AIRPORT TERMINAL
STATION (52 WEEKS) Kv:Kh = 1
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
REV. 0
FIGURE 9B

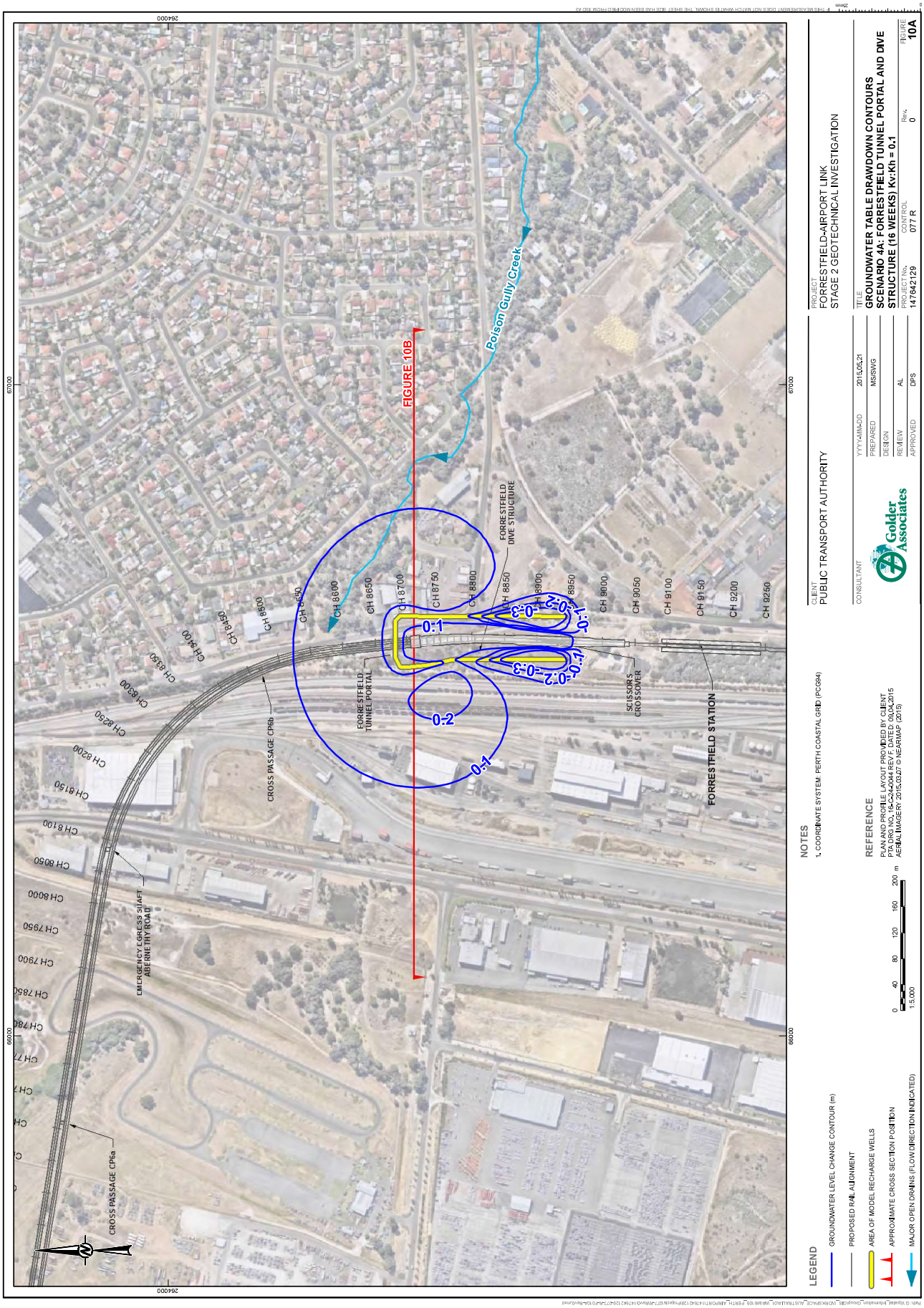
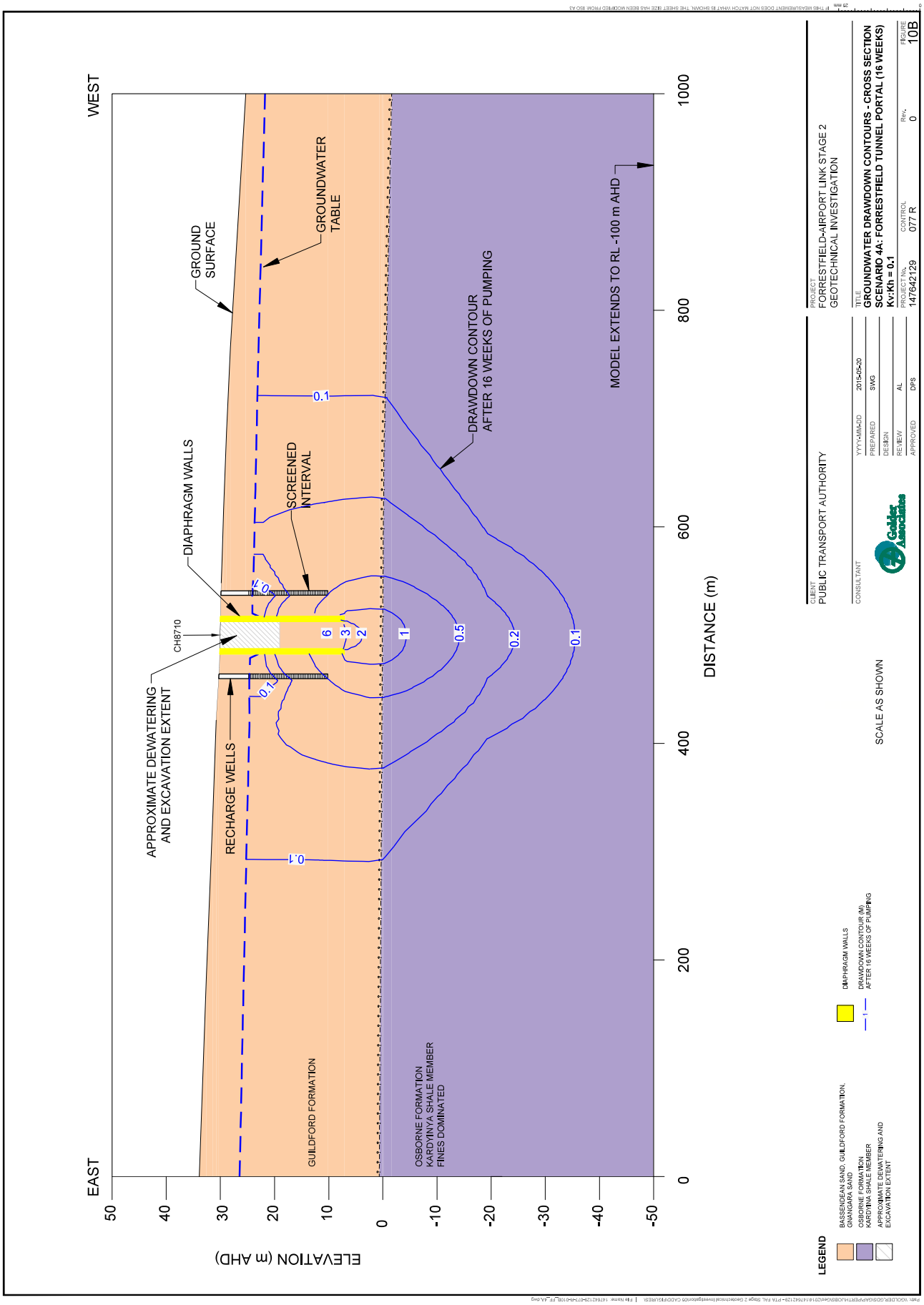


FIGURE 10B
Poison Gully Creek
FORRESTFIELD DIVE STRUCTURE
FORRESTFIELD TUNNEL PORTAL
CROSS PASSAGE CP6b
CROSS PASSAGE CP6a
EMERGENCY EGRESS SHAFT ABERNETHY ROAD
SCISSORS CROSSOVER
FORRESTFIELD STATION
0:1
0:2
0:3
CH 8600
CH 8650
CH 8700
CH 8750
CH 8800
CH 8850
CH 8900
CH 8950
CH 9000
CH 9050
CH 9100
CH 9150
CH 9200
CH 9250
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
AREA OF MODEL RECHARGE WELLS
APPROXIMATE CROSS SECTION POSITION
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY: CLIENT
PTA DRG NO.: 15-C-24-0044 REV: F, DATED: 09.04.2015
AERIAL IMAGERY: 2015.03.07 © NEARMAP (2015)
1:5,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 4A: FORRESTFIELD TUNNEL PORTAL AND DIVE STRUCTURE (16 WEEKS) Kv:Kh = 0.1
YYYY-MM-DD 2015.05.21
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 10A


WEST
EAST
GROUND SURFACE
GROUNDWATER TABLE
DIAPHRAGM WALLS
SCREENED INTERVAL
CH8710
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
RECHARGE WELLS
DRAWDOWN CONTOUR AFTER 16 WEEKS OF PUMPING
MODEL EXTENDS TO RL -100 m AHD
GUILDFORD FORMATION
OSBORNE FORMATION KARDYINYA SHALE MEMBER FINES DOMINATED
0.1
0.2
0.5
1
2
3
6
DISTANCE (m)
ELEVATION (m AHD)
LEGEND
BASSENDEAN SAND, GUILDFORD FORMATION, GNANGARA SAND
OSBORNE FORMATION KARDYINYA SHALE MEMBER
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M) AFTER 16 WEEKS OF PUMPING
SCALE AS SHOWN
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 4A: FORRESTFIELD TUNNEL PORTAL (16 WEEKS)
Kv:Kh = 0.1
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 10B

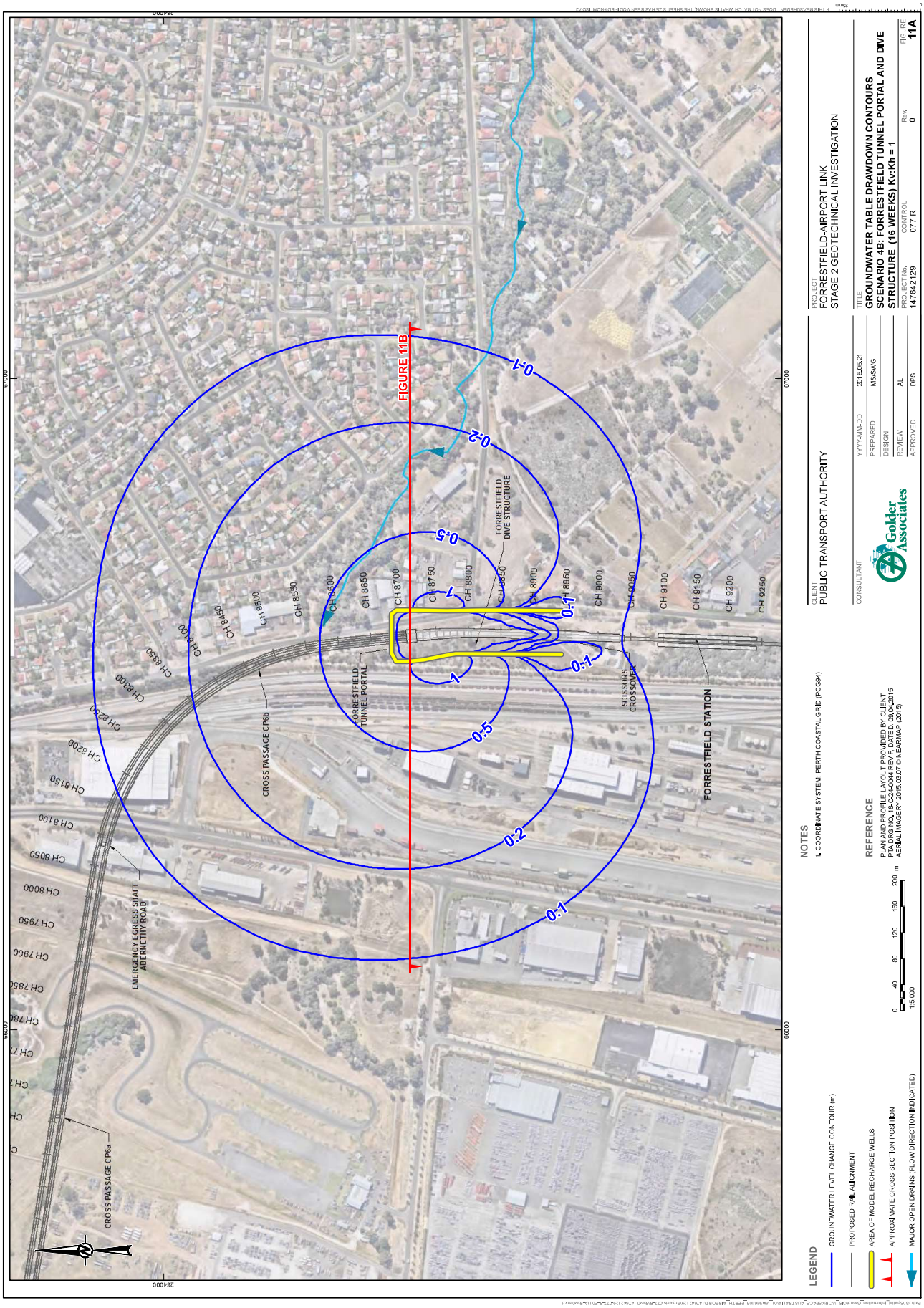
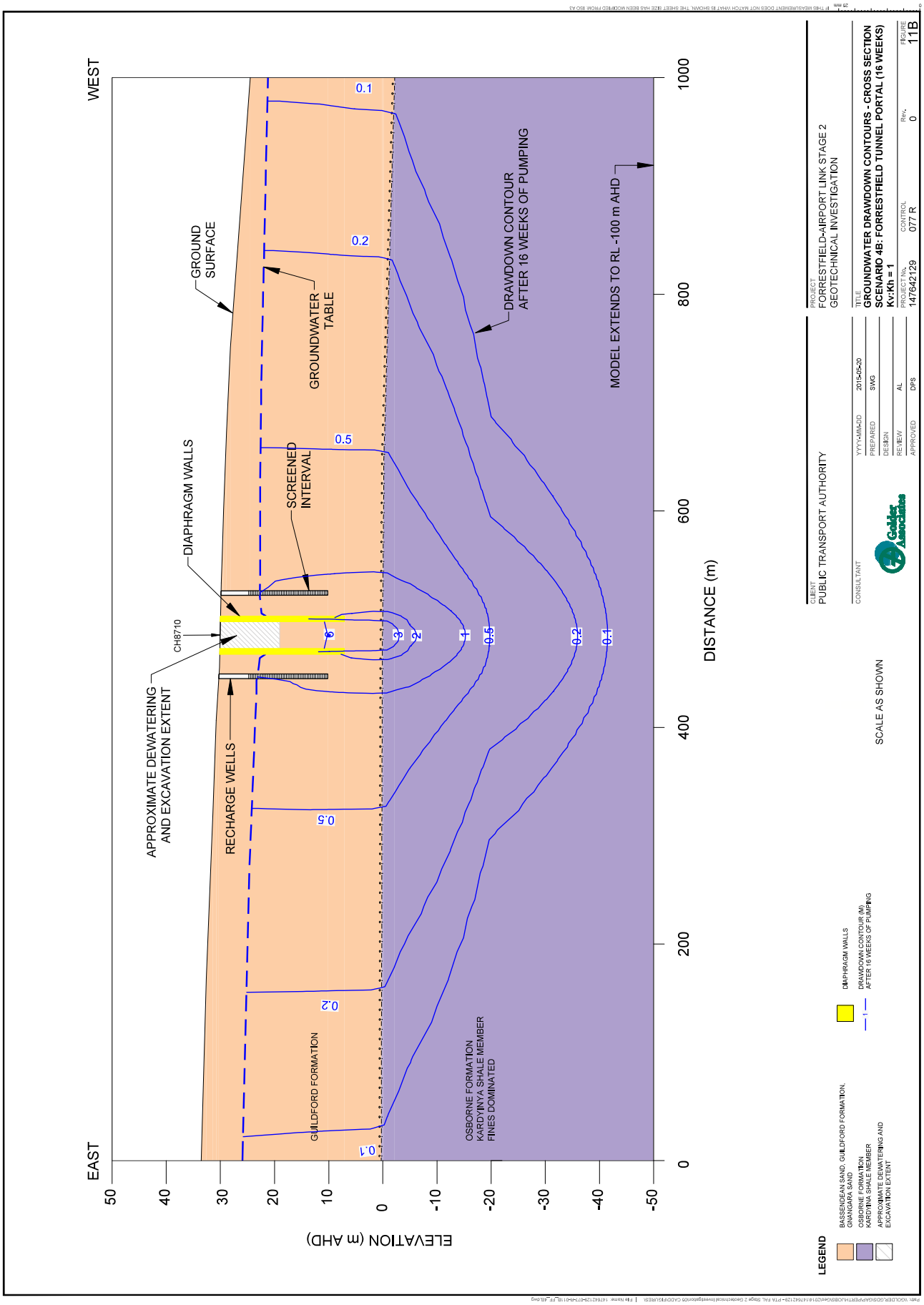
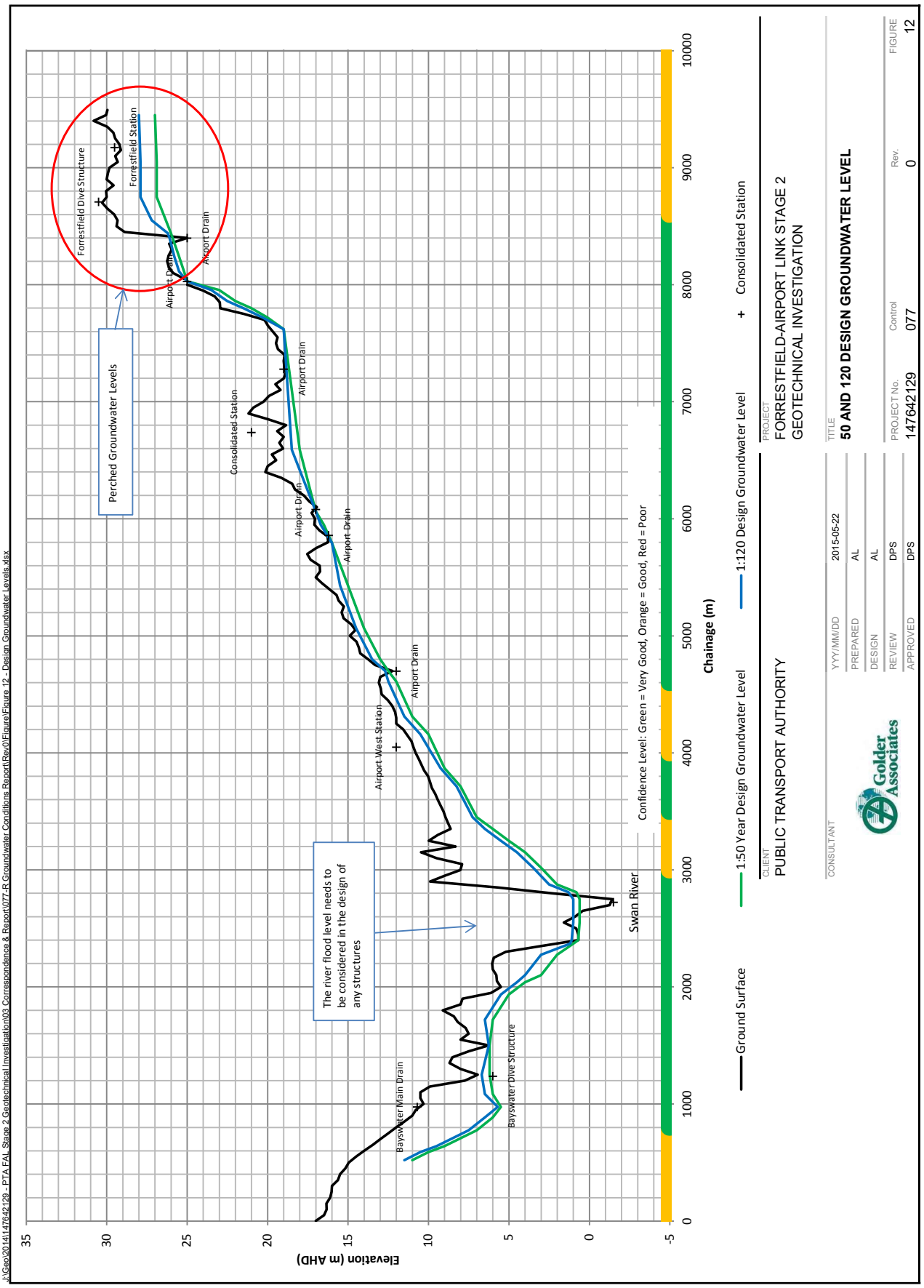


FIGURE 11B
0.1
0.2
0.5
1
FORRESTFIELD DIVE STRUCTURE
FORRESTFIELD TUNNEL PORTAL
CROSS PASSAGE CP6b
CROSS PASSAGE CP6a
EMERGENCY EGRESS SHAFT ABERNETHY ROAD
SCISSORS CROSSOVER
FORRESTFIELD STATION
CH 8450
CH 8500
CH 8550
CH 8600
CH 8650
CH 8700
CH 8750
CH 8800
CH 8850
CH 8900
CH 8950
CH 9000
CH 9050
CH 9100
CH 9150
CH 9200
CH 9250
CH 8100
CH 8150
CH 8200
CH 8250
CH 8300
CH 8350
CH 8400
CH 8450
CH 8500
CH 8550
CH 8600
CH 8650
CH 8700
CH 8750
CH 8800
CH 8850
CH 8900
CH 8950
CH 9000
CH 9050
CH 9100
CH 9150
CH 9200
CH 9250
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
AREA OF MODEL RECHARGE WELLS
APPROXIMATE CROSS SECTION POSITION
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-24-0044 REV F, DATED: 09.04.2015
AERIAL IMAGERY 2015.03.07 © NEARMAP (2015)
0 40 80 120 160 200 m
1:5,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.21
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 4B: FORRESTFIELD TUNNEL PORTAL AND DIVE STRUCTURE (16 WEEKS) Kv:Kh = 1
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 11A

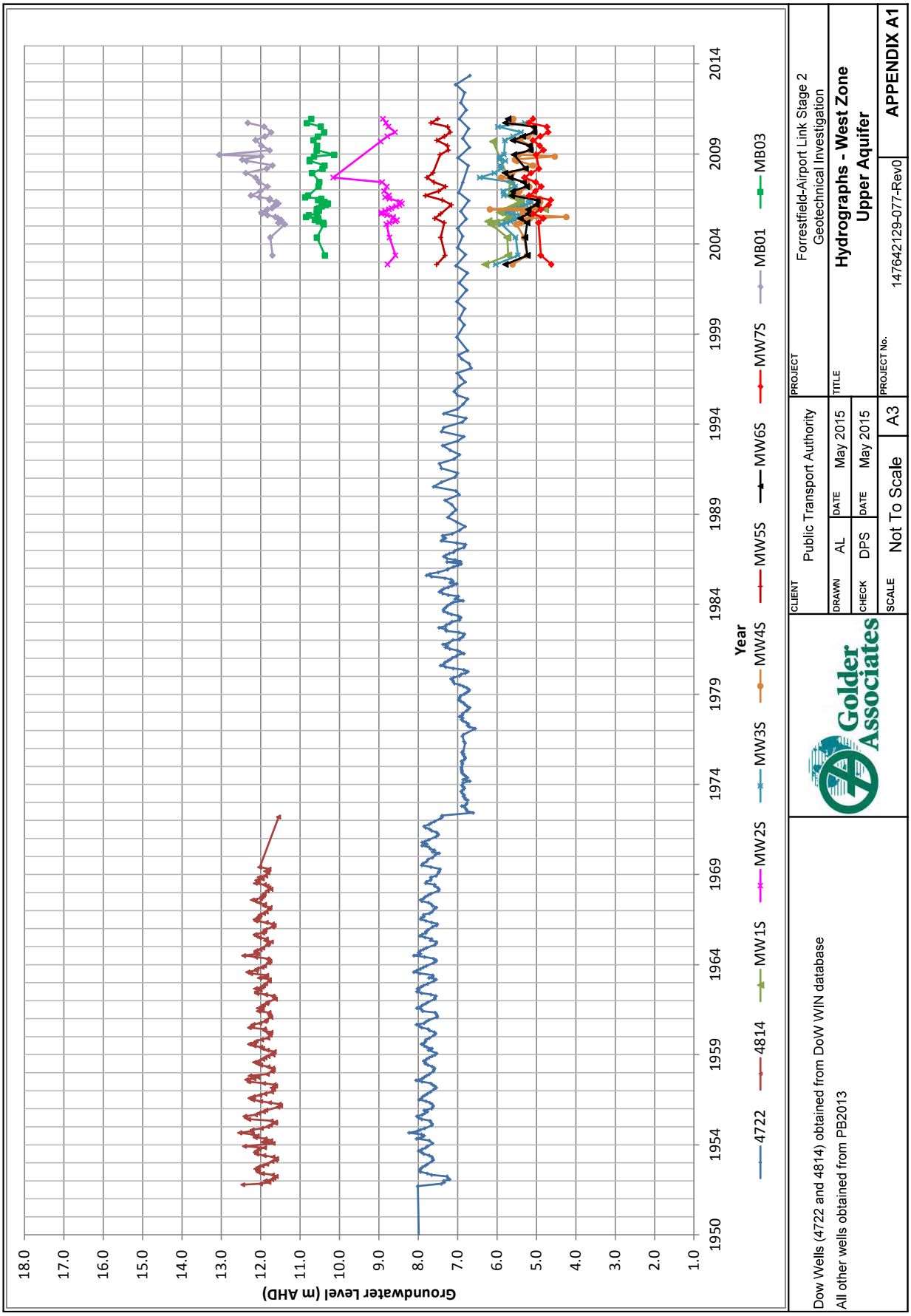

WEST
EAST
GROUND SURFACE
GROUNDWATER TABLE
DIAPHRAGM WALLS
SCREENED INTERVAL
CH8710
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
RECHARGE WELLS
DRAWDOWN CONTOUR AFTER 16 WEEKS OF PUMPING
MODEL EXTENDS TO RL -100 m AHD
GUILDFORD FORMATION
OSBORNE FORMATION KARDINYA SHALE MEMBER FINES DOMINATED
0.1
0.2
0.5
1
2
3
6
9
0.6
ELEVATION (m AHD)
DISTANCE (m)
50
40
30
20
10
0
-10
-20
-30
-40
-50
0
200
400
600
800
1000
LEGEND
BASSENDEAN SAND, GUILDFORD FORMATION, GNANGARA SAND
OSBORNE FORMATION KARDINYA SHALE MEMBER
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M) AFTER 16 WEEKS OF PUMPING
SCALE AS SHOWN
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 4B: FORRESTFIELD TUNNEL PORTAL (16 WEEKS)
Kv:Kh = 1
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
REV. 0
FIGURE 11B

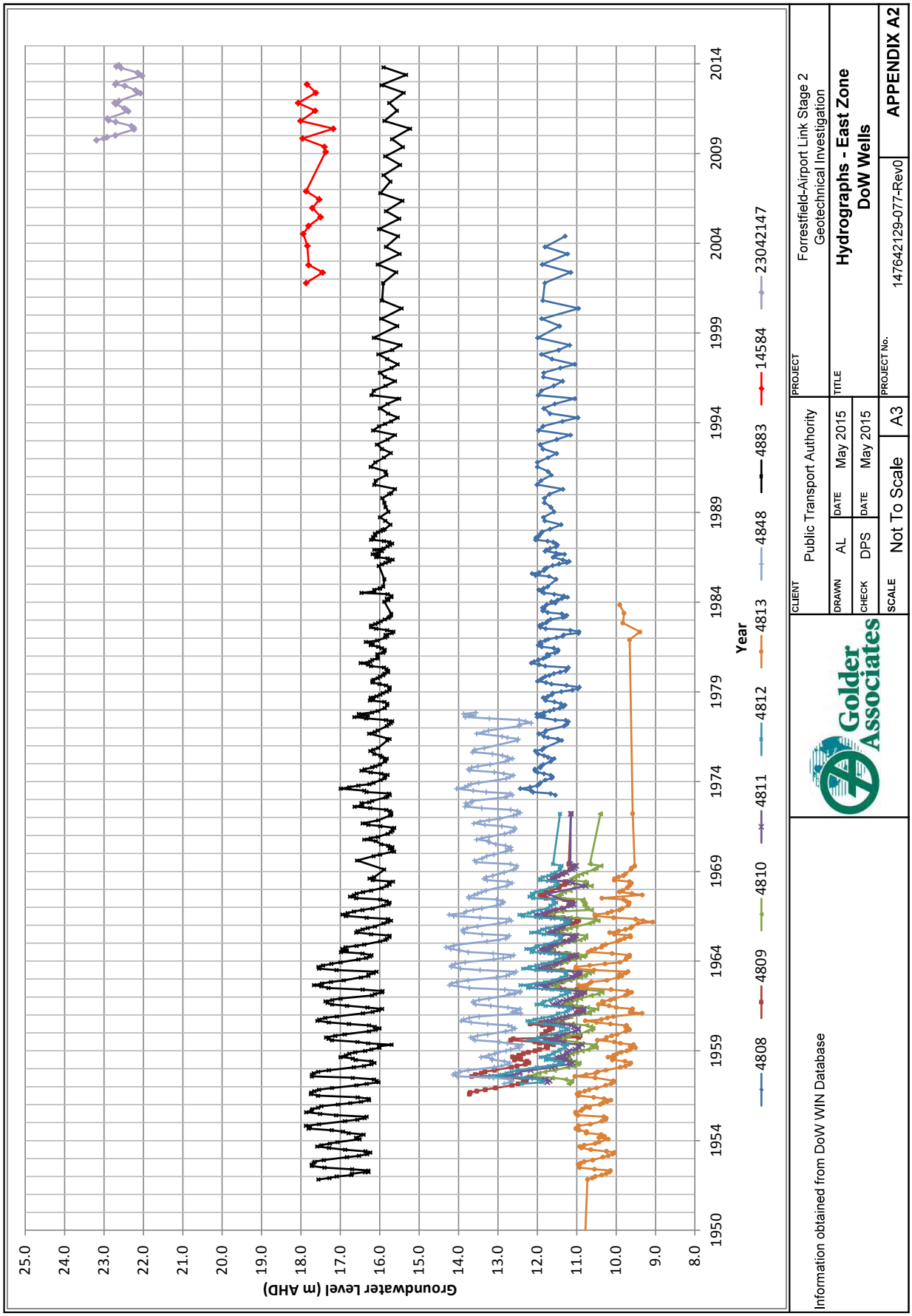

J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Figure\Figure 12 - Design Groundwater Levels.xlsx
Elevation (m AHD)
Chainage (m)
Perched Groundwater Levels
Forrestfield Dive Structure
Forrestfield Station
Airport Drain
Consolidated Station
Airport West Station
Bayswater Main Drain
Bayswater Dive Structure
Swan River
The river flood level needs to be considered in the design of any structures
Confidence Level: Green = Very Good, Orange = Good, Red = Poor
Ground Surface
1:50 Year Design Groundwater Level
1:120 Design Groundwater Level
Consolidated Station
CLIENT
PUBLIC TRANSPORT AUTHORITY
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
CONSULTANT
Golder Associates
TITLE
50 AND 120 DESIGN GROUNDWATER LEVEL
YYYY/MM/DD 2015-05-22
PREPARED AL
DESIGN AL
REVIEW DPS
APPROVED DPS
PROJECT No. 147642129
Control 077
Rev. 0
FIGURE 12

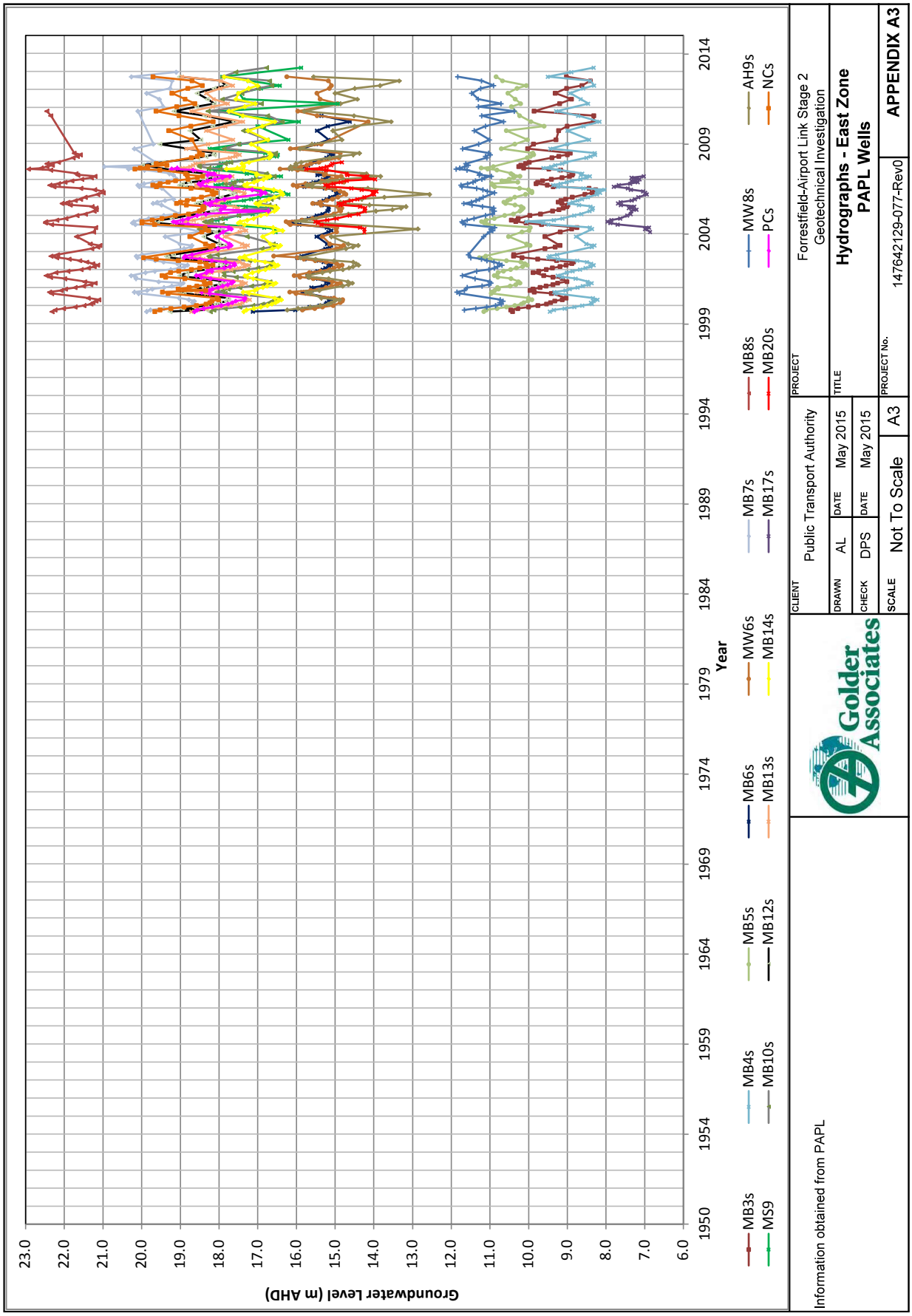
APPENDIX A

Historical Hydrographs

May 2015
Report No. 147642129-077-R-Rev0


Groundwater Level (m AHD)
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
1950
1954
1959
1964
1969
1974
1979
1984
1989
1994
1999
2004
2009
2014
Year
4722
4814
MW1S
MW2S
MW3S
MW4S
MW5S
MW6S
MW7S
MB01
MB03
Dow Wells (4722 and 4814) obtained from DoW WIN database
All other wells obtained from PB2013
Golder Associates
CLIENT
Public Transport Authority
PROJECT
Forrestfield-Airport Link Stage 2
Geotechnical Investigation
DRAWN
AL
DATE
May 2015
TITLE
Hydrographs - West Zone
Upper Aquifer
CHECK
DPS
DATE
May 2015
SCALE
Not To Scale
A3
PROJECT No.
147642129-077-Rev0
APPENDIX A1
J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Appendix A - Hydrographs\147642129-077-R-Rev0 - Appendix A - Hydrographs.xlsx


Groundwater Level (m AHD)
25.0
24.0
23.0
22.0
21.0
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
1950
1954
1959
1964
1969
1974
1979
1984
1989
1994
1999
2004
2009
2014
Year
4808
4809
4810
4811
4812
4813
4848
4883
14584
23042147
Information obtained from DoW WIN Database
Golder Associates
CLIENT
Public Transport Authority
PROJECT
Forrestfield-Airport Link Stage 2
Geotechnical Investigation
DRAWN
AL
DATE
May 2015
TITLE
Hydrographs - East Zone
DoW Wells
CHECK
DPS
DATE
May 2015
SCALE
Not To Scale
A3
PROJECT No.
147642129-077-Rev0
APPENDIX A2
J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Appendix A - Hydrographs\147642129-077-R-Rev0 - Appendix A - Hydrographs.xlsx


Groundwater Level (m AHD)
23.0
22.0
21.0
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
1950
1954
1959
1964
1969
1974
1979
1984
1989
1994
1999
2004
2009
2014
Year
MB3s
MB4s
MB5s
MB6s
MW6s
MB7s
MB8s
MW8s
AH9s
MS9
MB10s
MB12s
MB13s
MB14s
MB17s
MB20s
PCs
NCs
Information obtained from PAPL
Golder Associates
CLIENT
Public Transport Authority
PROJECT
Forrestfield-Airport Link Stage 2
Geotechnical Investigation
DRAWN
AL
DATE
May 2015
TITLE
Hydrographs - East Zone
PAPL Wells
CHECK
DPS
DATE
May 2015
SCALE
Not To Scale
A3
PROJECT No.
147642129-077-Rev0
APPENDIX A3
J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Appendix A - Hydrographs\147642129-077-R-Rev0 - Appendix A - Hydrographs.xlsx

APPENDIX B

March 2015 Groundwater Level Measurements

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX B

10 MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVEL MEASUREMENTS

Well ID	Chainage *	Offset (m) **	Type	Easting (PCG94)	Northing (PCG94)	Surface Elevation (m AHD)	Top of Casing Elevation (m AHD)	Monument Type	Borehole Depth (m)	Well or VWP Sensor Depth (m)	Screen (m bgl)	Material Screened	Aquifer	Groundwater or Piezometric Level	
											from - to			m b TOC	m AHD
LG 3-153	1435	-30	LG	60,395	266,740	9.17	9.80	Riser	20.0	13.0	6.0 - 13.0	Bassendean Sand	Upper	4.59	5.21
LG 3-149	1575	-20	LG	60,500	266,647	8.83	9.49	Riser	18.0	18.0	11.0 - 18.0	Perth Formation Clay Dominated	Aquitard	6.86	2.63
LG 2-18	2425	-18	LG	61,082	266,029	0.62	0.55	Riser	39.0	21.5	15.5 - 21.5	Swan River Alluvium	Aquitard	0.30	0.25
LG 3-131	2600	-19	LG	61,200	265,899	0.55	0.43	Gatic	19.5	19.5	15.5 - 19.5	Swan River Formation	Aquitard	0.26	0.17
LG 2-14	2825	35	LG	61,415	265,822	2.58	2.44	Gatic	35.0	31.9	16.9 - 31.9	Osborne Formation	Osborne	1.85	0.59
MW 3-180	70	0	MW	59,182	267,262	16.38	16.29	Gatic	6.5	6.5	3.5 - 6.5	Bassendean Sand	Upper	4.88	11.41
MW 3-175	500	-25	MW	59,650	267,164	15.06	14.98	Gatic	7.5	7.3	4.3 - 7.3	Bassendean Sand	Upper	5.17	9.81
MW 3-172	970	-20	MW	60,040	267,025	7.08	6.98	Gatic	4.5	2.7	1.2 - 2.7	Bassendean Sand	Upper	2.10	4.88
BH 2-26	1130	0	MW	60,187	266,966	10.14	10.73	Riser	35.0	35.0	26.0 - 35.0	Osborne Formation	Osborne	7.98	2.75
MW 3-165	1175	10	MW	60,242	266,946	8.54	9.12	Riser	17.0	5.0	2.0 - 5.0	Bassendean Sand	Upper	3.85	5.27
MW 3-166	1185	15	MW	60,232	266,948	9.23	9.85	Riser	20.0	5.5	2.5 - 5.5	Bassendean Sand	Upper	4.62	5.23
MW 3-159	1195	67	MW	60,287	266,976	9.86	10.56	Riser	6.2	5.2	2.2 - 5.2	Bassendean Sand	Upper	4.96	5.60
MW 2-02C	1230	8	MW	60,274	266,911	6.92	7.62	Riser	20.0	19.0	11.5 - 19.0	Perth Formation	Lower	6.05	1.57
MW 2-02A	1233	8	MW	60,277	266,911	6.90	7.67	Riser	26.0	26.0	22.0 - 26.0	Perth Formation	Lower	6.10	1.57
MW 2-02B	1235	48	MW	60,303	266,939	7.90	8.67	Riser	26.0	26.0	10.5 - 26.0	Perth Formation	Lower	6.98	1.69
MW 2-02D	1236	8	MW	60,279	266,909	6.87	7.63	Riser	6.9	6.7	0.7 - 6.7	Bassendean Sand	Upper	2.36	5.27
MW 3-156	1325	17	MW	60,338	266,863	7.89	8.56	Riser	17.0	17.0	15.0 - 17.0	Perth Formation Sand Dominated	Lower	6.96	1.60
MW 3-145	1805	-18	MW	60,660	266,483	9.77	9.69	Gatic	6.5	5.2	3.2 - 5.2	Bassendean Sand	Upper	4.47	5.22
MW 3-140	2195	43	MW	60,975	266,240	6.37	6.27	Gatic	6.0	6.0	3.5 - 6.0	Bassendean Sand	Upper	4.90	1.37
MW 3-137	2285	18	MW	61,015	266,157	5.28	5.22	Gatic	6.0	6.0	3.0 - 6.0	Perth Formation Sand Dominated	Upper	4.36	0.86
MW 3-136 (d)	2295	-18	MW	60,992	266,128	5.25	5.21	Gatic	28.5	15.5	12.5 - 15.5	Perth Formation Clay Dominated	Upper	4.38	0.83
MW 3-136 (s)	2295	-18	MW	60,993	266,129	5.25	5.22	Gatic	5.0	5.0	1.0 - 5.0	Perth Formation Sand Dominated	Upper	4.38	0.84
MW 3-132	2620	19	MW	61,237	265,914	0.83	0.79	Gatic	27.0	4.0	1.0 - 4.0	Swan River Formation	Superficial	0.46	0.33
MW 3-128	2815	30	MW	61,402	265,821	2.26	2.24	Gatic	37.0	4.0	1.0 - 4.0	Perth Formation Sand Dominated	Superficial	2.02	0.22
MW 3-125	3270	-64	MW	61,818	265,624	8.60	8.50	Gatic	5.3	5.3	2.3 - 5.3	Perth Formation Sand Dominated	Superficial	2.86	5.64
MW 3-121 (d)	3320	-28	MW	61,880	265,647	8.75	8.69	Gatic	27.1	22.0	19.5 - 23.7	Ascot Formation	Superficial	2.75	5.94
MW 3-121 (s)	3320	-28	MW	61,881	265,647	8.72	8.65	Gatic	6.0	5.5	1.0 - 5.5	Bassendean Sand	Superficial	2.59	6.06
MW 2-03	3330	-19	MW	61,888	265,654	8.75	8.68	Gatic	21.0	21.0	18.0 - 21.0	Osborne Formation	Osborne	2.69	5.99
MW 3-118	3695	-71	MW	62,232	265,524	9.66	9.57	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.89	7.68
MW 3-115	3705	39	MW	62,269	265,627	8.20	8.09	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.06	7.03
MW 3-109 (d)	3845	22	MW	62,403	265,579	9.84	9.75	Gatic	14.5	14.5	12.0 - 14.5	Ascot Formation	Superficial	2.09	7.66
MW 3-109 (i)	3850	22	MW	62,406	265,578	9.82	9.74	Gatic	10.0	10.0	8.0 - 10.0	Gnangara Sand	Superficial	2.06	7.68
MW 3-109 (s)	3855	22	MW	62,409	265,577	9.92	9.82	Gatic	4.5	4.5	1.5 - 4.5	Bassendean Sand	Superficial	2.14	7.68
MW 2-01D	3870	-3	MW	62,419	265,548	10.08	10.05	Gatic	18.0	17.0	11.0 - 17.0	Ascot Formation	Superficial	2.17	7.88
MW 3-108	3870	-44	MW	62,410	265,508	10.00	9.91	Gatic	20.0	17.5	14.5 - 17.5	Ascot Formation	Superficial	1.90	8.01
MW 2-01B	3910	-48	MW	62,447	265,496	10.12	10.05	Gatic	8.0	8.0	2.0 - 8.0	Bassendean Sand	Superficial	1.88	8.17
MW 2-01A	3925	-6	MW	62,473	265,533	10.32	10.28	Gatic	8.9	8.0	2.0 - 8.0	Bassendean Sand	Superficial	2.16	8.12
MW 2-01C	3925	-2	MW	62,473	265,537	10.33	10.29	Gatic	19.5	17.0	10.5 - 17.0	Ascot Formation	Superficial	2.19	8.10
MW 2-01E	3925	-4	MW	62,473	265,535	10.29	10.26	Gatic	25.0	23.0	20.0 - 23.0	Osborne Formation	Osborne	2.19	8.07
MW 3-105	3935	68	MW	62,498	265,603	10.12	10.02	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	2.08	7.94
MW 3-104	3970	23	MW	62,524	265,550	10.02	9.92	Gatic	18.1	17.1	14.1 - 17.1	Ascot Formation	Superficial	1.75	8.17
MW 3-103	3975	-49	MW	62,510	265,480	10.26	10.17	Gatic	14.0	4.5	1.5 - 4.5	Bassendean Sand	Superficial	1.83	8.34
BH 2-01	4035	-30	MW	62,575	265,483	10.44	11.01	Riser	28.0	18.5	11.0 - 18.5	Ascot Formation	Superficial	2.51	8.50
MW 3-100	4070	-48	MW	62,600	265,459	10.41	11.11	Riser	4.5	4.5	1.5 - 4.5	Bassendean Sand	Superficial	2.65	8.46
MW 3-099	4090	50	MW	62,645	265,547	10.29	10.19	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.83	8.36
MW 3-096	4335	-34	MW	62,867	265,412	11.96	11.86	Gatic	18.5	3.8	0.8 - 3.8	Bassendean Sand	Superficial	1.84	10.02
MW 3-094	4590	18	MW	63,125	265,402	12.96	12.98	Gatic	23.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	2.19	10.79
MW 3-093	4665	-46	MW	63,178	265,320	13.44	13.36	Gatic	4.5	4.5	1.5 - 4.5	Bassendean Sand	Superficial	2.15	11.21
MW 3-092	4675	42	MW	63,214	265,400	12.88	12.79	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.72	11.07
MW 3-089	4775	40	MW	63,309	265,362	13.50	13.42	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.77	11.65
MW 3-090	4825	-21	MW	63,330	265,284	14.00	13.88	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.63	12.25
MW 3-087	4970	-27	MW	63,445	265,208	14.81	14.71	Gatic	3.5	3.5	0.9 - 3.5	Bassendean Sand	Superficial	1.00	13.71
MW 1-03	5030	17	MW	63,522	265,208	14.68	14.59	Gatic	14.5	14.5	11.5 - 14.5	Ascot Formation	Superficial	1.89	12.70
MW 3-086 (d)	5035	15	MW	63,522	265,205	14.72	14.66	Gatic	28.0	22.0	20.0 - 22.0	Osborne Formation	Osborne	1.96	12.70
BH 1-25	6660	-5	MW	64,884	264,345	19.23	19.16	Gatic	48.0	42.0	29.0 - 42.0	Osborne Formation (Mirrabooka)	Osborne	2.28	16.88
MW 3-080 (d)	6660	30	MW	64,893	264,379	19.14	19.04	Gatic	20.0	20.0	17.0 - 20.0	Ascot Formation	Superficial	2.26	16.78
MW 3-080 (s)	6660	30	MW	64,891	264,378	19.17	19.08	Gatic	20.0	4.5	1.0 - 4.5	Bassendean Sand	Superficial	2.32	16.76
BH 1-21	6725	37	MW	64,955	264,371	19.03	19.54	Riser	36.0	24.9	15.9 - 24.9	Ascot Formation	Superficial	2.64	16.90
MW 3-077	6730	-27	MW	64,944	264,308	19.40	19.29	Gatic	14.0	5.0	1.5 - 5.0	Bassendean Sand	Superficial	2.38	16.91
MW 3-079	6730	-140	MW	64,915	264,197	20.38	20.32	Gatic	5.0	5.0	1.5 - 5.0	Bassendean Sand	Superficial	3.16	17.16
MW 1-02B	6780	-40	MW	64,989	264,283	19.45	20.08	Riser	28.5	26.0	14.0 - 26.0	Ascot Formation	Superficial	2.97	17.11
MW 1-02C	6785	-9	MW	65,005	264,311	18.63	19.32	Riser	11.0	11.0	5.0 - 11.0	Bassendean Sand	Superficial	2.30	17.02
MW 1-02D	6785	-6	MW	65,006	264,312	18.55	19.26	Riser	26.0	25.8	13.8 - 25.8	Ascot Formation	Superficial	2.17	17.09
MW 1-02A	6805	25	MW	65,032	264,340	18.56	19.05	Riser	36.0	26.0	14.0 - 26.0	Ascot Formation	Superficial	2.00	17.05
BH 1-13	7330	22	MW	65,544	264,238	18.90	19.55	Riser	36.0	16.5	15.5 - 16.5	Ascot Formation	Superficial	2.32	17.23
MW 3-065	7375	18	MW	65,588	264,226	18.72	19.22	Gatic	26.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	2.92	16.30
BH 1-12	7445	22	MW	65,658	264,219	18.94	19.60	Riser	35.0	23.5	15.5 - 23.5	Ascot Formation	Superficial	2.37	17.23
MW 3-064	7535	16	MW	65,746	264,200	19.09	19.56	Gatic	26.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	2.30	17.26

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FAL MASTER GROUNDATER LEVEL TABLE.xlsx

APPENDIX B

10 MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVEL MEASUREMENTS

Well ID	Chainage *	Offset (m) **	Type	Easting (PCG94)	Northing (PCG94)	Surface Elevation (m AHD)	Top of Casing Elevation (m AHD)	Monument Type	Borehole Depth (m)	Well or VWP Sensor Depth (m)	Screen (m bgl)	Material Screened	Aquifer	Groundwater or Piezometric Level	
											from - to			m b TOC	m AHD
MW 1-04	8110	15	MW	66,311	264,103	26.30	26.27	Gatic	31.5	31.5	28.5 - 31.5	Osborne Formation	Osborne	3.87	22.40
MW 3-060 (d)	8110	0	MW	66,308	264,087	26.33	26.29	Gatic	27.0	17.5	15.5 - 17.5	Guildford Formation	Superficial	3.86	22.43
MW 3-060 (s)	8110	0	MW	66,308	264,088	26.33	26.29	Gatic	4.5	4.0	1.0 - 4.0	Bassendean Sand	Perched	2.40	23.89
MW 3-059	8130	-33	MW	66,303	264,054	26.39	26.28	Gatic	4.0	3.5	1.0 - 3.5	Bassendean Sand	Perched	2.36	23.92
MW 3-055	8360	112	MW	66,599	264,019	28.81	28.71	Gatic	7.0	7.0	4.0 - 7.0	Guildford Formation	Superficial	5.95	22.76
MW 3-052	8545	-21	MW	66,564	263,781	28.35	28.97	Riser	17.0	10.0	7.0 - 10.0	Guildford Formation	Superficial	6.11	22.86
MW 3-051 (d)	8665	-40	MW	66,560	263,664	29.41	30.05	Riser	17.5	17.5	15.5 - 17.5	Guildford Formation	Superficial	7.18	22.87
MW 3-051 (s)	8665	-40	MW	66,560	263,662	29.39	30.01	Riser	17.5	7.0	4.0 - 7.0	Guildford Formation	Superficial	7.13	22.88
MW 1-01A	8710	13	MW	66,617	263,625	30.47	30.37	Gatic	46.2	32.0	8.0 - 32.0	Guildford Formation	Superficial	7.45	22.92
MW 1-01C	8720	12	MW	66,619	263,615	30.47	30.40	Gatic	45.0	32.3	7.0 - 32.3	Guildford Formation	Superficial	7.49	22.91
MW 1-01B	8725	-48	MW	66,558	263,604	29.33	29.26	Gatic	32.5	32.5	8.5 - 32.5	Guildford Formation	Superficial	6.37	22.89
MW 1-01D	8755	15	MW	66,624	263,578	30.19	30.07	Gatic	33.0	32.0	7.0 - 32.0	Guildford Formation	Superficial	7.15	22.92
MW 3-047	8780	18	MW	66,628	263,555	30.36	30.27	Riser	17.0	12.0	9.0 - 12.0	Guildford Formation	Superficial	7.32	22.95
MW 3-044 (d)	8825	-3	MW	66,606	263,509	29.54	30.27	Riser	14.0	14.0	11.0 - 14.0	Guildford Formation	Superficial	7.35	22.92
MW 3-044 (s)	8825	-3	MW	66,606	263,510	29.50	30.17	Riser	7.0	6.3	3.3 - 6.3	Guildford Formation	Perched	6.95	23.22
MW 3-038	8940	-57	MW	66,549	263,393	29.33	29.95	Riser	7.5	7.5	4.5 - 7.5	Guildford Formation	Superficial	7.07	22.88
MW 3-034	9010	200	MW	66,805	263,318	31.23	31.89	Riser	7.7	7.0	4.0 - 7.0	Guildford Formation	Perched	DRY	
MW 3-026	9170	-55	MW	66,542	263,164	28.39	29.02	Gatic	8.0	8.0	5.0 - 8.0	Guildford Formation	Superficial	6.22	22.80
MW 3-019	9250	75	MW	66,670	263,080	30.61	30.52	Gatic	6.5	6.5	3.5 - 6.5	Guildford Formation	Perched	4.32	26.20
MW 3-013	9315	506	MW	67,095	263,011	34.68	34.57	Gatic	17.5	17.3	16.3 - 17.3	Guildford Formation	Superficial	11.50	23.07
MW 3-012	9360	-58	MW	66,531	262,979	29.24	29.86	Riser	10.5	10.5	7.5 - 10.5	Guildford Formation	Superficial	6.97	22.89
MW 1-05	9595	-34	MW	66,548	262,742	28.42	28.42	Gatic	10.0	10.0	7.0 - 10.0	Guildford Formation	Superficial	5.01	23.41
MW 3-009	9630	6	MW	66,587	262,706	30.20	30.10	Gatic	8.0	6.9	3.9 - 5.9	Guildford Formation	Perched	5.56	24.54
MW 3-004	9755	-14	MW	66,561	262,579	30.00	29.89	Gatic	8.0	8.0	5.0 - 8.0	Guildford Formation	Perched	5.55	24.34
MW 3-001	9985	-32	MW	66,533	262,353	27.78	27.70	Gatic	6.0	6.0	3.0 - 6.0	Guildford Formation	Superficial	4.45	23.25
PW 2-02	1235	5	PW	60,275	266,907	6.91	7.40	Riser	27.0	26.0	10.5 - 26.0	Perth Formation	Lower	5.81	1.59
PW 2-01	3920	-4	PW	62,468	265,536	10.24	10.69	Riser	19.0	17.8	10.5 - 17.8	Ascot Formation	Superficial	2.57	8.12
PW 1-02	6790	-10	PW	65,009	264,309	18.60	18.99	Riser	28.5	26.0	12.0 - 26.0	Ascot Formation	Superficial	1.85	17.14

* Transposed perpendicular to alignment (approximate to nearest 5 m)

** Distance is measured to project centreline (between tunnels). Negative value is when monitoring point is located south or west of alignment. Positive value is when the monitoring point is located north or east of alignment

MW = Monitoring Well

VWP = Vibrating Wire Piezometer

LG = Gas Monitoring Well

PW = Pumping Well

NA = Not Applicable

NM = Not Measured

APPENDIX C

Summary of Analysis of PSD Samples

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX C
Particle Size Distribution Analysis

The hydraulic conductivity was calculated using Hazen's formula, shown below, which is an empirical method that uses data from the particle size distribution of a sample.

$$k = C_1 \times f(n) \times D_e^2 \times 975124$$

Where:

C_1 = 0.0047

f(n) = 1

$D_e = D_{10}$ in m

Hazen's method is usually used for samples with low fines content. Therefore samples with fines content greater than 10% were generally not analysed except for some samples from the Guildford and Perth Formation where samples with a fines content of up to 20% were analysed. It is therefore important to note that the derived hydraulic conductivities will be skewed toward giving higher hydraulic conductivity (i.e. the samples with large fines content which would result in low hydraulic conductivity have been excluded from analysis due to the limitation of the method).

The hydraulic conductivity was estimated for a total of 207 PSD test results. Table 1 presents a summary of the derived hydraulic conductivities.

Table 1: Hydraulic Conductivities Derived from Particle Size Distribution Analysis (m/d)

Geological Unit	CH0 to CH2800			CH2800 to CH4850			CH4850 to CH7600			CH7600 to CH9492			All			Number of Tests
	Range		Median	Range		Median	Range		Median	Range		Median	Range		Median	
Swan River Formation	3	9	6	-	-	-	-	-	-	-	-	-	3	9	6	4
Bassendean Sand	9	22	16	4	23	9	1	25	5	3	5	3	1	37	9	43
Perth Formation	1	23	2	5	14	9	-	-	-	-	-	-	1	23	5	25
Guildford Formation	-	-	-	1	16	3	1	36	3	1	25	2	1	36	2	79
Gnangara Sand	-	-	-	1	8	3	0.4	3	1	-	-	-	0.4	8	2	12
Ascot Formation	-	-	-	3	16	3	3	20	4	-	-	-	3	20	4	27
Osborne Formation Mirrabooka Member	-	-	-	-	-	-	0.3	20	9	-	-	-	0.3	20	9	17

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APPENDIX D

Summary of Slug Test Analysis

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX D
Summary of Slug Test Analysis

Single well hydraulic testing (slug testing) was carried out in 31 selected groundwater monitoring wells in March 2015.

The testing involved installing an automatic water level logger into the test well and introducing or removing a slug of PVC or water of known volume while measuring the resulting displacement in groundwater level at 1 second intervals. The groundwater level displacement data was then analysed using AQTESOLV Pro to provide an estimate of hydraulic conductivity (k) adjacent to the well screen.

Several tests were carried out at each well dependent on the time required for complete recovery of the initiated change in groundwater level. In most cases this comprised of three falling and three rising head tests.

Table 1 provides a summary of the estimated hydraulic conductivity for each well and for each hydrogeological unit. Table 1 only provides results for 29 of the 31 tests as the data collected at two of the wells was not suitable to estimate hydraulic conductivity.

Attachment D1 provides selected output files from AQTESOLV Pro of analysis of different groundwater displacement responses as examples:

- BH1-12 (Slug In, High k, Fast Response)
- MW1-03 (Slug Out, High k, Fast Response)
- MW1-02D (High k, Falling Head, Oscillatory Response)
- MW1-04 (Low k, Falling Head, Slow response)
- MW1-05 (Low k, Slug In, Moderate Response).

APPENDIX D
Summary of Slug Test Analysis

Table 1: Results of Slug Testing

Well ID	Location	Material Description at Well Screen	Well Screen Interval (m bgl)	Geological Unit (Formation)	Aquifer	Hydraulic Conductivity (m/d)			
						Individual Well		Hydrogeological Unit	
						Range	Median	Range	Median
MW2-02D	Bayswater	Sand (*f-c*)	0.7 – 6.7	Perth	Upper	5 – 6	6	0.3 – 35	11
MW2-02C	Bayswater	Sand (*c*)	11.5 – 19.0		Lower	10 – 32	27		
MW2-02A	Bayswater	Sandy clay/gravelly sand (*c*)	22.0 – 26.0			8 – 35	16		
MW2-02B	Bayswater	Sand (*f-m*),clayey sand(*f*), gravelly sand (*f-c*)	10.5 – 26.0			0.3 – 0.6	0.5		
LG2-18	Swan River	Clayey silt	15.5 – 21.5	Swan River	Aquitard	0.2 – 0.2	0.2	0.2	0.2
MW2-01A	Airport West	Sand (*m-c*), trace clay	2.0 – 8.0	Guildford	Superficial	1.1 – 2.2	2	0.1 – 12	2.5
MW2-01B	Airport West	Sand (*f-c*), trace clay	2.0 – 8.0			2.9 – 3.8	3		
MW1-02C	Consolidated	Sand (*f-c*)/clayey sand (*m-c*)	5.0 – 11.0			5 – 12	8		
MW1-01A	Forrestfield	Sand (*f-c*),silty sand (*f-m*)	8.0 – 32.0			0.1 – 0.3	0.2		
MW1-01B	Forrestfield	Sand (*f-c*), silty sand *(f-m)*, clayey sand (*f-m*)	8.5 – 32.5			0.1 – 1.5	0.4		
MW1-01C	Forrestfield	Clayey sand (*f-m*), sand (*f-c*), silty sand (*f-m*)	7.0 – 32.3			0.3 – 0.4	0.4		
MW1-01D	Forrestfield	Clayey sand (*f-m*), sand (*f-c*), clayey gravelly sand (*f*)	7.0 – 32.0			0.5 – 5.1	4		
MW1-05	Forrestfield	Sand (*f-m*), clayey sand (*f-m*)	7.0 – 10.0			0.1 – 0.2	0.2		

APPENDIX D
Summary of Slug Test Analysis

Well ID	Location	Material Description at Well Screen	Well Screen Interval (m bgl)	Geological Unit (Formation)	Aquifer	Hydraulic Conductivity (m/d)			
						Individual Well		Hydrogeological Unit	
						Range	Median	Range	Median
MW2-01D	Airport West	Silty gravelly sand (*f-c*)	11.0 – 17.0	Ascot	Superficial	2 – 26	6	1 – 75	7.5
MW2-01C	Airport West	Calcarenite (*m-c*)	10.5 – 17.0			3 – 23	18		
BH2-01	Airport West	Silty sand (*f*), sand (*f-m*), gravel (*f-c*)	11.0 – 18.5			2 – 3	3		
MW1-03	Airport	Silty sand Gravel (*f*)	11.5 – 14.5			5 – 8	7		
BH1-21	Consolidated	Silty sandy gravel (*f-c*), sand (*f-m*)	15.9 – 24.9			1 – 4	3		
MW1-02A	Consolidated	Gravelly silty sand (*f*), calcarenite (*f*)	14.0 – 26.0			2 – 8	5		
MW1-02D	Consolidated	Gravelly sand (*m-c*), calcarenite	13.8 – 25.8			15 – 21	17		
MW1-02B	Consolidated	Sandy gravel (*f*), gravelly sand (*f-c*)	14.0 – 26.0			3 – 12	12		
BH1-12	Consolidated	Sand (*f-c*), gravel (*f-c*), calcarenite (*f-c*)	15.5 – 23.5			5 – 12	8		
BH1-13	Consolidated	Sandy gravel (*f-c*)	15.5 – 16.5			20 – 75	50		
LG2-14	Swan River	Silty sandstone (*f*)	16.9 – 31.9	Osborne Formation (Kardinya Shale)	Osborne Aquitard	0.0002	0.0002	0.0002 – 0.008	0.0005
MW3-121 (d)	Airport West	Sandstone (*f*)	19.5 – 23.7			0.0005	0.0005		
MW2-01E	Airport West	Siltstone, claystone	20.0 – 23.0			0.0006	0.0006		
MW1-04	EE4	Sandy clay	28.5 – 31.5			0.008	0.008		
BH1-25	Consolidated	Sand (*f-c*), gravelly clayey sand (*f-c*)	29.0 – 42.0	Osborne Formation	Mirrabooka	1 – 4	3	1 – 7	4.5
MW1-02E	Consolidated	Sandy Gravel (*f-c*), calcarenite	27.0 – 34.0			4 – 7	6		

f,- fine grained, *m* – medium grained, *c* – coarse grained, m bgl – meters below ground level

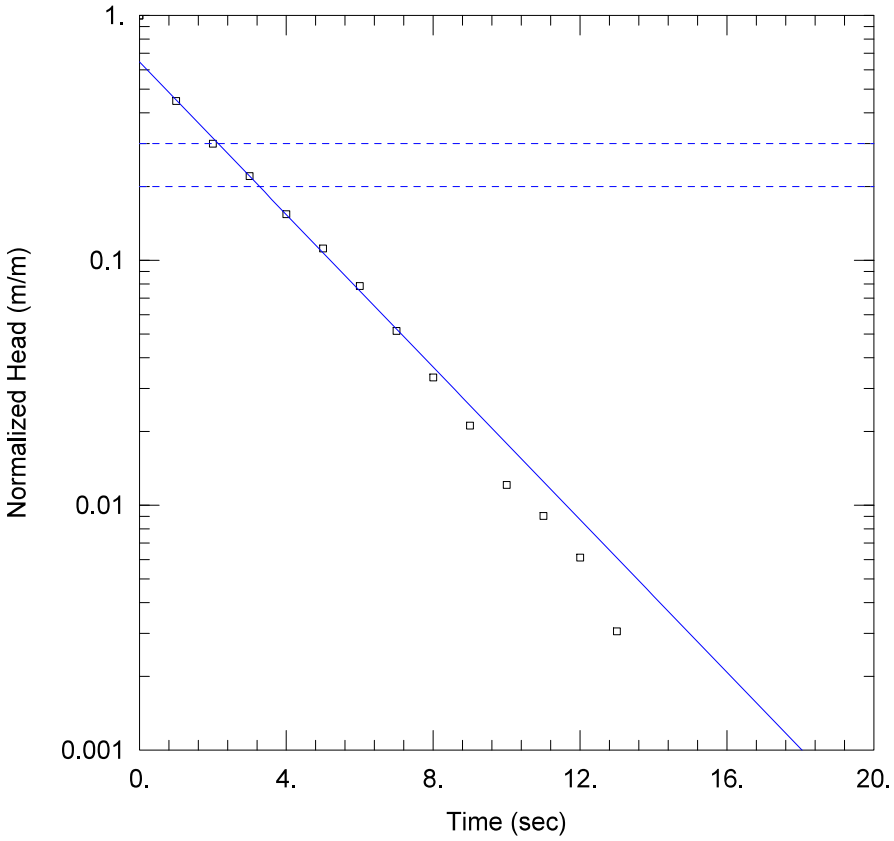


APPENDIX D
Summary of Slug Test Analysis

ATTACHMENT D1
Selected AQTESOLV Solutions from Slug Testing

May 2015
Reference No. 147642129-077-R-Rev0

Golder Associates



WELL TEST ANALYSIS

Data Set: J:\...\BH1-12 slug in 2_AL.aqt
Date: 04/08/15 Time: 09:15:44

PROJECT INFORMATION

Test Well: BH1-12

AQUIFER DATA

Saturated Thickness: 21.81 m Anisotropy Ratio (Kz/Kr): 0.1

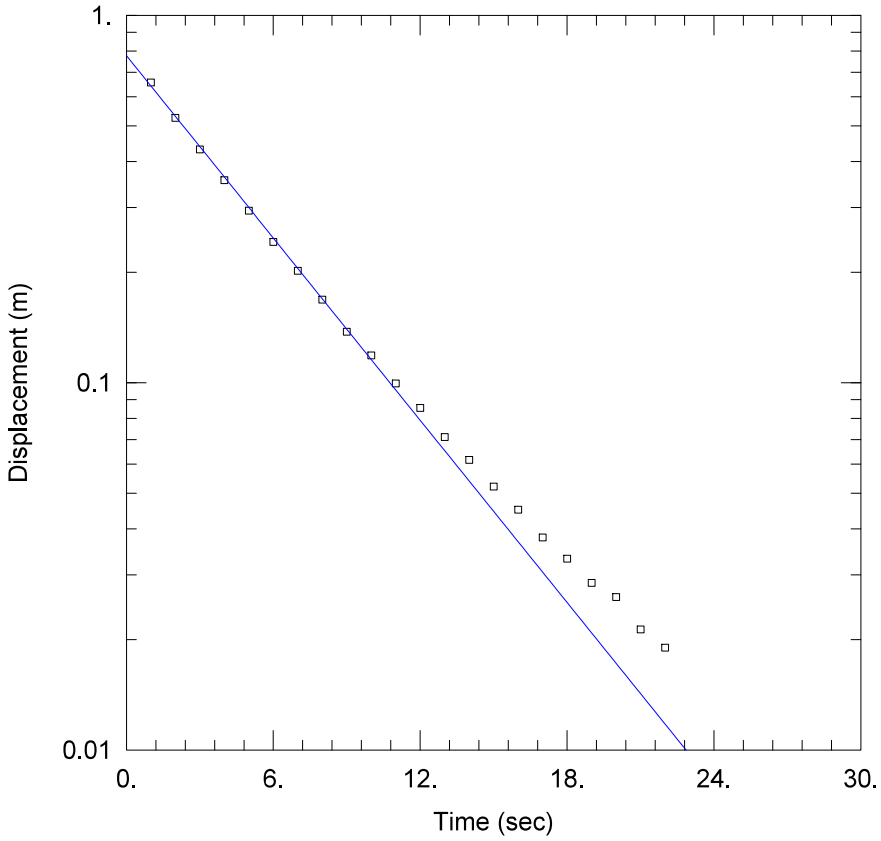
WELL DATA (BH1-12)

Initial Displacement: 0.7852 m
Total Well Penetration Depth: 21.81 m
Casing Radius: 0.0253 m
Static Water Column Height: 21.81 m
Screen Length: 9. m
Well Radius: 0.06 m
Gravel Pack Porosity: 0.

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 6.427 m/day y0 = 0.5072 m



WELL TEST ANALYSIS

Data Set: J:\...\MW1-03 slug out 1.aqt
Date: 04/08/15 Time: 09:27:58

PROJECT INFORMATION

Test Well: MW1-03

AQUIFER DATA

Saturated Thickness: 11.74 m Anisotropy Ratio (Kz/Kr): 0.1

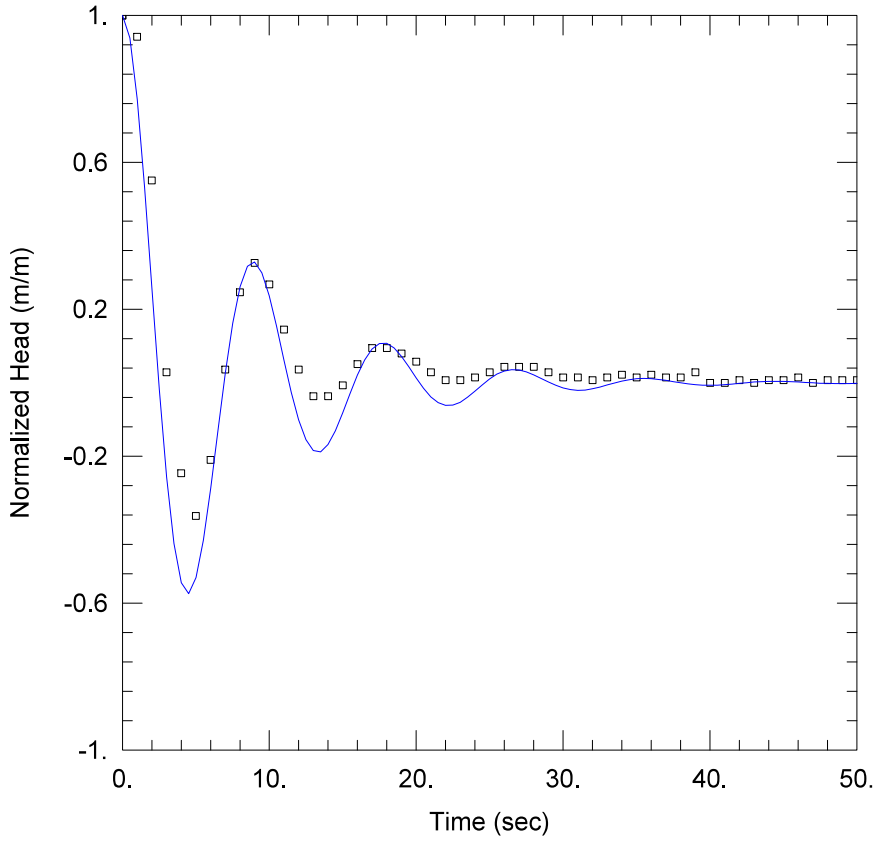
WELL DATA (MW1-03)

Initial Displacement: 1.233 m
Total Well Penetration Depth: 11.74 m
Casing Radius: 0.0253 m
Static Water Column Height: 11.74 m
Screen Length: 3.5 m
Well Radius: 0.0253 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 6.883 m/day y0 = 0.7768 m



WELL TEST ANALYSIS

Data Set: J:\...\MW1-02D - Falling Head 1_AL.aqt
Date: 04/08/15 Time: 09:29:14

PROJECT INFORMATION

Test Well: MW1-02D

AQUIFER DATA

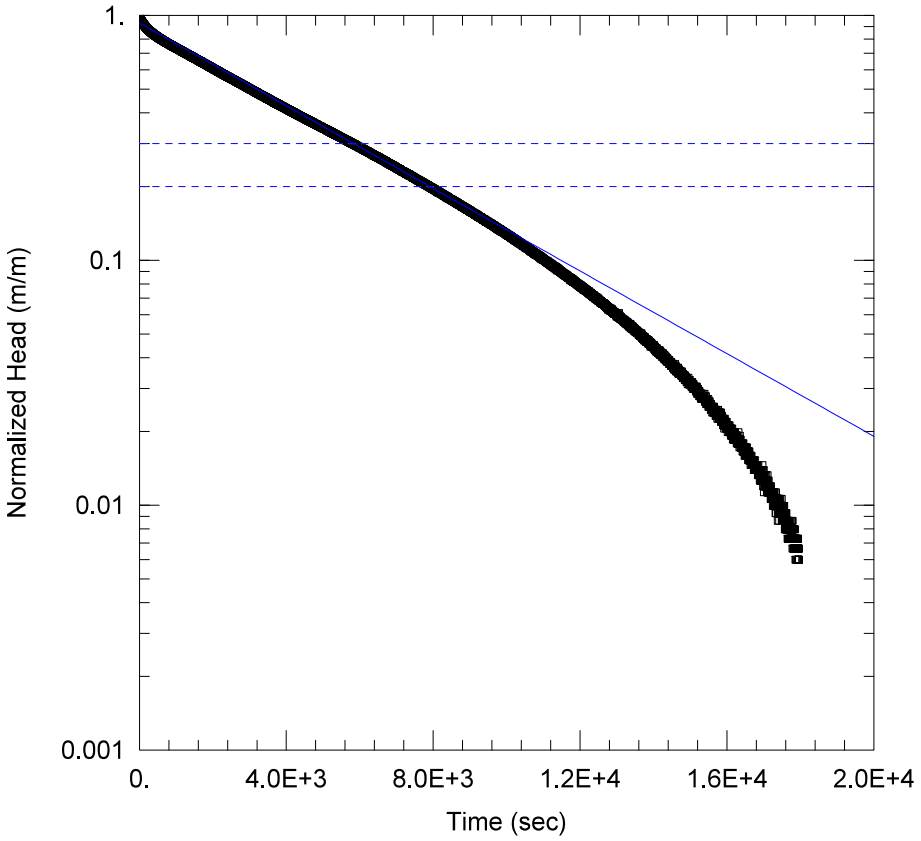
Saturated Thickness: 29.78 m Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (MW1-02D)

Initial Displacement: 0.3273 m
Total Well Penetration Depth: 23.78 m
Casing Radius: 0.0253 m
Static Water Column Height: 23.78 m
Screen Length: 12. m
Well Radius: 0.06 m

SOLUTION

Aquifer Model: Unconfined
K = 20.53 m/day
Solution Method: Springer-Gelhar
Le = 19. m



WELL TEST ANALYSIS

Data Set: J:\...\MW1-04-Confined.aqt
Date: 04/08/15 Time: 09:20:55

PROJECT INFORMATION

Test Well: MW1-04

AQUIFER DATA

Saturated Thickness: 4.5 m Anisotropy Ratio (Kz/Kr): 0.1

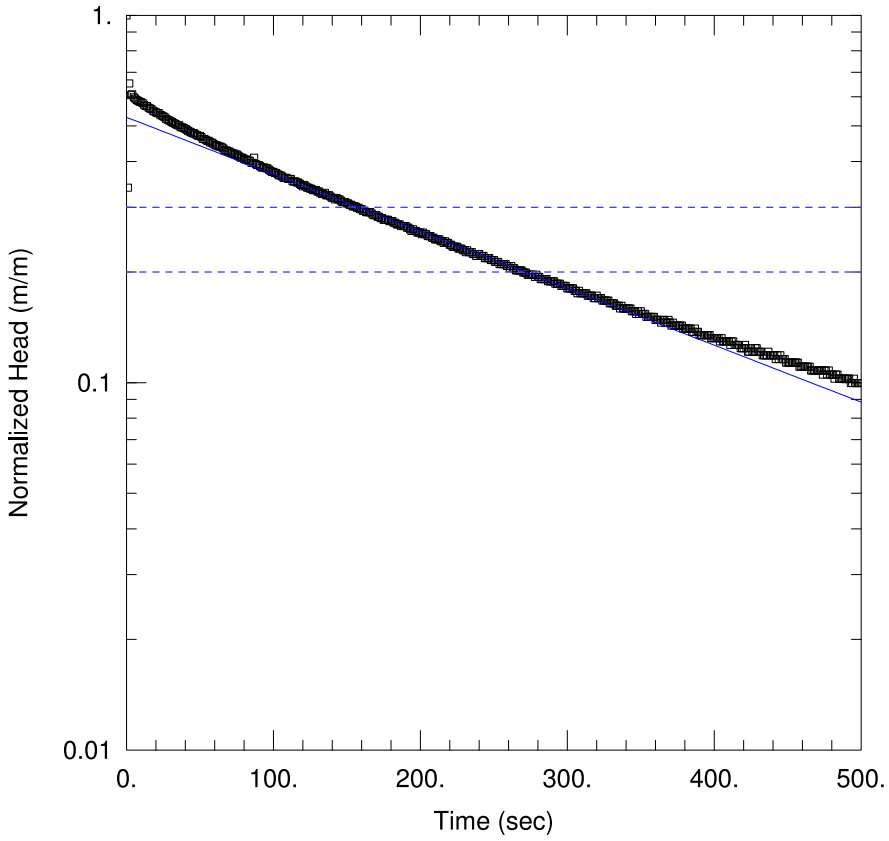
WELL DATA (MW1-04)

Initial Displacement: 3.578 m
Total Well Penetration Depth: 27.89 m
Casing Radius: 0.0253 m
Static Water Column Height: 27.89 m
Screen Length: 3.5 m
Well Radius: 0.06 m

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice

K = 0.006694 m/day y0 = 3.333 m



WELL TEST ANALYSIS

Data Set: J:\...\MW1-05- Slug In_AL.aqt
Date: 05/16/15 Time: 16:56:23

PROJECT INFORMATION

Test Well: MW1-05

AQUIFER DATA

Saturated Thickness: 4.87 m
Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (MW1-05)

Initial Displacement: 0.8801 m
Total Well Penetration Depth: 4.87 m
Casing Radius: 0.0253 m
Static Water Column Height: 4.87 m
Screen Length: 3. m
Well Radius: 0.06 m
Gravel Pack Porosity: 0.

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice
K = 0.1633 m/day
y0 = 0.4643 m

APPENDIX E

Summary of Test Pumping Analysis

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX E
Summary of Test Pumping Analysis

1.0 INTRODUCTION

This Appendix describes and presents the results from the four pumping tests carried out as part of the Stage 2 Geotechnical Investigation.

The following pumping tests were completed at the following sites:

- PW1-01 at Forrestfield Dive Structure – 8 to 12 December 2014
- PW1-02 at Consolidated Airport Station – 5 to 9 January 2015
- PW2-01 at Airport West Station – 19 to 22 January 2015
- PW2-02 at Bayswater Dive Structure – 2 to 6 February 2015.

A 26D licence under the Rights in Water and Irrigation Act 1914 was obtained to install all the pumping test wells and to carry out all the pumping tests. This licence was issued by the Department of Water (DoW) and allowed for the installation, development and testing of the pumping test wells. Additional approvals were obtained for disposal of the groundwater abstracted during the pumping tests. The required disposal approvals are site specific and are discussed specifically for each site in the sections below.

2.0 GENERAL METHODOLOGY

2.1 Pumping Tests

The pumping tests were completed by McArthur Drilling and Pumping (MDP) under the supervision of Golder in general accordance with AS 2368-1990 – "Test pumping of water wells". The following tests were carried out at each of the four sites:

- Step-Rate – The test involved pumping the test well at four different pumping rates (with the rate constant during each step, but increasing between steps) with each step lasting around one hour. The purpose of this test is to assess the pumping rate for the constant rate test.
- Constant Rate – This test involved pumping the test well at a constant rate for up to 72 hours depending on the response observed in the pumping and monitoring wells during the test.
- Recovery – The recovery occurred immediately following the cessation of the constant rate test (i.e. the start of the test is when the pump is turned off for the constant rate test) and was continued until approximately 95% recovery had occurred.

Some changes to the methodology did occur at each site due to site specific conditions and logistics. The design of each pumping test focused on maximising the return of information from each of the four pumping test locations.

2.2 Monitoring

Monitoring of both groundwater level and quality was carried out as part of the pumping tests in accordance with AS 2368-1990. In summary, the groundwater monitoring consisted of:

- Background barometric pressure and groundwater levels in monitoring and pumping wells were monitored prior to the start of each pumping test using automated barometric and groundwater level loggers.
- Groundwater level monitoring in the pumping well and at least four monitoring wells was completed during each pumping test using time-synchronised, automated groundwater level loggers logging at 1 minute intervals.

- Manual measurement of groundwater level in the pumping wells and the monitoring wells closest to the pumping well was undertaken at the intervals outlined in AS 2368-1990 as a quality assurance control and to verify automated water level logger information during each pumping test. In addition, groundwater levels in other monitoring wells with automated loggers was manually measured during the pumping tests as time permitted.
- Barometric pressure was measured for the duration of the pumping tests using an automated pressure logger that was synchronised with the groundwater level loggers to allow for barometric correction of the groundwater level logger data.
- Flow volume and discharge rate from the pumping well was measured using a digital flow meter at the wellhead.
- Water quality field monitoring of pH, electrical conductivity (EC), and temperature was carried out on the abstracted groundwater.
- Sampling of discharge water for laboratory water quality analysis for major ions, metals, nutrients, TPH, BTEX, PAH, MAH and pesticides was undertaken as a minimum at the beginning of each pumping test and at the end of each pumping test. A sample was also taken halfway through the test at Airport West (PW2-01) at the request of Swan River Trust (SRT).

Some variations in the monitoring at each site did occur due to site specific conditions and logistics.

2.3 Groundwater Disposal

The groundwater abstracted during pumping well development was either reinfiltrated on site (Bayswater), pumped to a turkeys nest for construction use by others (Consolidated Airport Terminal Station) or pumped to the stormwater drain (Forrestfield and Airport West). The method of disposal was evaluated on a case by case basis. Approval from the relevant asset owner and regulators was obtained before proceeding with each pumping test.

2.4 Analysis

The pumping test data was analysed analytically [1, 2] using multiple analytical solutions and type curves and numerically using Visual MODFLOW. A basic numerical 3D groundwater model developed using Visual MODFLOW for the area around each pumping well was used to match the pumping test data by manually changing hydraulic conductivity and storage.

3.0 PW 1-01 (FORRESTFIELD DIVE STRUCTURE)

The results from the Stage 2 Geotechnical Investigation indicate that the regional Superficial Aquifer at the Forrestfield Dive Structure is comprised of Guildford Formation with a depth of approximately 32 m from current ground surface. The pumping well and associated monitoring wells were installed to the bottom of the Guildford Formation and screened across the whole aquifer (screen lengths of approximately 24 m to 27 m).

3.1 Site and Pumping Test Layout

Pumping tests were carried out at PW 1-01 between 8 and 12 December 2014 and included:

- A step test with pumping rates set at 6, 8, 10 and 12 L/s. The first three steps lasted for 60 minutes each, while the fourth step was stopped after 8 minutes due to the groundwater level drawdown reaching the pump.

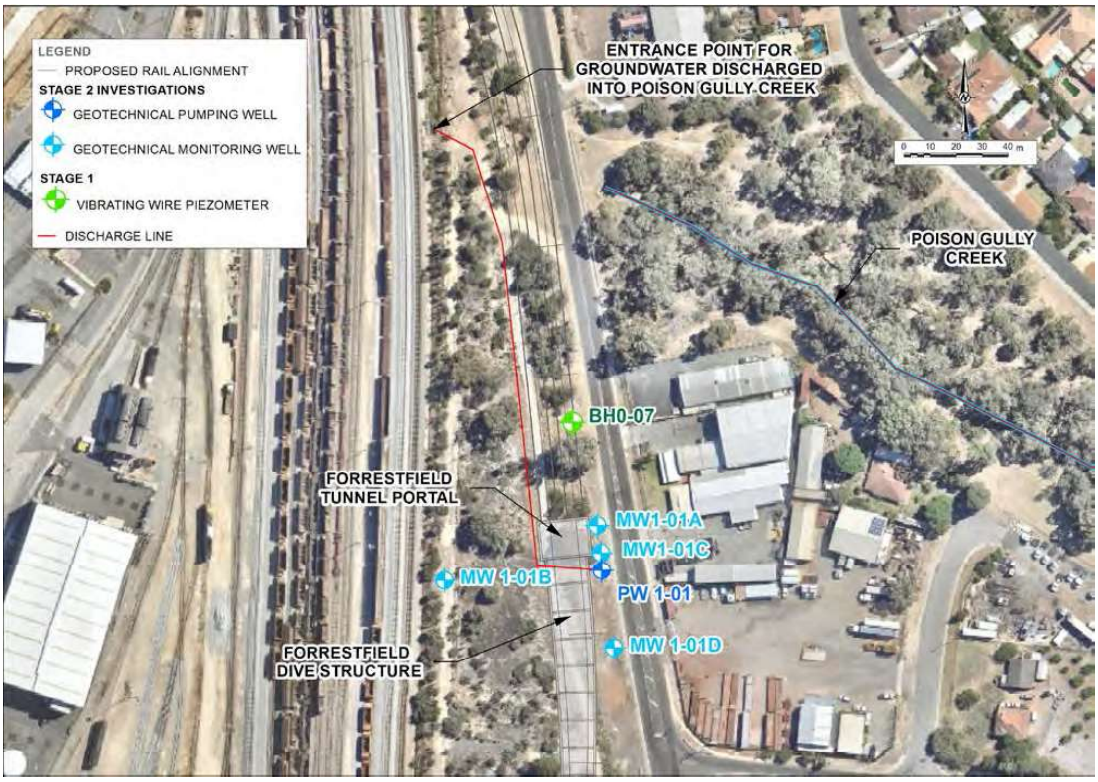
[1] Kruseman, G.P. and N.A. de Ridder, 1994. Analysis and Evaluation of Pumping Test Data. Second Edition. Publication 47. International Institute for Land Reclamation and Improvement, Wageningen, The Netherlands, 1994.

[2] Cooper, H.H. and C.E. Jacob, 1946. A generalized graphical method for evaluating formation constants and summarizing well field history. Am. Geophys. Union Trans. 27: 526-534.

APPENDIX E
Summary of Test Pumping Analysis

- A 72-hour constant rate test with a discharge rate of approximately 10 L/s; followed by
- A 9-hour recovery test.

Sketch 1 shows the site layout of the pumping test well (PW1-01), four dedicated monitoring wells (MW1-01A to MW1-01D) and Vibrating Wire Piezometers (VWPs) (BH0-07) while Table 1 presents the distances and directions of the different monitoring locations from the pumping test well.



Sketch 1: Site Layout for pumping test at (PW 1-01)

Table 1: Distance and Direction from the Pumping Test Well to the Monitoring Locations

Well ID	Type	Distance to Pumping Well (m)	General Direction to Pumping Well	Screened Geological Unit
PW1-01	Pumping Well	-	-	GF
MW1-01-A	Dedicated Monitoring Well	18	North	GF
MW1-01-B	Dedicated Monitoring Well	61	West	GF
MW1-01-C	Dedicated Monitoring Well	7	North	GF
MW1-01-D	Dedicated Monitoring Well	30	South	GF
BH0-07	Vibrating Wire Piezometer	58	North	GF
BH1-02	Vibrating Wire Piezometer	74	South	GF
BH1-04	Vibrating Wire Piezometer	197	North	GF

* Location of BH1-02 and BH1-04 are not shown on Sketch 1 due to the distance from PW1-01. Refer to Figure 5 in the main report for location of these VWPs.

3.2 Discharge Location

The abstracted groundwater during the pumping test was discharged to Poison Gully Creek at the confluence just before it becomes a man-made drain and enters part of the Perth Airport Drain system (Sketch 1). Discharge approval was obtained from the Shire of Kalamunda.

3.3 Analysis and Results

Figures E1–1 to E1–2 summarise the pumping test results while Table 2 presents the estimated hydraulic properties from the different analysis methods.

Table 2: Results from Pumping Test Analysis – Forrestfield Dive Structure

Method	Transmissivity (m^2/d)	Storage Coefficient	Figures
Distance Drawdown	180	6.0×10^{-3}	E1-3
Groundwater Model	104	2.5×10^{-3}	E1–4 to E1–5
Analytical Type Curves	94–316	-	E1–6 to E1–15 *

* Selected type curves only

The results indicate:

- The groundwater level drawdown in the pumping well reached almost 16 m toward the end of the test and the groundwater level almost reached steady-state conditions at the end of the pumping test
- The groundwater level drawdown in the monitoring wells ranged between 0.9 m and 3.2 m, depending on their distance from the pumping well.
- Four hours after starting the Constant Rate test the flow rate was adjusted as it was decreasing. The increase in flow rate resulted in a step change in the groundwater level within the test well. However, the groundwater level decline was similar before and after the step change, suggesting that the step change in level within the pumping well was due to removal of bore storage when the pumping rate was increased.
- The distance drawdown curve (Figure E1–3) indicates that
  - There is a relatively good correlation between distance (from the pumping well) and drawdown, which suggests generally homogenous conditions.
  - The expected groundwater level drawdown in the aquifer just outside the pumping well is around 6.5 m, indicating that 9.5 m of the 16 m of drawdown in the well was caused by well loss.
- There is a reasonably good match between observed and modelled groundwater level drawdown (E1–4 and E1–5). There was found to be only a small difference when varying the vertical hydraulic conductivity in the model, which is likely due to the wells being screened over the full aquifer thickness and the presence of relatively homogenous conditions.
- The estimated range in the derived storage coefficient suggests semi-confined to confined conditions.

3.4 Groundwater Quality

Figure E1–16 shows the field monitoring results of pH and Electrical Conductivity (EC) from the Constant Rate Test while Table E1 presents the groundwater quality laboratory results from samples taken at the beginning and end of the pumping test. The results indicate:

- pH is acidic and ranged between 5.5 and 6.0 during the pumping test with no particular trend, excluding one outlier.

- EC was generally steady throughout the pumping test at around 1000 uS/cm, indicating slightly brackish water.
- Total Nitrogen was 44 mg/L and 33 mg/L at the beginning and end of test respectively, indicating high nutrient concentrations.
- Total iron increased from below detection limit at the beginning of the test to 0.018 mg/L at the end of the test, which is well below SRT criterion of 1 mg/L.
- BTEX, MAH, PAH, hydrocarbons and pesticides were not detected in either of the two samples.

4.0 PW1-02 (CONSOLIDATED AIRPORT TERMINAL STATION)

The results from the Stage 2 Geotechnical Investigation indicated that the Superficial Aquifer at this location consists of the Bassendean Sand, Guildford Formation and Ascot Formation, overlying the Mirrabooka Member of the Osborne Formation. The Superficial Aquifer is approximately 24 m thick at the Consolidated Airport Terminal Station, of which approximately 12 m consists of the Ascot Formation.

The pumping test well was installed and screened across the full thickness of the Ascot Formation with two of the dedicated monitoring wells (MW1-02B and MW1-02D) installed in the Ascot Formation, one monitoring well in the Bassendean Sand and Guildford Formation (MW1-02C) and one well (MW1-02E) in the Mirrabooka Aquifer.

4.1 Site and Pumping Test Layout

Pumping tests were carried out at PW 1-02 between 5 and 9 January 2015 and included:

- A step test with pumping rates set at 6, 8, 10, 12 and 13.5 L/s. The first four steps lasted for 60 minutes each, while the fifth step was stopped after 3 minutes due to excessive drawdown and the pump sucking in air.
- CRT 1 - A 52-hour constant rate test with a discharge rate of approximately 12.0 L/s followed by an 18.5-hour recovery test.
- CRT 2 - A 6-hour constant rate test with a discharge rate of approximately 9.5 L/s followed by a 1.5 hour recovery test.

CRT 1 was found to pump some air during the pumping test which resulted in some fluctuations in the groundwater level inside the pumping well. Following the completion of the recovery after CRT 1 it was therefore decided to run a short second test (CRT2) at a lower pumping rate to see if less groundwater level fluctuation would be observed in the pumping well.

Sketch 2 shows the site layout of the pumping test well (PW1-02), monitoring wells and VWPs while Table 3 presents the distances and directions of the different monitoring locations from the pumping test well.

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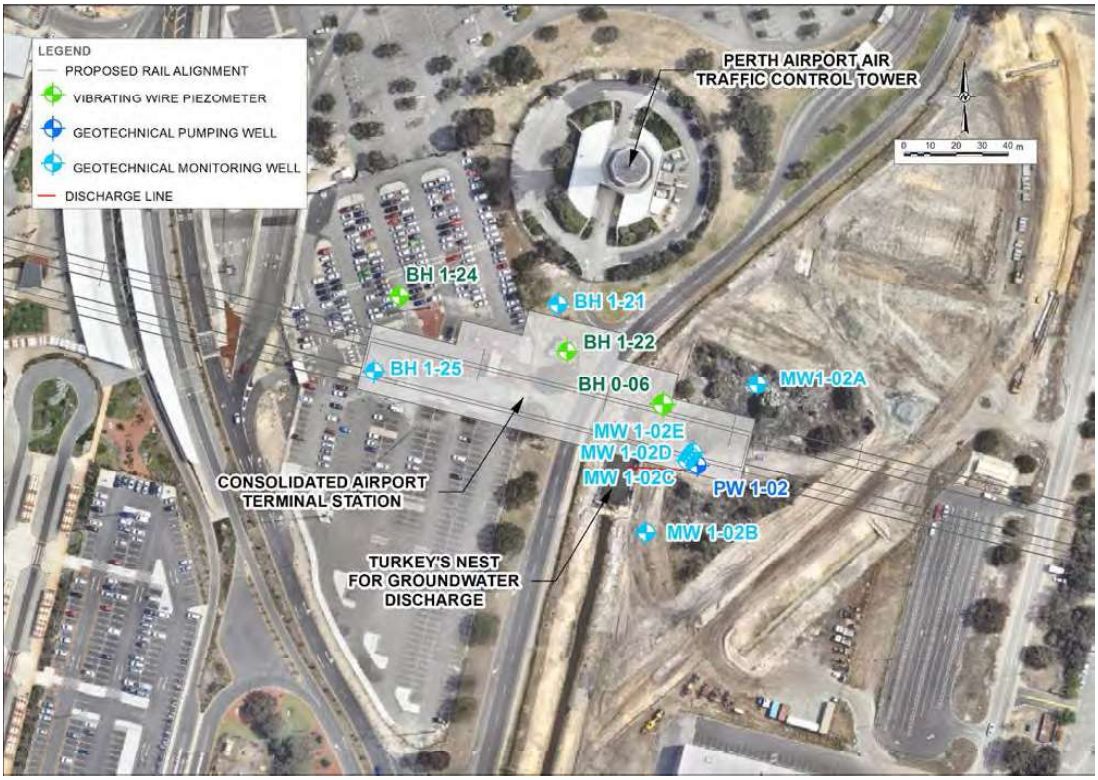
Summary of Test Pumping Analysis



Sketch 2: Site Layout for pumping test at PW 1-02

Table 3: Distance and Direction from the Pumping Test Well to the Monitoring Locations

Well ID	Type	Distance to Pumping Well (m)	General Direction to Pumping Well	Screened Geological Unit
PW1-02	Pumping Well	-	-	AF
MW1-02-A	Dedicated Monitoring Well	39	Northeast	AF
MW1-02-B	Dedicated Monitoring Well	33	Southwest	AF
MW1-02-C	Dedicated Monitoring Well	5	West	BS, GF, GS
MW1-02-D	Dedicated Monitoring Well	5	West	AF
MW1-02-E	Dedicated Monitoring Well	4	Northwest	OFm
BH1-21	Monitoring Well	82	Northwest	AF
BH1-25	Monitoring Well	130	West	OFm
BH0-06	Vibrating Wire Piezometer (two sensors)	27	Northwest	GF and AF
BH1-22	Vibrating Wire Piezometer (two sensors	67	Northwest	AF and OFm
BH1-24	Vibrating Wire Piezometer (three sensors)	132	West	GF, AF and OFm



APPENDIX E
Summary of Test Pumping Analysis

4.2 Discharge Location

The abstracted groundwater during the pumping test was discharged into a turkeys nest located approximately 30 m west PW1-02 (Sketch 2). The water from the turkeys nest was used during the day by Densford Civil for dust suppression. Normally Densford Civil obtained their construction water from a reticulation bore located approximately 50 m north of the site. However, the pumping rate from PW1-02 was sufficient to supply Densford Civil with their water demand and therefore the reticulation bore could be turned off during the test. While the pump test was being carried out, the turkeys nest overflowed into the Perth Airport Drain (Sketch 2) during the night when the water was not being used by Densford Civil. Discharge approval into the Perth Airport Drain was obtained from PAPL.

4.3 Analysis and Results

Figures E2–1 to E2–3 summarise the pumping test results while Table 4 presents the estimated hydraulic properties from the different analysis methods.

Table 4: Results from Pumping Test Analysis – Consolidated Airport Terminal Station

Method	Transmissivity (m^2/d)	Storage Coefficient	Figures
Distance Drawdown	271–281	1×10^{-2} - 5×10^{-2}	E2–4 to E2–5
Groundwater Model	274	6×10^{-4}	E2–6
Analytical Type Curves	136–690 *	-	E2–7 to E2–20 *

* Selected type curves only

The results indicate:

- The drawdown in piezometric level in the pumping well reached approximately 17 m and 9 m during Test 1 and Test 2, respectively, and the groundwater level reached steady-state within only a few hours of pumping.
- The groundwater and piezometric level drawdown in the monitoring wells ranged between 0.1 m and 2.3 m, depending on the distance of the monitoring well from the pumping well. This suggests that there is a large well loss in the pumping well (according to the distance drawdown curves the expected groundwater level drawdown in the aquifer just outside the pumping well is estimated to be around 4.5 m and 3.5 m instead of the observed 17 m and 9 m, for the two tests respectively).
- MW1-02E and BH1-25, both screened in the underlying Osborne Formation, show significant hydraulic connection with the Ascot Formation. MW1-02E shows a very similar response to pumping when compared with MW1-02D which is screened in the Ascot Formation and located next to MW1-02E.
- BH1-25 shows a small response to pumping at a distance of 130 m from the test well. The hydraulic connection between the Osborne Formation and the Ascot Formation is supported by the distance drawdown plot where the measurements for both formations plot on a straight line.
- The drawdown in piezometric level at BH1-21 (located closest to the Perth Airport control tower) was approximately 0.5 m at the end of the test.
- MW1-02C, screened in the overlying Bassendean Sand, Guildford Formation and Gnangara Sand, shows a similar but reduced response to pumping when compared with MW1-02D and MW2-01E. This response indicates some connection with the Ascot Formation, semi-confined conditions and a lower vertical hydraulic conductivity.
- A good match between observed and modelled groundwater level drawdown (Figure E2–6) was obtained. The Ascot Formation was found to have the greatest hydraulic conductivity. The best match was obtained when using a vertical to horizontal hydraulic conductivity ratio of 0.1 (= k_v/k_h).

- The estimated range in storage coefficient suggests semi-confined to confined conditions.

4.3.1 Groundwater Quality

Figure E2–21 shows the field monitoring results of pH and Electrical Conductivity (EC) from the Constant Rate tests while Table E1 presents the groundwater quality laboratory results from samples taken at the beginning and end of the pumping test. The results indicate:

- pH was stable throughout the pumping tests ranging between 7.5 and 8.0, indicating slightly alkaline conditions, excluding one outlier.
- EC was stable throughout the pumping tests ranging between ranging between 600 and 800 uS/cm, indicating fresh water.
- Total Nitrogen was between 0.16 mg/L and 0.1 mg/L at the beginning and end of each test respectively, which is below the SRT criterion for disposal to Swan River (1 mg/L). However, total phosphorous (0.2 mg/L and 0.13 mg/L) was above the SRT criterion (0.1 mg/L).
- Total iron was low and ranged between 0.57 mg/L and 0.87 mg/L, which is below the SRT criterion of 1 mg/L.
- BTEX, MAH, PAH, hydrocarbons and pesticides were not detected in either of the two samples.

5.0 PW 2-01 (AIRPORT WEST STATION)

The results from the Stage 2 Geotechnical Investigation indicated that the Superficial Aquifer at Airport West Station consists of Bassendean Sand, Guildford Formation, Gnangara Sand and Ascot Formation, overlying the Kardinya Shale of the Osborne Formation. The Superficial Aquifer is approximately 16 m thick at the Airport West Station, of which approximately 8 m consists of the Ascot Formation. The Osborne Formation below the Airport West Station consists of Kardinya Shale which was tested to have a very low hydraulic conductivity. A pumping test in the Osborne Formation at this location was therefore considered to be not necessary. The most permeable unit at the Airport West Station is the Ascot Formation and it was therefore decided to undertake the pumping test in a pumping well screened across the Ascot Formation. The pumping test well was installed and screened across the full thickness of the Ascot Formation with two of the dedicated monitoring wells (MW2-01C and MW2-01D) installed in the Ascot Formation, one monitoring well in the Kardinya Shale (MW2-01E) and two wells in the Bassendean Sand (MW2-01A and MW2-01B). MW2-01B was located on the other side of Belmont Main Drain to assess the effect the drain had on the pumping test results.

5.1 Site and Pumping Test Layout

Pumping tests were carried out at PW 2-01 between 19 and 22 January 2015 and included:

- A step-test with pumping rates set at 2, 3, 4, 5 and 5.5 L/s. The steps lasted for 60 minutes each.
- Constant Rate Test 1 (CRT 1) - A 26-hour constant rate test with a discharge rate of approximately 5.0 L/s followed by a 5-hour recovery test.
- Constant Rate Test 1 (CRT 2) - A 22-hour constant rate test with a discharge rate of approximately 6.1 L/s followed by a 3.5-hour recovery test.

During CRT 1 reticulation systems from garden bores in adjacent properties and parks were found to affect the groundwater levels in the monitoring wells and VWPs. In addition the groundwater levels reached steady state within a short period of time indicating that the aquifer system could be pumped at a higher pumping rate. The short period of time to reach steady state may also be due to a recharge boundary associated with the nearby drain.

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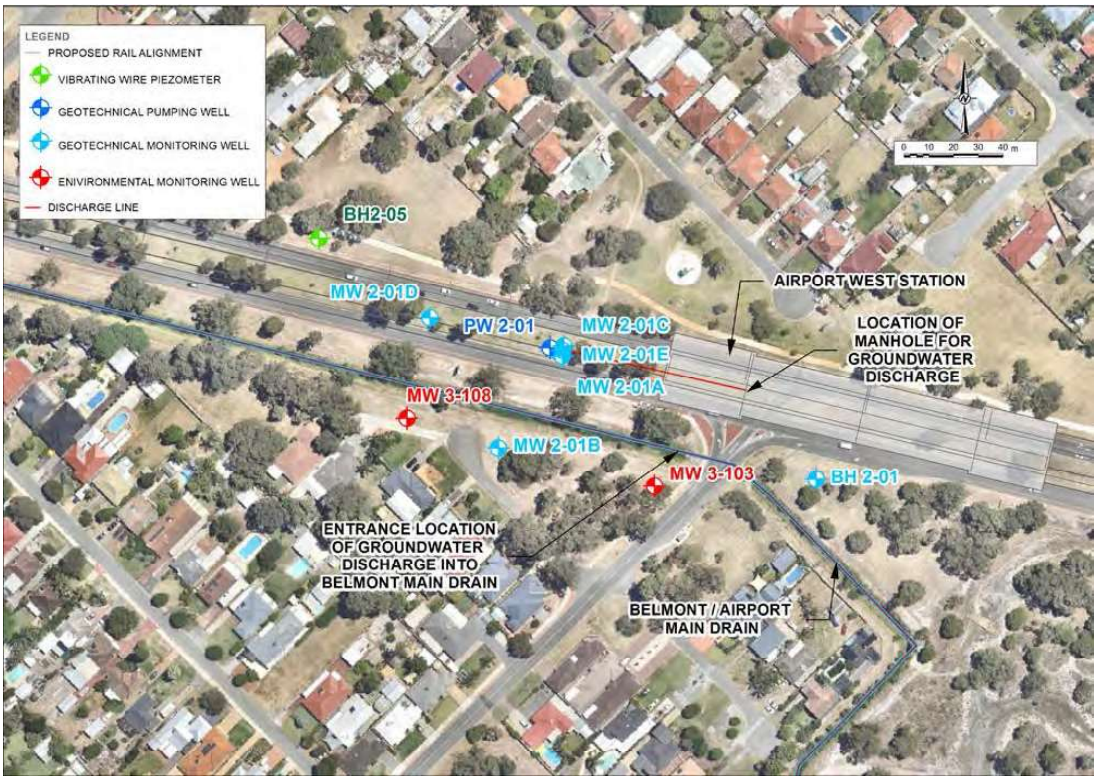
Summary of Test Pumping Analysis

CRT 2 was carried out after it was arranged with the City of Belmont to turn off their reticulation systems and the test was started in the middle of the morning when the household reticulation bores were no longer pumping. A higher pumping rate was chosen based on the results of the first test.

Sketch 3 shows the site layout of the pumping test well (PW1-02), monitoring wells and VWPs while Table 5 presents the distances and directions of the different monitoring locations from the pumping test well.



Sketch 3: Site Layout for pumping test at PW 2-01.

Table 5: Distance and Direction from the Pumping Test Well to the Monitoring Locations

Well ID	Type	Distance to Pumping Well (m)	General Direction to Pumping Well	Screened Geological Unit
PW2-01	Pumping Well	-	-	AF
MW2-01A	Dedicated Monitoring Well	6	East	BS, GF, GS
MW2-01B	Dedicated Monitoring Well	45	South	BS, GF, GS
MW2-01C	Dedicated Monitoring Well	6	East	AF
MW2-01D	Dedicated Monitoring Well	50	West	AF
MW2-01E	Dedicated Monitoring Well	6	East	OFs
BH2-01	Monitoring Well	120	Southeast	AF
MW3-108	Monitoring Well	64	Southwest	AF
MW3-103	Monitoring Well	70	Southeast	BS
BH2-05	Vibrating Wire Piezometer	103	Northwest	GS and AF

5.2 Discharge Location

The abstracted groundwater during the pumping test was discharged into the Belmont Main Drain via a man hole located on the road island where the pumping well was installed (Sketch 3). Approval for the discharge of the groundwater was given by the Water Corporation, the City of Belmont and Swan River Trust on the condition that water quality monitoring and sampling was also undertaken in the Belmont Main Drain. pH and EC was therefore measured in the morning and evening in the Belmont Drain during the CRT at two locations:

- Corner of Brearley Ave and Second Street, approximately 5 m downstream of the discharge point into the drain.
- Corner of Brearley Ave and First Street, approximately 365 m downstream of the discharge point into the drain.

Three water samples were collected at the corner of Brearley Ave and Second Street, at the beginning, halfway through and at the end of the pumping tests.

5.3 Analysis and Results

Figures E3–1 to E3–3 summarise the pumping test results while Table 6 presents the estimated hydraulic properties from the different analysis methods.

Table 6: Results from Pumping Test Analysis – Airport West Station

Method	Transmissivity (m^2/d)	Storage Coefficient	Figures
Distance Drawdown	74–76	6×10^{-3}	E3–4 to E3–5
Groundwater Model	76	2×10^{-4}	E3–6
Analytical Type Curves	39–180 *	-	E3–7 to E3–15 *

* Selected type curves only

The results indicate:

- The drawdown in piezometric level in the pumping well reached approximately 8.5 m and 11 m during Test 1 and Test 2, respectively, and the groundwater level reached steady-state within only one hour of pumping.
- The groundwater and piezometric level drawdown in the monitoring wells ranged between 0.1 m and 4.0 m, depending on the distance of the monitoring well from the pumping well.
- According to the distance drawdown curves the expected groundwater level drawdown in the aquifer just outside the pumping well is estimated to be 7 m and 8 m instead of the observed 8.5 m and 11 m, in the two CRT respectively.
- The monitoring wells, particularly MW3-108 and MW2-01B showed a clear effect from pumping of nearby household reticulation bores, resulting in temporary drawdown of up to 0.5 m in these wells.
- MW2-01E (screened in the underlying Osborne Formation) showed a different response to any of the other monitoring locations with a steady decrease in piezometric pressure throughout the tests. At the end of the tests the piezometric head in the Osborne Formation had been reduced by approximately 2 m. After the pumping ceased only a very small recovery (<0.1 m) occurred suggesting a very low hydraulic conductivity and slow or limited recharge of the Osborne Formation.

- MW3-103 which is shallow (4.5 m deep) and located south of Belmont Main Drain showed no noticeable response to the pumping, suggesting that the Belmont Main Drain acts a positive head boundary to the groundwater table. MW3-108 and MW2-01B, which are also located south of the Belmont Main Drain but installed deeper in the Ascot Formation, showed a reduction in piezometric level of up to approximately 1.2 m, indicating that a piezometric pressure reduction occured below the Belmont Main drain and that the radius of influence extended beyond the drain.
- MW2-01A, screened in the overlying Bassendean Sand, Guildford Formation and Gnangara Sand, shows a similar but reduced response to pumping when compared with MW2-01C. This response indicates some connection with the Ascot Formation, semi-confined conditions and a lower vertical hydraulic conductivity.
- A reasonable match between observed and modelled groundwater level drawdown (Figure E3-6) was obtained in the groundwater model. The Ascot Formation was found to have the greatest hydraulic conductivity. The best match was obtained when using a vertical to horizontal hydraulic conductivity ratio of 0.1 (= k_v/k_h).
- The groundwater model indicates that water inflow from the Belmont Main Drain into the aquifer during the pumping tests could have been up to 0.4 L/s.
- The estimated range in storage coefficient suggests confined conditions.

5.4 Groundwater Quality

Figure E3–16 and E3–17 show the field monitoring results of pH and Electrical Conductivity (EC) from the Constant Rate Test while Table E1 presents the groundwater quality laboratory results from samples taken at the beginning and end of the pumping test. The results indicate:

- pH was generally steady through the pumping test with the following ranges, excluding outliers:
  - 6.9 to 7.4 in PW 2-01 (average of 7.25)
  - 6.7 to 7.4 in Belmont Drain at Second Street (average of 7.19)
  - 7.6 to 7.3 in Belmont Drain at First Street (average of 7.43)
  - pH was within the upper and lower bounds of the Swan River Trust criteria.
  - pH of the discharge water remained within one pH unit of the receiving environment.
- EC was brackish in the both the discharge water and the drain with the following ranges:
  - 1191 to 1339 uS/cm in PW2-01 (average of 1313 uS/cm)
  - 628 to 969 uS/cm in Belmont Drain at Second Street (average of 820 uS/cm)
  - 608 to 955 uS/cm in Belmont Drain at First Street (average of 820 uS/cm)
  - EC was higher in the discharge water than in the drain
  - The EC increased in the drain from around 600 uS/cm prior to discharge to approximately 900 to 950 uS/cm during discharge into the drain
  - The EC decreased at the discharge to pre-discharge concentrations within 0.5 hour of ceasing the discharge into the drain.
- Total Nitrogen ranged between 0.31 mg/L and 0.13 mg/L at the beginning and end of test respectively, which is below the SRT criteria for disposal to Swan River (1 mg/L).

- Total iron remained stable throughout the pumping and ranged between 2.5 mg/L and 2.8 mg/L.
- BTEX, MAH, PAH, hydrocarbons and pesticides were not detected in either of the two samples.

The results indicated that discharge to Belmont Main Drain did not have a detrimental effect on the receiving environment.

6.0 PW 2-02 (BAYSWATER DIVE STRUCTURE)

The results from the Stage 2 Geotechnical Investigation indicated that the Superficial Aquifer at the Bayswater Dive Structure consists of two aquifers (Upper and Lower Aquifer), separated by a 5 m thick impermeable aquitard. The Superficial Aquifer is approximately 25 m thick (including the aquitard) and is underlain by the Kardinya Shale of the Osborne Formation. The Upper Aquifer consists mainly of Bassendean Sand while the Lower Aquifer consists of Perth Formation. The Lower Aquifer is approximately 15 m thick at the Bayswater Dive Structure portal.

The pumping test well was installed and screened across the full thickness of the Lower Aquifer with three of the dedicated monitoring wells (MW2-02A, MW2-02B and MW2-02C) installed in the Lower Aquifer and one monitoring well (MW2-02D) in the Upper Aquifer. Two other environmental monitoring wells (MW3-165 and MW3-166) were installed in the Upper Aquifer while one geotechnical borehole drilled into the Kardinya Shale was converted to a monitoring well (BH2-26).

6.1 Site and Pumping Test Layout

Pumping tests were carried out at PW 2-02 between 2 and 6 February 2015 and included:

- A step test with pumping rates set at 10, 14, 16 and 18 L/s. The first three steps lasted 60 minutes, while the fourth step was stopped after 4 minutes due to excessive drawdown.
- A 32-hour constant rate test with a discharge rate of approximately 12 L/s followed by a 46-hour recovery test.

Sketch 4 shows the site layout of the pumping test well (PW2-02), monitoring wells and VWPs while Table 7 presents the distances and directions of the different monitoring locations from the pumping test well.

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Summary of Test Pumping Analysis



Sketch 4: Site Layout for pumping test at PW 2-02

Table 7: Distance and Direction from the Pumping Test Well to the Monitoring Locations

Well ID	Type	Distance to Pumping Well (m)	General Direction to Pumping Well	Screened Geological Unit	Aquifer
PW2-02	Pumping Well	-	-	PF	Lower
MW2-02A	Dedicated Monitoring Well	4	North	PF	Lower
MW2-02B	Dedicated Monitoring Well	42	Northeast	PF	Lower
MW2-02C	Dedicated Monitoring Well	5	North	PF	Lower
MW2-02D	Dedicated Monitoring Well	4	Northeast	BS	Upper
BH2-26	Monitoring Well	106	Northwest	OFf	Osborne Aquitard
MW3-156	Monitoring Well	77	Southeast	PF	Lower
MW3-165	Monitoring Well	51	Northwest	BS	Upper
MW3-166	Monitoring Well	60	Northwest	BS	Upper
BH0-02	Vibrating Wire Piezometer#	16	Northwest	BS, PF	Upper and Lower
BH2-27	Vibrating Wire Piezometer#	165	Northwest	BS, PF	Upper and Aquitard
BH2-22 *	Vibrating Wire Piezometer#	501	Southeast	PF, OFf	Lower and Osborne Aquitard
BH2-19 *	Vibrating Wire Piezometer#	1054	Southeast	PF, PF	Lower

* Locations of BH2-19 and BH2-22 are not shown on Sketch 1 due to the distance from PW2-02. Refer to Figure 5 in the main report for location of these VWPs. # Each VWP has two sensors installed.

6.2 Discharge Location

Originally the plan was to discharge the pumped groundwater into the Bayswater Main Drain via a stormwater drain located to the south of the site. However, approval could not be obtained from the City of Bayswater and Swan River Trust due to the known groundwater contamination in the area from the CSBP site (located to the northeast of the Bayswater Dive structure). Therefore, arrangements were instead made with the Water Corporation to discharge the pumped water to the sewer (approval to discharge up to 15 L/s to the sewer was granted by the Water Corporation). However, on the day of the connection to the sewer discharge point (manhole), it was found that it had been covered and that it would take several months before access could be obtained.

Approval to infiltrate on site was then sought and obtained from the City of Bayswater and the site owner (Western Power) and the groundwater abstracted during the pumping tests was therefore re-infiltrated on site (Sketch 4). The water was infiltrated into two infiltration basins with discharge being alternated between the basins when they had filled. The capacity of the infiltration basins was found to be insufficient as the pumping test progressed and it was therefore decided to start alternating the discharge between the infiltration basins and a natural low depression south of the main infiltration basin. After 32 hours of pumping it was considered that the pumping test data was sufficient for analysis and the pumping test was therefore terminated due to issues with the available infiltration capacity.

6.3 Analysis and Results

Figures E4–1 to E4–2 summarise the pumping test results while Table 8 presents the estimated hydraulic properties from the different analysis methods.

Table 8: Results from Pumping Test Analysis – Bayswater Dive Structure

Method	Transmissivity (m^2/d)	Storage Coefficient	Figures
Distance Drawdown	120	7.0×10^{-5}	E4–3
Groundwater Model	60–113	1.5×10^{-5} - 7.5×10^{-5}	E4–4 and E4–5
Analytical Type Curves	21–300 *	-	E4–6 to E4–13 *

* Selected type curves only

The results indicate:

- The drawdown in piezometric level in the pumping well reached almost 14 m toward the end of the test and the piezometric level almost reached steady-state conditions at the end of the pumping test. The drawdown in the monitoring wells and VWPs ranged between 0.1 m and 9.2 m.
- There was no groundwater level drawdown response in the Upper Aquifer due to the pumping of the Lower Aquifer. Instead groundwater level rose in some of the wells due to the infiltration of the water back into the Upper Aquifer. This indicates that there is no noticeable connection between the two aquifers.
- BH2-26 (screened in the Osborne Formation), showed a small delayed response to the pumping, indicating limited connection of the Lower Aquifer with the Osborne Formation.
- The recovery of the piezometric level after pumping ceased took a much longer time than for the other pumping tests described in this Appendix, suggesting that there is limited direct recharge to the Lower Aquifer.
- The distance drawdown curve (Figure E4–3) indicates that there is a medium correlation between distance (from the pumping well) and drawdown, suggesting heterogeneous conditions. MW2-02B falls far from the curve, which is also the well that showed a significantly lower hydraulic conductivity during slug testing than the other wells.

- The groundwater model was not able to provide a good match for all observed piezometric levels by using a homogenous aquifer approach. The closer wells (MW2-01C and BH0-02) showed a better match with a higher transmissivity, while the wells and VWPs located further away (BH2-19, BH2-22 and MW2-02B) showed a better match with a lower transmissivity (E4–4 and E4–5). There was found to be only a small difference in the calculated values when varying the vertical hydraulic conductivity in the model, which is likely due to the wells being screened over the full aquifer thickness.
- The type curve analysis provided larger transmissivity for the pumping curves and lower transmissivity for the recovery curves, again indicating slow or limited recharge to the Lower Aquifer. MW2-02C reached steady-state more quickly than any other well, suggesting that the hydraulic conductivity could be higher in the upper part of the Lower Aquifer.

6.4 Groundwater Quality

Figure E4–14 shows the field monitoring results of pH and Electrical Conductivity (EC) from the Constant Rate Test while Table E1 presents the groundwater quality laboratory results from samples taken at the beginning and end of the pumping test. The results indicate:

- pH decreased from 7.5 to 7.0 within the first 400 minutes of the pumping test after which it remained stable at about 7.0, suggesting neutral conditions.
- EC ranged between 500 and 700 uS/cm during the test indicating fresh water. There was a slight increasing trend from the beginning of the test to 1600 minutes into the test, after which EC decreased again.
- Total Nitrogen was 0.33 mg/L and 0.49 mg/L at the beginning and end of test, respectively, which is below the SRT criterion for disposal to Swan River (1 mg/L).
- Total iron increased from 3 mg/L at the beginning of the test to 22 mg/L at the end of the test.
- BTEX, MAH, PAH, hydrocarbons and pesticides were not detected in either of the collected samples.

26/05/2015

TABLE E1 - SUMMARY OF WATER QUALITY RESULTS

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			Area	Forrestfield		Airport West									Consolidated		Bayswater Dive Structure		
			Well ID	PW1-01	PW1-01	PW2-01	SW08	PW2-01	Drain	PW2-01	Drain	PW2-01	PW2-01 DUP	Drain	PW1-02	PW1-02	MW2-02A	MW2-02C	PW2-02
			Test Time	Beginning	End	Before		Beginning		Mid		End			Beginning	End	Before		End
			Sample Date	8/12/2014	11/12/2014	5/12/2014		19/1/2015		21/1/2015		22/1/2015			6/1/2015	8/1/2015	5/12/2014		4/2/2015
			Matrix	Groundwater		Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater		Water	Groundwater		Groundwater		
pH	pH Units	6.0-8.5	0	6.3	6.5	9.1	8.0	7.9	7.7	7.7	7.7	7.8		7.6	8.0	8.1	7.1	7.4	7.0
Total Dissolved Solids Dried at 175-185°C	mg/L		10	540	520	770	360	720	390	740	540	740		460	410	420	250	380	420
Conductivity @ 25 C	µS/cm		2	930	930			1300	670	1300	920	1300		660	720	750			670
Salinity	mg/L		2	610	610			860	440	860	600	850		430	470	490			440
Total Suspended Solids Dried at 103-105°C	mg/L		5			7	<5										<5	9	
Turbidity	NTU		0.5			5.7	3.5										4.1	15	
Hydroxide Alkalinity as OH	mg/L		5	<5	<5	<5	<5	<5	<5	<5	<5	<5		<5	<5	<5	<5	<5	<5
Carbonate Alkalinity as CO3	mg/L		1	<1	<1	57	<1	<1	<1	<1	<1	<1		<1	<1	<1	<1	<1	<1
Bicarbonate Alkalinity as HCO3	mg/L		5	25	22	170	110	340	110	330	200	330		110	180	180	100	150	93
Total Alkalinity as CaCO3	mg/L		5	21	18	240	90	280	88	270	160	270		90	150	150	84	120	76
Acidity to pH 8.3	mg CaCO3/L	>40	5	39	33	<5	<5	16	10	18	10	17		9	7	<5	16	19	50
Chloride, Cl	mg/L		1	130	120	260	130	260	140	260	180	260		140	110	120	91	110	92
Sulphate, SO4	mg/L		1	69	65	24	30	<1	18	<1	20	<1		29	58	58	6	20	110
Sulphide	mg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		0	<0.1	<0.1			<0.1
Sodium, Na	mg/L		0.5	110	100	150	67	140	76	130	92	140		63	67	66	55	110	59
Calcium, Ca	mg/L		0.2	10.0	10.0	95	28	94.0	29	86.0	51	87.0		27	57.0	53.0	14	8.7	30.0
Magnesium, Mg	mg/L		0.1	28.0	28.0	11	12	18.0	14	16.0	14	16.0		12	11.0	12.0	7.5	5.5	14.0
Potassium, K	mg/L		0.1	4.7	4.7	9.7	5.0	6.1	4.8	5.5	5.0	6.3		4.9	4.0	4.0	4.3	3.5	5.5
Total Hardness by Calculation	mg CaCO3/L		1	140	140	280	120	310	130	280	180	290		120	190	180	66	45	130
Total Organic Carbon	mg/L		0.2			21	24										4.1	7.2	
Total Cyanide	mg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	0.1	<0.004			<0.004
Thiocyanate	mg/L		0.1	<0.1	<0.1			<0.1	0	0.4	1	0.3		1	<0.1	<0.1			<0.1
Chlorophyll a	mg/L		0.0005			<0.0005	0.0010										0.0031	<0.0005	
NUTRIENTS																			
Nitrite, NO_2 as NO_2	mg/L		0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Nitrate, NO_3 as NO_3	mg/L		0.2	190	150	<0.2	0.4	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Nitrate/Nitrite Nitrogen, NOx as N	v		0.05	44	33			<0.05	0.1	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05			<0.05
Ammonia, NH_3	mg/L		0.05	0.07	0.25	0.17	<0.05	0.52	<0.05	0.54	0.19	0.55	0.54	<0.05	<0.05	0.05	0.15	0.13	0.32
Total Kjeldahl Nitrogen	mg/L		0.05	<0.05	<0.05	0.82	0.85	0.70	<0.05	0.76	0.76	0.77	0.82	0.89	0.13	0.10	0.30	0.33	0.47
Total Nitrogen (calc)	mg/L	1	0.05	44.00	33.00	0.83	0.94	0.72	0.06	0.77	0.76	0.78	0.82	0.89	0.16	0.10	0.30	0.33	0.49
Total Phosphorus (Kjeldahl Digestion)	mg/L	0.1	0.01	0.09	0.07	1.10	<0.01	0.31	<0.01	0.12	0.06	0.13	0.12	0.01	0.20	0.13	0.76	0.12	0.07
Filterable Reactive Phosphorus	mg/L		0.00	0.09	0.06			0.04	0.01	0.09	0.05	0.09	0.09	0.004	0.08	0.07			<0.002
DISSOLVED METALS																			
Aluminium, Al	mg/L	0.15	0.02	0.01	<0.005	<0.02	0.10	0.01	0.12	<0.005	0.07	<0.005		0.08	0.04	<0.005	<0.02	0.03	<0.005
Arsenic, As	mg/L		0.02	<0.001	<0.001	<0.02	<0.02	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	0.002	0.002	<0.02	<0.02	0.003
Boron, B	mg/L		0.05	0.05	0.06	0.09	0.07	0.03	0.03	0.02	0.02	0.03		0.04	0.02	0.02	<0.05	0.07	0.02
Cadmium, Cd	mg/L		0.001	<0.0001	<0.0001	<0.001	<0.001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001		<0.0001	<0.0001	<0.0001	<0.001	<0.001	<0.0001
Chromium, Cr	mg/L		0.005	<0.001	<0.001	<0.005	<0.005	0.002	0.001	0.002	0.002	0.002		<0.001	<0.001	<0.001	<0.005	<0.005	<0.001
Copper, Cu	mg/L		0.005	0.001	0.008	<0.005	<0.005	<0.001	0.001	<0.001	<0.001	<0.001		0.001	<0.001	<0.001	<0.005	<0.005	<0.001
Iron, Fe	mg/L		0.02	0.02	0.03	<0.02	0.23	2.10	0.33	2.70	1.10	1.10		0.18	0.13	0.51	2.90	0.25	20.00
Manganese, Mn	mg/L		0.005	0.031	0.002	0.026	0.007	0.093	0.007	0.028	0.014	0.027		<0.001	0.038	0.034	0.44	0.094	0.064
Molybdenum, Mo	mg/L		0.01	<0.001	<0.001	<0.01	<0.01	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.01	<0.01	<0.001
Lead, Pb	mg/L		0.02	0.001	0.002	<0.02	<0.02	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.02	<0.02	<0.001
Nickel, Ni	mg/L		0.005	0.001	<0.001	<0.005	<0.005	<0.001	0.001	<0.001	<0.001	<0.001		0.002	<0.001	<0.001	<0.005	<0.005	0.001
Selenium, Se	mg/L		0.05	<0.002	<0.002	<0.05	<0.05	<0.002	<0.002	<0.002	<0.002	<0.002		<0.002	<0.002	<0.002	<0.05	<0.05	<0.002
Tin, Sn	mg/L		0.05	<0.001	<0.001	<0.05	<0.05	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.05	<0.05	<0.001
Zinc, Zn	mg/L		0.01	0.009	0.065	<0.01	0.01	<0.005	<0.005	<0.005	0.01	<0.005		<0.005	<0.005	<0.005	<0.01	<0.01	0.008
Mercury	mg/L		0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005		<0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005
TOTAL METALS																			
Total Aluminium	mg/L	1	0.02	0.08	0.01	0.18	0.27	0.26	0.13	0.02	0.09	0.01		0.11	0.08	0.02	0.03	1.50	0.01
Total Arsenic	mg/L		0.02	<0.001	<0.001	<0.02	<0.02	0.001	<0.001	<0.001	<0.001	<0.001		<0.001	0.002	0.002	<0.02	<0.02	0.003
Total Boron	mg/L		0.05	0.07	0.08	0.09	0.07	0.03	0.03	0.03	0.04	0.03		0.04	0.03	0.03	<0.05	0.07	0.03
Total Cadmium	mg/L		0.001	<0.0001	<0.0001	<0.001	<0.001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001		<0.0001	<0.0001	<0.0001	<0.001	<0.001	<0.0001
Total Chromium	mg/L		0.005	<0.001	<0.001	0.008	<0.005	0.007	0.001	0.003	0.002	0.002		0.001	<0.001	<0.001	<0.005	<0.005	<0.001
Total Copper	mg/L		0.005	0.002	0.003	<0.005	<0.005	<0.001	0.001	<0.001	<0.001	<0.001		0.001	<0.001	<0.001	<0.005	0.006	<0.001
Ferrous Iron, Fe2+	mg/L		0.05	<0.05	<0.05			2.50	0.37	2.70	0.99	2.80		0.33	0.51	0.49			22.00
Ferric Iron, Fe3+	mg/L		0.05	<0.05	0.018			<0.05	<0.05	<0.05	0.32	<0.05		<0.05	0.06	0.38			1.00
Total Iron	mg/L	1	0.02	0.03	0.02	0.10	0.43	2.50	0.41	2.70	1.30	2.80		0.33	0.57	0.87	3.00	1.20	22.00
Total Manganese	mg/L		0.005	0.033	0.002	0.032	0.008	0.096	0.008	0.028	0.016	0.028		0.008	0.042	0.037	0.46	0.099	0.071
Total Molybdenum	mg/L		0.01	<0.001	<0.001	<0.01	<0.01	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.01	<0.01	<0.001
Total Lead	mg/L		0.02	0.002	<0.001	<0.02	<0.02	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.02	<0.02	<0.001
Total Nickel	mg/L		0.005	0.001	<0.001	0.005	<0.005	0.001	0.001	<0.001	<0.001	<0.001		0.002	0.001	<0.001	<0.005	<0.005	0.001
Total Selenium	mg/L		0.05	<0.002	<0.002	<0.05	<0.05	<0.002	<0.002	<0.002	<0.002	<0.002		<0.002	<0.002	<0.002	<0.05	<0.05	<0.002
Total Tin	mg/L		0.05	<0.001	<0.001	<0.05	<0.05	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.05	<0.05	<0.001
Total Zinc	mg/L		0.01	0.012	0.008	<0.01	<0.01	<0.005	0.007	<0.005	0.01	<0.005		0.009	<0.005	0.029	<0.01	<0.01	0.011
Total Mercury	mg/L		0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001		<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001

TABLE E1 - SUMMARY OF WATER QUALITY RESULTS

			Area	Forrestfield		Airport West									Consolidated		Bayswater Dive Structure		
			Well ID	PW1-01	PW1-01	PW2-01	SW08	PW2-01	Drain	PW2-01	Drain	PW2-01	PW2-01 DUP	Drain	PW1-02	PW1-02	MW2-02A	MW2-02C	PW2-02
			Test Time	Beginning	End	Before		Beginning		Mid		End			Beginning	End	Before		End
			Sample Date	8/12/2014	11/12/2014	5/12/2014		19/1/2015		21/1/2015		22/1/2015			6/1/2015	8/1/2015	5/12/2014		4/2/2015
			Matrix	Groundwater		Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater		Water	Groundwater		Groundwater		
HYDROCARBONS																			
Benzene (F0)	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
TRH C6-C9	µg/L		40	<40	<40	71	<40	<40	<40	<40	<40	<40		<40	<40	<40	<40	<40	<40
TRH C6-C10	µg/L		50	<50	<50			<50	<50	<50	<50	<50		<50	<50				<50
TRH C6-C10 minus BTEX (F1)	µg/L		50	<50	<50	79	<50	<50	<50	<50	<50	<50		<50	<50	<50	<50	<50	<50
TRH C10-C14	µg/L		50	55	<50	<50	<50	<50	<50	<50	<50	<50		<50	<50	<50	<50	<50	<50
TRH C15-C28	µg/L		200	<200	<200	<200	<200	<200	<200	<200	<200	<200		<200	<200	<200	<200	<200	<200
TRH C29-C36	µg/L		200	<200	<200	<200	<200	<200	<200	<200	<200	<200		<200	<200	<200	<200	<200	<200
TRH >C10-C16 (F2)	µg/L		60	<60	<60	<60	<60	<60	<60	<60	<60	<60		<60	<60	<60	<60	<60	<60
TRH >C16-C34 (F3)	µg/L		500	<500	<500	<500	<500	<500	<500	<500	<500	<500		<500	<500	<500	<500	<500	<500
TRH >C34-C40 (F4)	µg/L		500	<500	<500	<500	<500	<500	<500	<500	<500	<500		<500	<500	<500	<500	<500	<500
VOLATILES																			
Benzene	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Toluene	µg/L		0.5	<0.5	<0.5	1.3	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Ethylbenzene	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
m/p-xylene	µg/L		1	<1	<1	<1	<1	<1	<1	<1	<1	<1		<1	<1	<1	<1	<1	<1
o-xylene	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Total Xylenes			1.5	<1.5	<1.5			<1.5	<1.5	<1.5	<1.5	<1.5		<1.5	<1.5	<1.5			<1.5
Styrene (Vinyl benzene)			0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Naphthalene	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
MtBE (Methyl-tert-butyl ether)				<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
PESTICIDES																			
Hexachlorobenzene (HCB)	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Alpha BHC	µg/L			<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
Lindane (gamma BHC)	µg/L		0.002	<0.05	<0.05	<0.002	<0.002	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.002	<0.002	<0.05
Beta BHC	µg/L			<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
Heptachlor	µg/L		0.002	<0.02	<0.02	<0.002	<0.002	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.002	<0.002	<0.02
Delta BHC	µg/L			<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
Aldrin	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Isodrin	µg/L			<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			<0.02
Heptachlor epoxide	µg/L		0.002	<0.02	<0.02	<0.002	<0.002	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.002	<0.002	<0.02
Gamma Chlordane	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Alpha Chlordane	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Alpha Endosulfan	µg/L		0.005	<0.02	<0.02	<0.005	<0.005	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.005	<0.005	<0.02
p,p'-DDE	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Dieldrin	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Endrin	µg/L		0.004	<0.02	<0.02	<0.004	<0.004	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.004	<0.004	<0.02
p,p'-DDD	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Beta Endosulfan	µg/L		0.005	<0.02	<0.02	<0.005	<0.005	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.005	<0.005	<0.02
p,p'-DDT	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Endrin Aldehyde	µg/L			<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			<0.02
Endosulfan sulphate	µg/L		0.005	<0.02	<0.02	<0.005	<0.005	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.005	<0.005	<0.02
Methoxychlor	µg/L		0.100	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
Mirex	µg/L		0.010	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Endrin Ketone	µg/L		0.010	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Oxychlordane	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Azinphos-methyl (Guthion)	µg/L		0.050	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Bromophos Ethyl	µg/L		0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Chlorpyrifos (Chlorpyrifos Ethyl)	µg/L		0.009	<0.01	<0.01	<0.009	<0.009	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.009	<0.009	<0.01
Diazinon (Dimpylate)	µg/L		0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Dichlorvos	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Dimethoate	µg/L		0.1	<0.15	<0.15	<0.1	<0.1	<0.15	<0.15	<0.15	<0.15	<0.15		<0.15	<0.15	<0.15	<0.1	<0.1	<0.15
Ethion	µg/L		0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Fenitrothion	µg/L		0.1	<0.2	<0.2	<0.1	<0.1	<0.2	<0.2	<0.2	<0.2	<0.2		<0.2	<0.2	<0.2	<0.1	<0.1	<0.2
Malathion	µg/L		0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Methidathion	µg/L		0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Parathion-ethyl (Parathion)	µg/L		0.004	<0.01	<0.01	<0.004	<0.004	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.004	<0.004	<0.01
PCBs																			
PCB Congener C28	µg/L		0.02	<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			<0.02
PCB Congener C52	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
PCB Congener C101	µg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	<0.004	<0.004			<0.004
PCB Congener C118	µg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	<0.004	<0.004			<0.004
PCB Congener C153	µg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	<0.004	<0.004			<0.004
PCB Congener C138	µg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	<0.004	<0.004			<0.004
PCB Congener C180	µg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	<0.004	<0.004			<0.004

City of Kalamunda

1005

Ordinary Council Meeting 12 October 2021 Attachments

Attachment 10.5.2.3

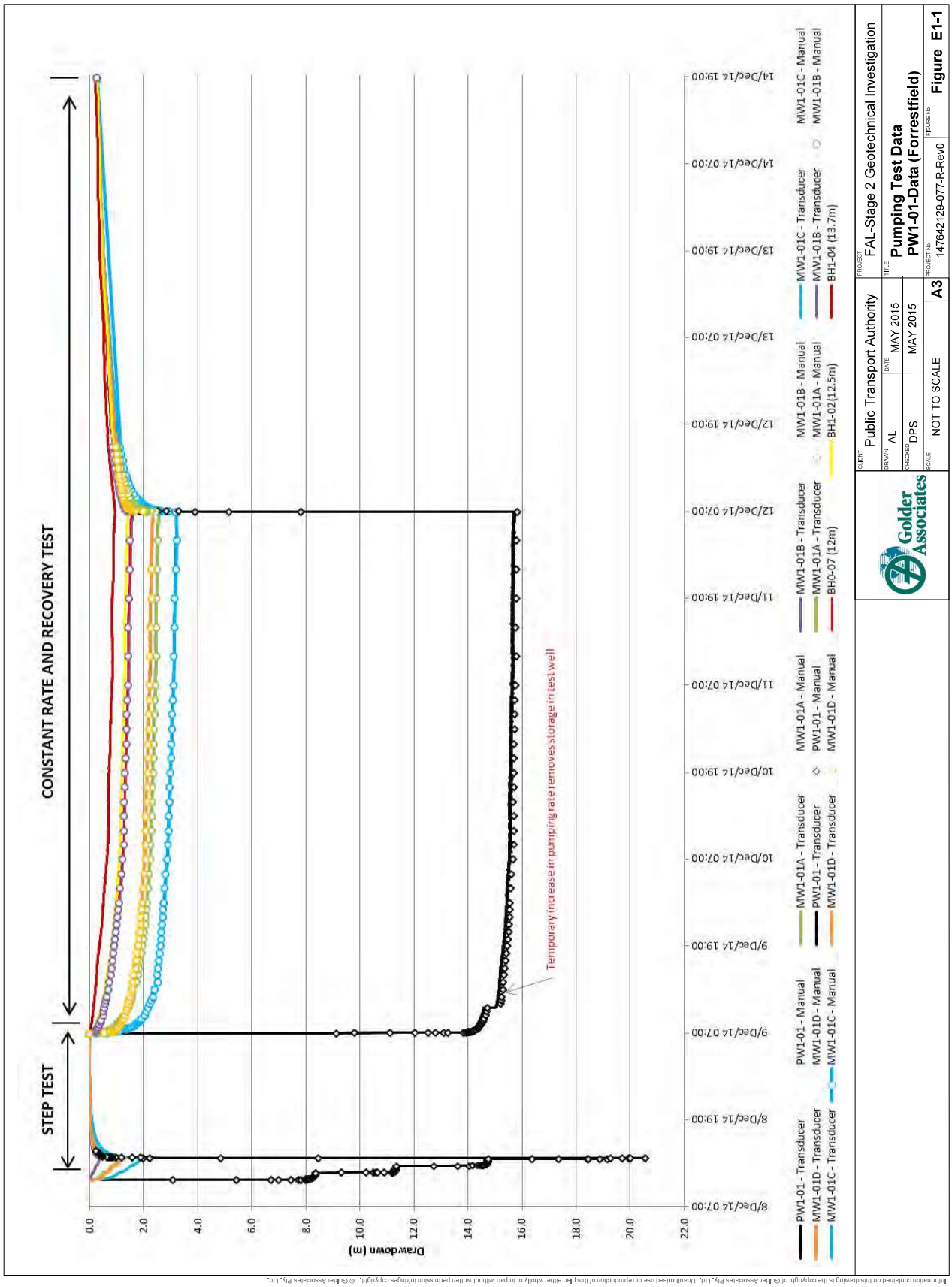
TABLE E1 - SUMMARY OF WATER QUALITY RESULTS

			Area	Forrestfield		Airport West									Consolidated		Bayswater Dive Structure		
			Well ID	PW1-01	PW1-01	PW2-01	SW08	PW2-01	Drain	PW2-01	Drain	PW2-01	PW2-01 DUP	Drain	PW1-02	PW1-02	MW2-02A	MW2-02C	PW2-02
			Test Time	Beginning	End	Before		Beginning		Mid		End			Beginning	End	Before		End
			Sample Date	8/12/2014	11/12/2014	5/12/2014		19/1/2015		21/1/2015		22/1/2015			6/1/2015	8/1/2015	5/12/2014		4/2/2015
			Matrix	Groundwater		Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater		Water	Groundwater		Groundwater		
PAH																			
Naphthalene	µg/L		0.02	<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			0.03
2-methylnaphthalene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
1-methylnaphthalene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Acenaphthylene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Acenaphthene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Fluorene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Phenanthrene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Anthracene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Fluoranthene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Pyrene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(a)anthracene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Chrysene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(k)fluoranthene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(b&j)fluoranthene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(a)pyrene	µg/L		0.005	<0.005	<0.005			<0.005	<0.005	<0.005	<0.005	<0.005		<0.005	<0.005	<0.005			<0.005
Indeno(1,2,3-cd)pyrene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Dibenzo(a&h)anthracene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(ghi)perylene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(b&j&k)fluoranthene	µg/L		0.02	<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			<0.02
Total PAH (18)	µg/L		0.1					<0.1	<0.1	<0.1	<0.1	<0.1		<0.1					<0.1
Phenols																			
4-chloro-3-methylphenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
2-chlorophenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
2,4-dichlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
2,6-dichlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Pentachlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
2,5-dichlorophenol	µg/L		0.05	<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
2,3,4,5-tetrachlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
2,3,4,6/2,3,5,6-tetrachlorophenol	µg/L		0.02	<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			<0.02
2,4,5-trichlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
2,4,6-trichlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Phenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	0.1	<0.1	<0.1		<0.1	0.1	<0.1			0.3
2-methyl phenol (o-cresol)	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
3/4-methyl phenol (m/p-cresol)	µg/L		0.2	<0.2	<0.2			<0.2	<0.2	<0.2	<0.2	<0.2		<0.2	<0.2	<0.2			<0.2
2,4-dimethylphenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
2,4-dinitrophenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
2-methyl-4,6-dinitrophenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
2-nitrophenol	µg/L		0.05	<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
4-nitrophenol	µg/L		0.05	<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
Dinex (2-cyclohexyl-4,6-dinitrophenol)	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
Dinoseb	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
Carbamates																			
Carbofuran	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Carbaryl	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Triazines																			
Simazine	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Atrazine	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Propazine	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Terbuthylazine	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Metribuzin	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Prometryn	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Terbutryn	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Cyanazine	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Hexazinone	µg/L		1	<1	<1			<1	<1	<1	<1	<1		<1	<1	<1			<1
E.coli & Coliforms																			
E. coli	CFU/100mL		1	<1	<1			<1	490	<1	170	<1		500	<1	<1			<1
Thermotolerant Coliforms	CFU/100mL		1	<1	<1			<1	720	<1	250	<1		500	<1	<1			<1



APPENDIX E
Summary of Test Pumping Analysis

ATTACHMENT E1
Constant Rate Test – Forrestfield Dive Structure (PW1-01)



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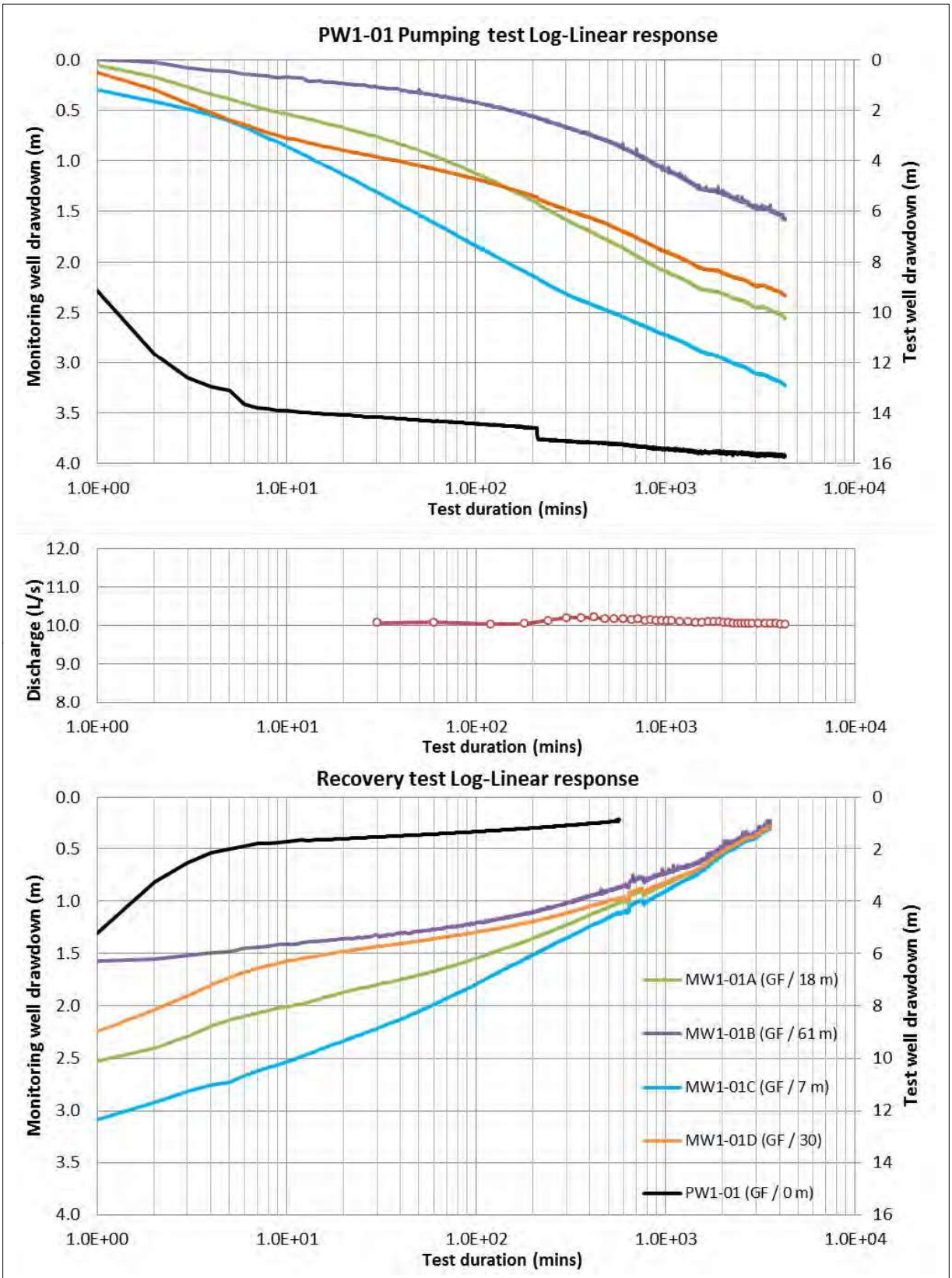

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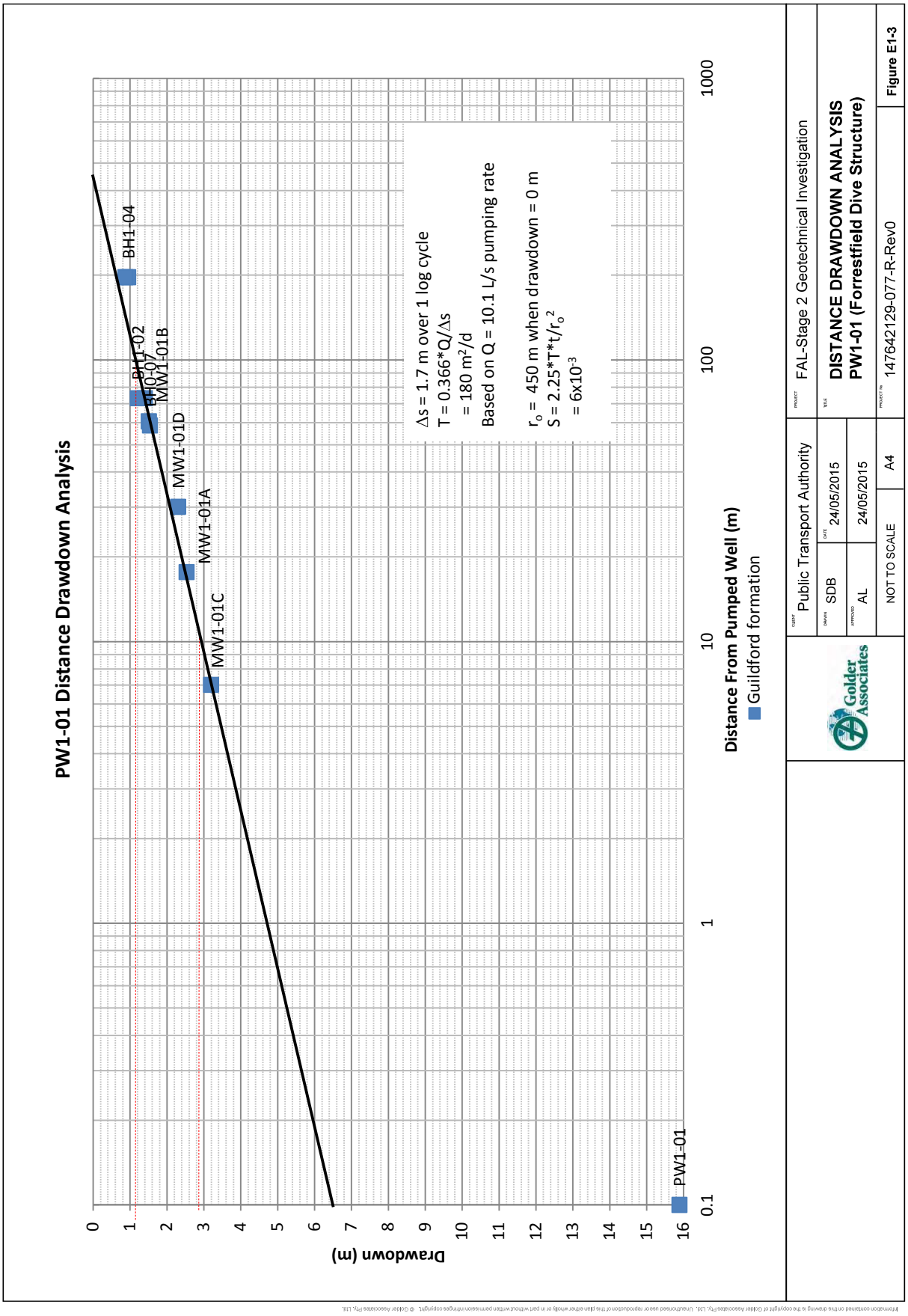
Golder Associates
Level 3, 1 Havelock St West Perth WA 6005
Ph: +618 9213 7600 Fx: +618 9213 7611

CLIENT	Public Transport Authority	PROJECT	FAL-Stage 2 Geotechnical Investigation
DRAWN	SDB	DATE 23/05/2015	TITLE **Pumping Test Data PW1-01 (Forrestfield Dive Structure)**
CHECKED	AL	DATE 23/05/2015	
SCALE	NOT TO SCALE	A4	PROJECT No 147642129-077-R-Rev0
FIGURE No	**Figure E1-2**		



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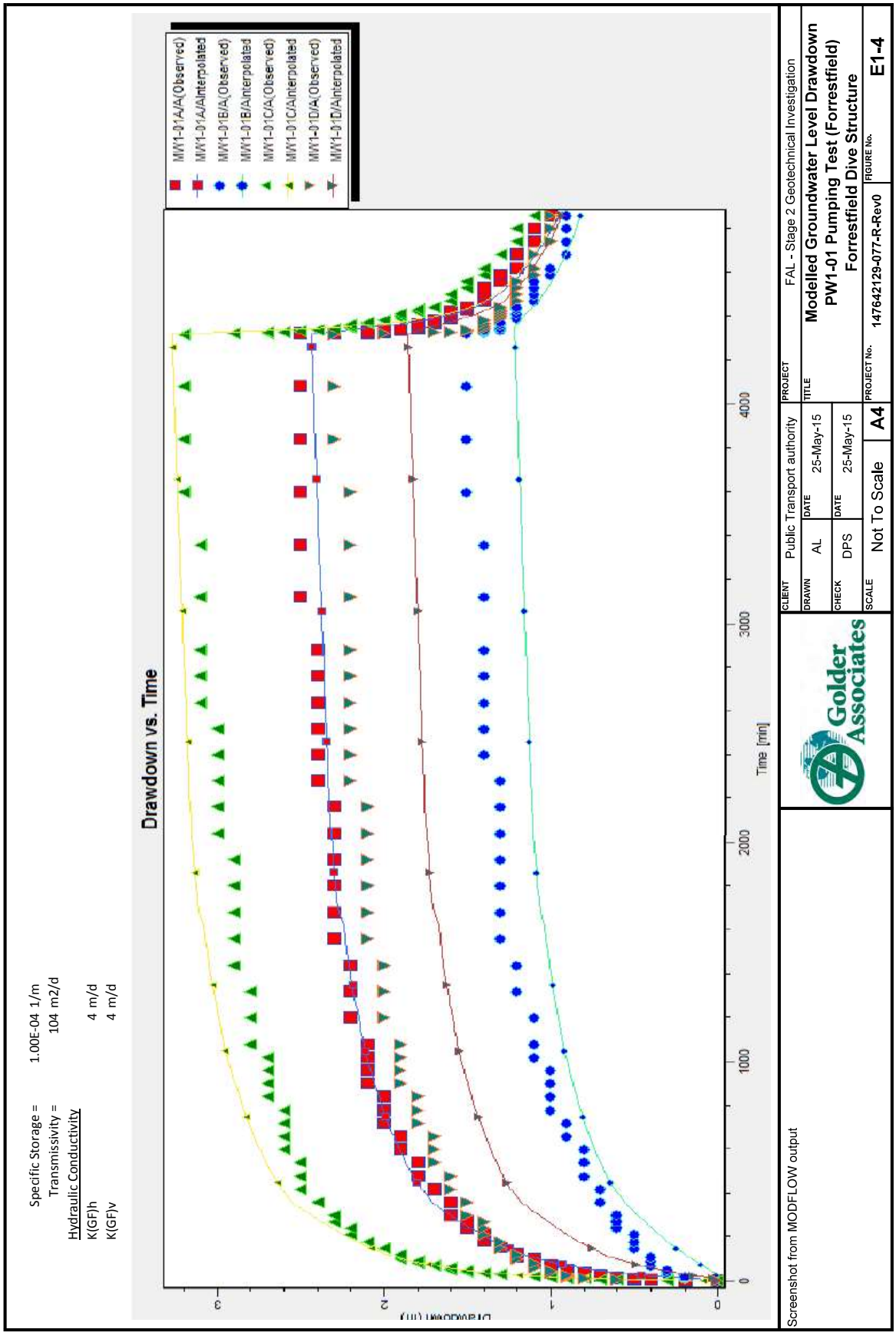
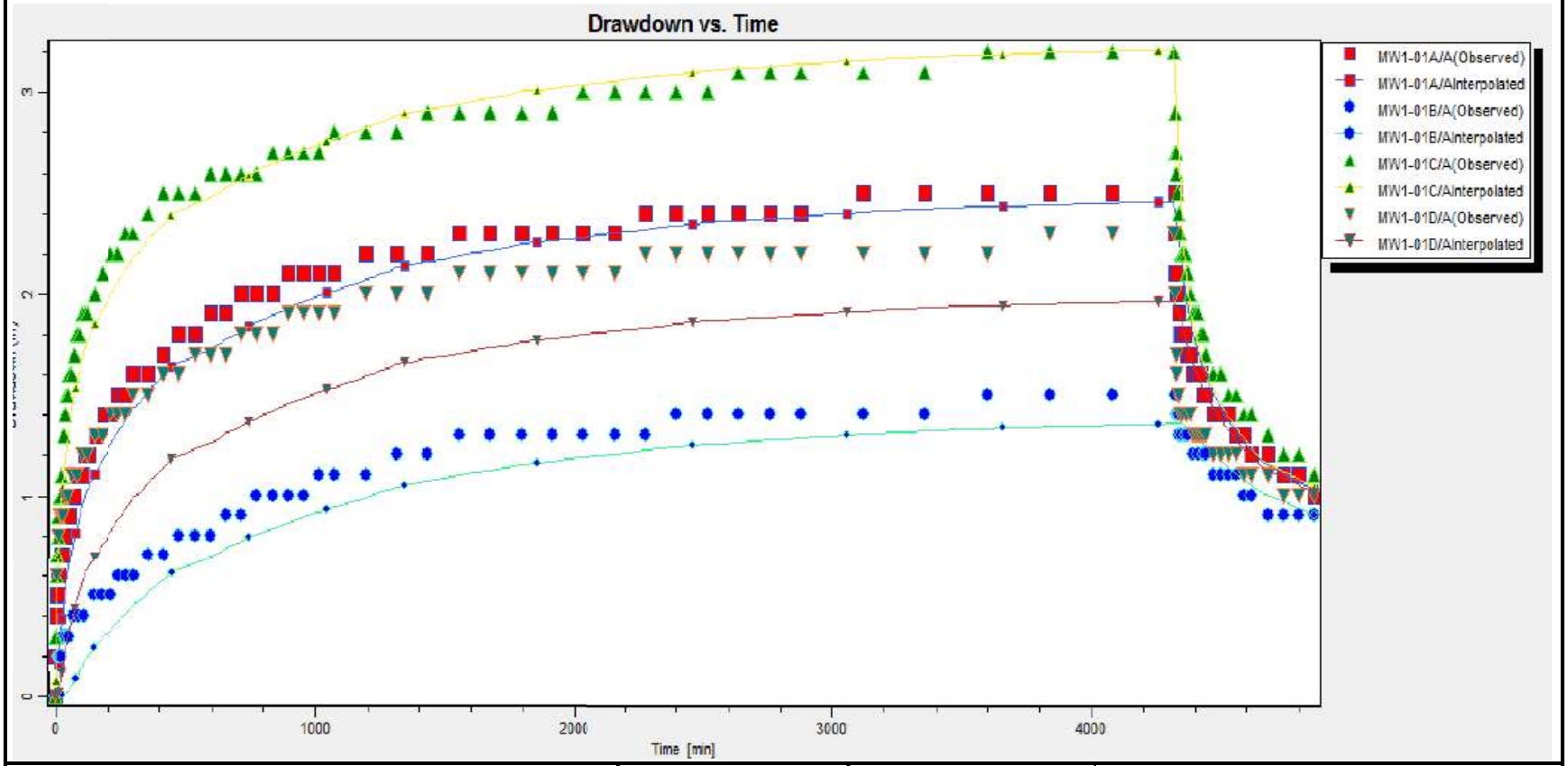

Drawdown vs. Time
Time [min]
MW1-01A/A(Observed)
MW1-01A/A(Interpolated)
MW1-01E/A(Observed)
MW1-01E/A(Interpolated)
MW1-01C/A(Observed)
MW1-01C/A(Interpolated)
MW1-01D/A(Observed)
MW1-01D/A(Interpolated)
Specific Storage = 1.00E-04 1/m
Transmissivity = 104 m2/d
Hydraulic Conductivity
K(GF)h 4 m/d
K(GF)v 4 m/d
Screenshot from MODFLOW output
CLIENT Public Transport authority
DRAWN AL
DATE 25-May-15
CHECK DPS
DATE 25-May-15
SCALE Not To Scale
A4
PROJECT FAL - Stage 2 Geotechnical Investigation
TITLE Modelled Groundwater Level Drawdown PW1-01 Pumping Test (Forrestfield) Forrestfield Dive Structure
PROJECT No. 147642129-077-R-Rev0
FIGURE No. E1-4
J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Appendix E - Test Pumping Results\Attachment E1\Figure E1-4&5.xlsx

Specific Storage =	1.00E-04	1/m
Transmissivity =	104	m2/d
Hydraulic Conductivity		
K(GF)h	4	m/d
K(GF)v	1	m/d



Screenshot from MODFLOW output

Golder Associates

CLIENT	Public Transport authority	PROJECT	FAL - Stage 2 Geotechnical Investigation
DRAWN	AL	DATE	25-May-15
CHECK	DPS	DATE	25-May-15
TITLE	Modelled Groundwater Level Drawdown PW1-01 Pumping Test (Forrestfield) Forrestfield Dive Structure		
SCALE	Not To Scale	A4	
PROJECT No.	147642129-077-R-Rev0	FIGURE No.	E1-5

J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Appendix E - Test Pumping Results\Attachment E1\Figure E1-4&5.xlsx

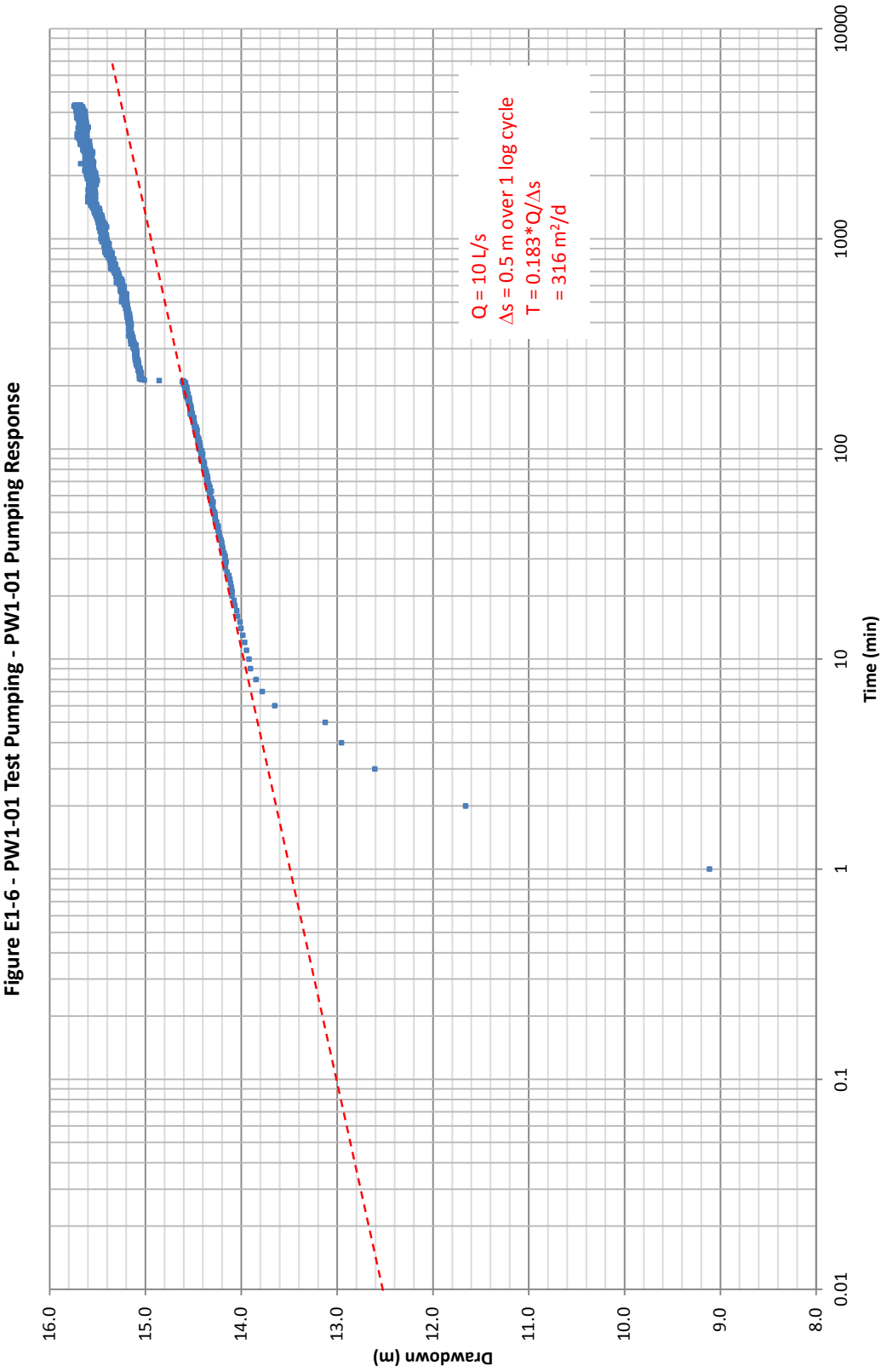


Figure E1-6 - PW1-01 Test Pumping - PW1-01 Pumping Response
Drawdown (m)
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 10 L/s
Δs = 0.5 m over 1 log cycle
T = 0.183*Q/Δs
= 316 m²/d

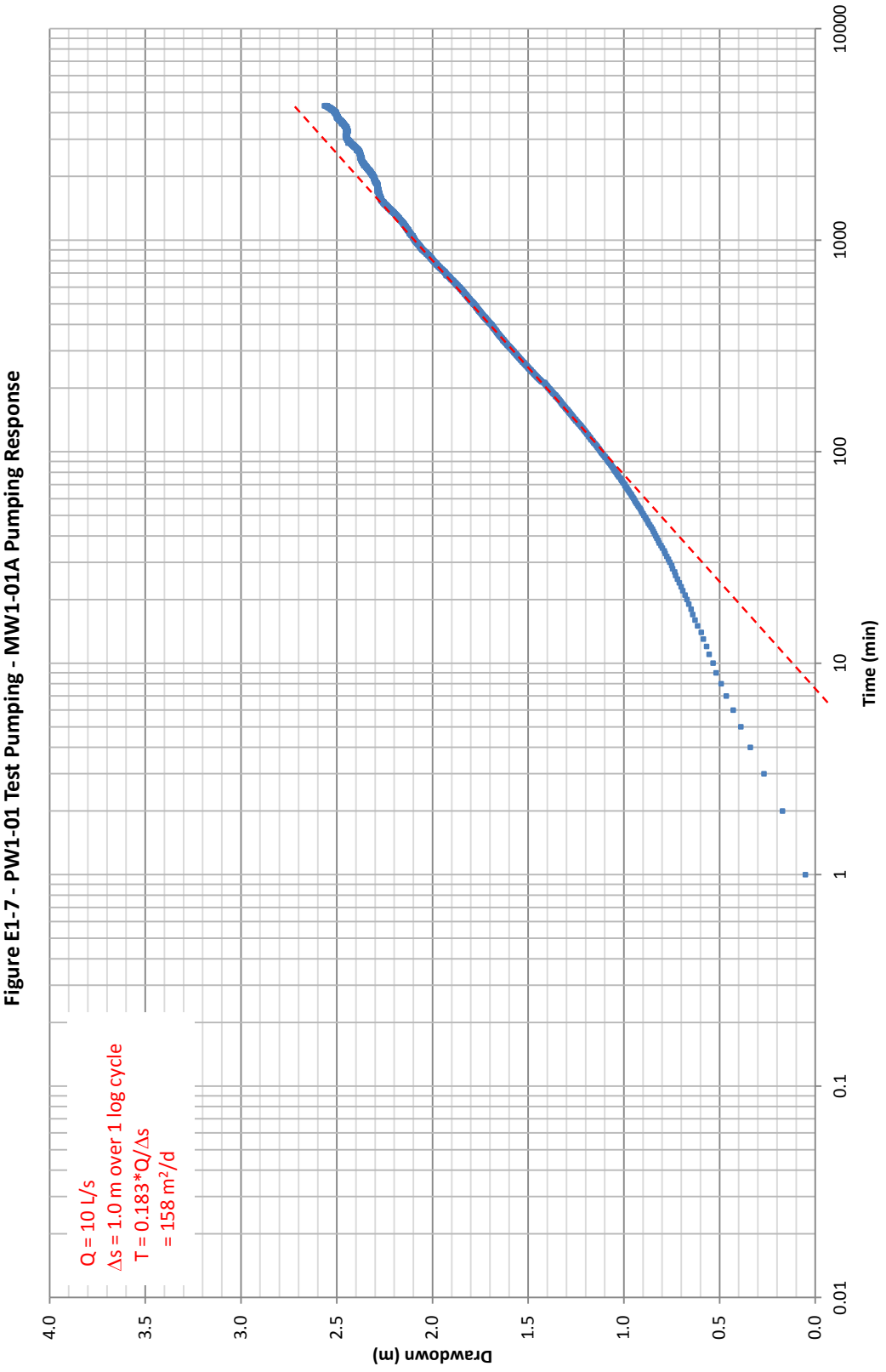


Figure E1-7 - PW1-01 Test Pumping - MW1-01A Pumping Response
Drawdown (m)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 10 L/s
Δs = 1.0 m over 1 log cycle
T = 0.183*Q/Δs
= 158 m²/d

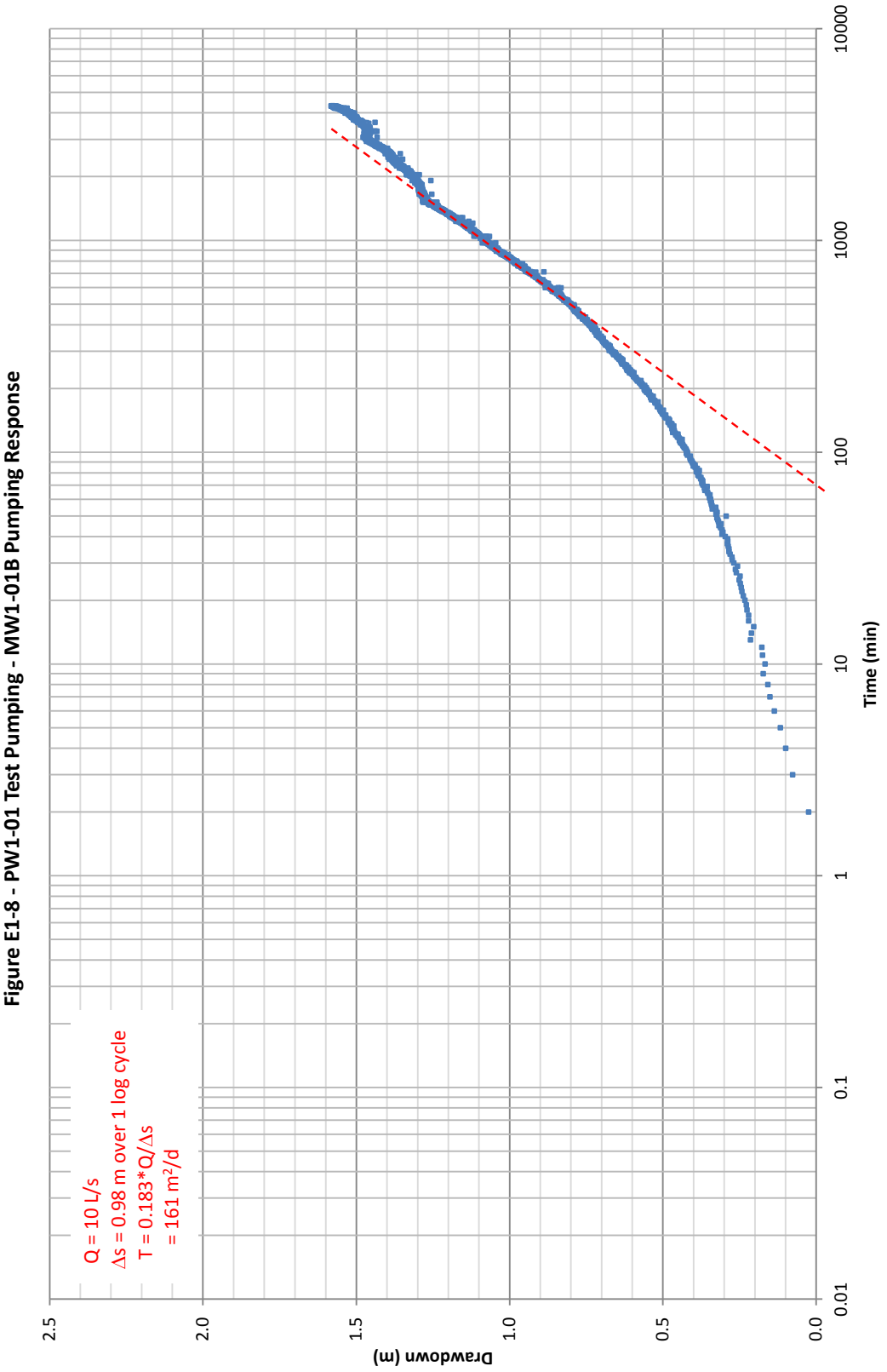


Figure E1-8 - PW1-01 Test Pumping - MW1-01B Pumping Response

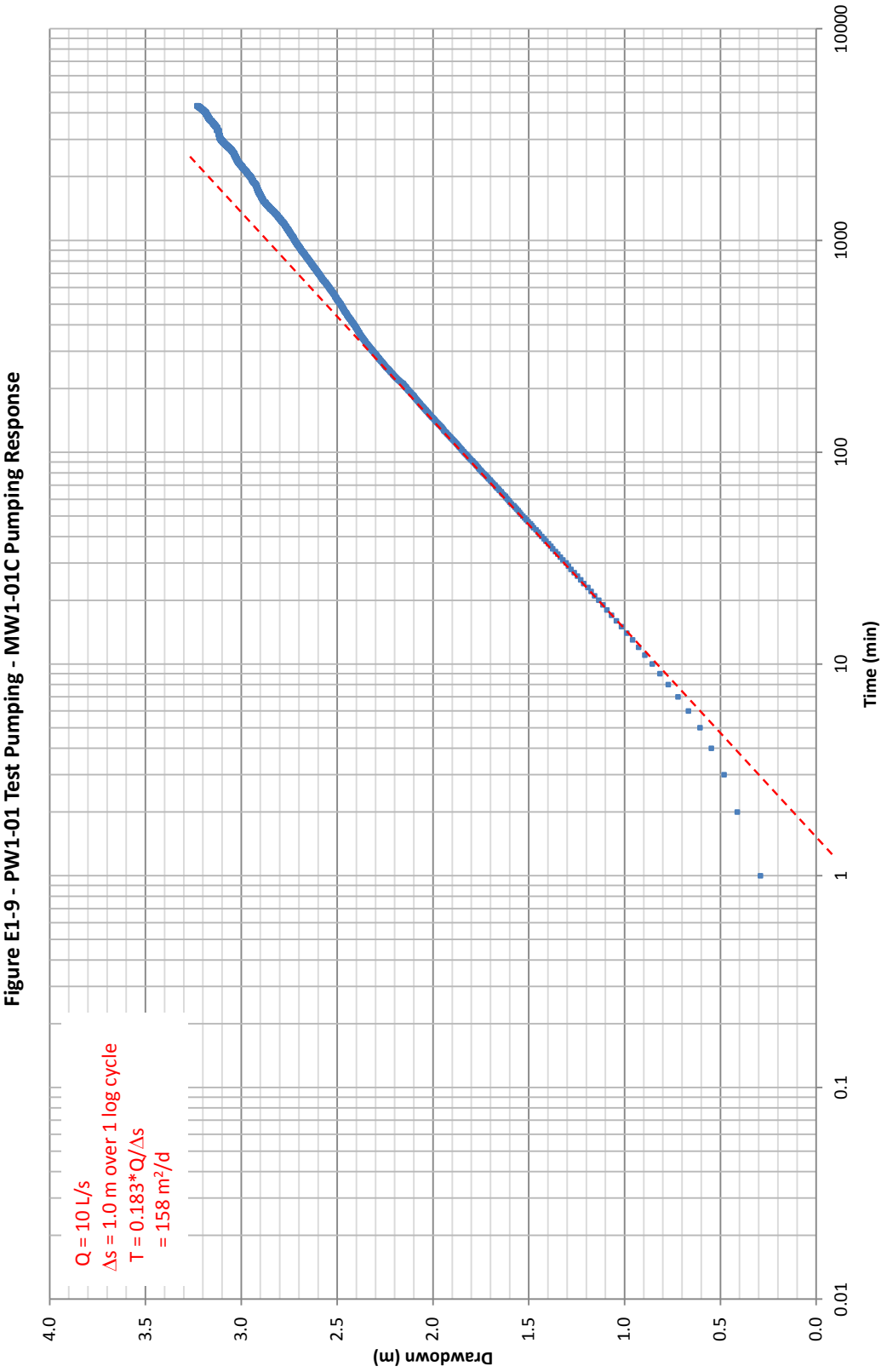


Figure E1-9 - PW1-01 Test Pumping - MW1-01C Pumping Response
Drawdown (m)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 10 L/s
Δs = 1.0 m over 1 log cycle
T = 0.183*Q/Δs
= 158 m²/d

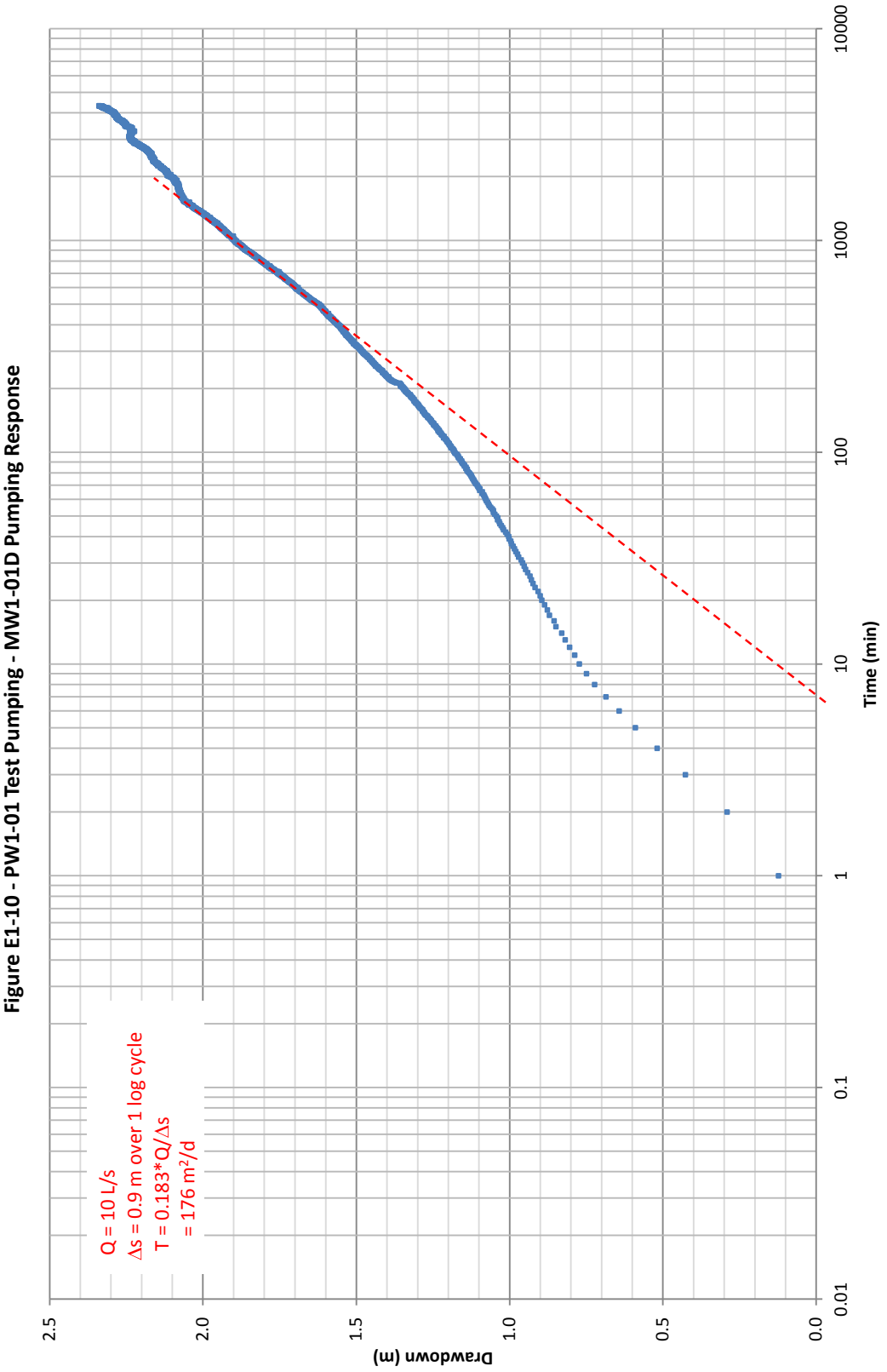


Figure E1-10 - PW1-01 Test Pumping - MW1-01D Pumping Response
Drawdown (m)
2.5
2.0
1.5
1.0
0.5
0.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 10 L/s
Δs = 0.9 m over 1 log cycle
T = 0.183*Q/Δs
= 176 m²/d

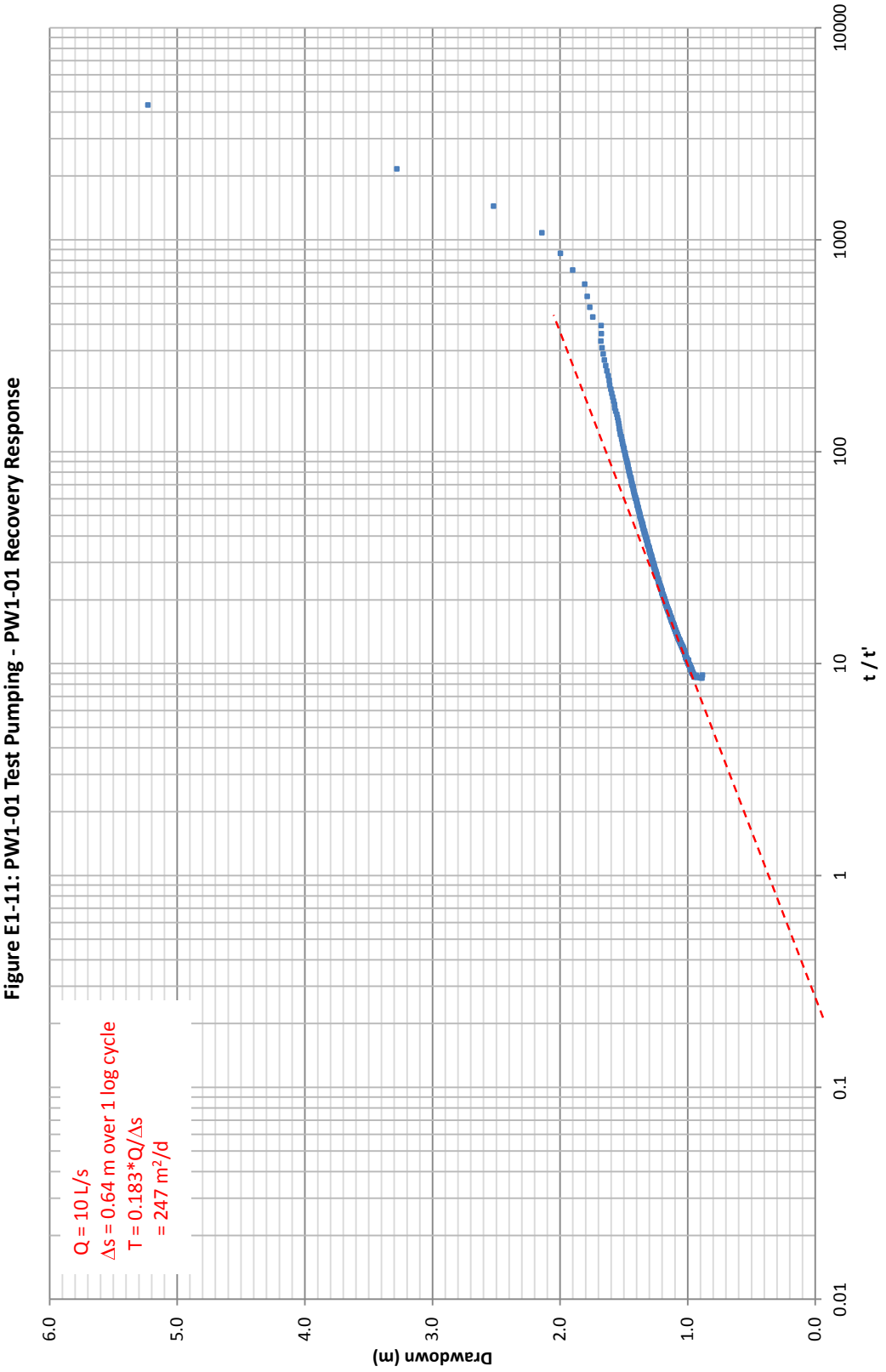


Figure E1-11: PW1-01 Test Pumping - PW1-01 Recovery Response

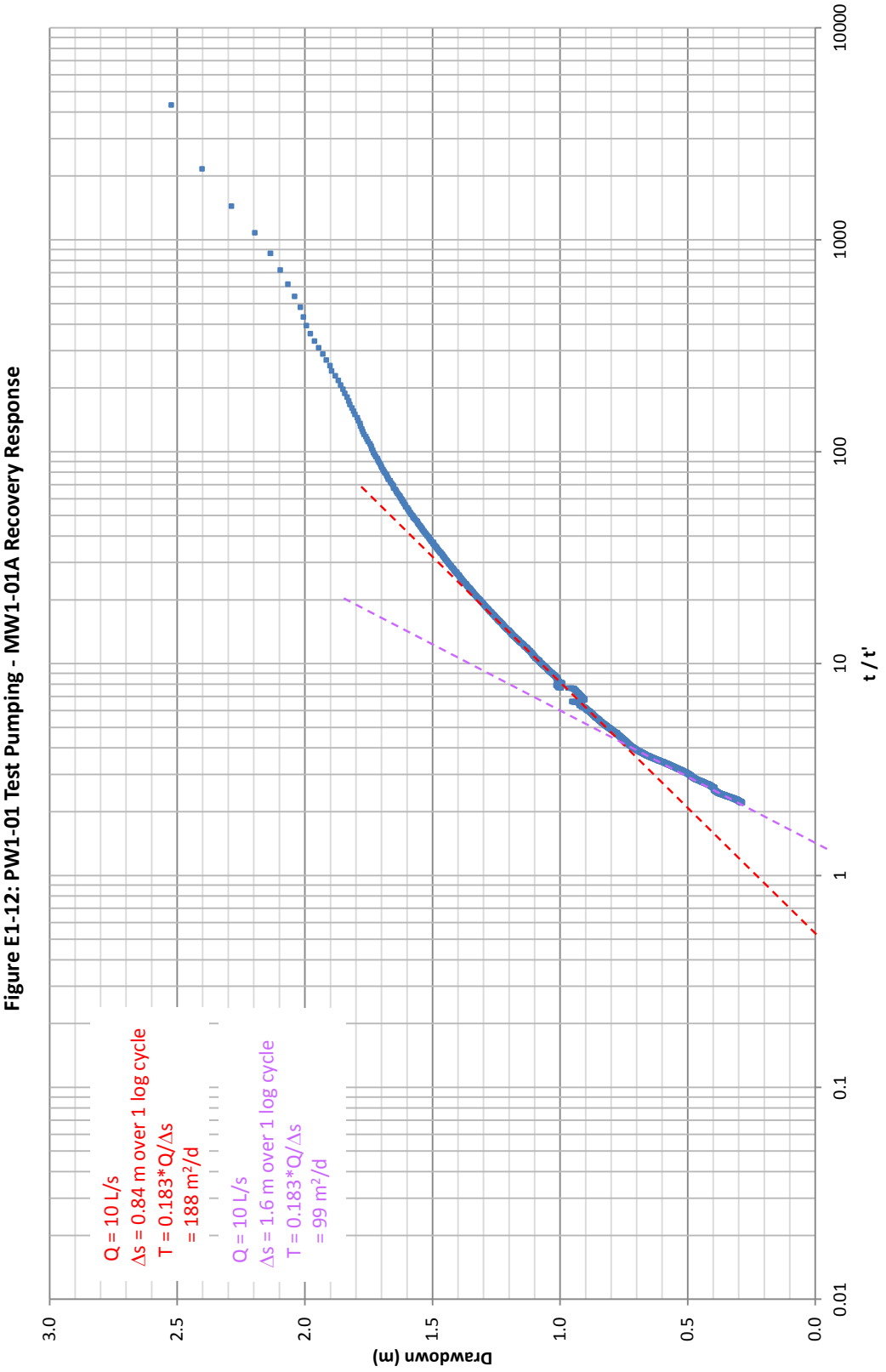


Figure E1-12: PW1-01 Test Pumping - MW1-01A Recovery Response

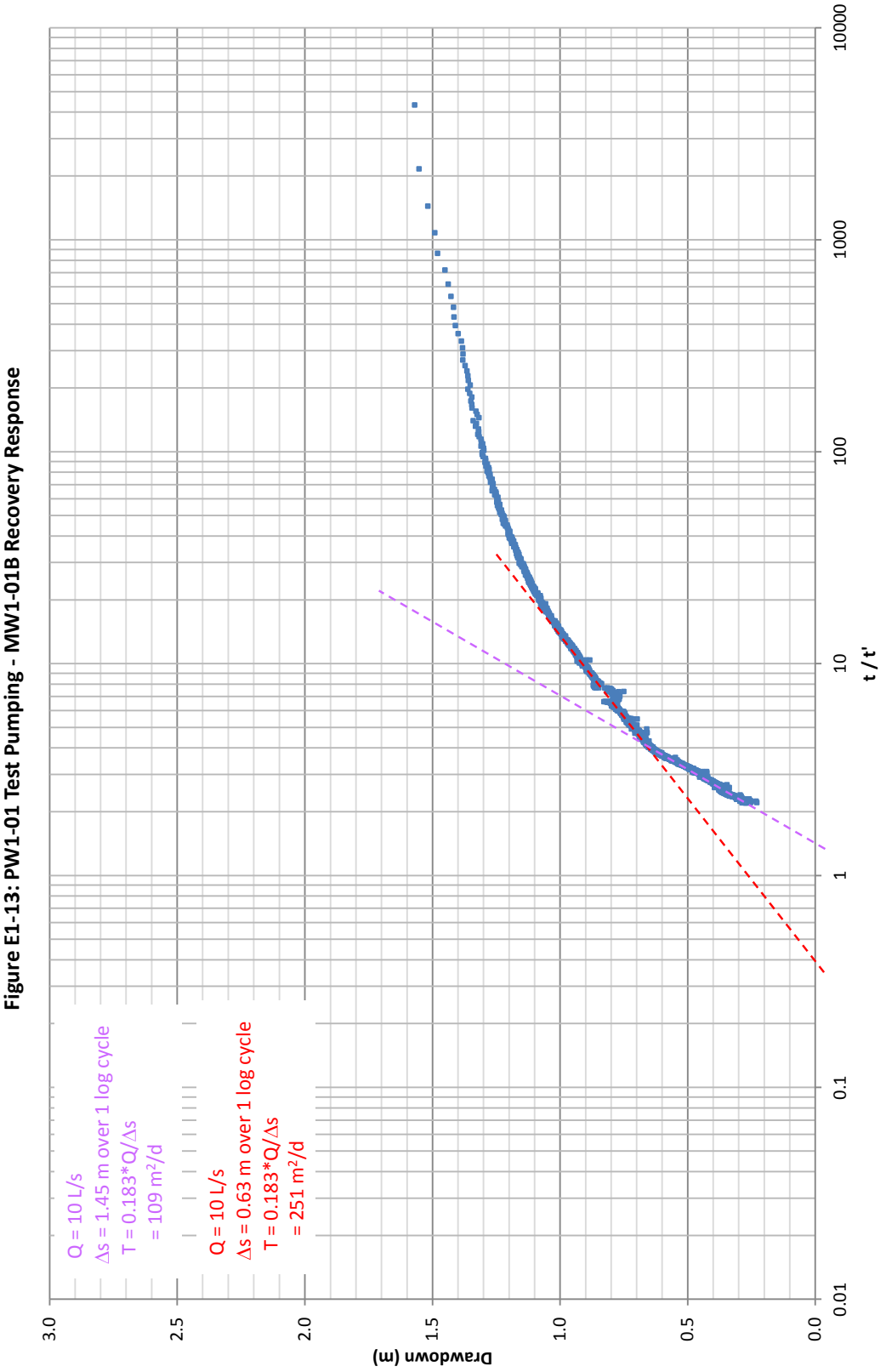


Figure E1-13: PW1-01 Test Pumping - MW1-01B Recovery Response

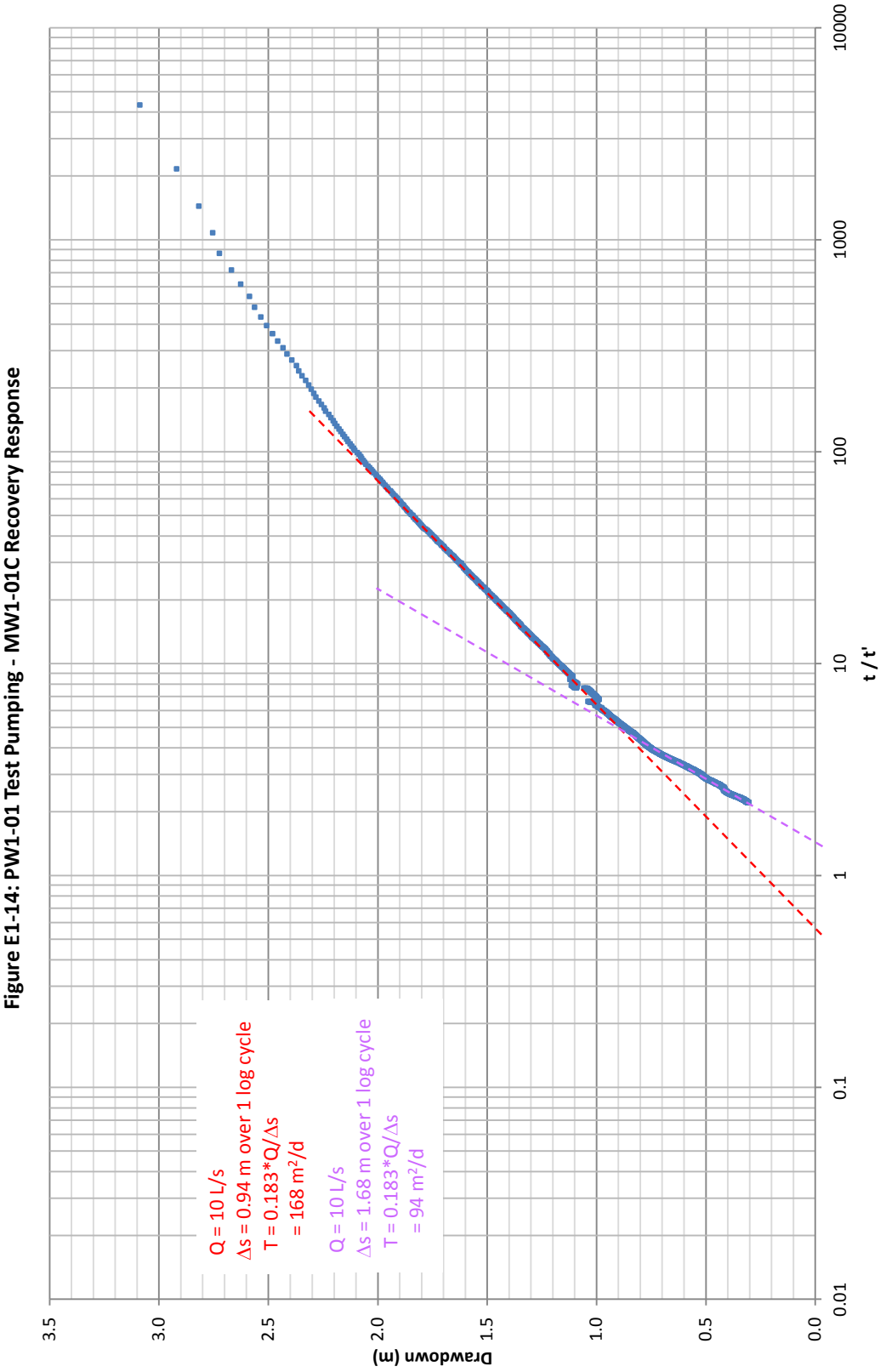


Figure E1-14: PW1-01 Test Pumping - MW1-01C Recovery Response

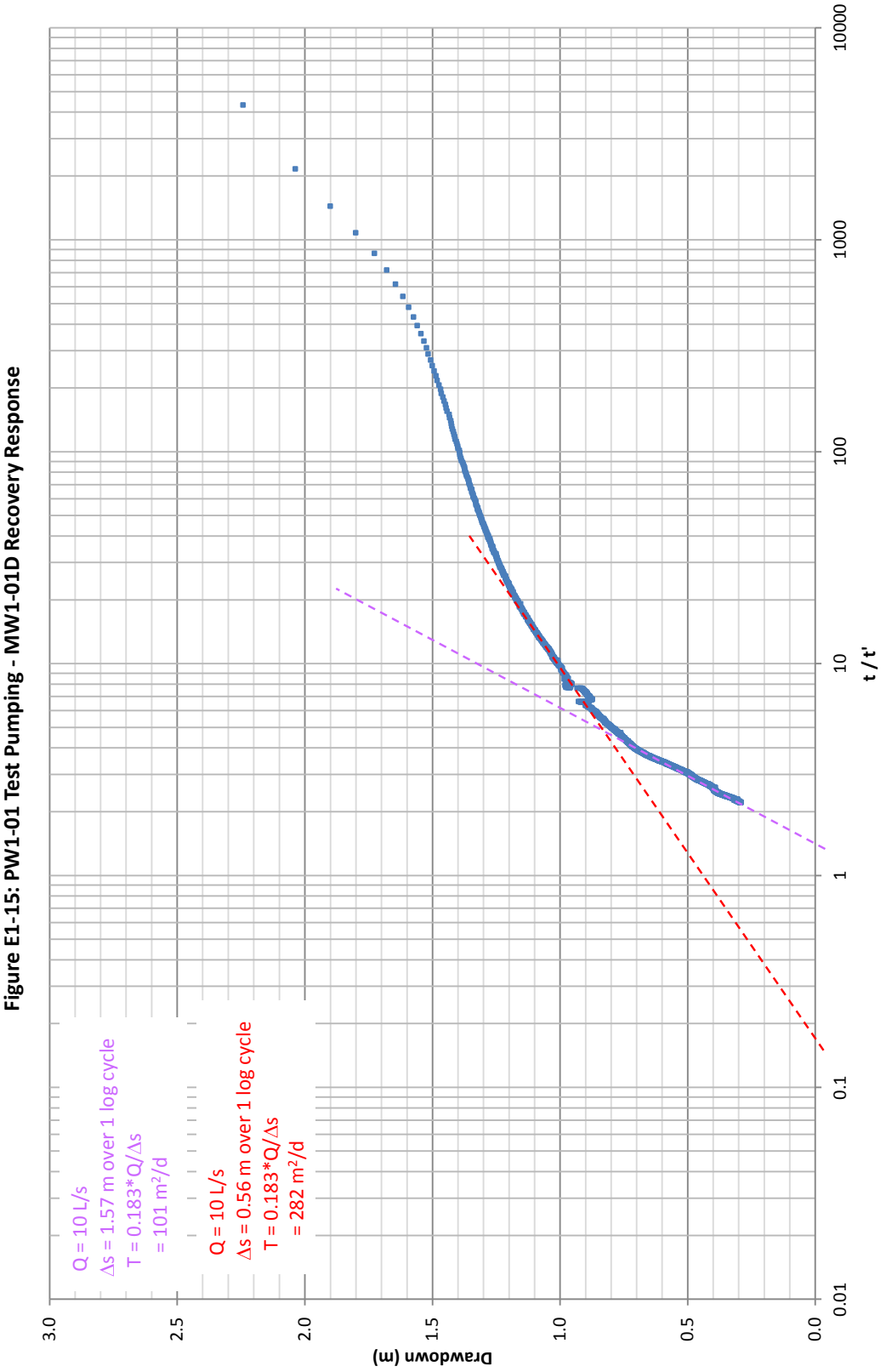


Figure E1-15: PW1-01 Test Pumping - MW1-01D Recovery Response

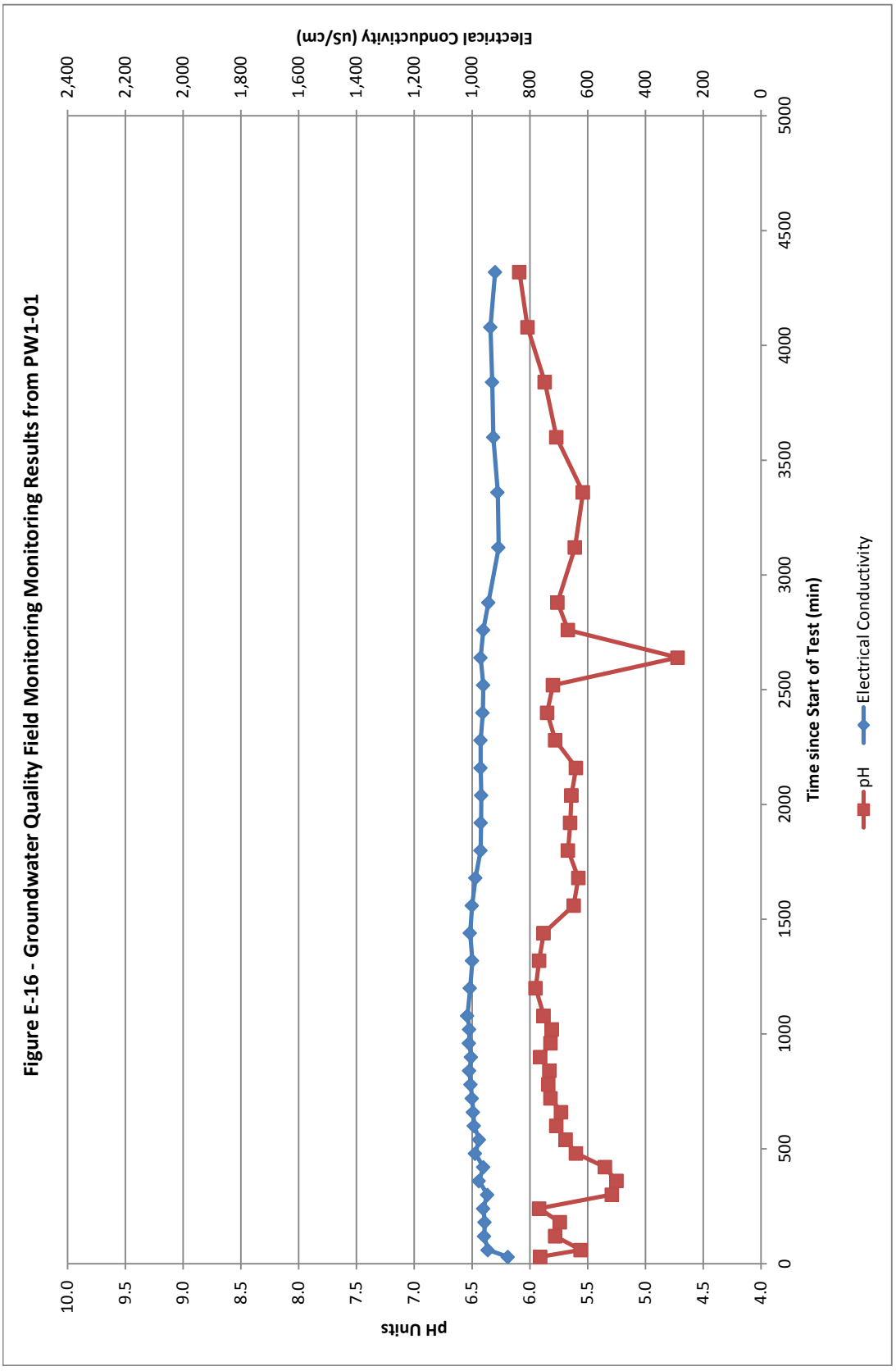
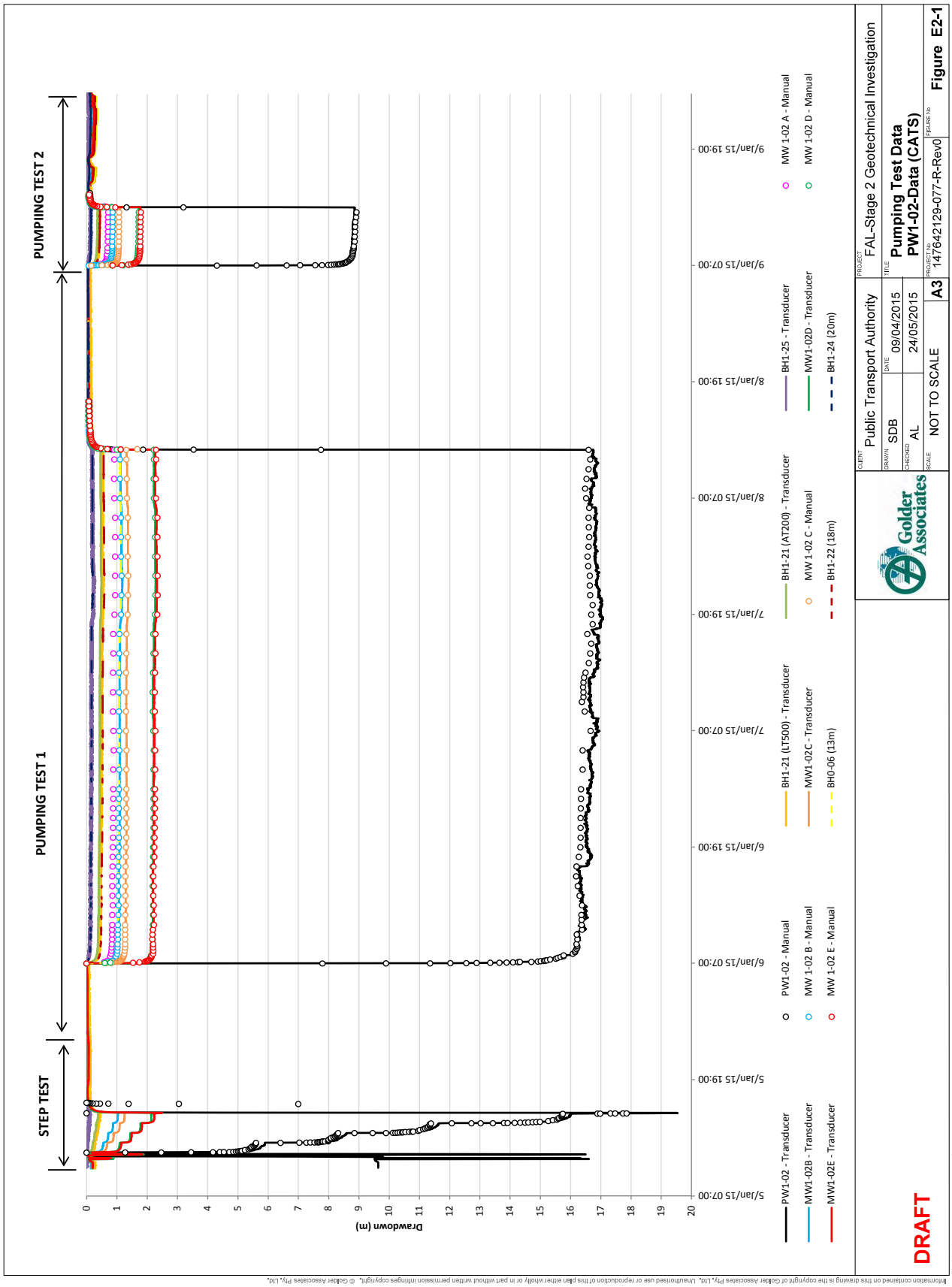


Figure E-16 - Groundwater Quality Field Monitoring Results from PW1-01
pH Units
Electrical Conductivity (uS/cm)
Time since Start of Test (min)
pH
Electrical Conductivity
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
2,400
2,200
2,000
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000



APPENDIX E
Summary of Test Pumping Analysis

ATTACHMENT E2
Constant Rate Test – Consolidated Terminal Station (PW1-02)


STEP TEST
PUMPING TEST 1
PUMPIING TEST 2
Drawdown (m)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
5/Jan/15 07:00
5/Jan/15 19:00
6/Jan/15 07:00
6/Jan/15 19:00
7/Jan/15 07:00
7/Jan/15 19:00
8/Jan/15 07:00
8/Jan/15 19:00
9/Jan/15 07:00
9/Jan/15 19:00
PW1-02 - Transducer
MW1-02B - Transducer
MW1-02E - Transducer
PW1-02 - Manual
MW 1-02 B - Manual
MW 1-02 E - Manual
BH1-21 (LT500) - Transducer
MW1-02C - Transducer
BH0-06 (13m)
BH1-21 (AT200) - Transducer
MW 1-02 C - Manual
BH1-22 (18m)
BH1-25 - Transducer
MW1-02D - Transducer
BH1-24 (20m)
MW 1-02 A - Manual
MW 1-02 D - Manual
DRAFT
Golder Associates
CLIENT
Public Transport Authority
DRAWN
SDB
DATE
09/04/2015
CHECKED
AL
24/05/2015
SCALE
NOT TO SCALE
A3
PROJECT
FAL-Stage 2 Geotechnical Investigation
TITLE
Pumping Test Data
PW1-02-Data (CATS)
PROJECT No
147642129-077-R-Rev0
FIGURE No
Figure E2-1
File Location: t:\powerpoint\golder_s3_landscape.pptm
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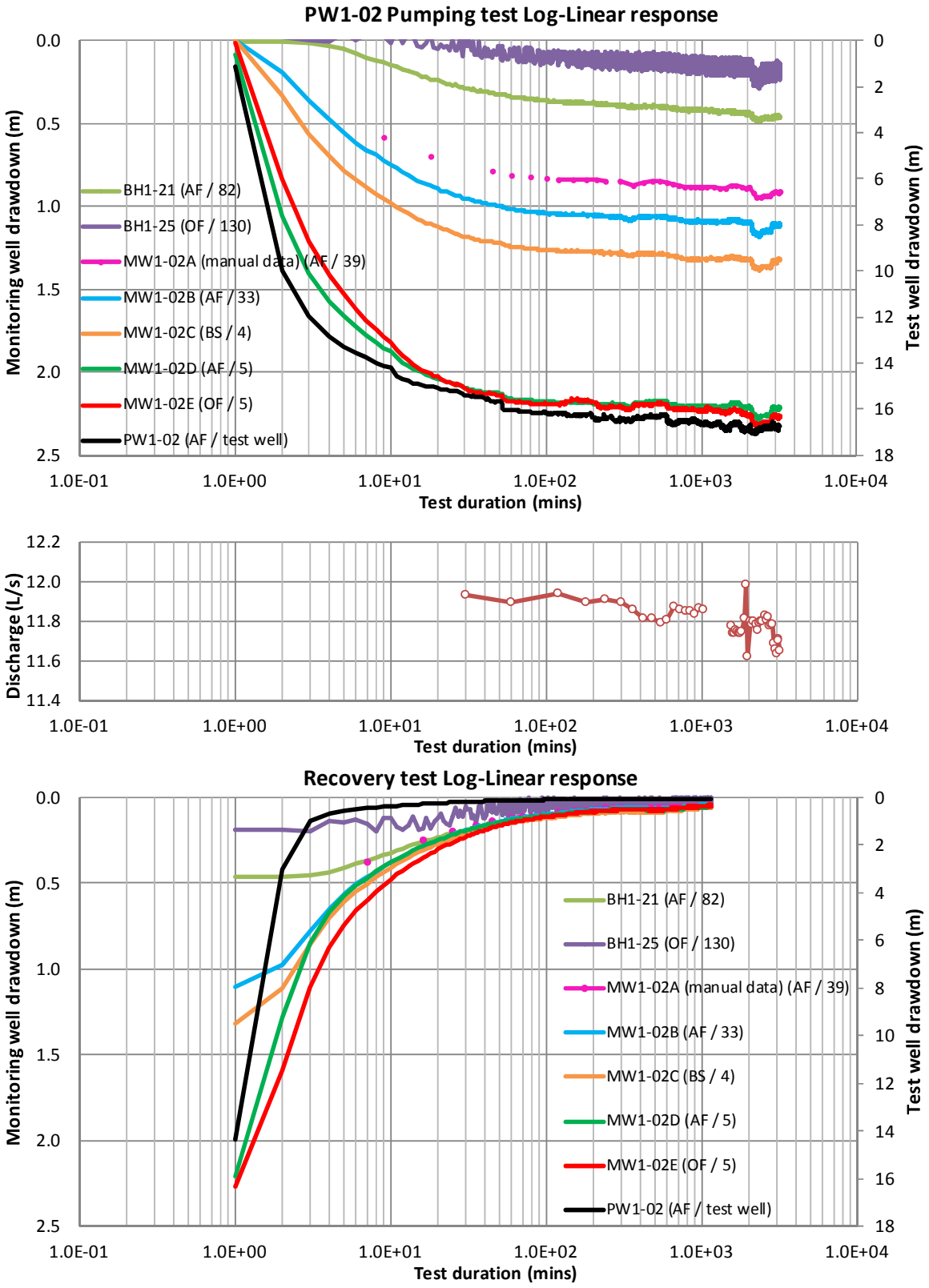

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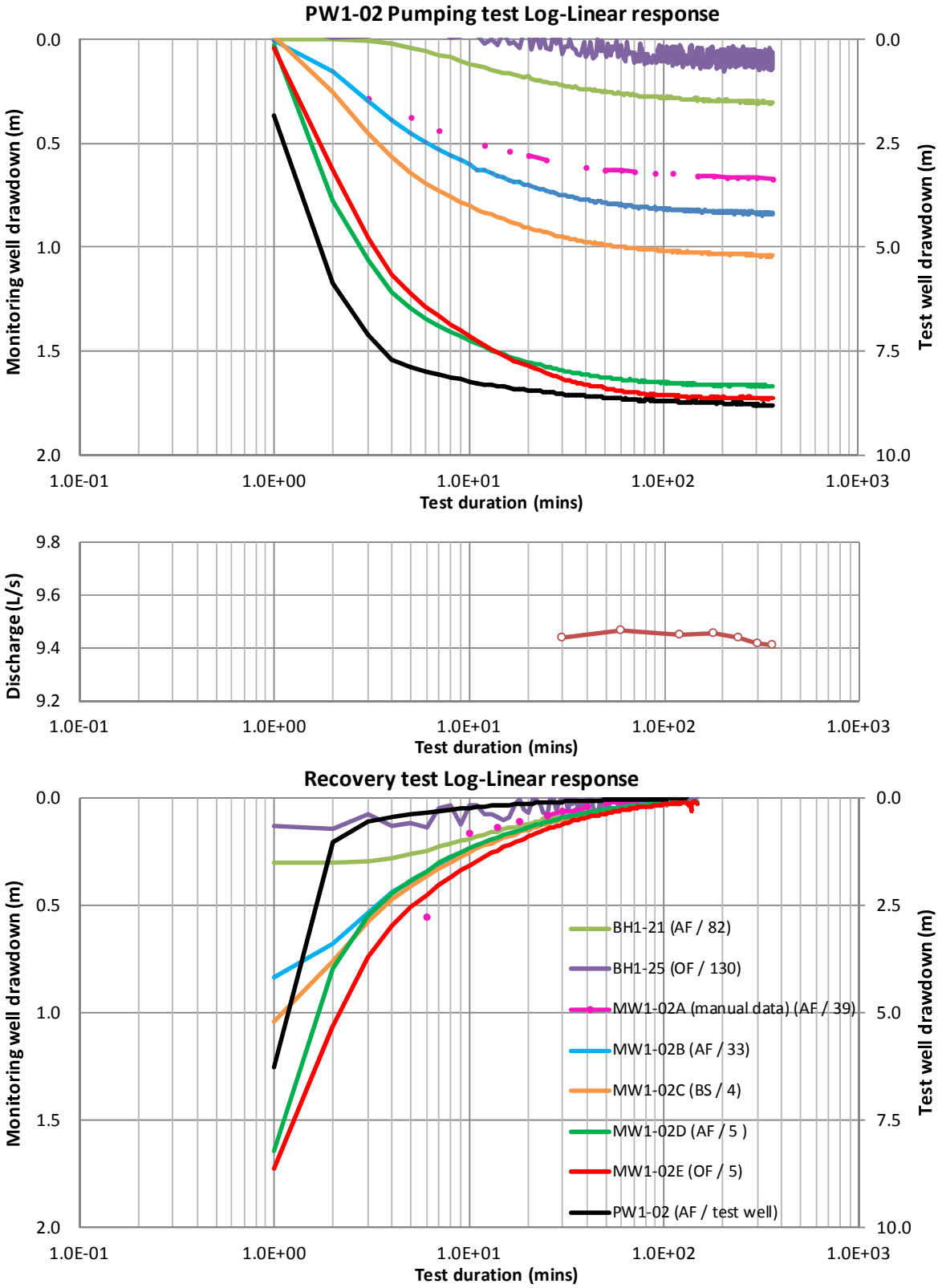
Golder Associates
Level 3, 1 Havelock St West Perth WA 6005
Ph: +618 9213 7600 Fx: +618 9213 7611

CLIENT	Public Transport Authority	PROJECT	FAL-Stage 2 Geotechnical Investigation
DRAWN	SDB	DATE 24/05/2015	TITLE Pumping Test Data PW1-02-Test 1 (Consolidated Station)
CHECKED	AL	DATE 24/05/2015	
SCALE	NOT TO SCALE	A4	PROJECT No 147642129-077-R-Rev0
FIGURE No	Figure E2-2		

File Location:



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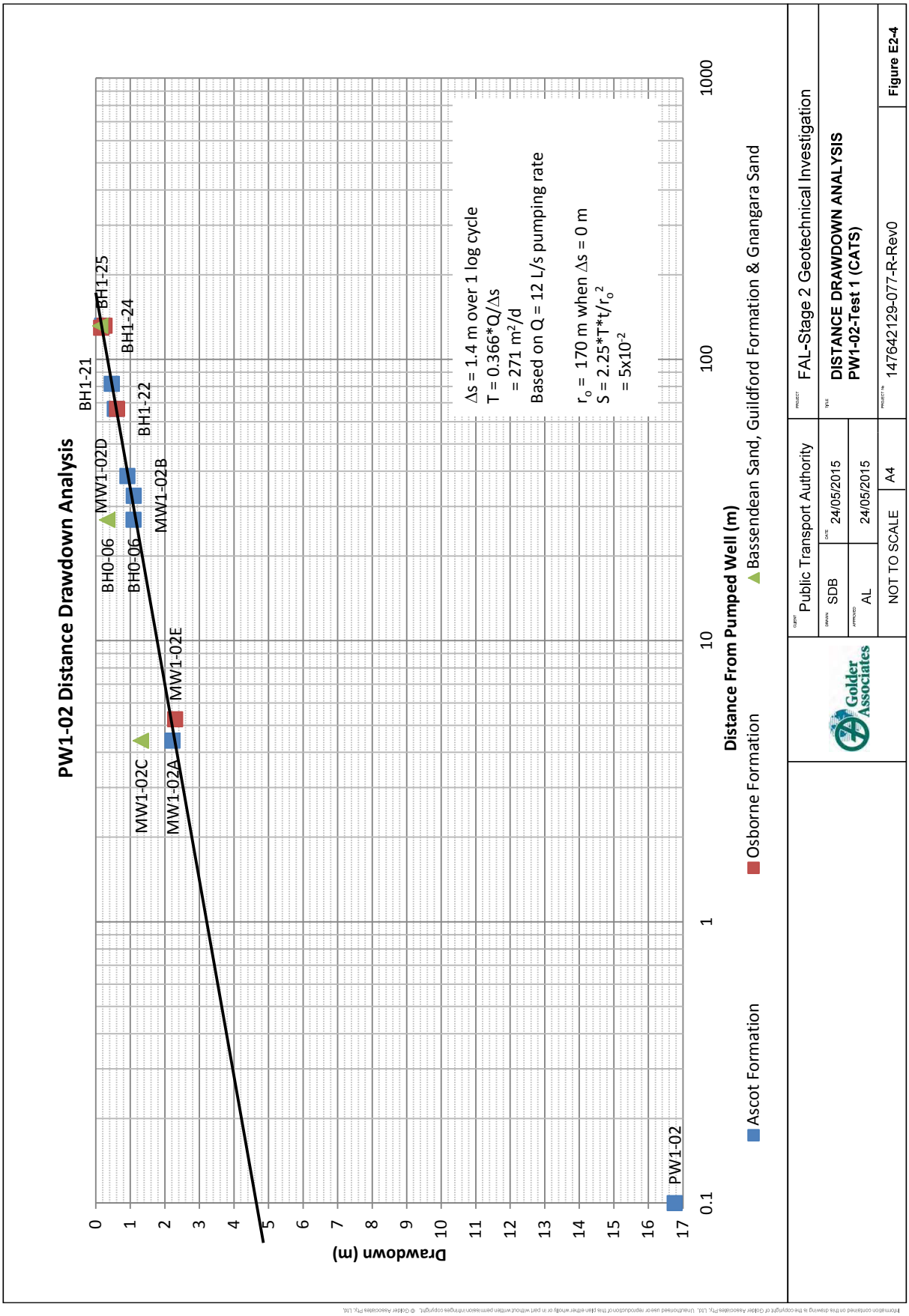
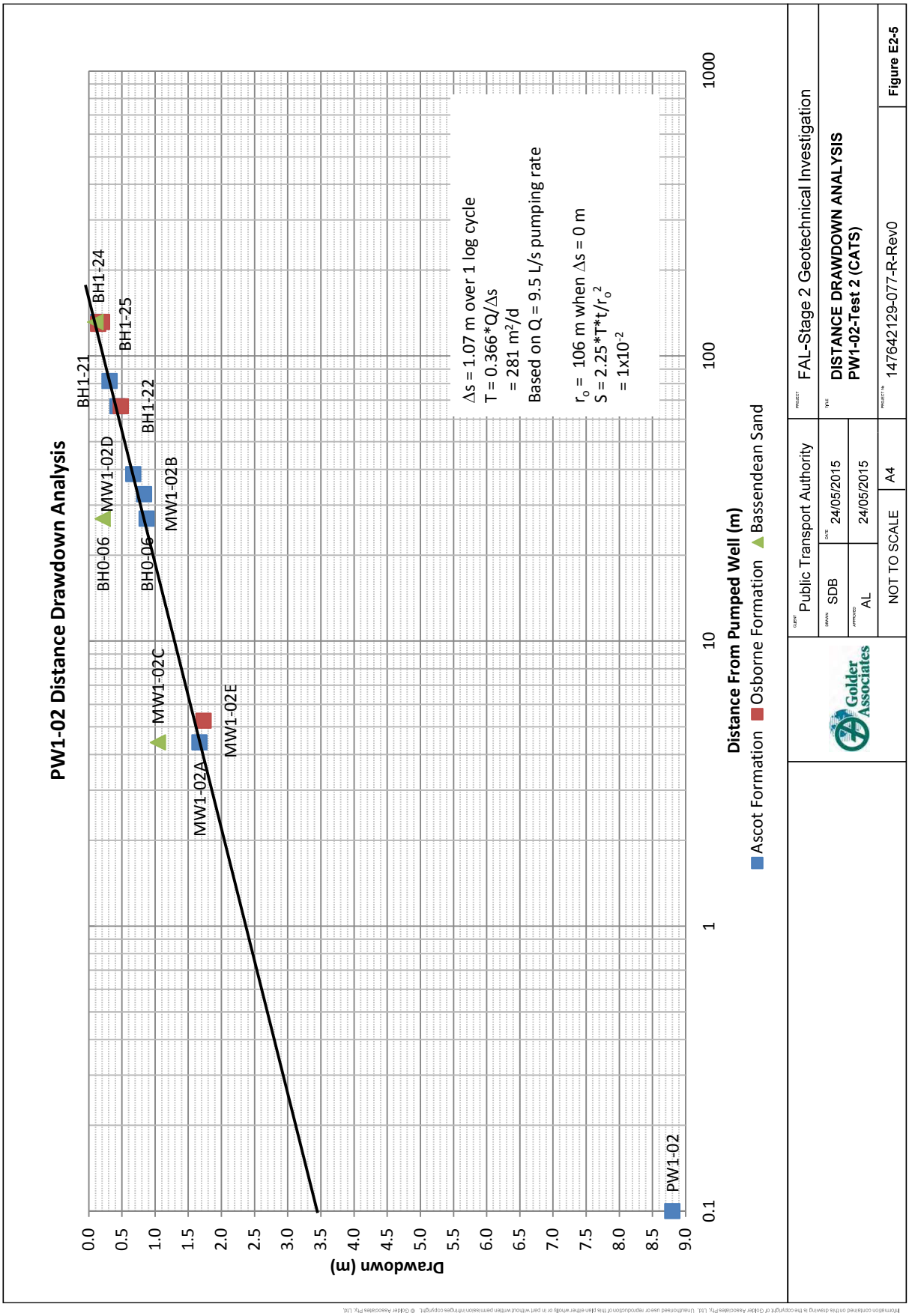
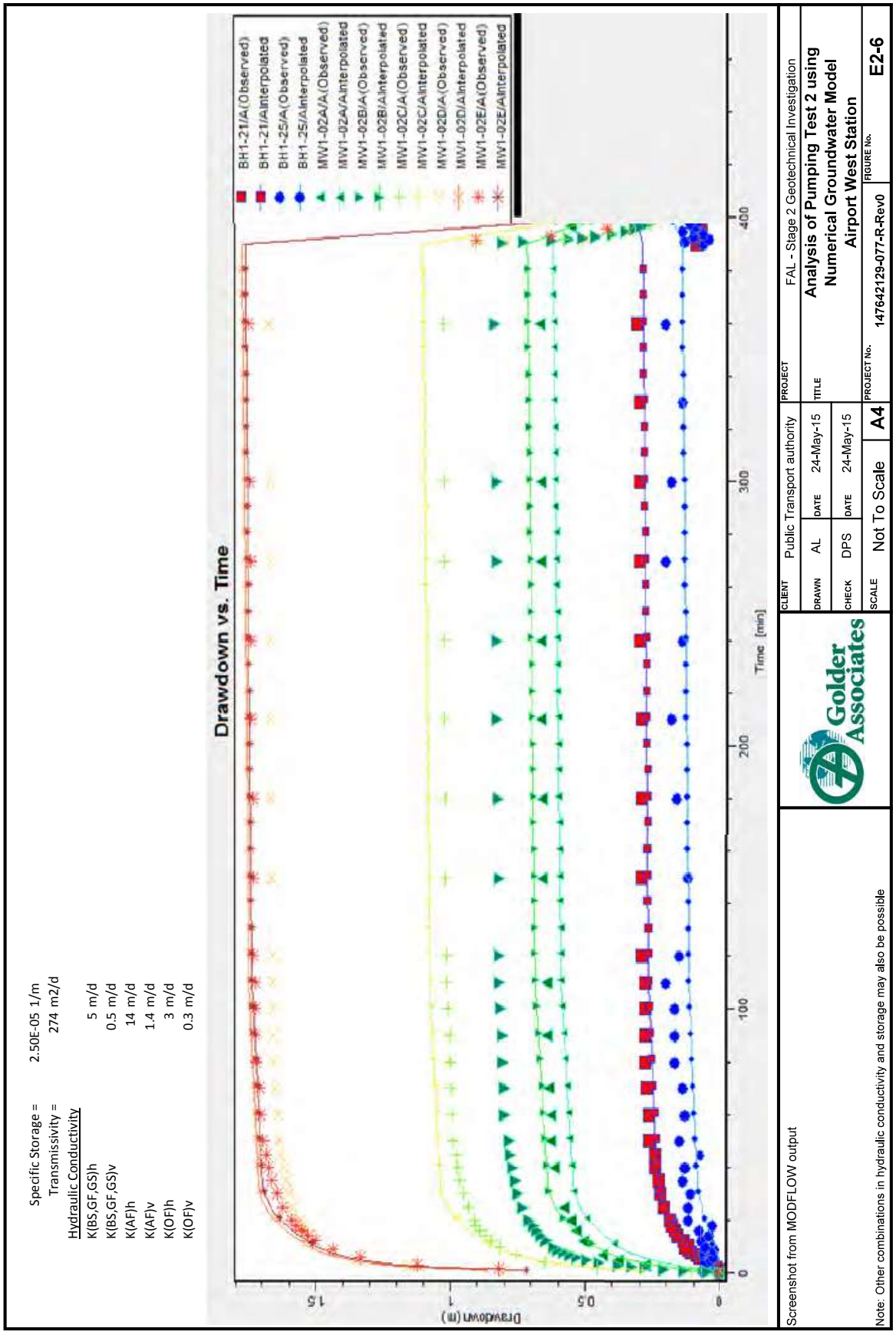
Golder Associates Level 3, 1 Havelock St West Perth WA 6005 Ph: +618 9213 7600 Fx: +618 9213 7611	CLIENT: Public Transport Authority		PROJECT: FAL-Stage 2 Geotechnical Investigation
	DRAWN: SDB	DATE: 24/05/2015	TITLE: **Pumping Test Data**
	CHECKED: AL	DATE: 24/05/2015	**PW1-02-Test 2 (Consolidated Station)**
	SCALE: NOT TO SCALE	A4	PROJECT No: 147642129-077-R-Rev0 / FIGURE No: **Figure E2-3**

File Location:


PW1-02 Distance Drawdown Analysis
Drawdown (m)
Distance From Pumped Well (m)
Ascot Formation
Osborne Formation
Bassendean Sand, Guildford Formation & Gnangara Sand
PW1-02
MW1-02A
MW1-02C
MW1-02E
BH0-06
BH0-06
MW1-02D
MW1-02B
BH1-21
BH1-22
BH1-24
BH1-25
Δs = 1.4 m over 1 log cycle
T = 0.366*Q/Δs
= 271 m²/d
Based on Q = 12 L/s pumping rate
r_o = 170 m when Δs = 0 m
S = 2.25*T*t/r_o²
= 5x10⁻²
Public Transport Authority
FAL-Stage 2 Geotechnical Investigation
SDB
24/05/2015
AL
24/05/2015
NOT TO SCALE
A4
DISTANCE DRAWDOWN ANALYSIS
PW1-02-Test 1 (CATS)
147642129-077-R-Rev0
Figure E2-4


PW1-02 Distance Drawdown Analysis
Drawdown (m)
Distance From Pumped Well (m)
Ascot Formation
Osborne Formation
Bassendean Sand
PW1-02
MW1-02A
MW1-02C
MW1-02E
BH0-06
MW1-02D
BH0-06
MW1-02B
BH1-21
BH1-22
BH1-25
BH1-24
Δs = 1.07 m over 1 log cycle
T = 0.366*Q/Δs
= 281 m²/d
Based on Q = 9.5 L/s pumping rate
r_o = 106 m when Δs = 0 m
S = 2.25*T*t/r_o²
= 1x10⁻²
Public Transport Authority
SDB
24/05/2015
AL
24/05/2015
NOT TO SCALE
A4
FAL-Stage 2 Geotechnical Investigation
DISTANCE DRAWDOWN ANALYSIS
PW1-02-Test 2 (CATS)
147642129-077-R-Rev0
Figure E2-5


Drawdown vs. Time
Drawdown (m)
Time (min)
BH1-21/A(Observed)
BH1-21/AInterpolated
BH1-25/A(Observed)
BH1-25/AInterpolated
MW1-02A/A(Observed)
MW1-02A/AInterpolated
MW1-02B/A(Observed)
MW1-02B/AInterpolated
MW1-02C/A(Observed)
MW1-02C/AInterpolated
MW1-02D/A(Observed)
MW1-02D/AInterpolated
MW1-02E/A(Observed)
MW1-02E/AInterpolated
Specific Storage = 2.50E-05 1/m
Transmissivity = 274 m2/d
Hydraulic Conductivity
K(BS,GF,GS)h 5 m/d
K(BS,GF,GS)v 0.5 m/d
K(AF)h 14 m/d
K(AF)v 1.4 m/d
K(OF)h 3 m/d
K(OF)v 0.3 m/d
Screenshot from MODFLOW output
Note: Other combinations in hydraulic conductivity and storage may also be possible
Golder Associates
CLIENT Public Transport authority
DRAWN AL DATE 24-May-15
CHECK DPS DATE 24-May-15
SCALE Not To Scale A4
PROJECT FAL - Stage 2 Geotechnical Investigation
TITLE Analysis of Pumping Test 2 using Numerical Groundwater Model Airport West Station
PROJECT No. 147642129-077-R-Rev0
FIGURE No. E2-6
C:\Users\alundorf\Desktop\FAL\APPENDIX E\Appendix E - Test Pumping Results\Attachment E2\Figure E2-6.xlsx

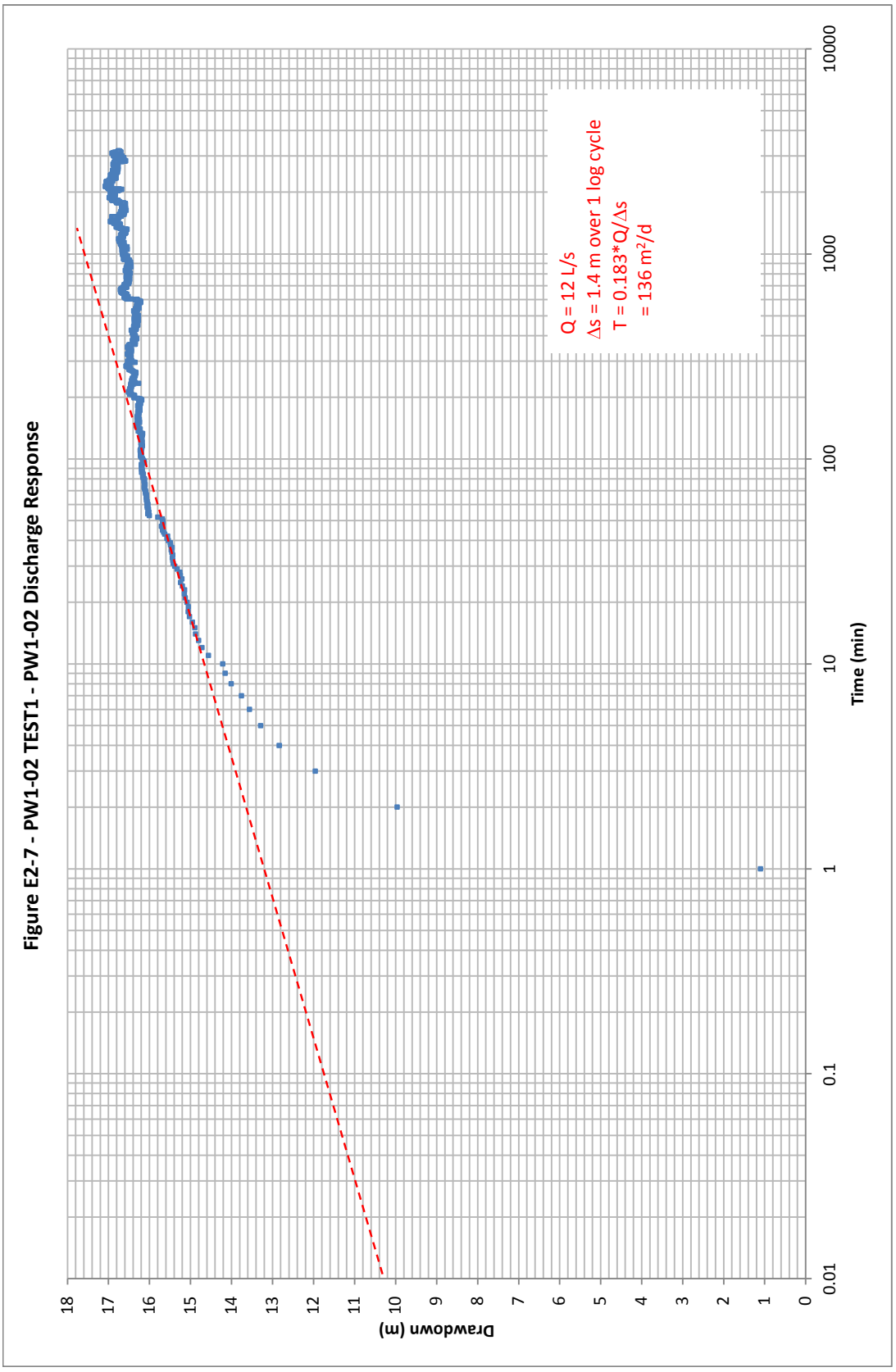


Figure E2-7 - PW1-02 TEST1 - PW1-02 Discharge Response
Drawdown (m)
Time (min)
Q = 12 L/s
Δs = 1.4 m over 1 log cycle
T = 0.183*Q/Δs
= 136 m²/d

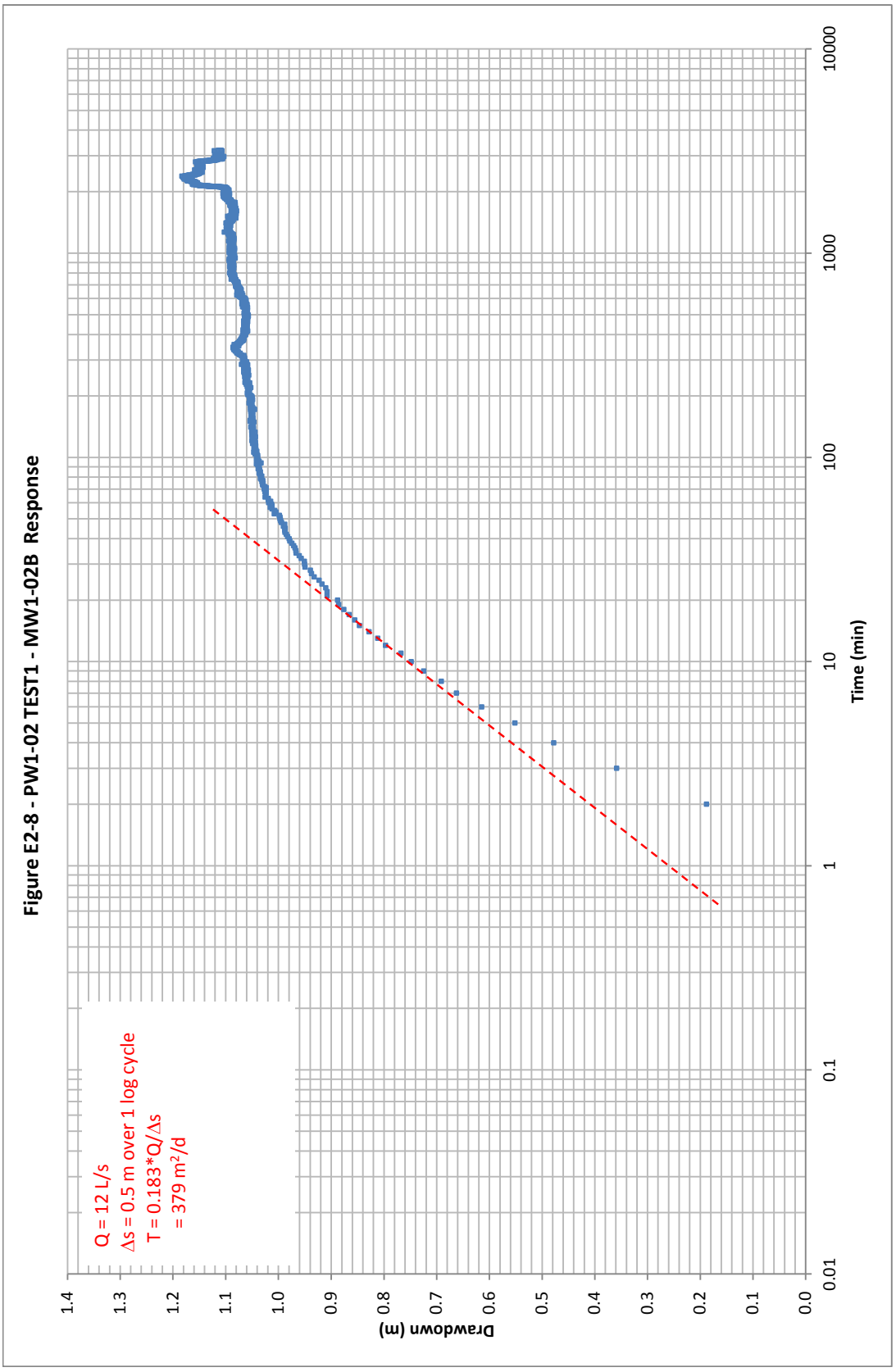


Figure E2-8 - PW1-02 TEST1 - MW1-02B Response
Drawdown (m)
Time (min)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
0.01
0.1
1
10
100
1000
10000
Q = 12 L/s
Δs = 0.5 m over 1 log cycle
T = 0.183*Q/Δs
= 379 m²/d

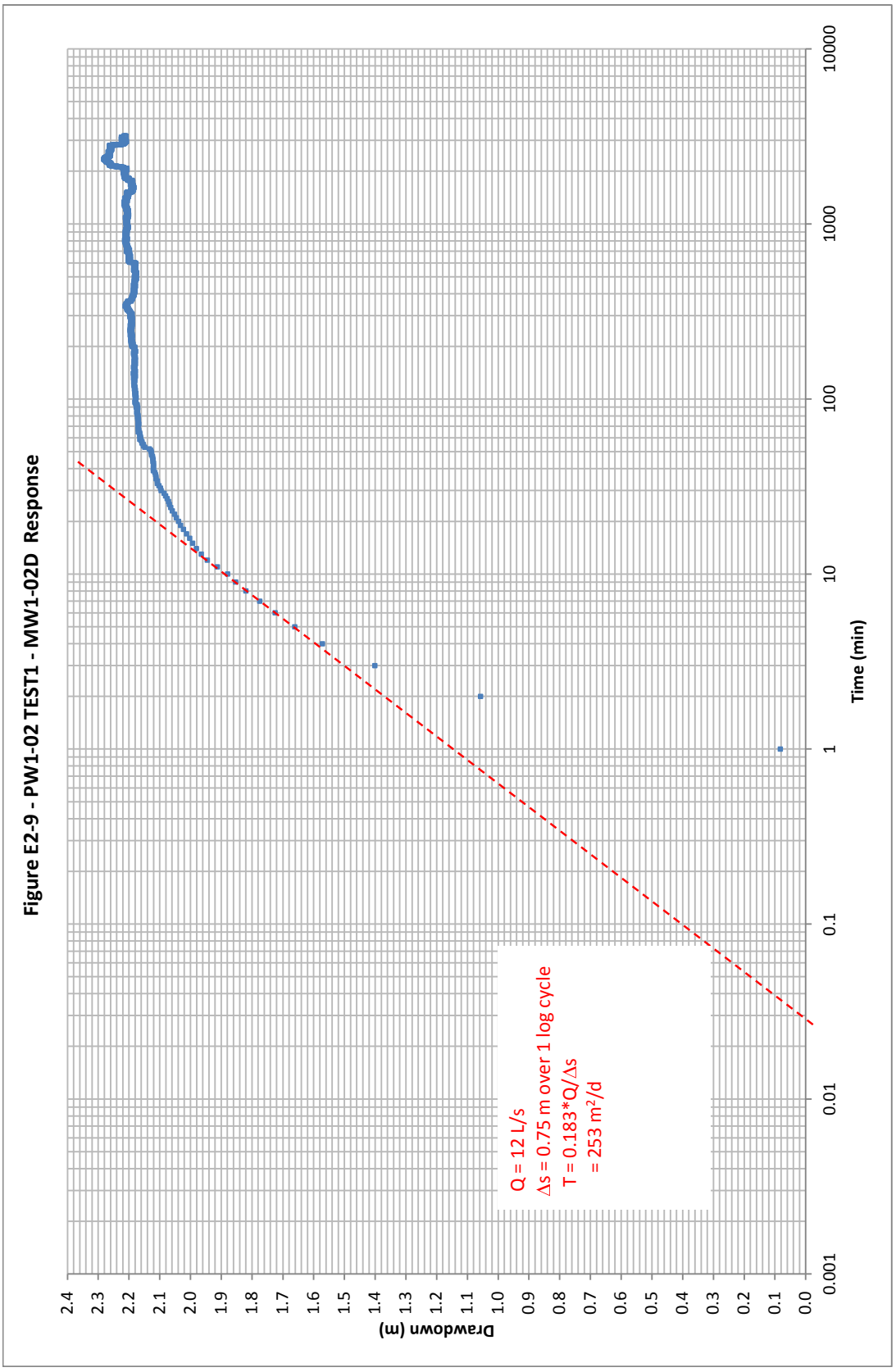


Figure E2-9 - PW1-02 TEST1 - MW1-02D Response
Drawdown (m)
Time (min)
Q = 12 L/s
Δs = 0.75 m over 1 log cycle
T = 0.183*Q/Δs
= 253 m²/d

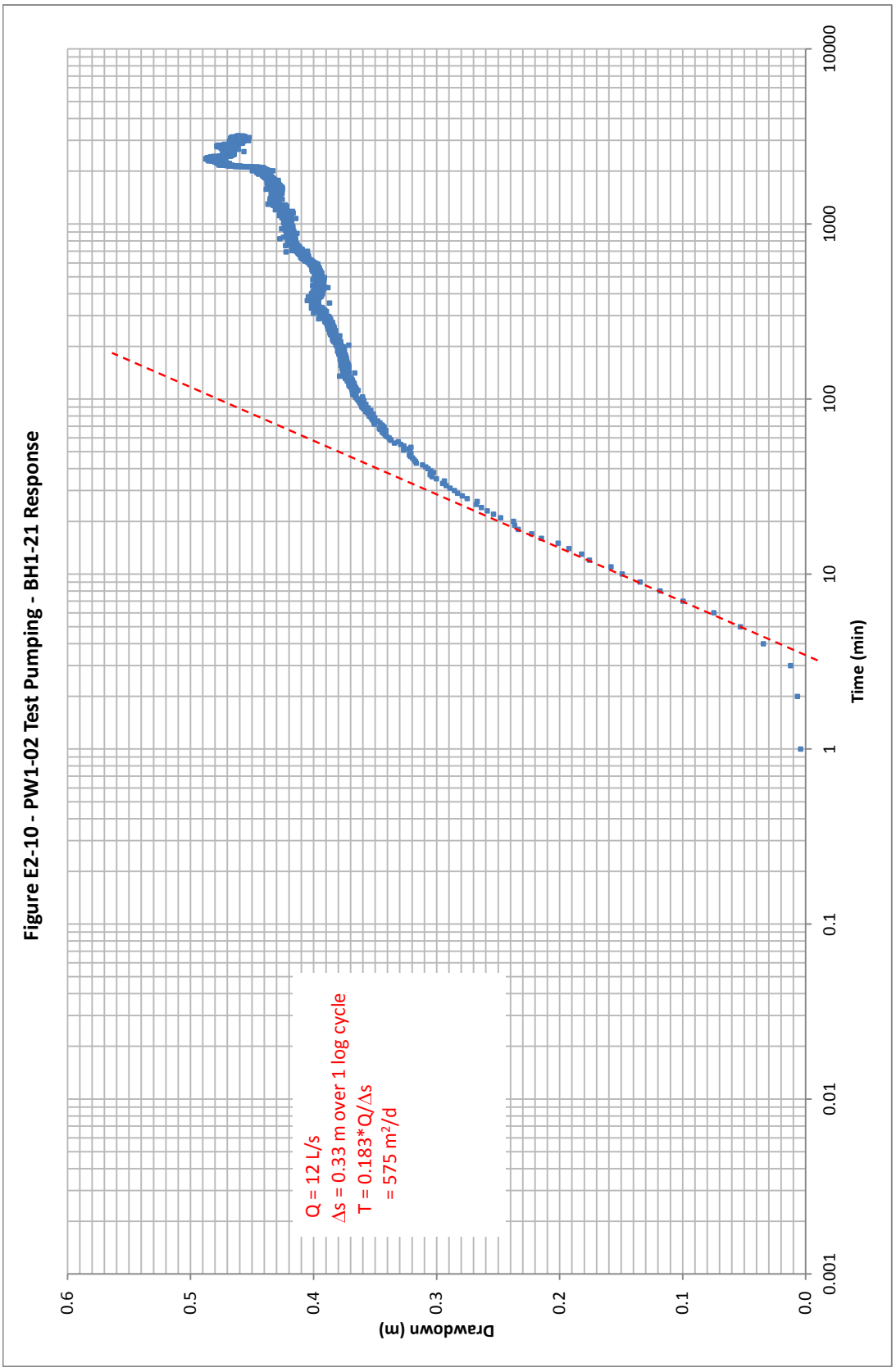


Figure E2-10 - PW1-02 Test Pumping - BH1-21 Response
Drawdown (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Time (min)
0.001
0.01
0.1
1
10
100
1000
10000
Q = 12 L/s
Δs = 0.33 m over 1 log cycle
T = 0.183*Q/Δs
= 575 m²/d

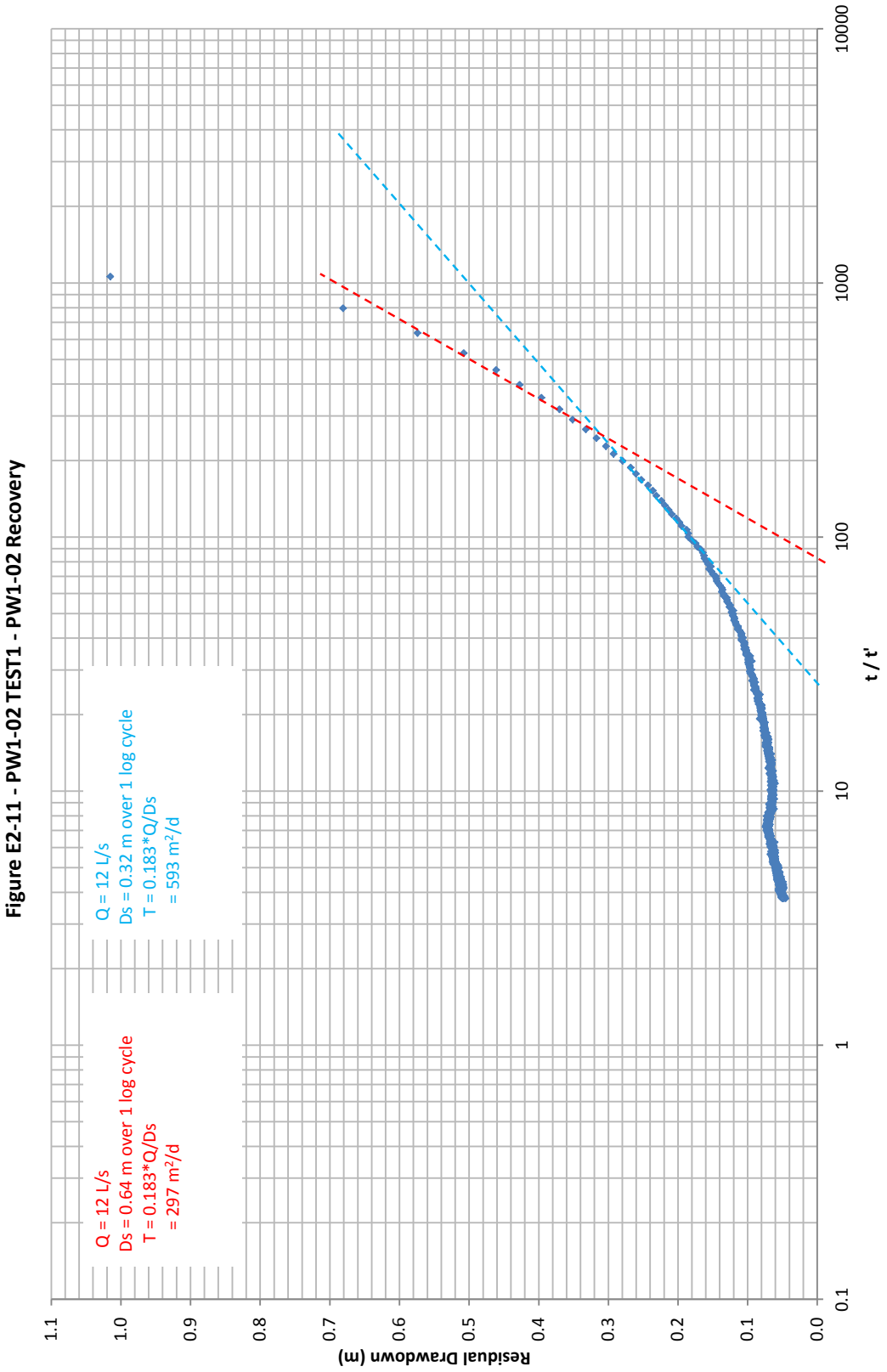


Figure E2-11 - PW1-02 TEST1 - PW1-02 Recovery

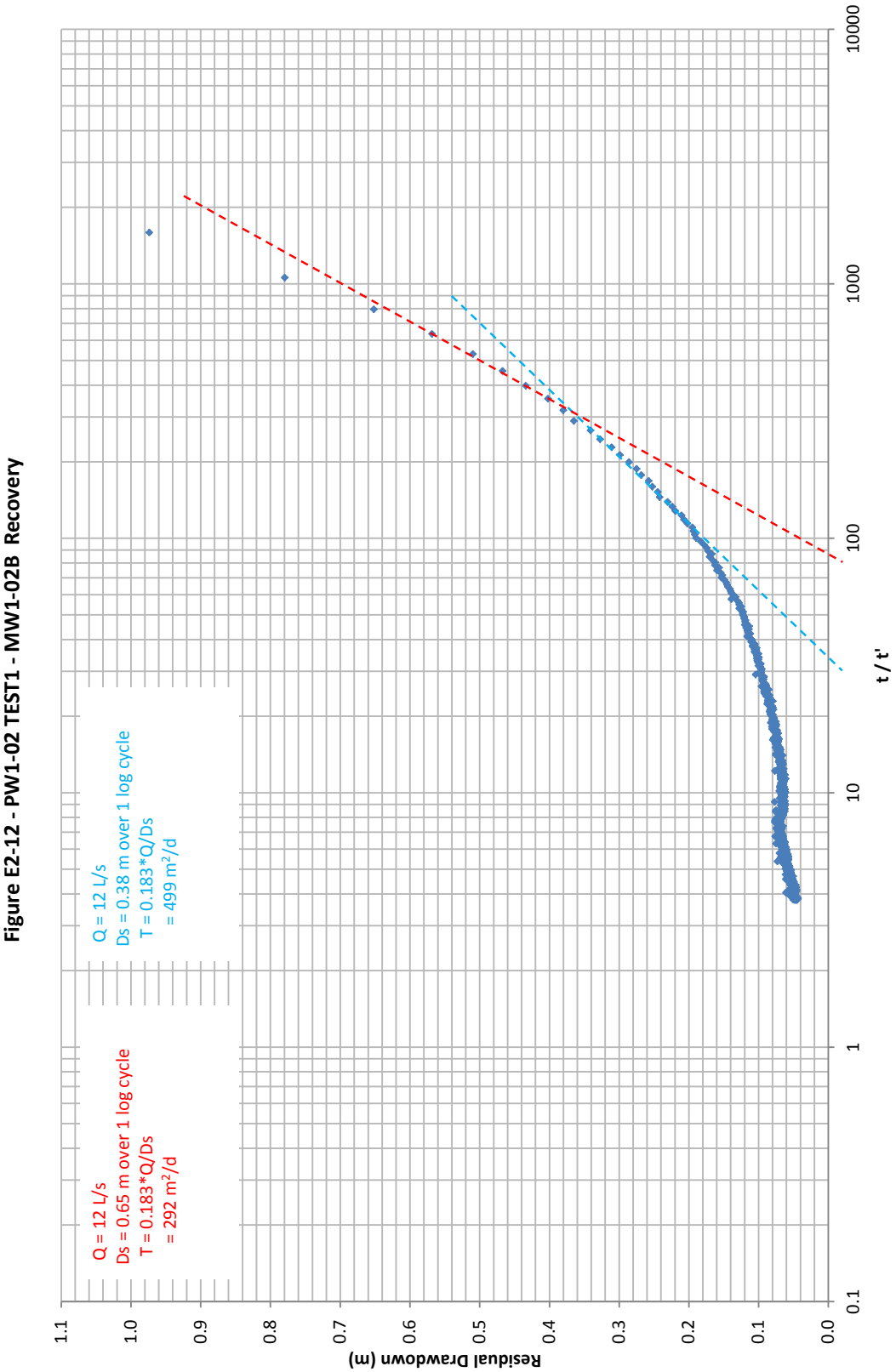


Figure E2-12 - PW1-02 TEST1 - MW1-02B Recovery
Residual Drawdown (m)
t / t'
Q = 12 L/s
Ds = 0.65 m over 1 log cycle
T = 0.183*Q/Ds
= 292 m²/d
Q = 12 L/s
Ds = 0.38 m over 1 log cycle
T = 0.183*Q/Ds
= 499 m²/d

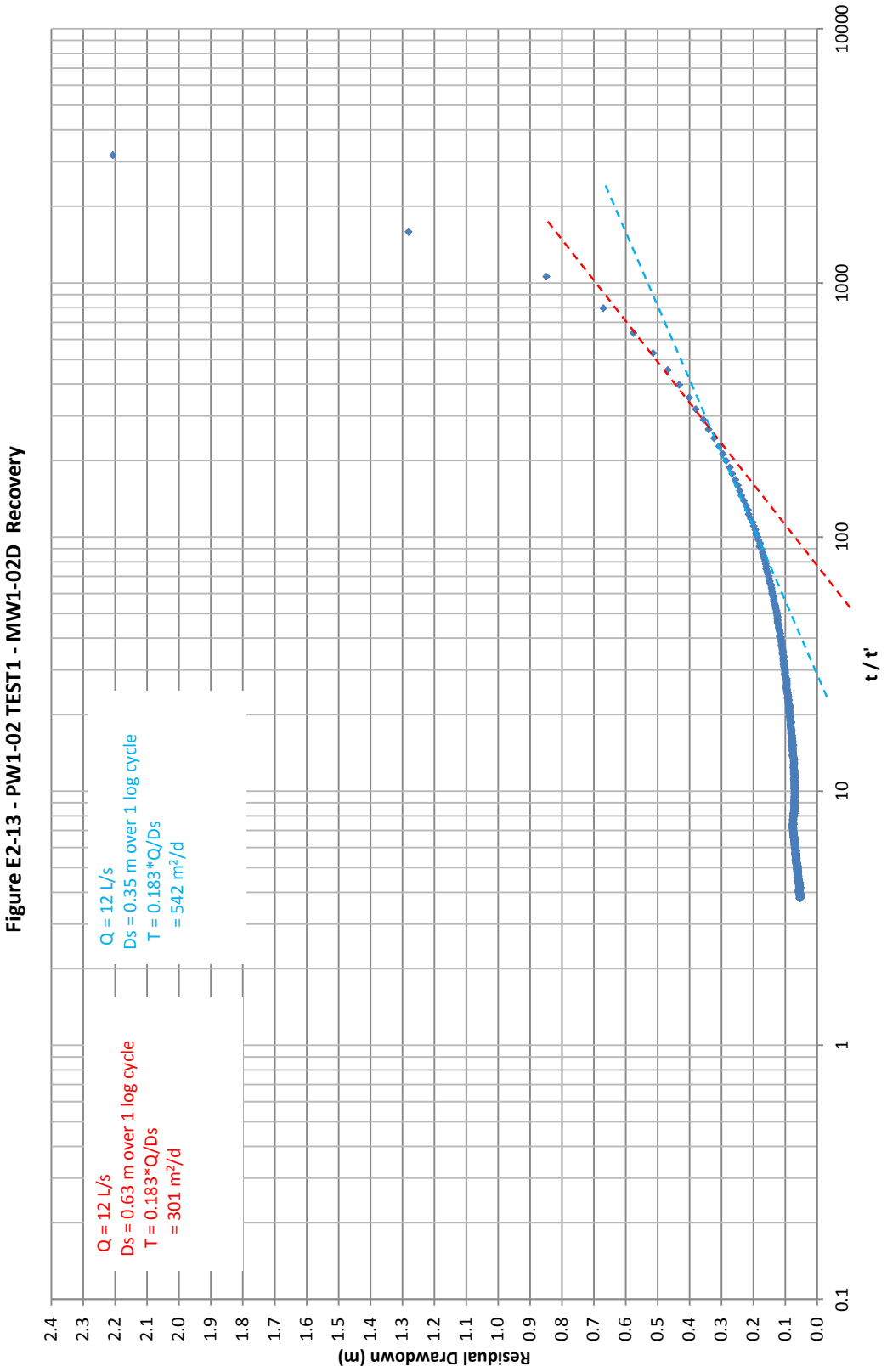


Figure E2-13 - PW1-02 TEST1 - MW1-02D Recovery
Residual Drawdown (m)
t / t'
Q = 12 L/s
Ds = 0.63 m over 1 log cycle
T = 0.183*Q/Ds
= 301 m²/d
Q = 12 L/s
Ds = 0.35 m over 1 log cycle
T = 0.183*Q/Ds
= 542 m²/d

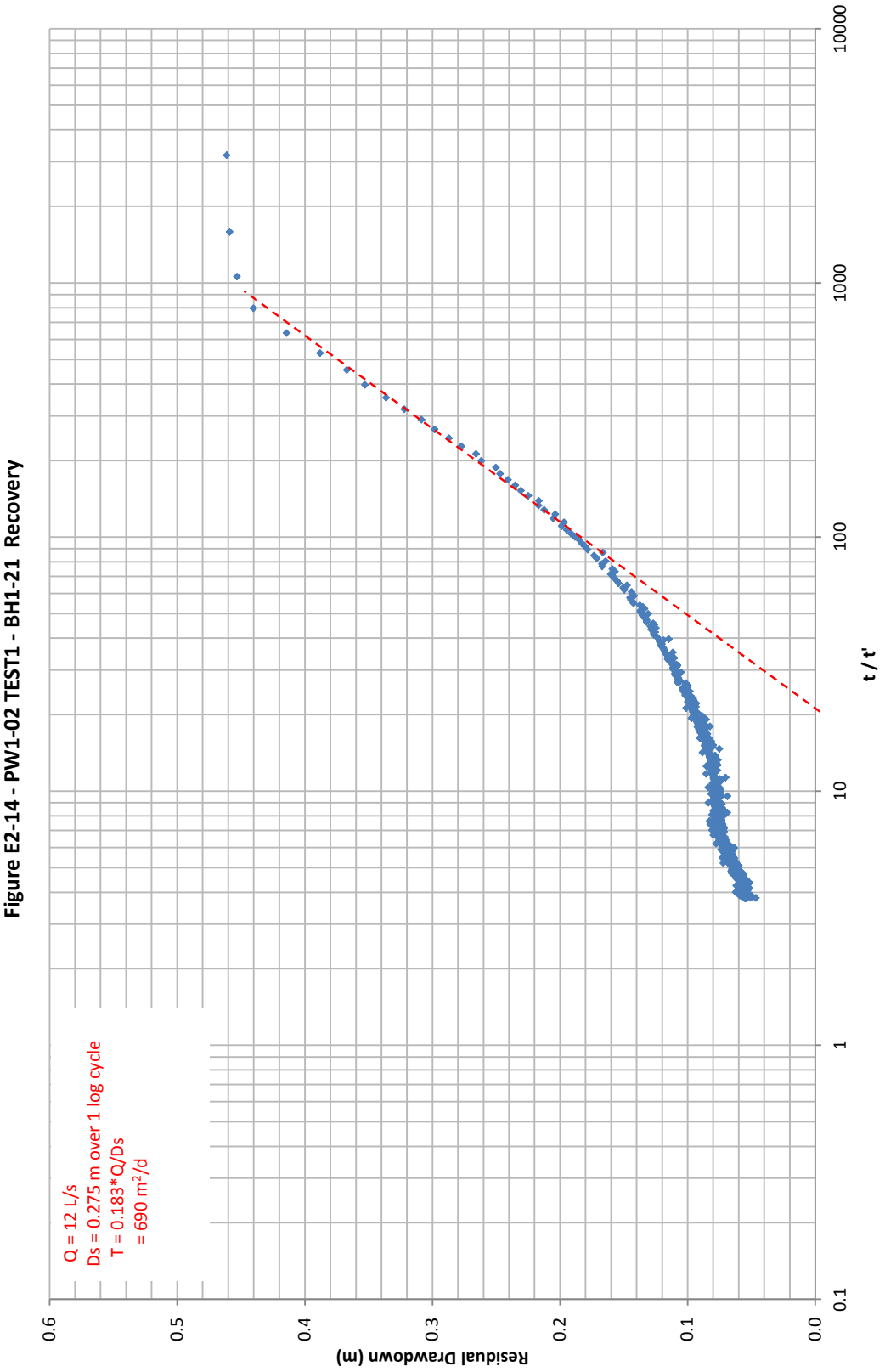


Figure E2-14 - PW1-02 TEST1 - BH1-21 Recovery

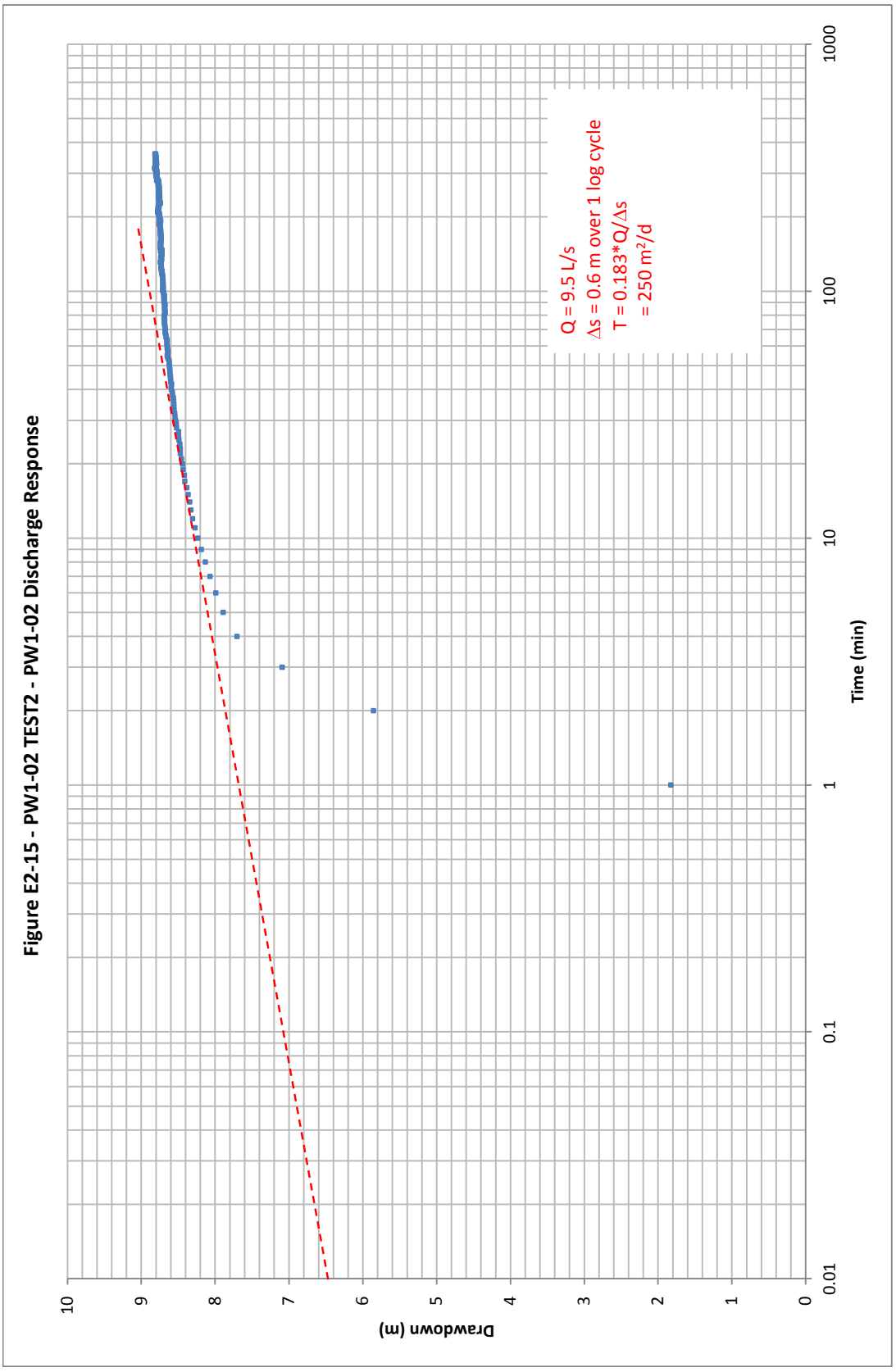


Figure E2-15 - PW1-02 TEST2 - PW1-02 Discharge Response
Drawdown (m)
0
1
2
3
4
5
6
7
8
9
10
Time (min)
0.01
0.1
1
10
100
1000
Q = 9.5 L/s
Δs = 0.6 m over 1 log cycle
T = 0.183*Q/Δs
= 250 m²/d

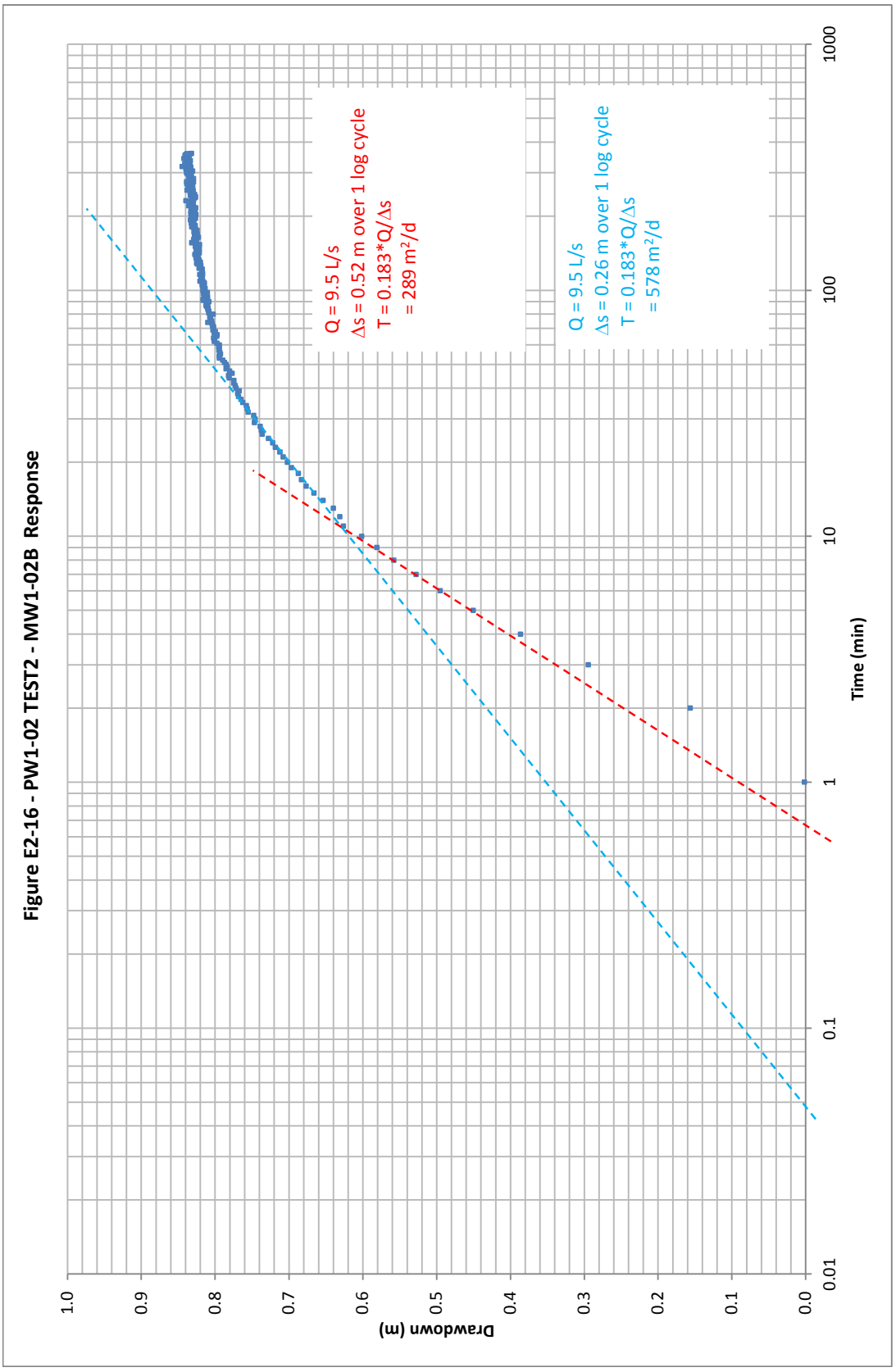


Figure E2-16 - PW1-02 TEST2 - MW1-02B Response
Drawdown (m)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0.01
0.1
1
10
100
1000
Time (min)
Q = 9.5 L/s
Δs = 0.52 m over 1 log cycle
T = 0.183*Q/Δs
= 289 m²/d
Q = 9.5 L/s
Δs = 0.26 m over 1 log cycle
T = 0.183*Q/Δs
= 578 m²/d

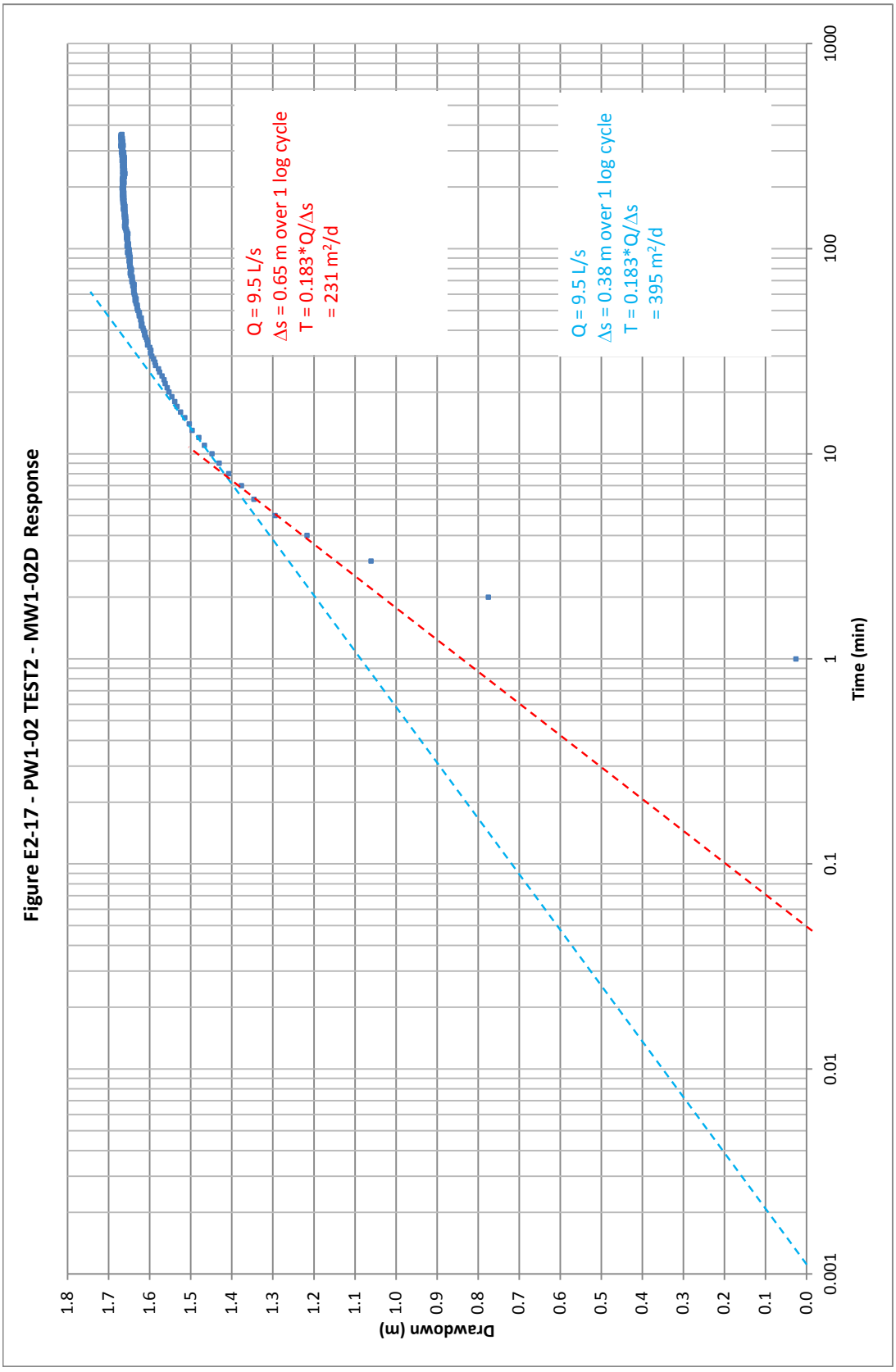


Figure E2-17 - PW1-02 TEST2 - MW1-02D Response
Drawdown (m)
Time (min)
Q = 9.5 L/s
Δs = 0.65 m over 1 log cycle
T = 0.183*Q/Δs
= 231 m²/d
Q = 9.5 L/s
Δs = 0.38 m over 1 log cycle
T = 0.183*Q/Δs
= 395 m²/d

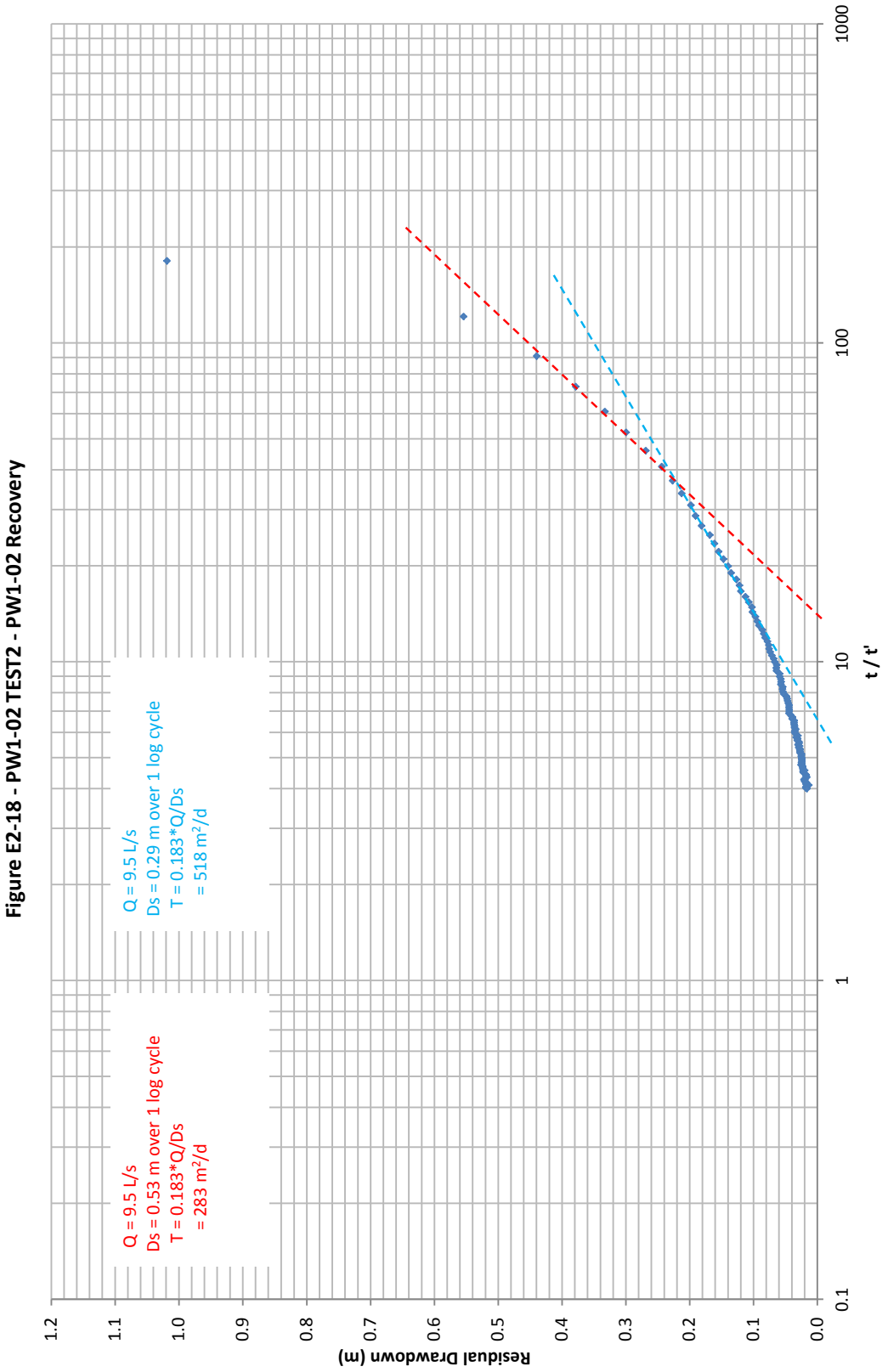


Figure E2-18 - PW1-02 TEST2 - PW1-02 Recovery

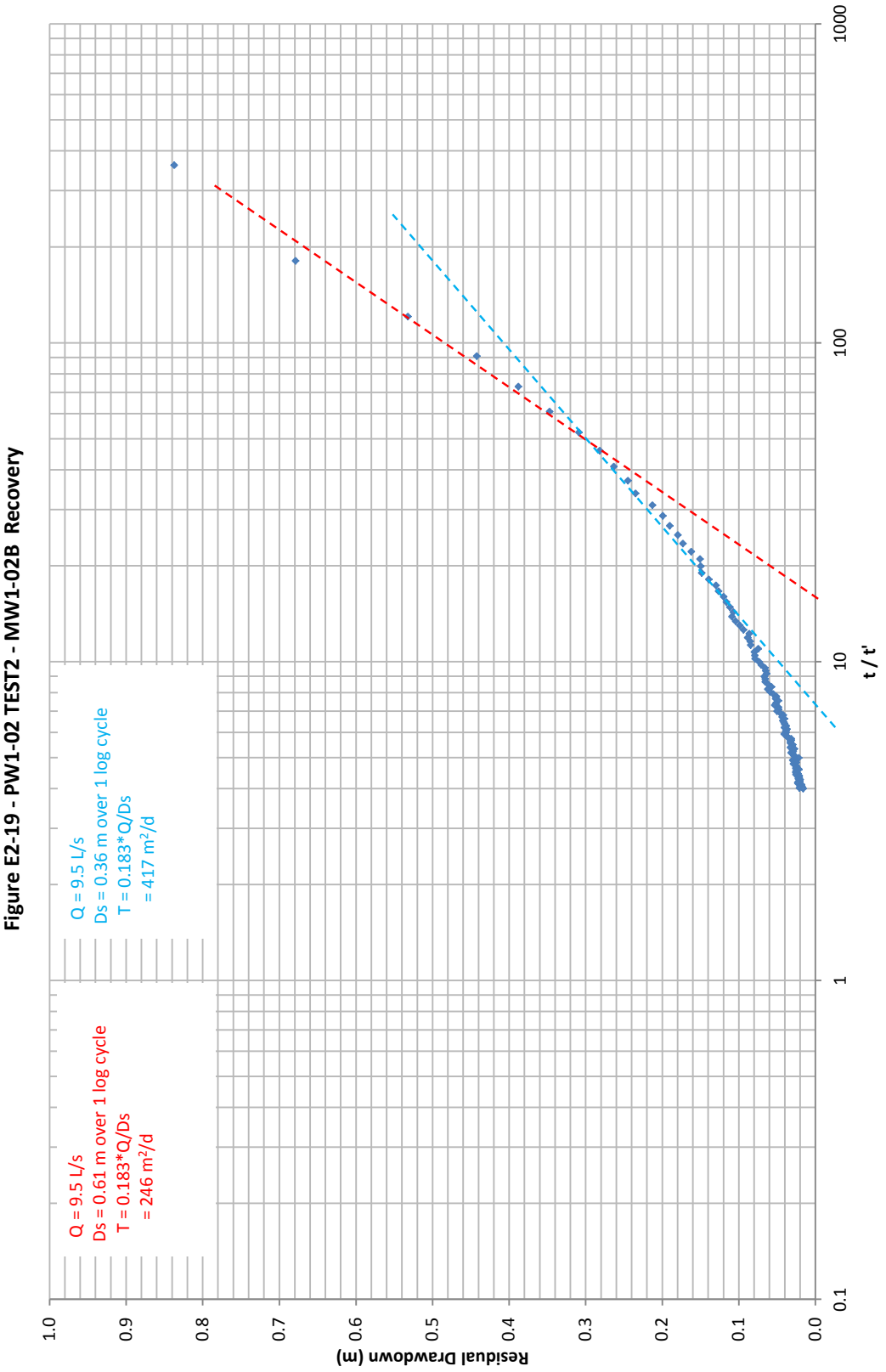


Figure E2-19 - PW1-02 TEST2 - MW1-02B Recovery
Residual Drawdown (m)
t / t'
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0.1
1
10
100
1000
Q = 9.5 L/s
Ds = 0.61 m over 1 log cycle
T = 0.183*Q/Ds
= 246 m²/d
Q = 9.5 L/s
Ds = 0.36 m over 1 log cycle
T = 0.183*Q/Ds
= 417 m²/d

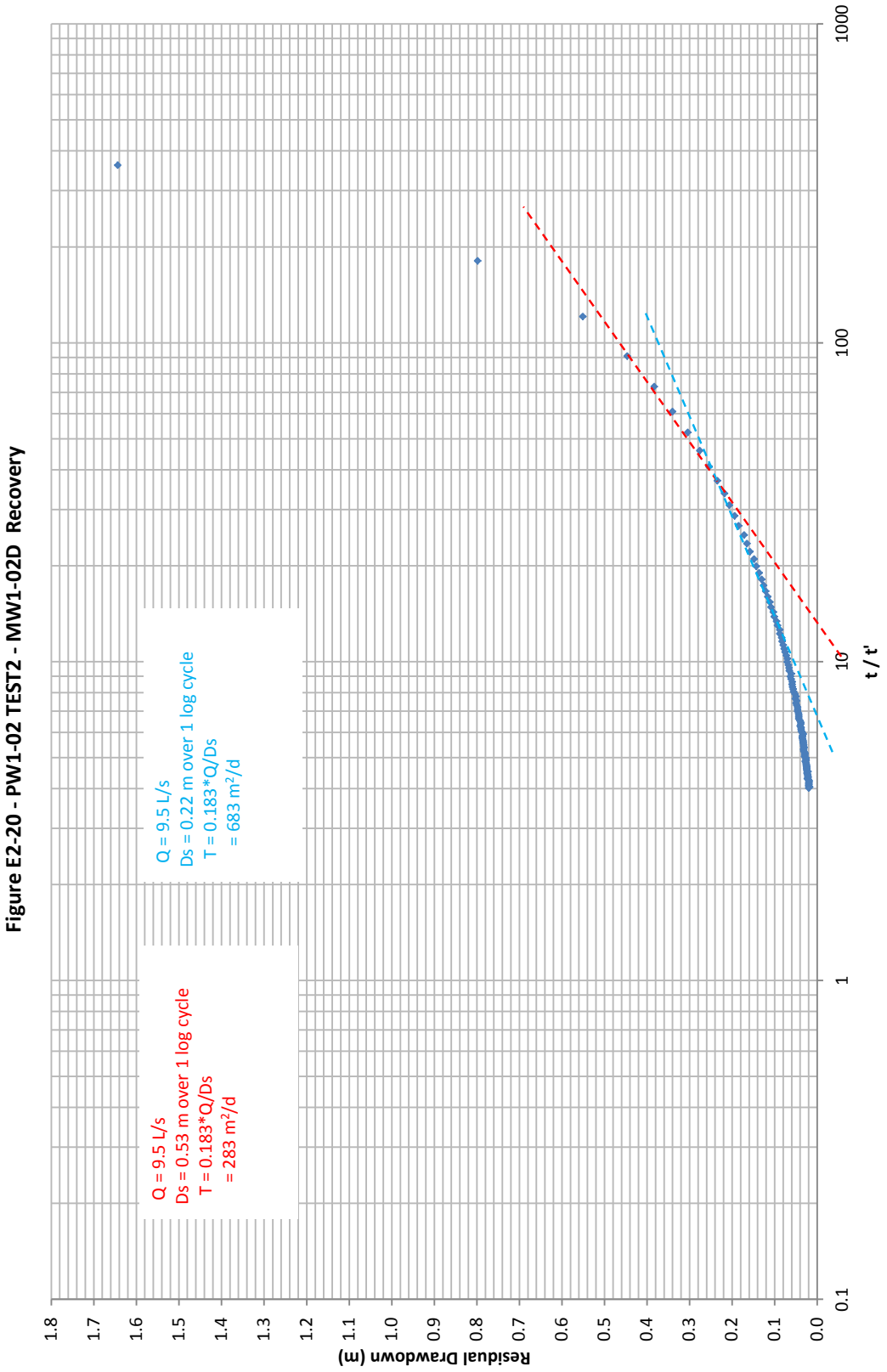


Figure E2-20 - PW1-02 TEST2 - MW1-02D Recovery
Residual Drawdown (m)
t / t'
Q = 9.5 L/s
Ds = 0.53 m over 1 log cycle
T = 0.183*Q/Ds
= 283 m²/d
Q = 9.5 L/s
Ds = 0.22 m over 1 log cycle
T = 0.183*Q/Ds
= 683 m²/d

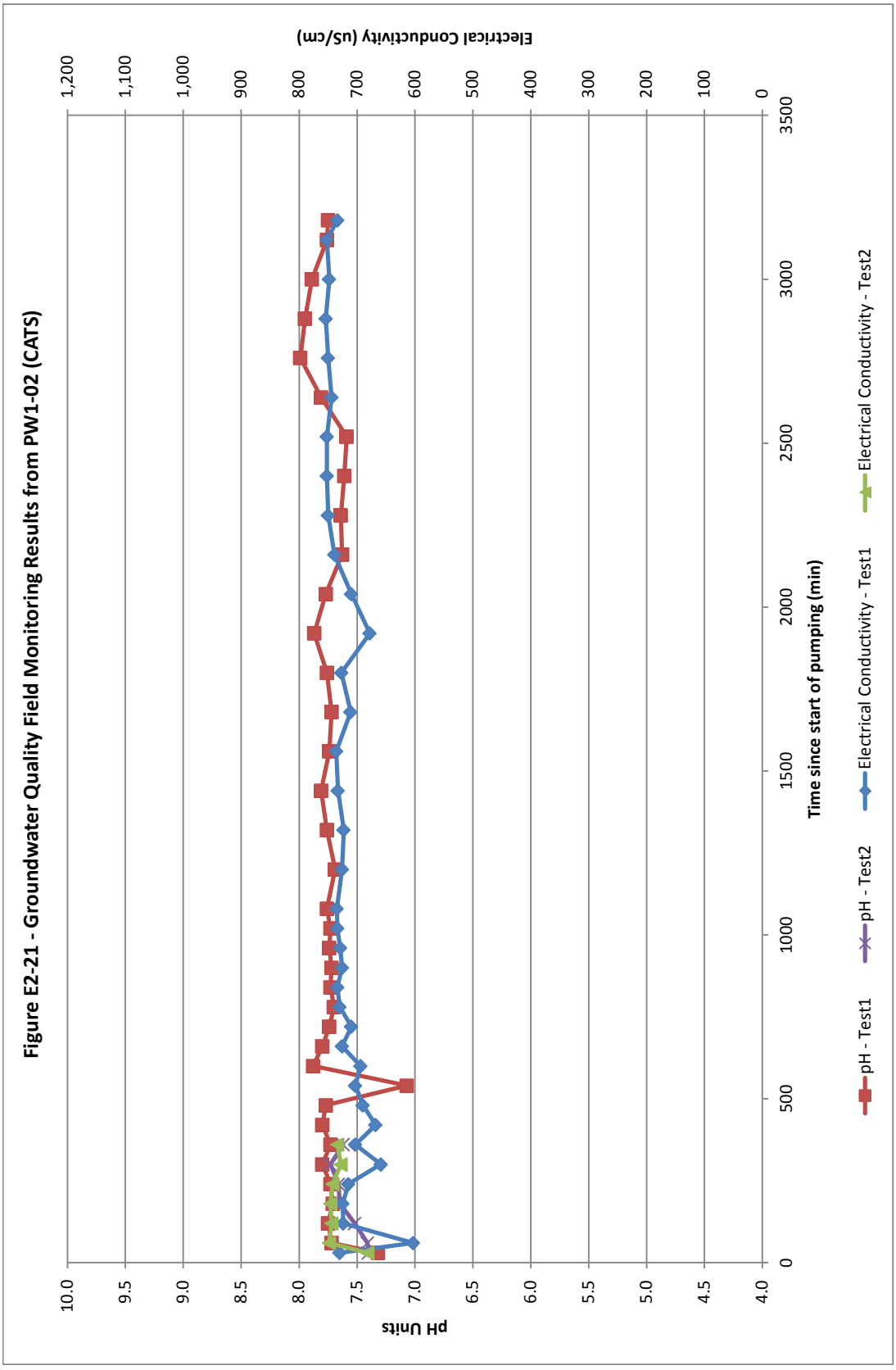
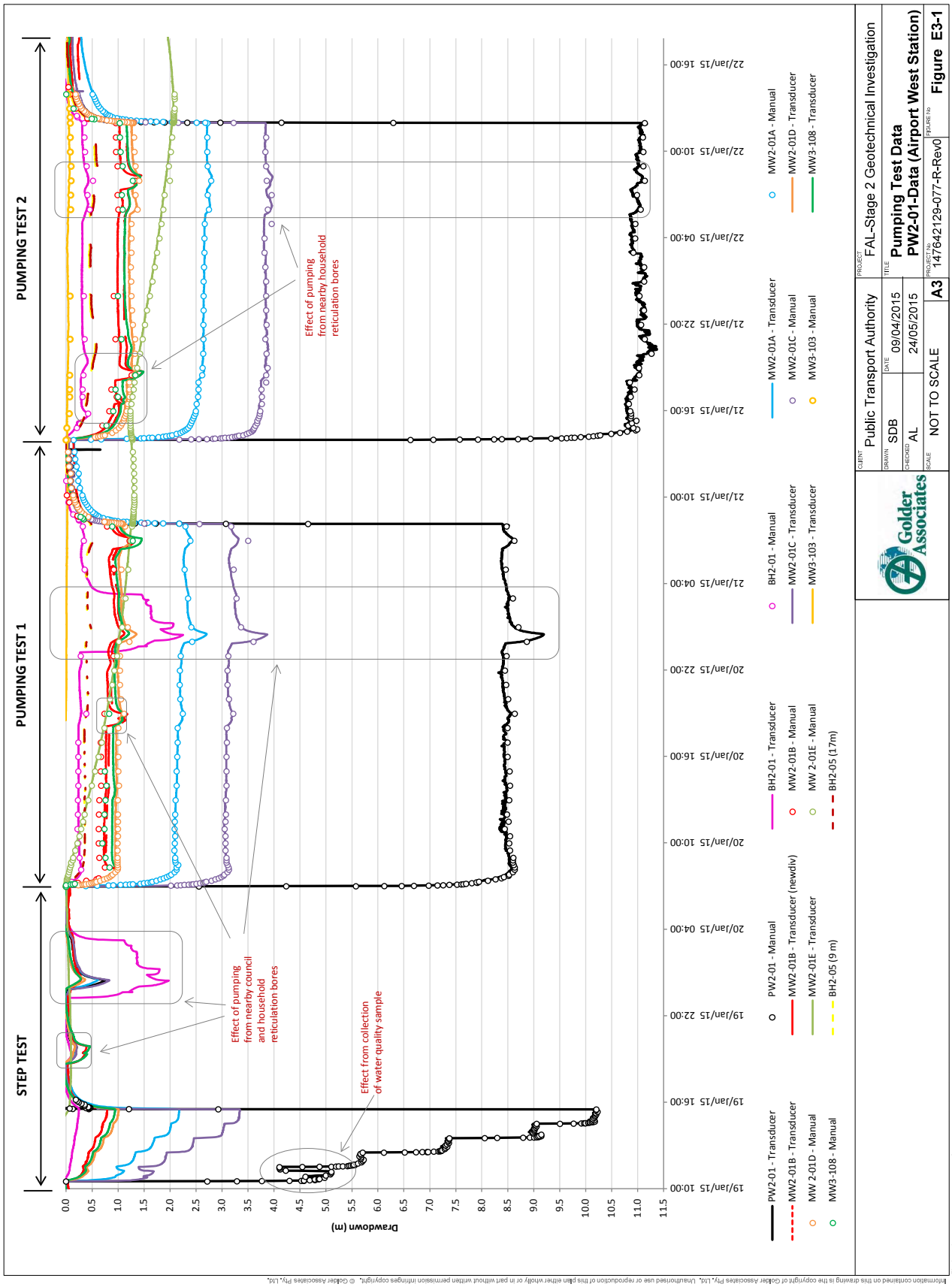


Figure E2-21 - Groundwater Quality Field Monitoring Results from PW1-02 (CATS)
pH Units
Electrical Conductivity (uS/cm)
Time since start of pumping (min)
pH - Test1
pH - Test2
Electrical Conductivity - Test1
Electrical Conductivity - Test2



APPENDIX E
Summary of Test Pumping Analysis

ATTACHMENT E3
Constant Rate Test – Airport West Station (PW2-01)



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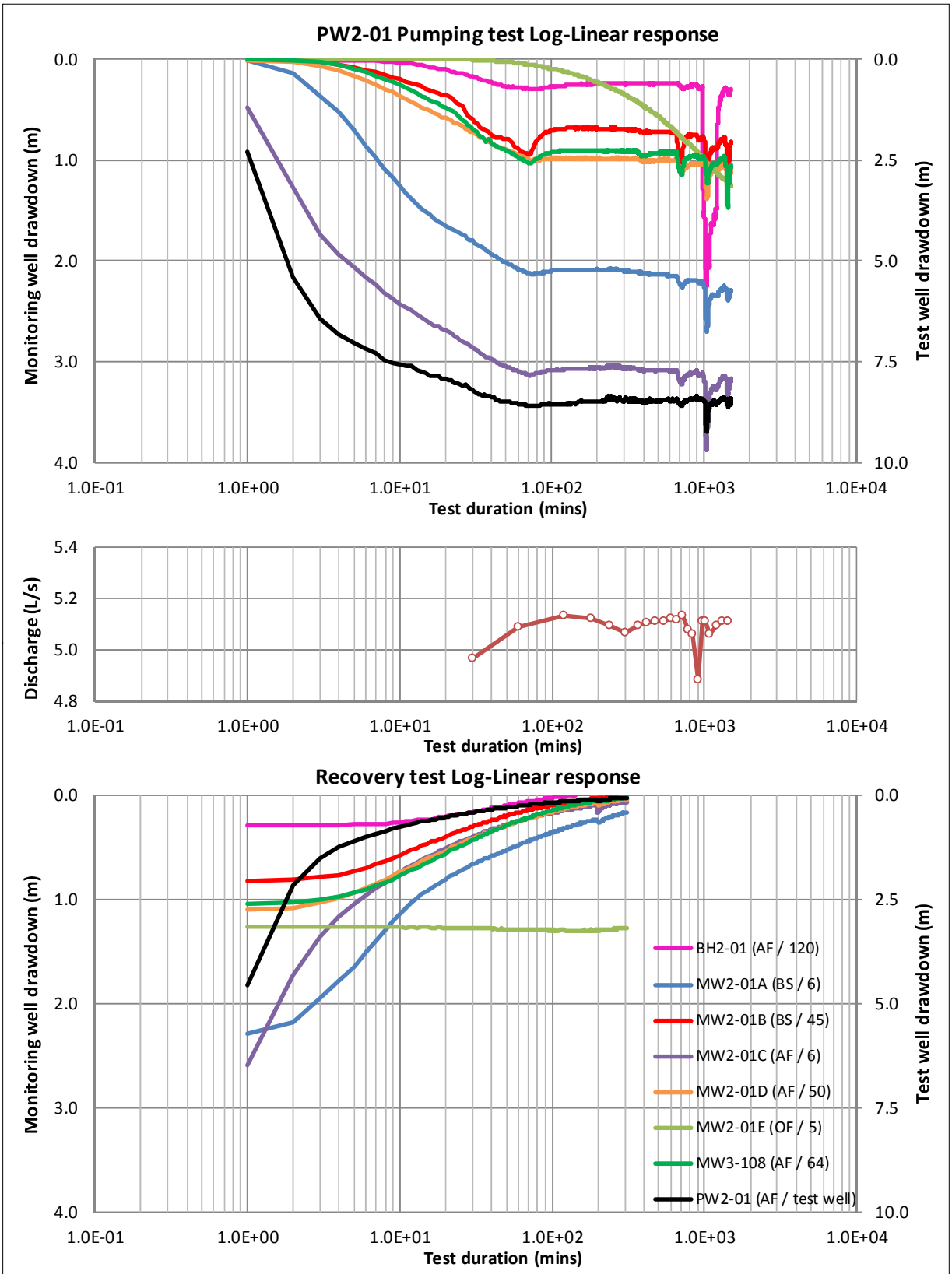

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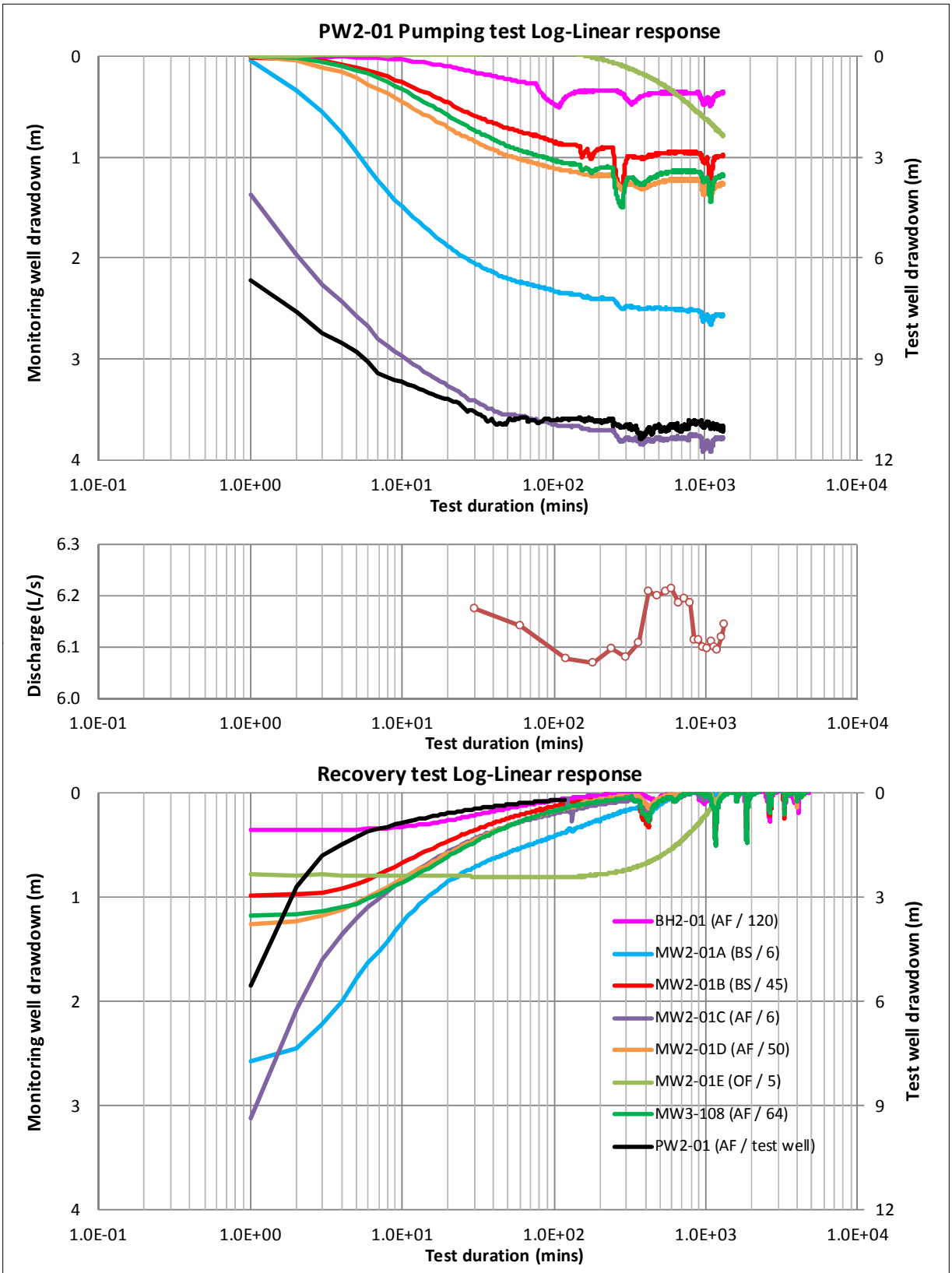
Golder Associates
Level 3, 1 Havelock St West Perth WA 6005
Ph: +618 9213 7600 Fx: +618 9213 7611

CLIENT	Public Transport Authority	PROJECT	FAL-Stage 2 Geotechnical Investigation
DRAWN	SDB	DATE 24/05/2015	TITLE Pumping Test Data
CHECKED	AL	DATE 24/05/2015	PW2-01-Test 1 (Airport West Station)
SCALE	NOT TO SCALE	A4	PROJECT No 147642129-077-R-Rev0
			FIGURE No Figure E3-2

File Location:



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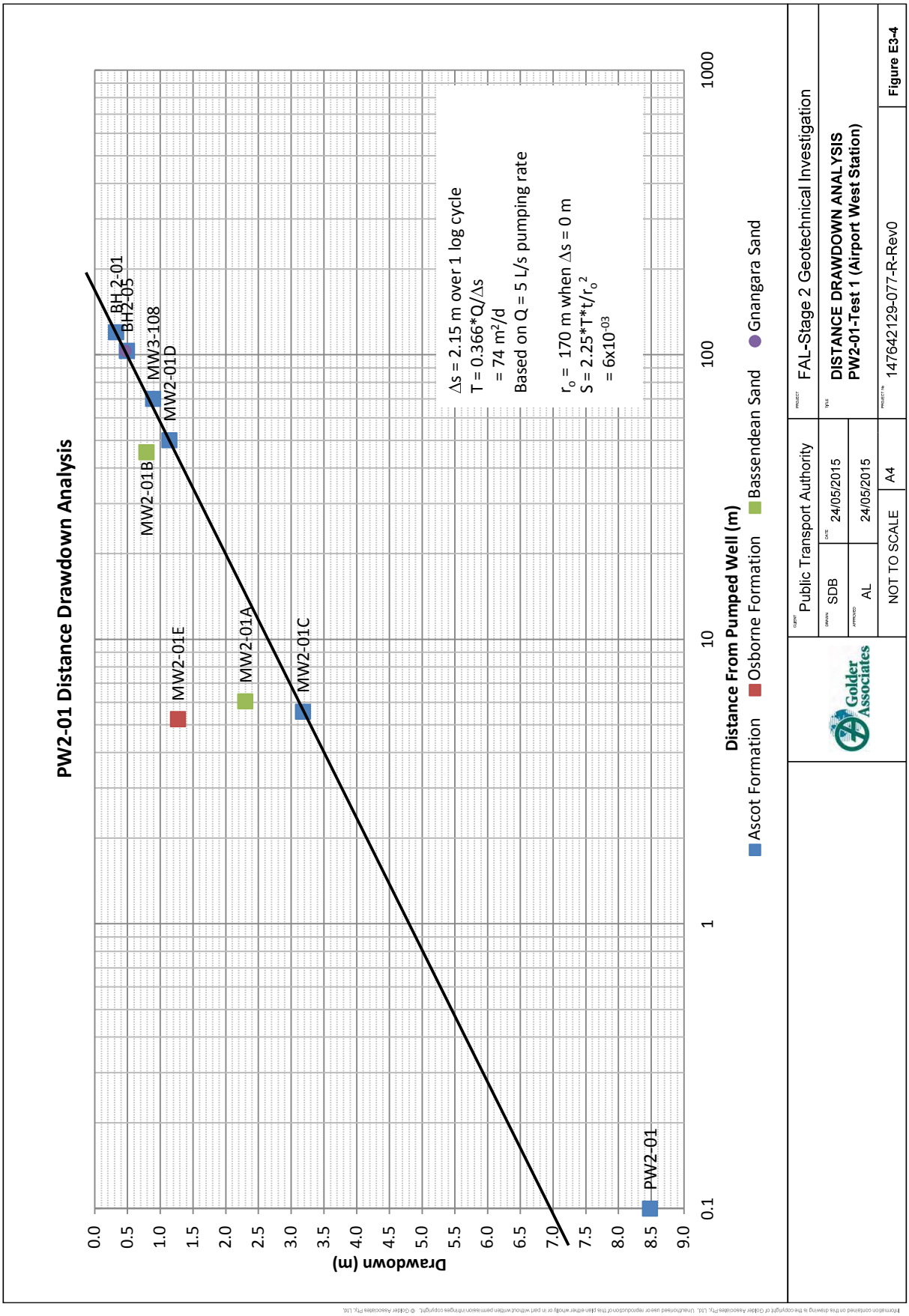
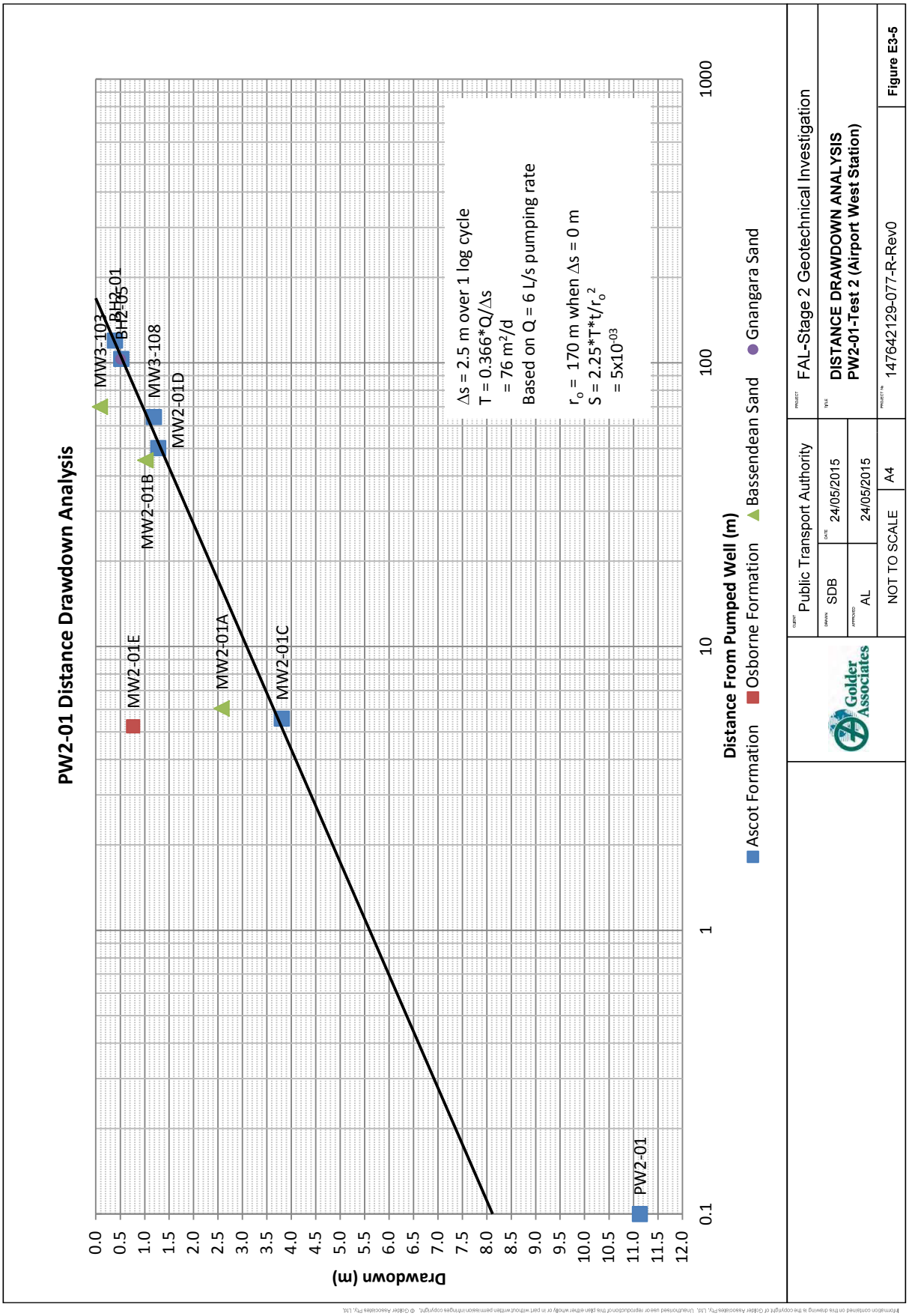
Golder Associates
Level 3, 1 Havelock St West Perth WA 6005
Ph: +618 9213 7600 Fx: +618 9213 7611

CLIENT	Public Transport Authority	PROJECT	FAL-Stage 2 Geotechnical Investigation
DRAWN	SDB	DATE 24/05/2015	TITLE **Pumping Test Data**
CHECKED	AL	DATE 24/05/2015	**PW2-01-Test 2 (Airport West Station)**
SCALE	NOT TO SCALE	A4	PROJECT No 147642129-077-R-Rev0
FIGURE No	**Figure E3-3**		

File Location:

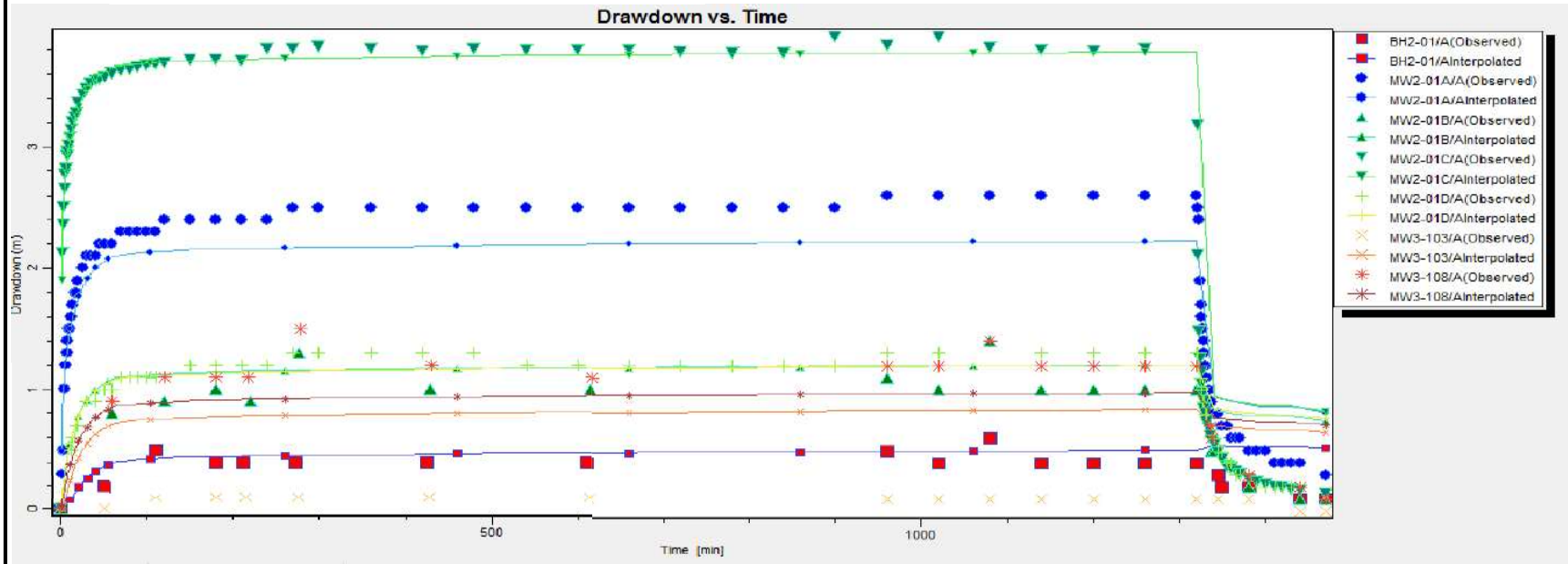

PW2-01 Distance Drawdown Analysis
Drawdown (m)
Distance From Pumped Well (m)
Ascot Formation
Osborne Formation
Bassendean Sand
Gnangara Sand
PW2-01
MW2-01C
MW2-01A
MW2-01E
MW2-01B
MW3-108
MW2-01D
BH2-05
BH2-01
Δs = 2.15 m over 1 log cycle
T = 0.366*Q/Δs
= 74 m²/d
Based on Q = 5 L/s pumping rate
r₀ = 170 m when Δs = 0 m
S = 2.25*T*t/r₀²
= 6x10⁻⁰³
Public Transport Authority
FAL-Stage 2 Geotechnical Investigation
DISTANCE DRAWDOWN ANALYSIS
PW2-01-Test 1 (Airport West Station)
SDB
AL
24/05/2015
24/05/2015
NOT TO SCALE
A4
147642129-077-R-Rev0
Figure E3-4


PW2-01 Distance Drawdown Analysis
Drawdown (m)
Distance From Pumped Well (m)
Ascot Formation
Osborne Formation
Bassendean Sand
Gnangara Sand
MW3-103
BH2-05
BH2-01
MW3-108
MW2-01D
MW2-01B
MW2-01A
MW2-01E
MW2-01C
PW2-01
Δs = 2.5 m over 1 log cycle
T = 0.366*Q/Δs
= 76 m²/d
Based on Q = 6 L/s pumping rate
r_o = 170 m when Δs = 0 m
S = 2.25*T*t/r_o²
= 5x10⁻⁰³
Public Transport Authority
FAL-Stage 2 Geotechnical Investigation
DISTANCE DRAWDOWN ANALYSIS
PW2-01-Test 2 (Airport West Station)
SDB
24/05/2015
AL
24/05/2015
NOT TO SCALE
A4
147642129-077-R-Rev0
Figure E3-5

Specific Storage =	1.00E-05	1/m
Transmissivity =	76	m2/d
Hydraulic Conductivity		
K(BS,GF,GS)h	3	m/d
K(BS,GF,GS)v	0.3	m/d
K(AF)h	8	m/d
K(AF)v	0.8	m/d
K(OF)h	0.0005	m/d
K(OF)v	0.00005	m/d
River Conductance =	9	m2/d

River Leakage

Time (min)	Rate (L/S)
10	0.1
20	0.2
30	0.3
40	0.3
50	0.3
120	0.4
320	0.4
520	0.4
920	0.4
1120	0.4
1320	0.4
1470	0.4



Screenshot from MODFLOW output

Note: Other combinations in hydraulic conductivity and storage could may be possible

Golder Associates

CLIENT	Public Transport authority			PROJECT	FAL - Stage 2 Geotechnical Investigation
DRAWN	AL	DATE	24-May-15	TITLE	Analysis of Pumping Test 2 using Numerical Groundwater Model Airport West Station
CHECK	DPS	DATE	24-May-15		
SCALE	Not To Scale	A4		PROJECT No.	147642129-077-R-Rev0
				FIGURE No.	E3-6

C:\Users\alundorf\Desktop\FAL\APPENDIX E\Appendix E - Test Pumping Results\Attachment E3\Figure E3-6.xlsx

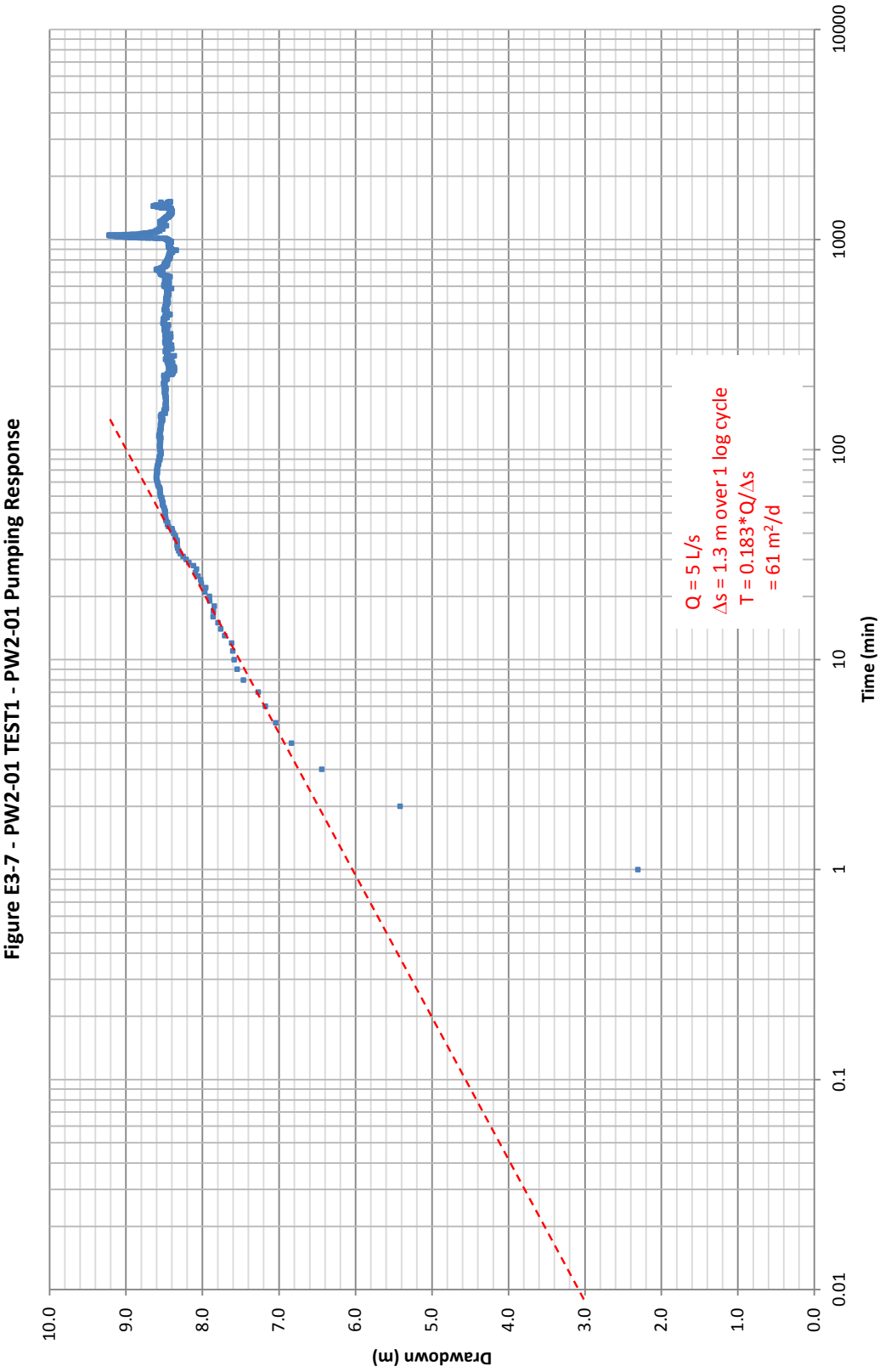


Figure E3-7 - PW2-01 TEST1 - PW2-01 Pumping Response
Drawdown (m)
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 5 L/s
Δs = 1.3 m over 1 log cycle
T = 0.183*Q/Δs
= 61 m²/d

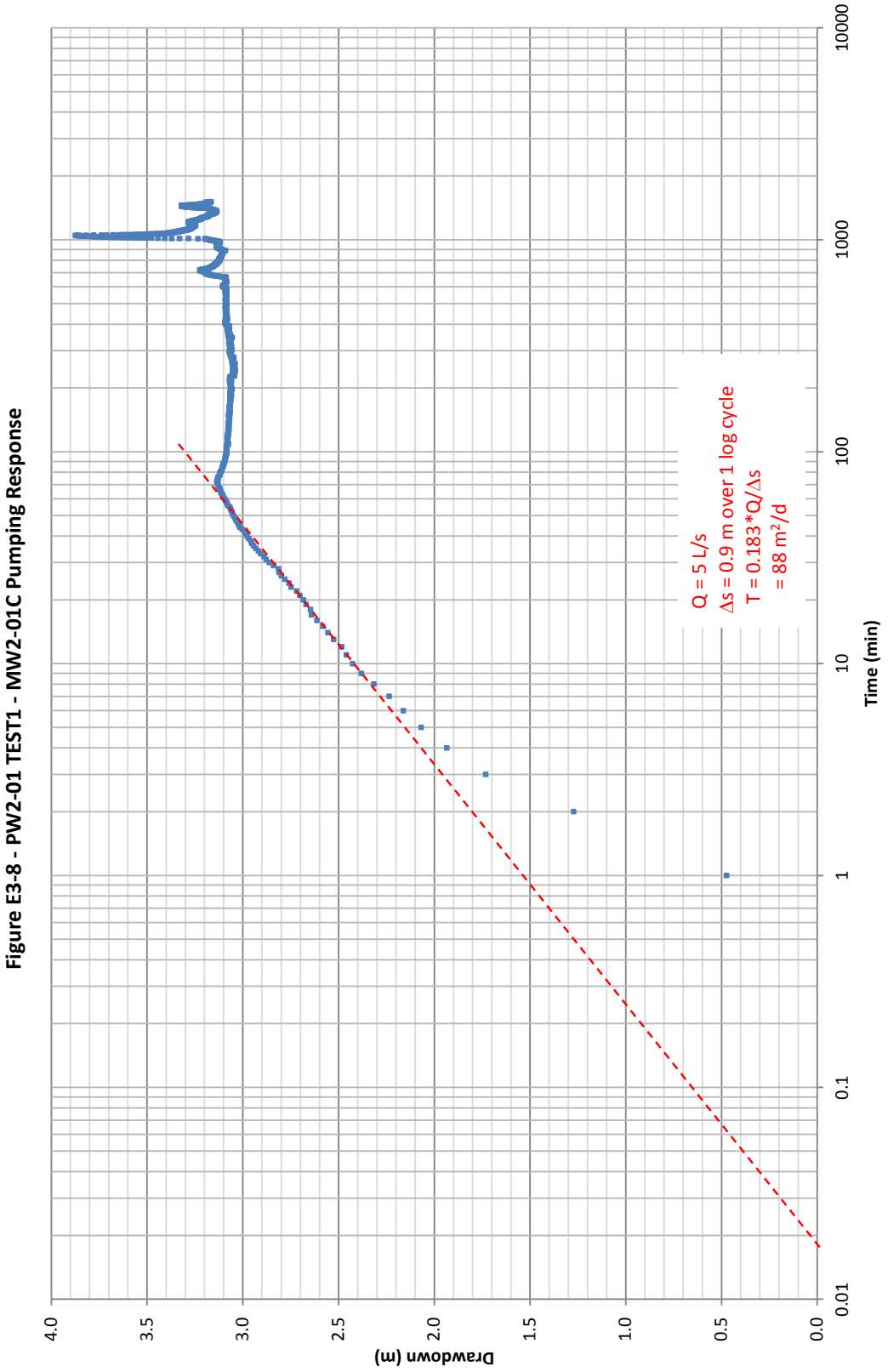


Figure E3-8 - PW2-01 TEST1 - MW2-01C Pumping Response
Drawdown (m)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 5 L/s
Δs = 0.9 m over 1 log cycle
T = 0.183*Q/Δs
= 88 m²/d

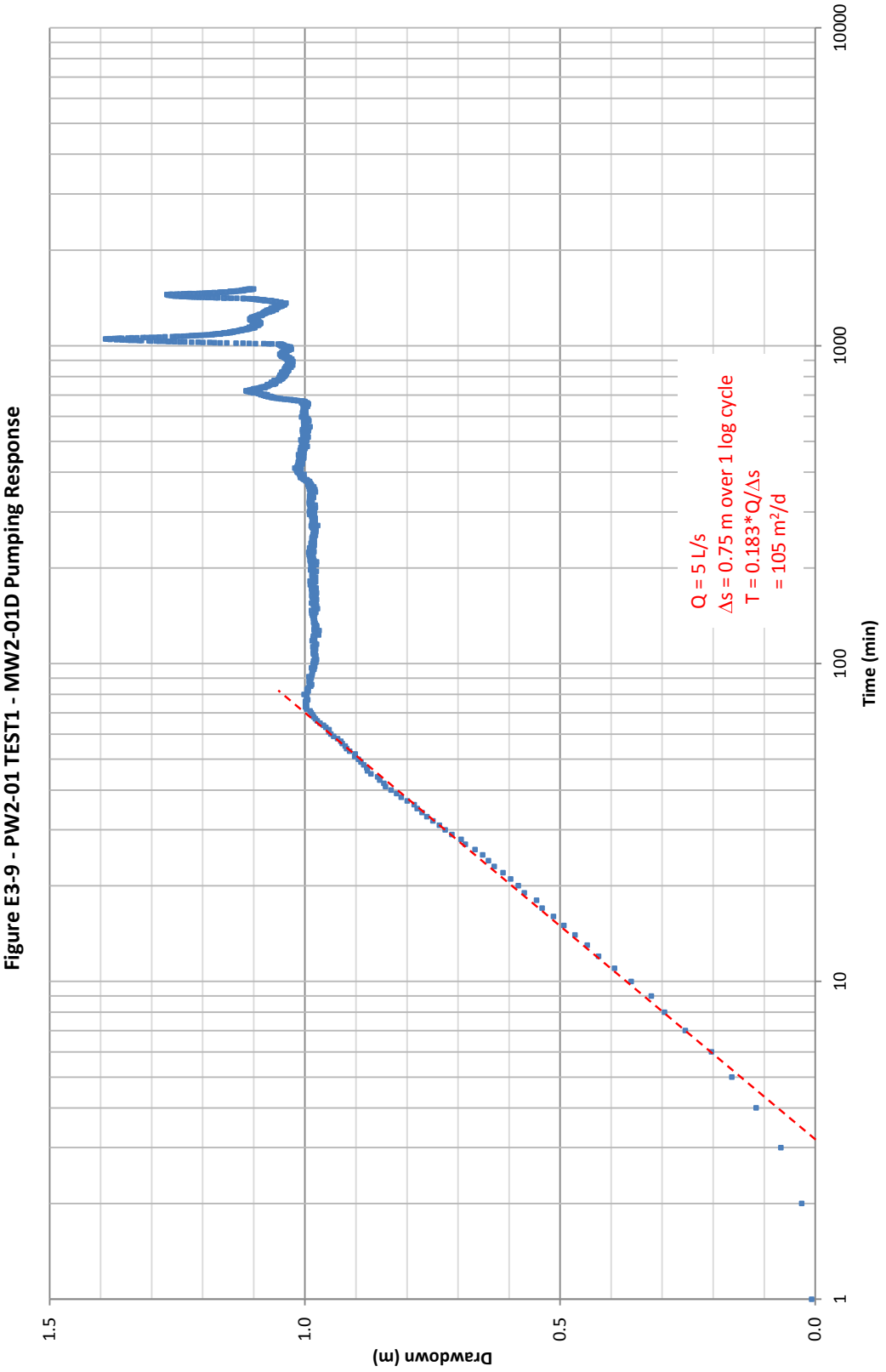


Figure E3-9 - PW2-01 TEST1 - MW2-01D Pumping Response
Drawdown (m)
1.5
1.0
0.5
0.0
Time (min)
1
10
100
1000
10000
Q = 5 L/s
Δs = 0.75 m over 1 log cycle
T = 0.183*Q/Δs
= 105 m²/d

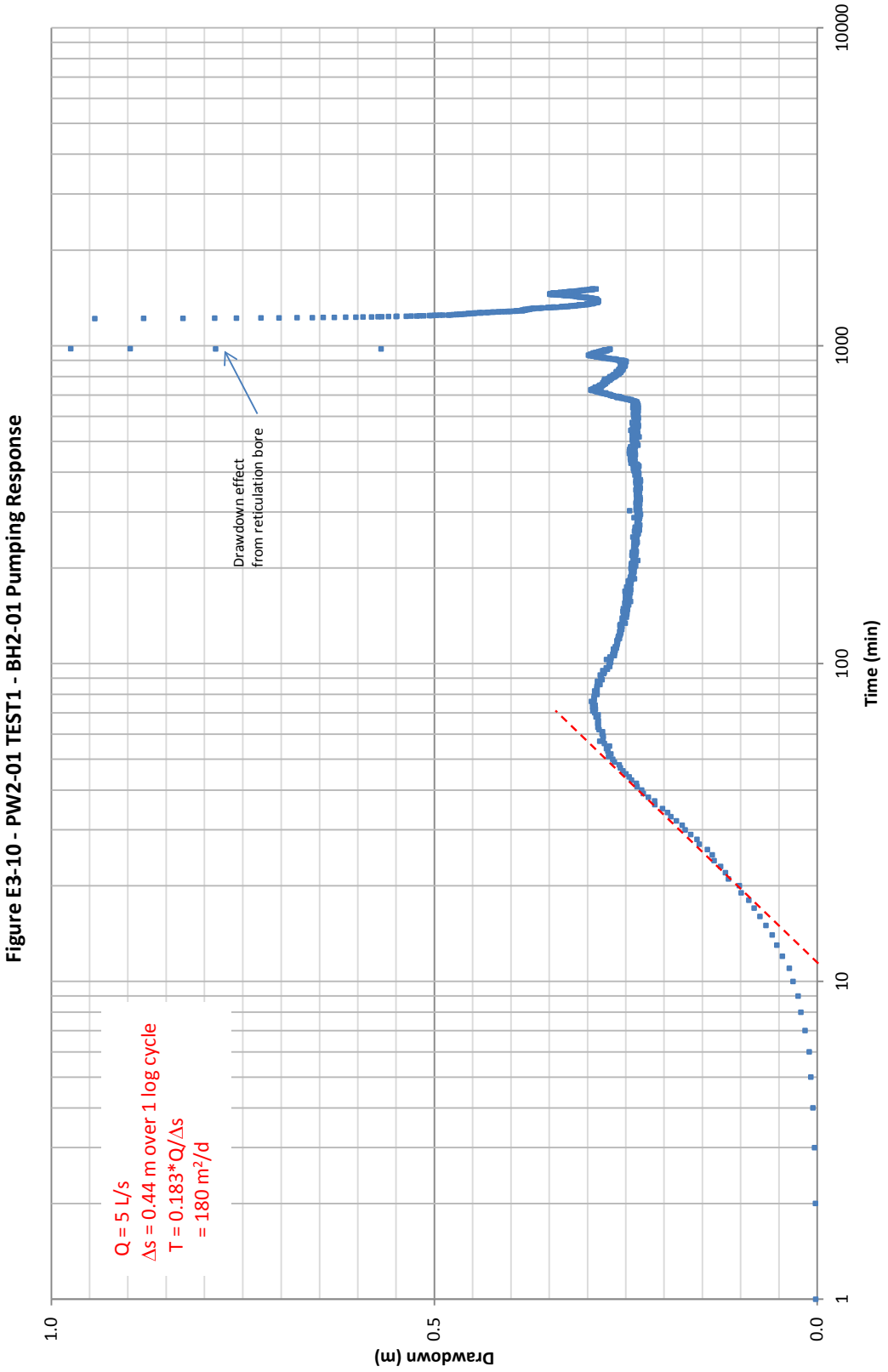


Figure E3-10 - PW2-01 TEST1 - BH2-01 Pumping Response
Drawdown (m)
1.0
0.5
0.0
Time (min)
1
10
100
1000
10000
Drawdown effect
from reticulation bore
Q = 5 L/s
Δs = 0.44 m over 1 log cycle
T = 0.183*Q/Δs
= 180 m²/d

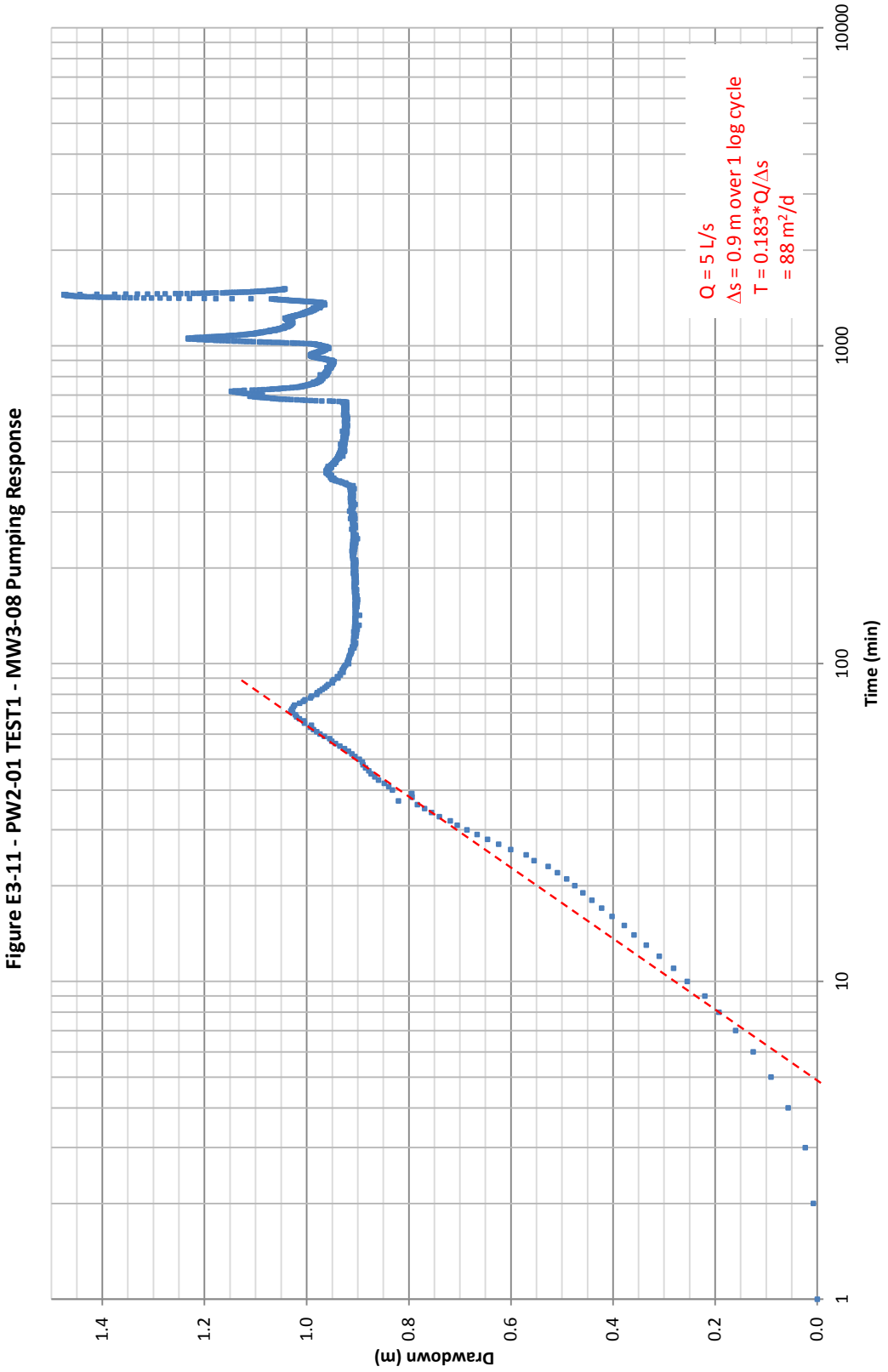


Figure E3-11 - PW2-01 TEST1 - MW3-08 Pumping Response
Drawdown (m)
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
Time (min)
1
10
100
1000
10000
Q = 5 L/s
Δs = 0.9 m over 1 log cycle
T = 0.183*Q/Δs
= 88 m²/d

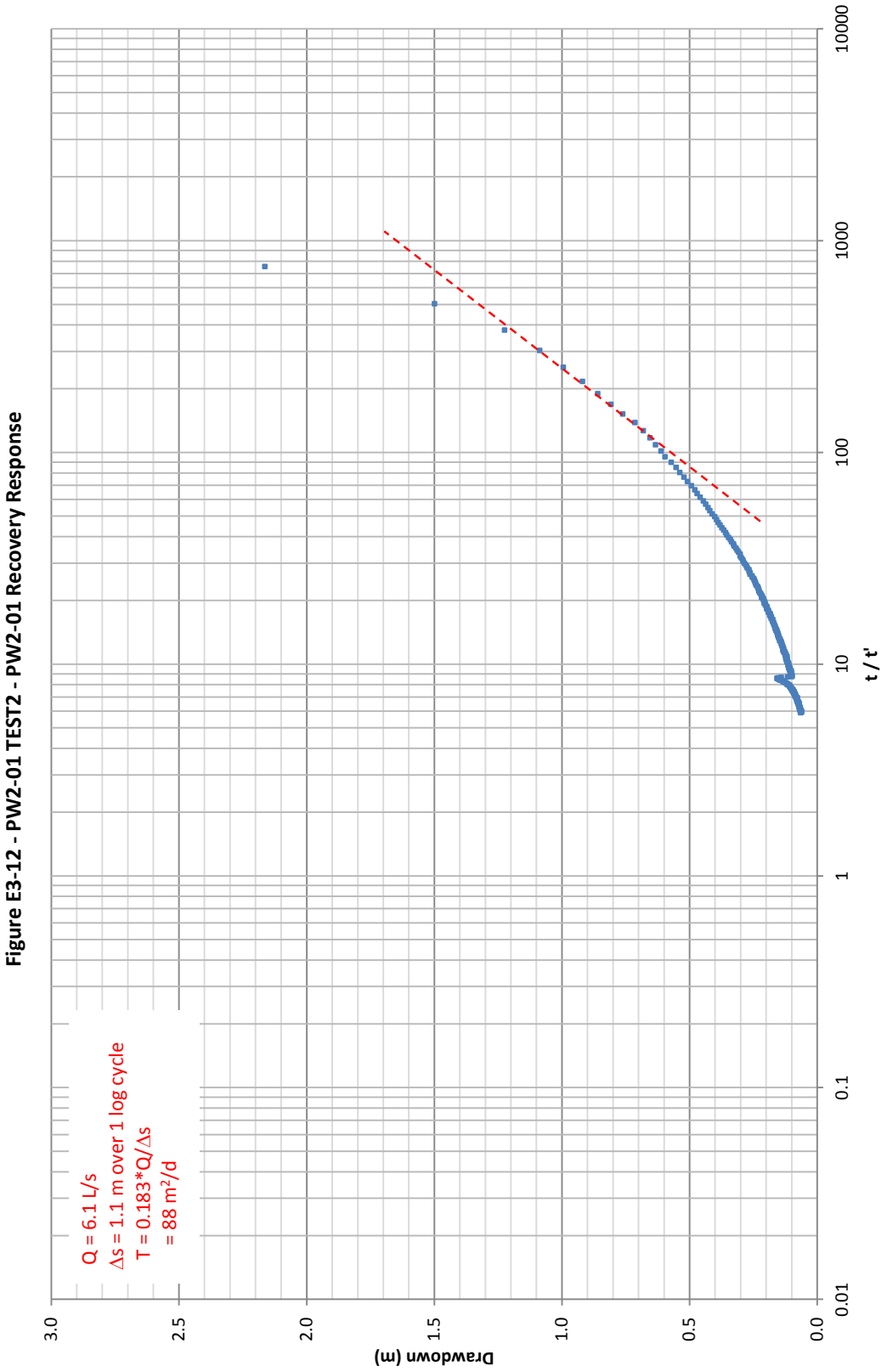


Figure E3-12 - PW2-01 TEST2 - PW2-01 Recovery Response

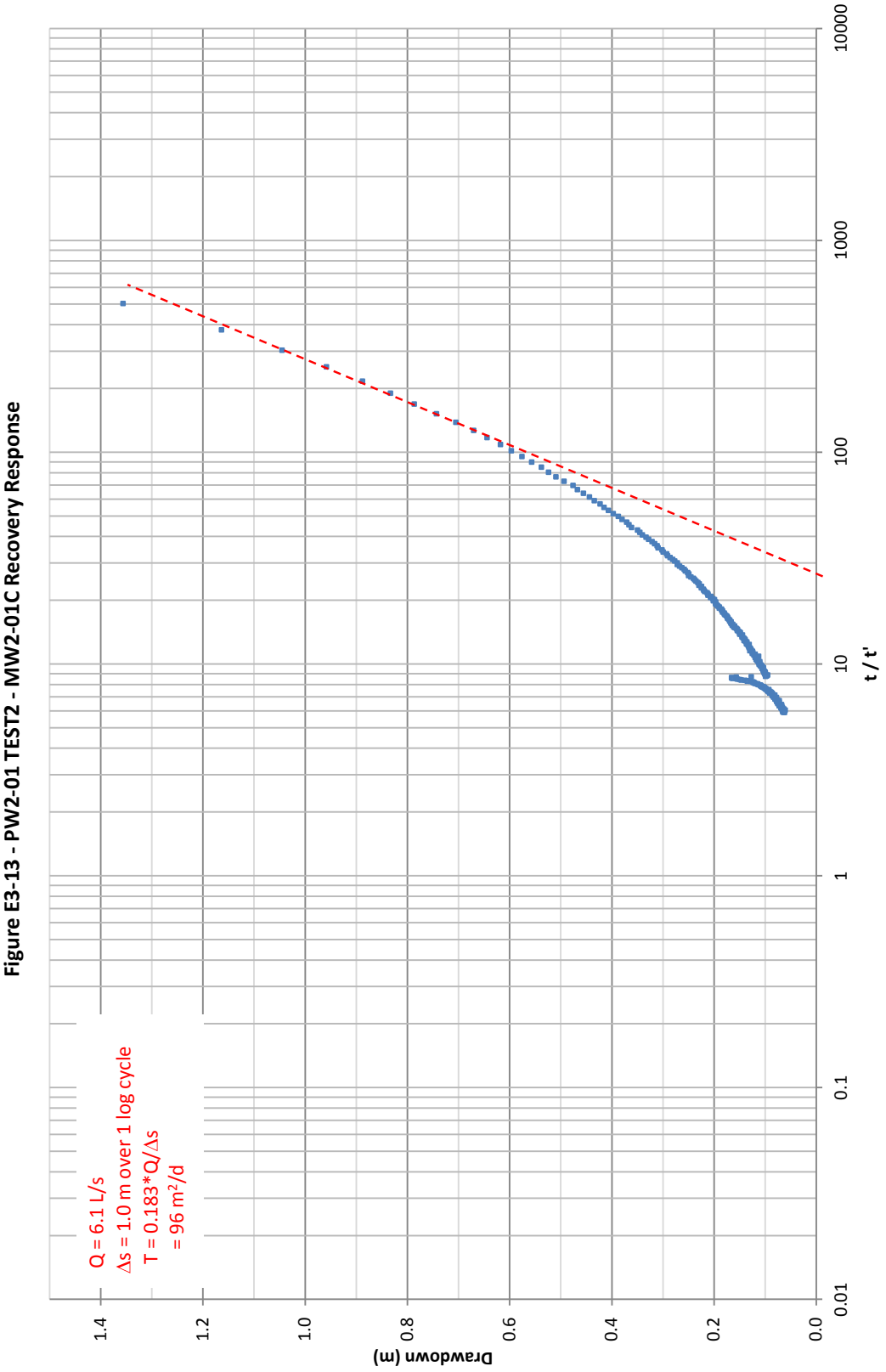


Figure E3-13 - PW2-01 TEST2 - MW2-01C Recovery Response
Drawdown (m)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.01
0.1
1
10
100
1000
10000
t / t'
Q = 6.1 L/s
Δs = 1.0 m over 1 log cycle
T = 0.183*Q/Δs
= 96 m²/d

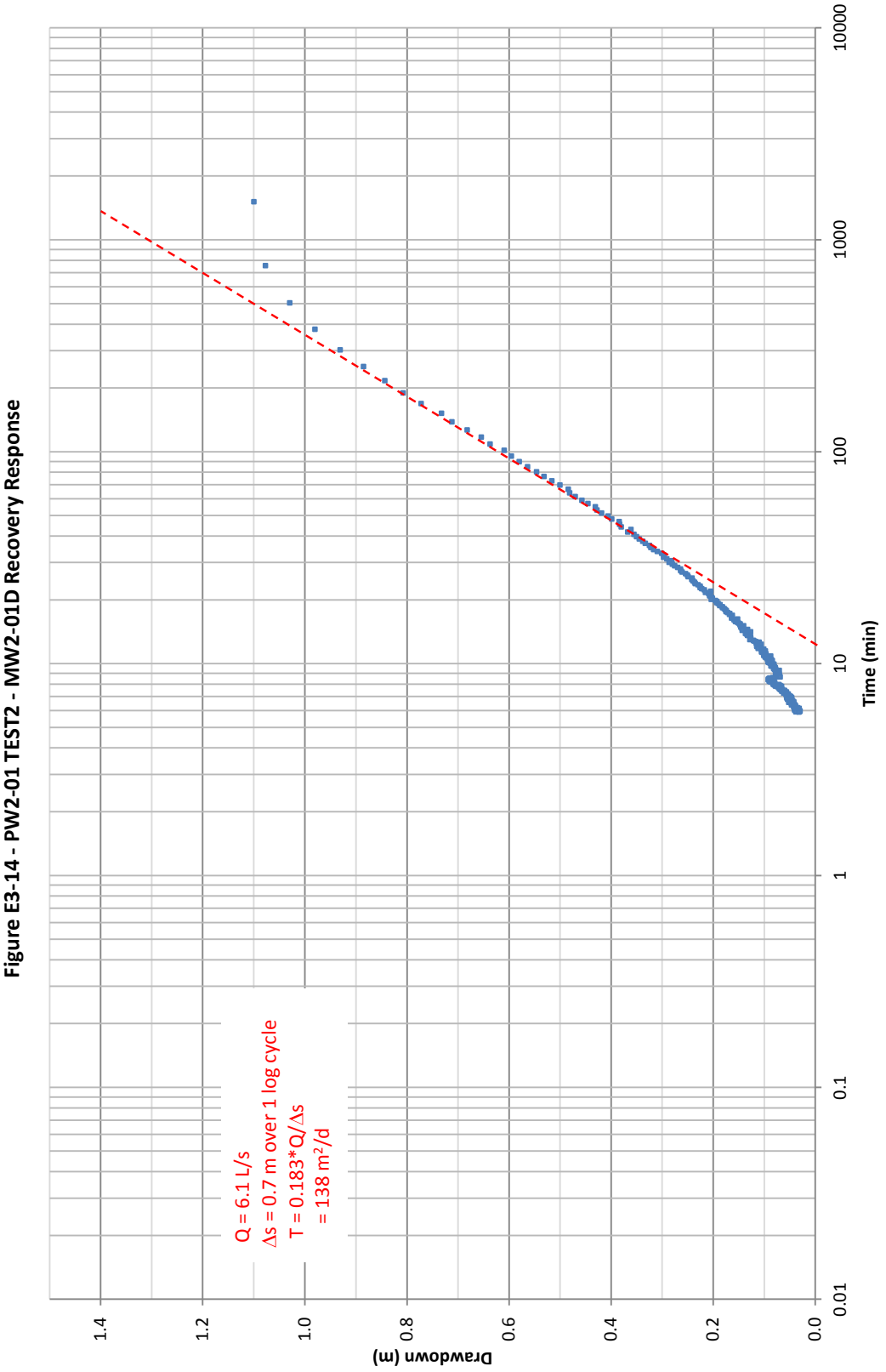


Figure E3-14 - PW2-01 TEST2 - MW2-01D Recovery Response
Drawdown (m)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.01
0.1
1
10
100
1000
10000
Time (min)
Q = 6.1 L/s
Δs = 0.7 m over 1 log cycle
T = 0.183*Q/Δs
= 138 m²/d

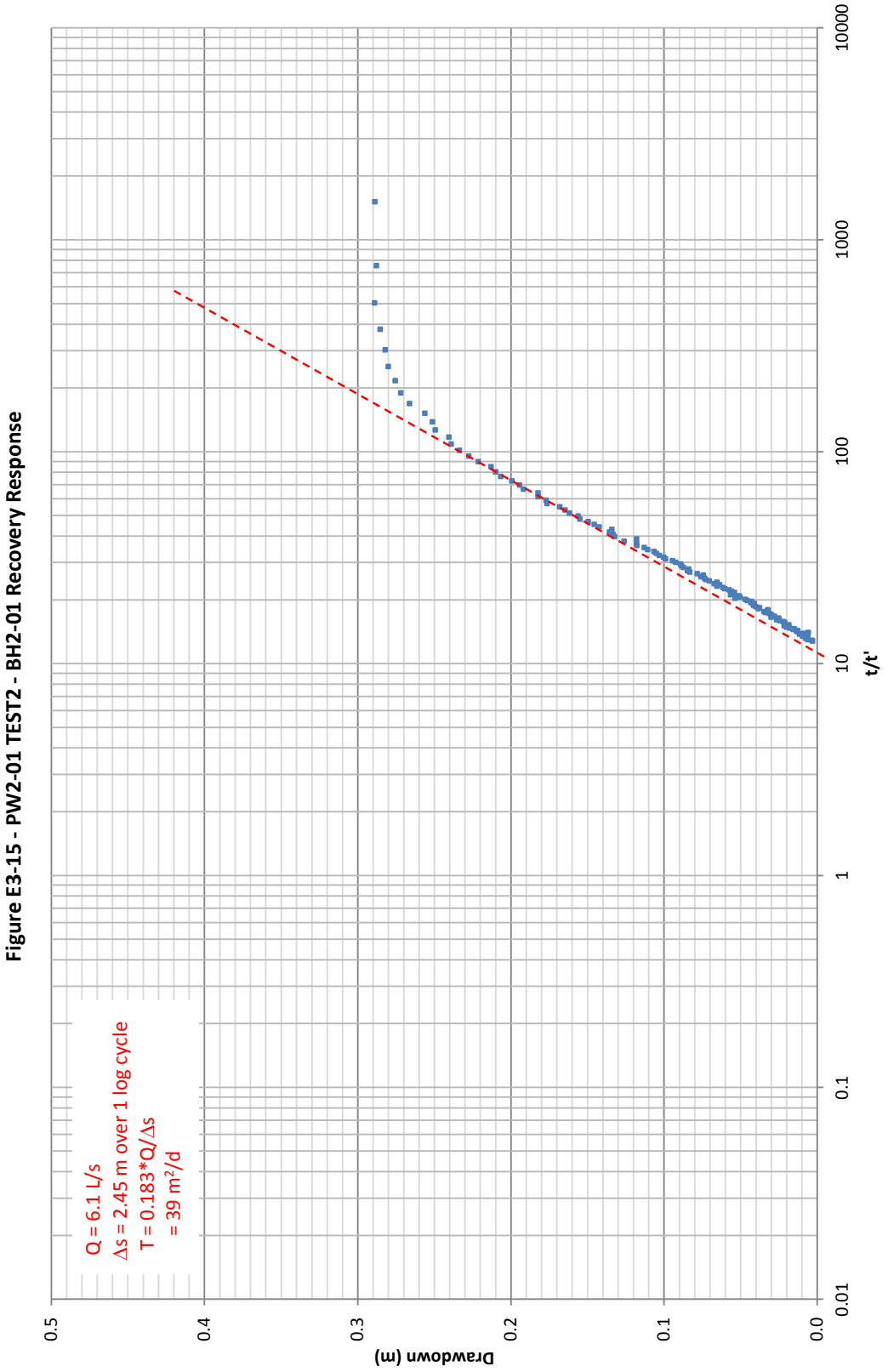


Figure E3-15 - PW2-01 TEST2 - BH2-01 Recovery Response

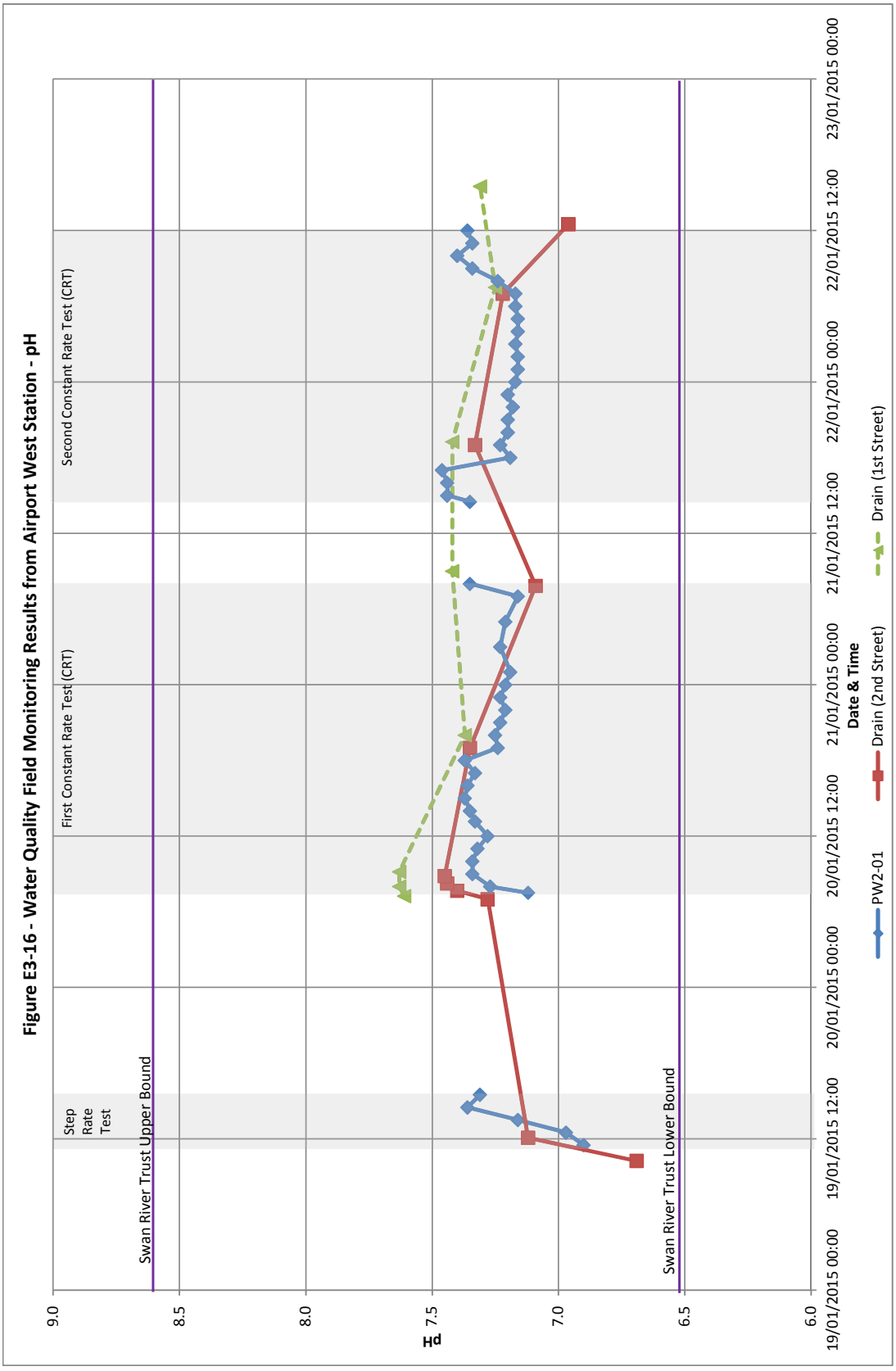


Figure E3-16 - Water Quality Field Monitoring Results from Airport West Station - pH
Step Rate Test
First Constant Rate Test (CRT)
Second Constant Rate Test (CRT)
Swan River Trust Upper Bound
Swan River Trust Lower Bound
pH
9.0
8.5
8.0
7.5
7.0
6.5
6.0
19/01/2015 00:00
19/01/2015 12:00
20/01/2015 00:00
20/01/2015 12:00
21/01/2015 00:00
21/01/2015 12:00
22/01/2015 00:00
22/01/2015 12:00
23/01/2015 00:00
Date & Time
PW2-01
Drain (2nd Street)
Drain (1st Street)

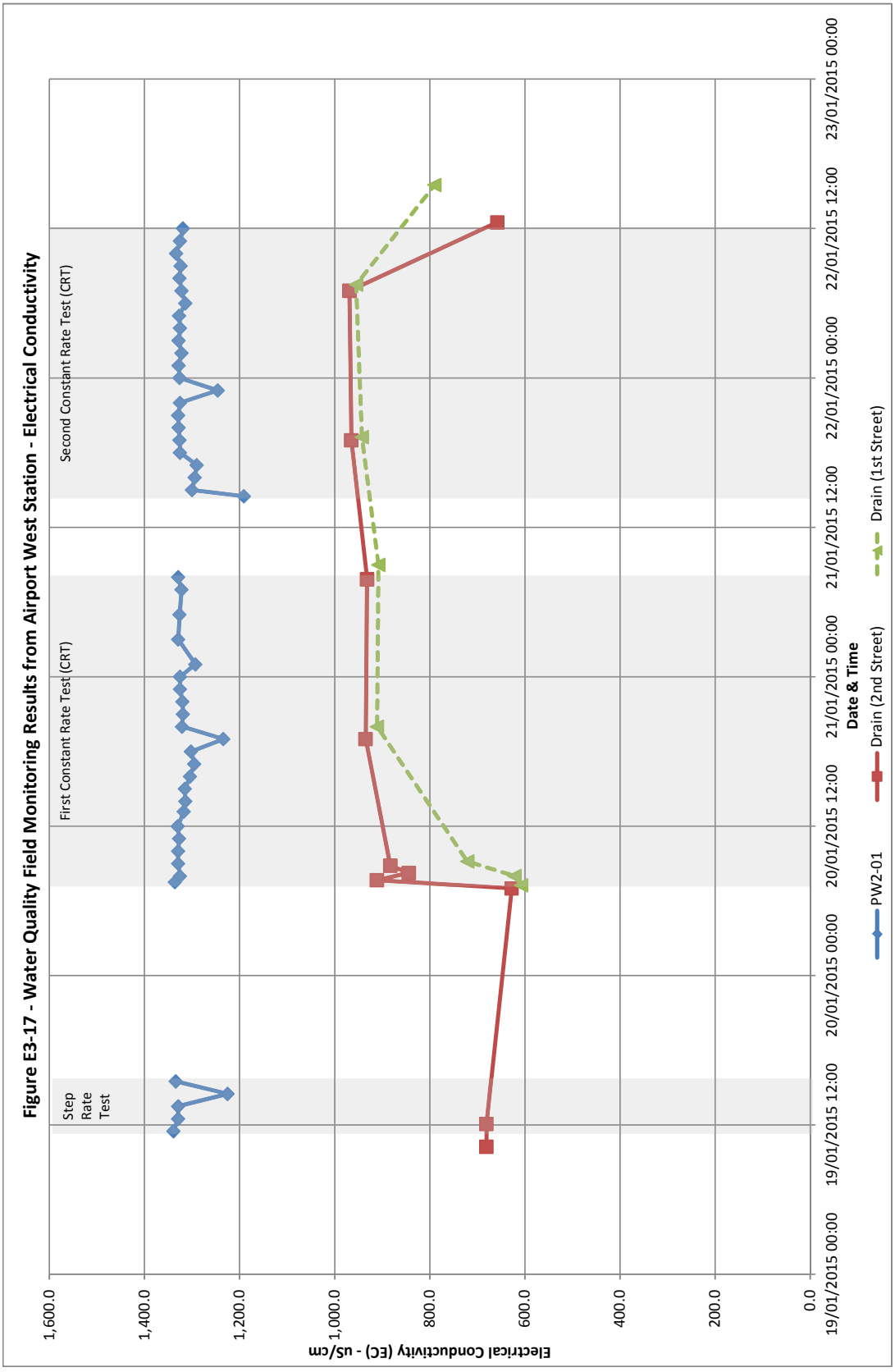
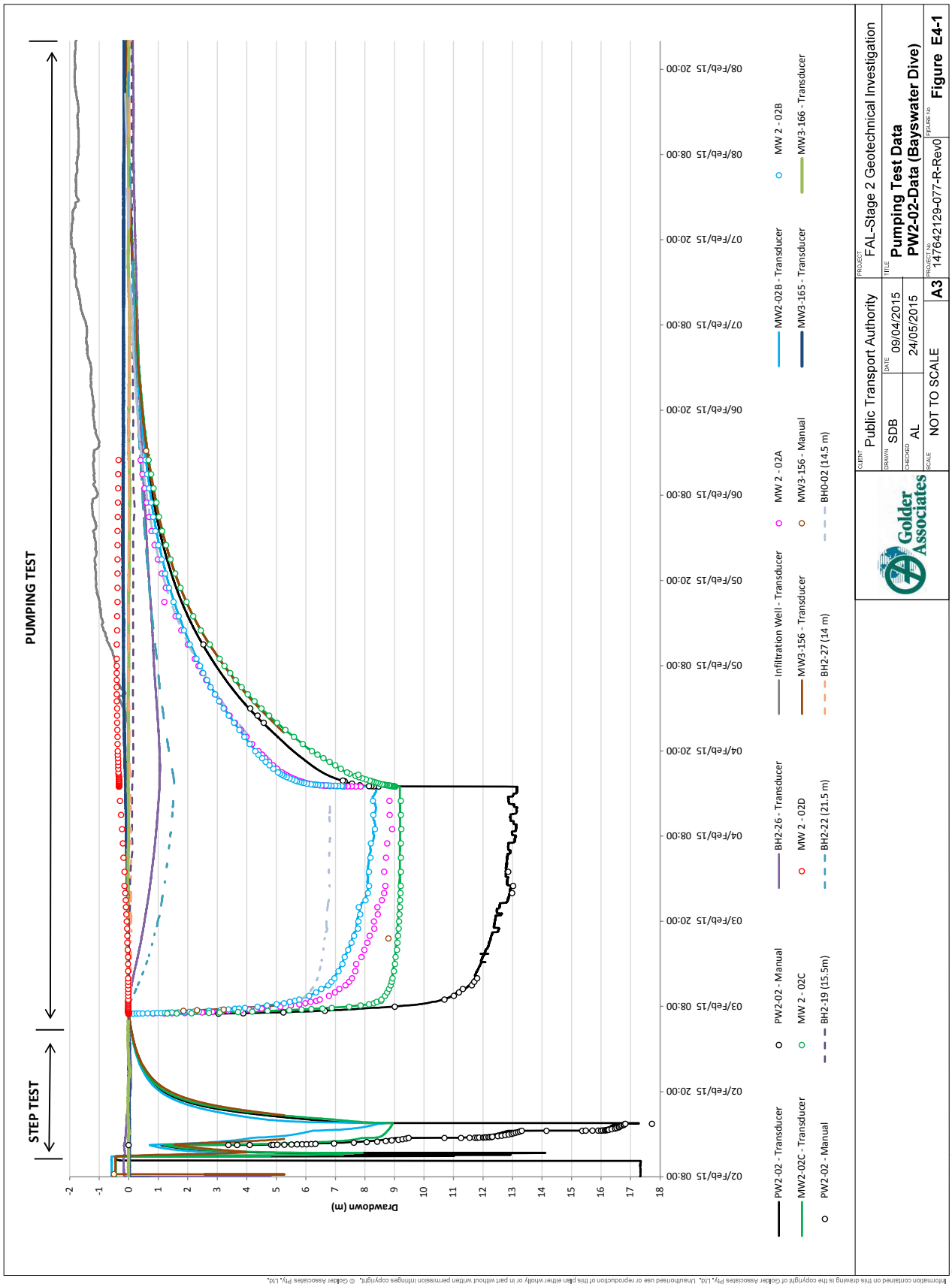


Figure E3-17 - Water Quality Field Monitoring Results from Airport West Station - Electrical Conductivity
Step Rate Test
First Constant Rate Test (CRT)
Second Constant Rate Test (CRT)
Electrical Conductivity (EC) - uS/cm
1,600.0
1,400.0
1,200.0
1,000.0
800.0
600.0
400.0
200.0
0.0
19/01/2015 00:00
19/01/2015 12:00
20/01/2015 00:00
20/01/2015 12:00
21/01/2015 00:00
21/01/2015 12:00
22/01/2015 00:00
22/01/2015 12:00
23/01/2015 00:00
Date & Time
PW2-01
Drain (2nd Street)
Drain (1st Street)



APPENDIX E
Summary of Test Pumping Analysis

ATTACHMENT E4
Constant Rate Test – Bayswater Dive Structure (PW2-02)



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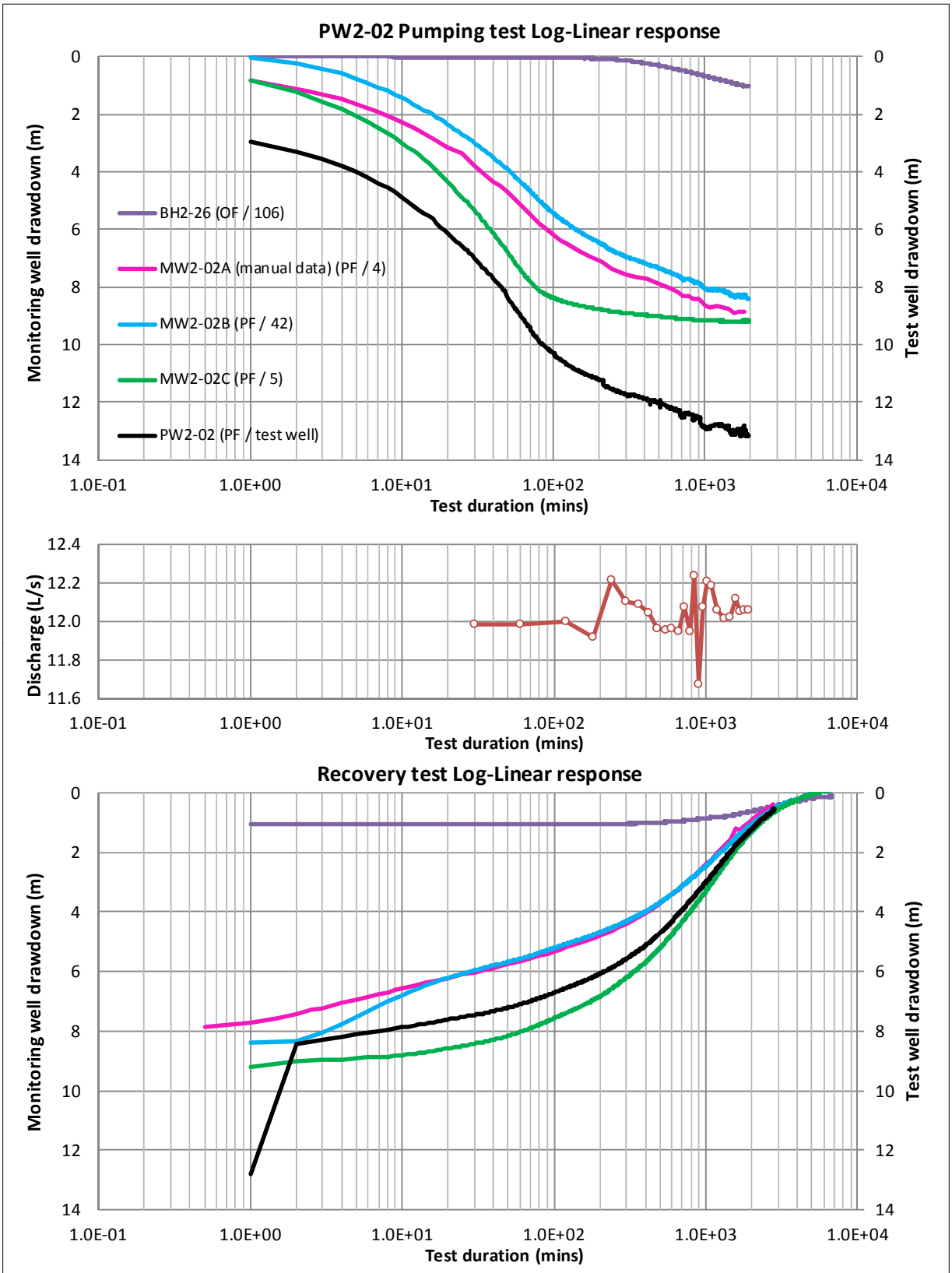

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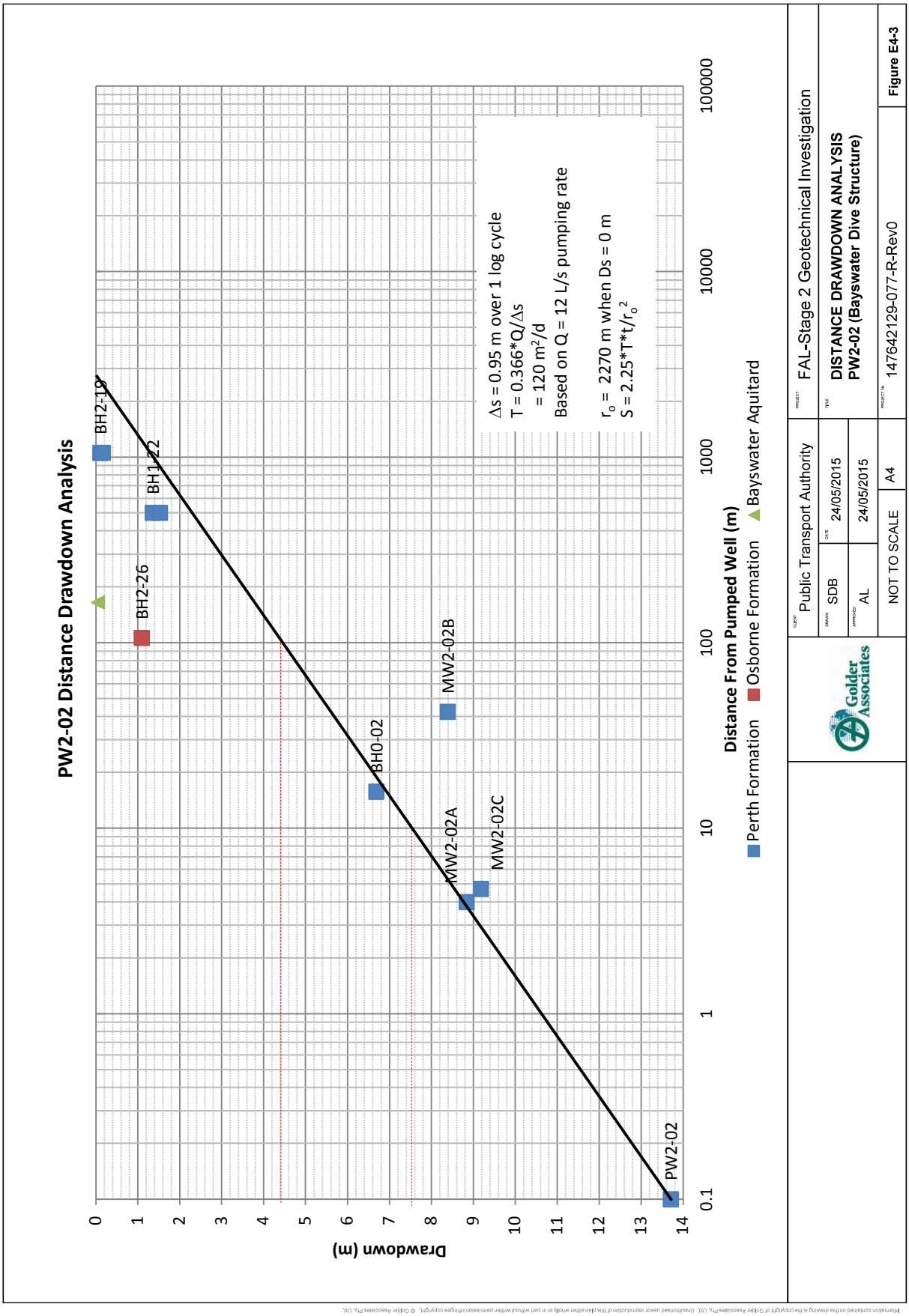
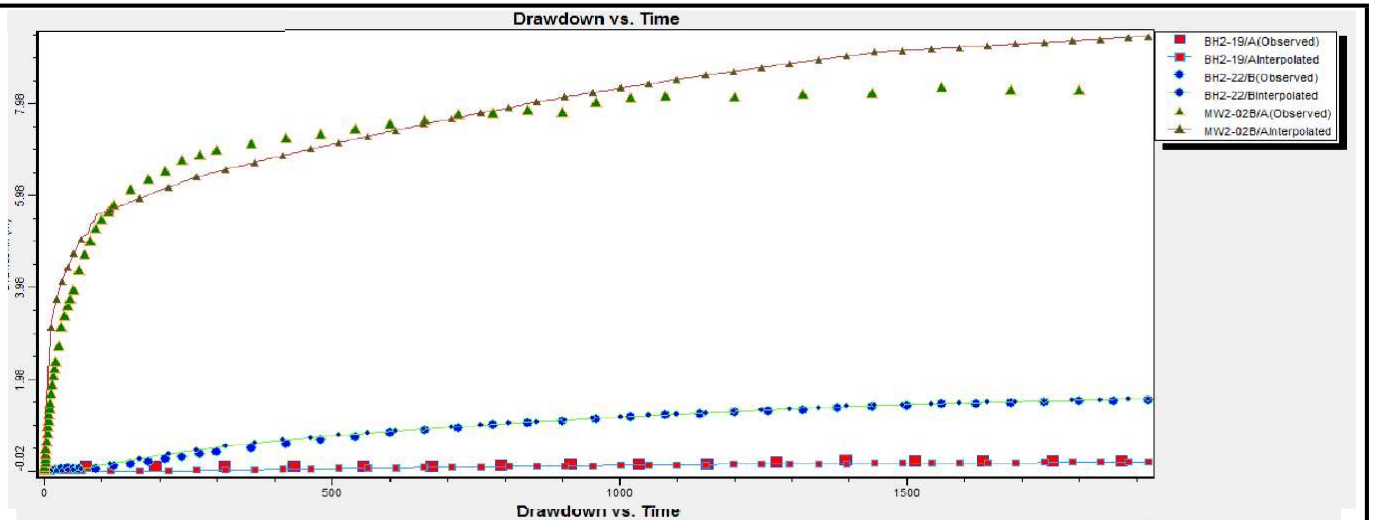
Golder Associates Level 3, 1 Havelock St West Perth WA 6005 Ph: +618 9213 7600 Fx: +618 9213 7611	CLIENT: Public Transport Authority	PROJECT: FAL-Stage 2 Geotechnical Investigation	
	DRAWN: SDB / DATE: 24/05/2015	TITLE: **Pumping Test Data**	
	CHECKED: AL / DATE: 24/05/2015	**PW2-02 (Bayswater Dive Structure)**	
	SCALE: NOT TO SCALE / A4	PROJECT No: 147642129-077-R-Rev0	FIGURE No: **Figure E4-2**

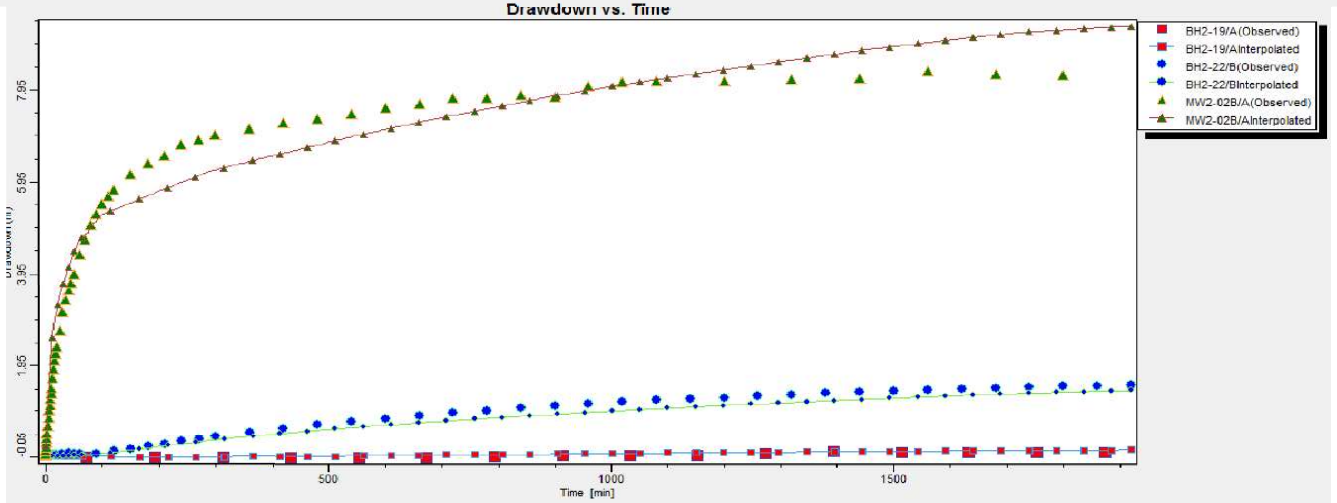
File Location:


PW2-02 Distance Drawdown Analysis
Drawdown (m)
Distance From Pumped Well (m)
Perth Formation
Osborne Formation
Bayswater Aquitard
BH2-19
BH1-22
BH2-26
MW2-02B
BH0-02
MW2-02A
MW2-02C
PW2-02
Δs = 0.95 m over 1 log cycle
T = 0.366*Q/Δs
= 120 m²/d
Based on Q = 12 L/s pumping rate
r_o = 2270 m when Ds = 0 m
S = 2.25*T*t/r_o²
Public Transport Authority
FAL-Stage 2 Geotechnical Investigation
DISTANCE DRAWDOWN ANALYSIS
PW2-02 (Bayswater Dive Structure)
SDB
24/05/2015
AL
24/05/2015
NOT TO SCALE
A4
147642129-077-R-Rev0
Figure E4-3

Specific Storage =	1.00E-06 1/m
Transmissivity =	60 m2/d
Hydraulic Conductivity	
K(GF)h	4 m/d
K(GF)v	0.4 m/d



Specific Storage =	5.00E-06 1/m
Transmissivity =	60 m2/d
Hydraulic Conductivity	
K(GF)h	4 m/d
K(GF)v	4 m/d



Screenshot from MODFLOW output

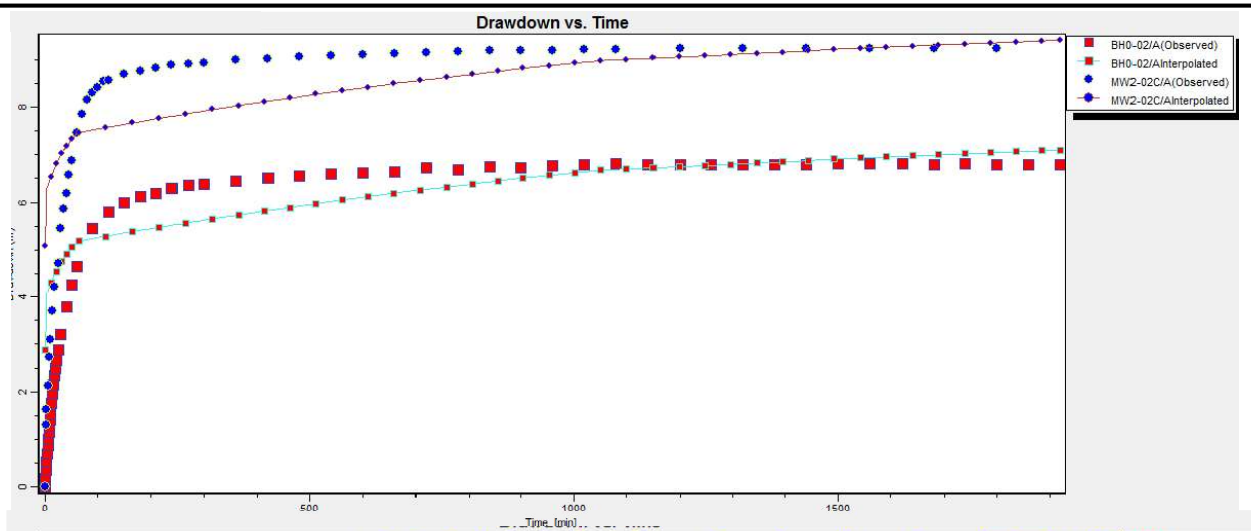
Graphs for BH2-19, BH2-22 and MW2-02B (further from PW2-02)

Golder Associates

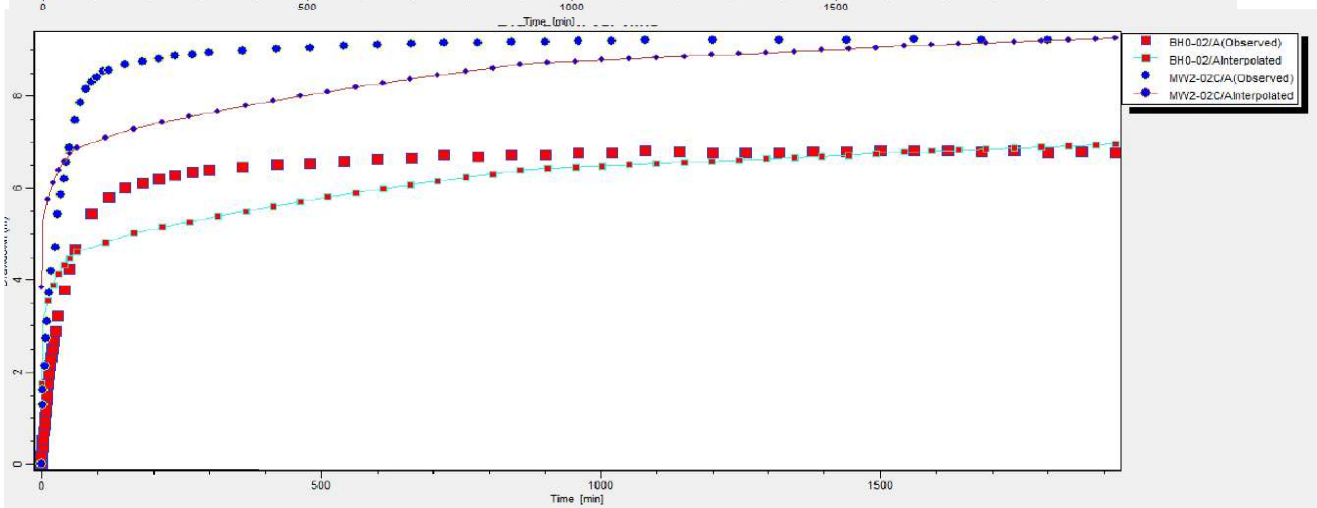
CLIENT	Public Transport authority	PROJECT	FAL - Stage 2 Geotechnical Investigation
DRAWN	AL	DATE	24-May-15
CHECK	DPS	DATE	24-May-15
SCALE	Not To Scale	A4	
TITLE	**Modelled Groundwater Level Drawdown PW2-02 Pumping Test (Bayswater) Forrestfield Dive Structure**		
PROJECT No.	147642129-077-R-Rev0	FIGURE No.	**E4-4**

C:\Users\alundorf\Desktop\FAL\APPENDIX E\Appendix E - Test Pumping Results\Attachment E4\Figure E4-4&E4-5.xlsx

Specific Storage = 1.00E-06 1/m
Transmissivity = 113 m2/d
Hydraulic Conductivity
K(GF)h 7.5 m/d
K(GF)v 0.75 m/d



Specific Storage = 5.00E-06 1/m
Transmissivity = 113 m2/d
Hydraulic Conductivity
K(GF)h 7.5 m/d
K(GF)v 7.5 m/d



Screenshot from MODFLOW output

Graphs for BH0-02 and MW2-02C (Closer to PW2-02)

CLIENT	Public Transport authority	PROJECT	FAL - Stage 2 Geotechnical Investigation
DRAWN	AL / DATE 24-May-15	TITLE	Modelled Groundwater Level Drawdown PW2-02 Pumping Test (Bayswater) Forrestfield Dive Structure
CHECK	DPS / DATE 24-May-15		
SCALE	Not To Scale / A4	PROJECT No.	147642129-077-R-Rev0
		FIGURE No.	E4-5

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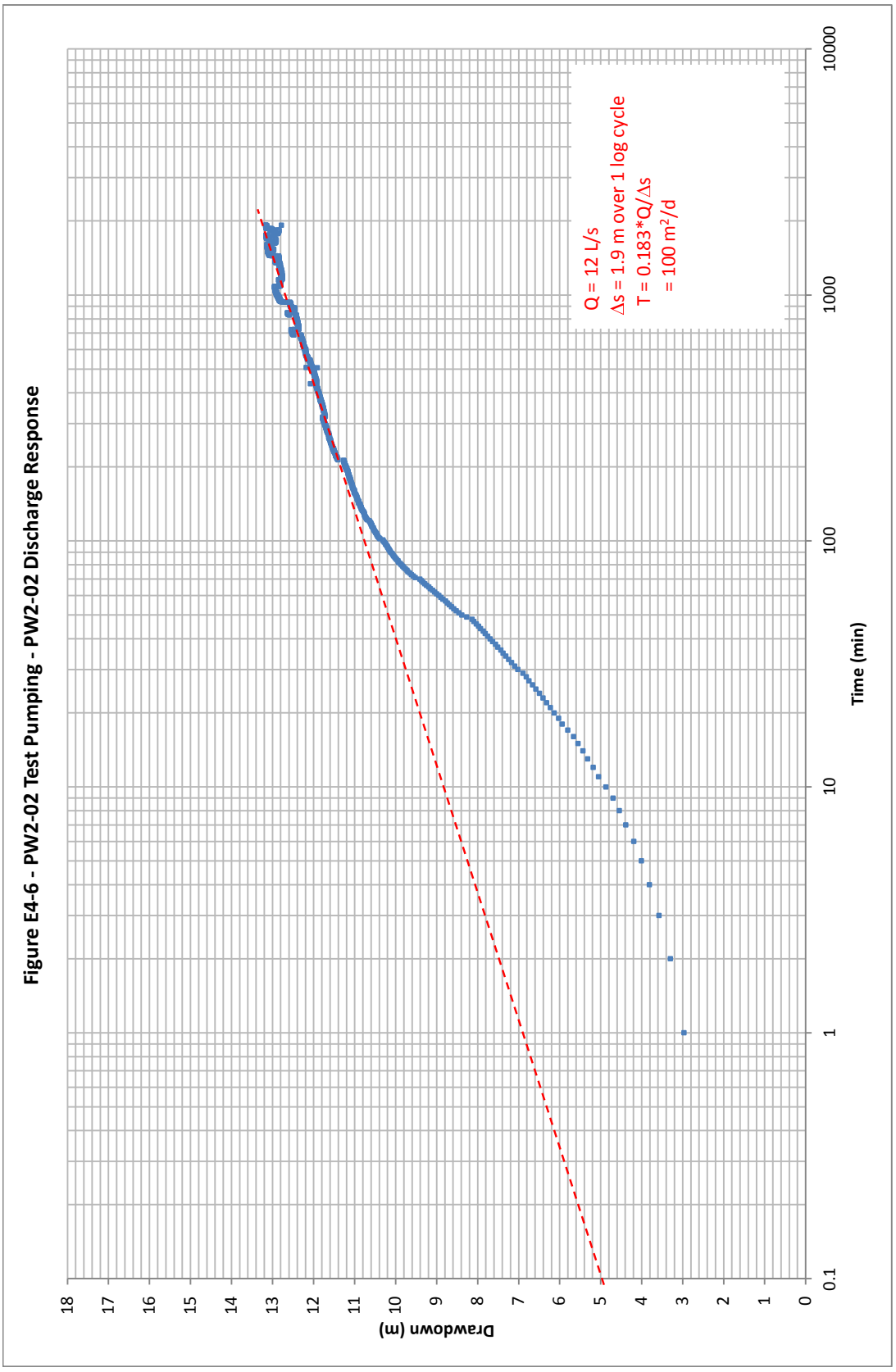


Figure E4-6 - PW2-02 Test Pumping - PW2-02 Discharge Response
Drawdown (m)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
Time (min)
0.1
1
10
100
1000
10000
Q = 12 L/s
Δs = 1.9 m over 1 log cycle
T = 0.183*Q/Δs
= 100 m²/d

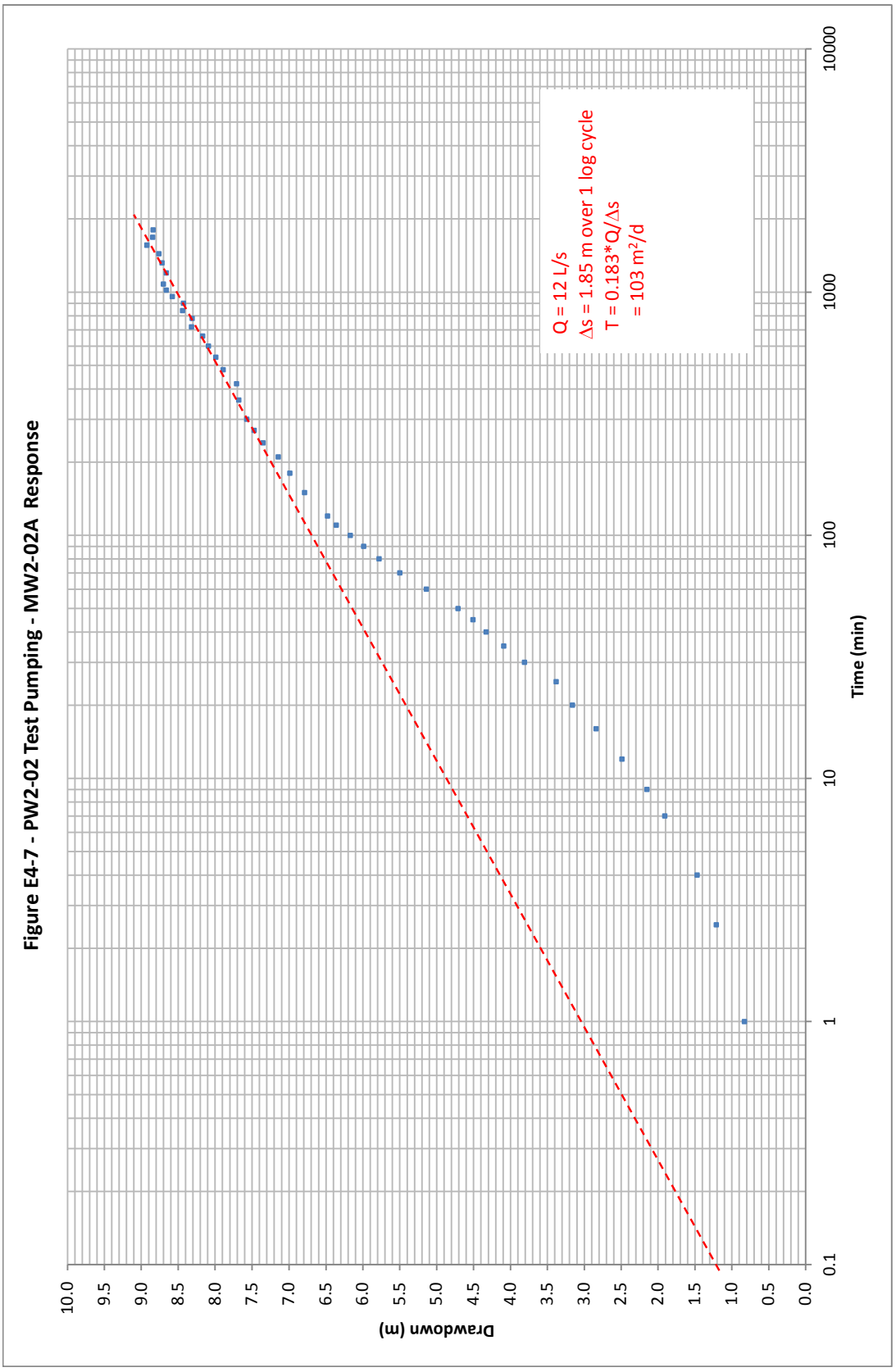


Figure E4-7 - PW2-02 Test Pumping - MW2-02A Response
Drawdown (m)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Time (min)
0.1
1
10
100
1000
10000
Q = 12 L/s
Δs = 1.85 m over 1 log cycle
T = 0.183*Q/Δs
= 103 m²/d

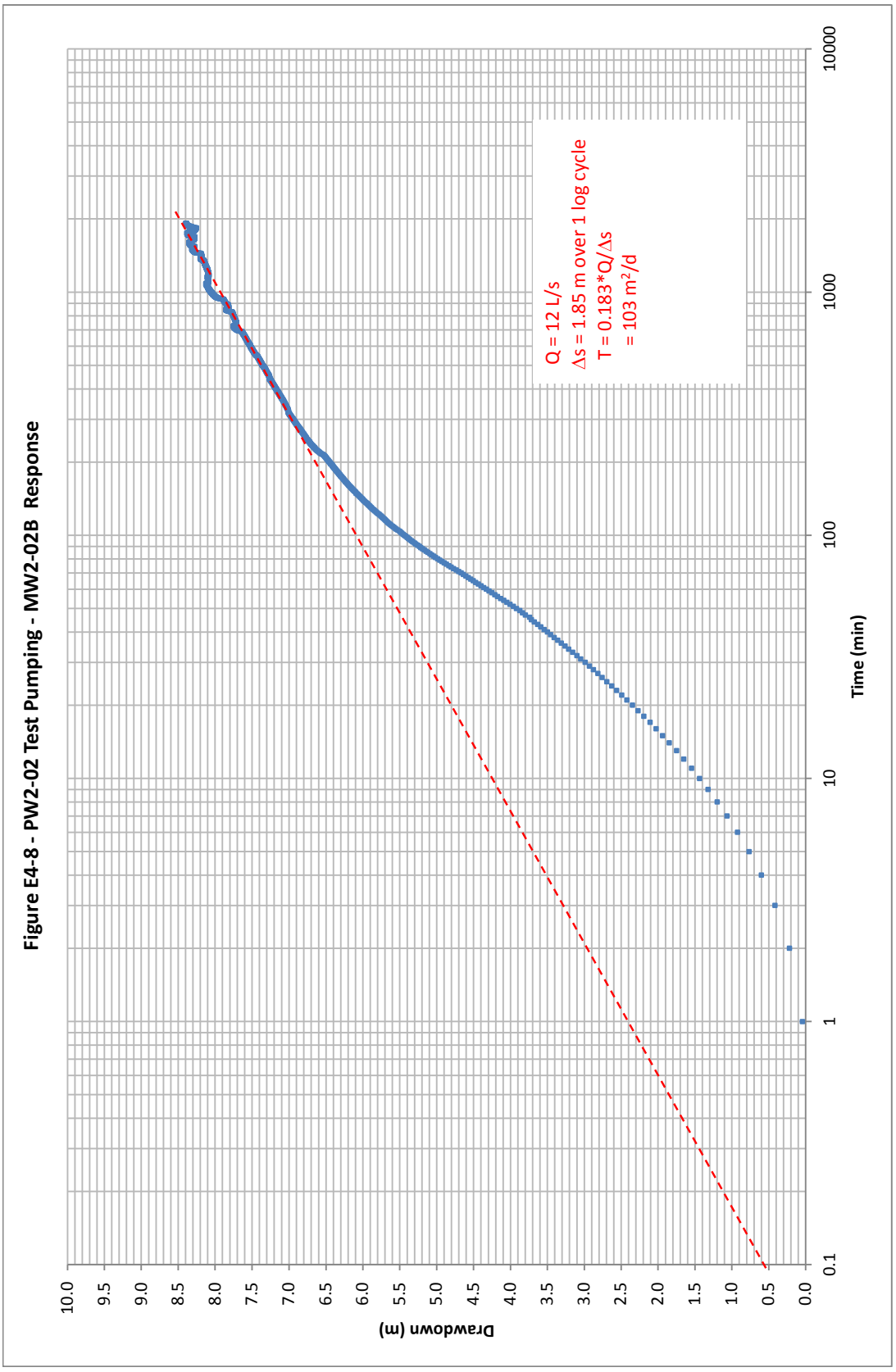


Figure E4-8 - PW2-02 Test Pumping - MW2-02B Response
Drawdown (m)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
1
10
100
1000
10000
Time (min)
Q = 12 L/s
Δs = 1.85 m over 1 log cycle
T = 0.183*Q/Δs
= 103 m²/d

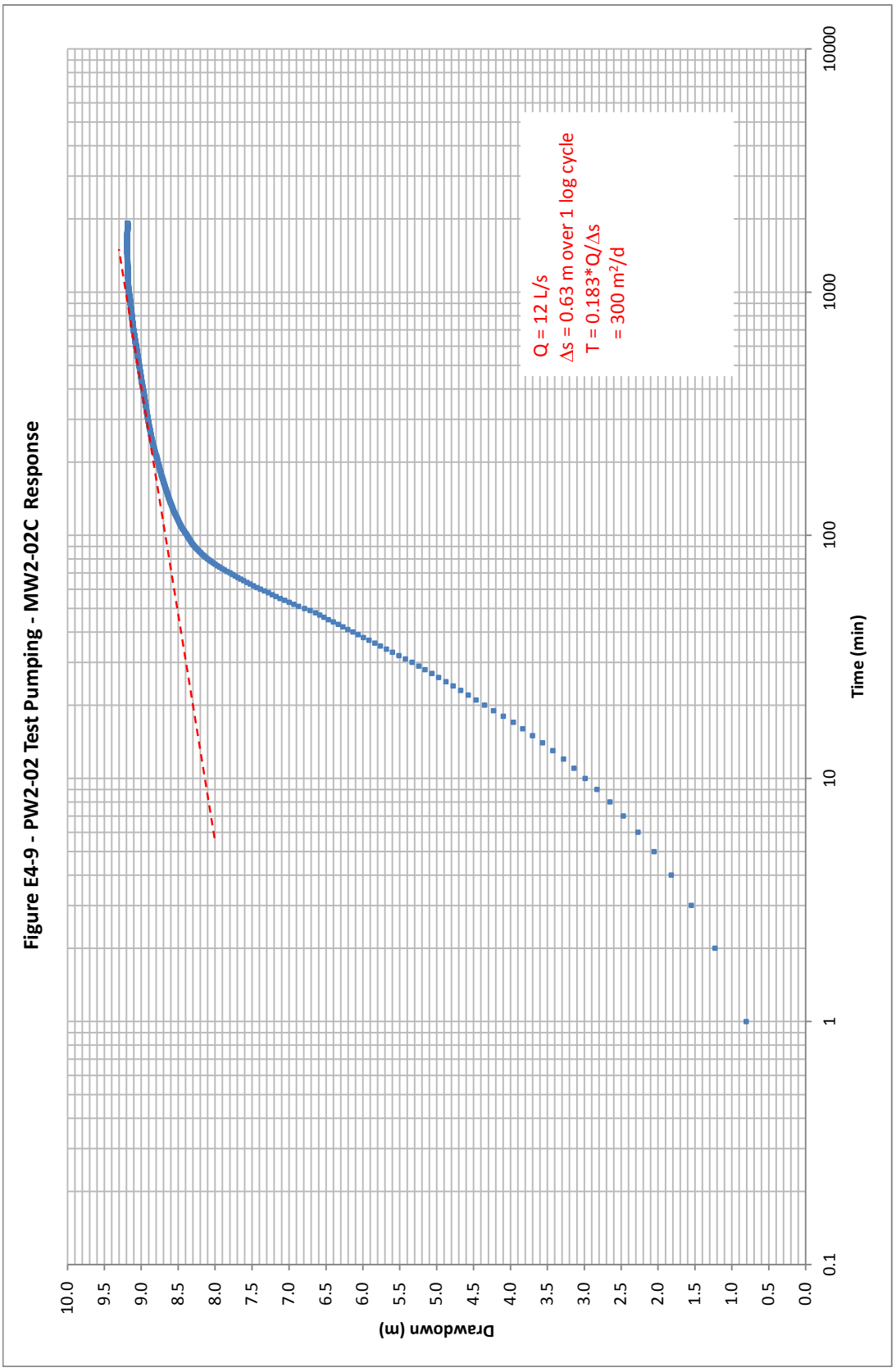


Figure E4-9 - PW2-02 Test Pumping - MW2-02C Response
Drawdown (m)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
1
10
100
1000
10000
Time (min)
Q = 12 L/s
Δs = 0.63 m over 1 log cycle
T = 0.183*Q/Δs
= 300 m²/d

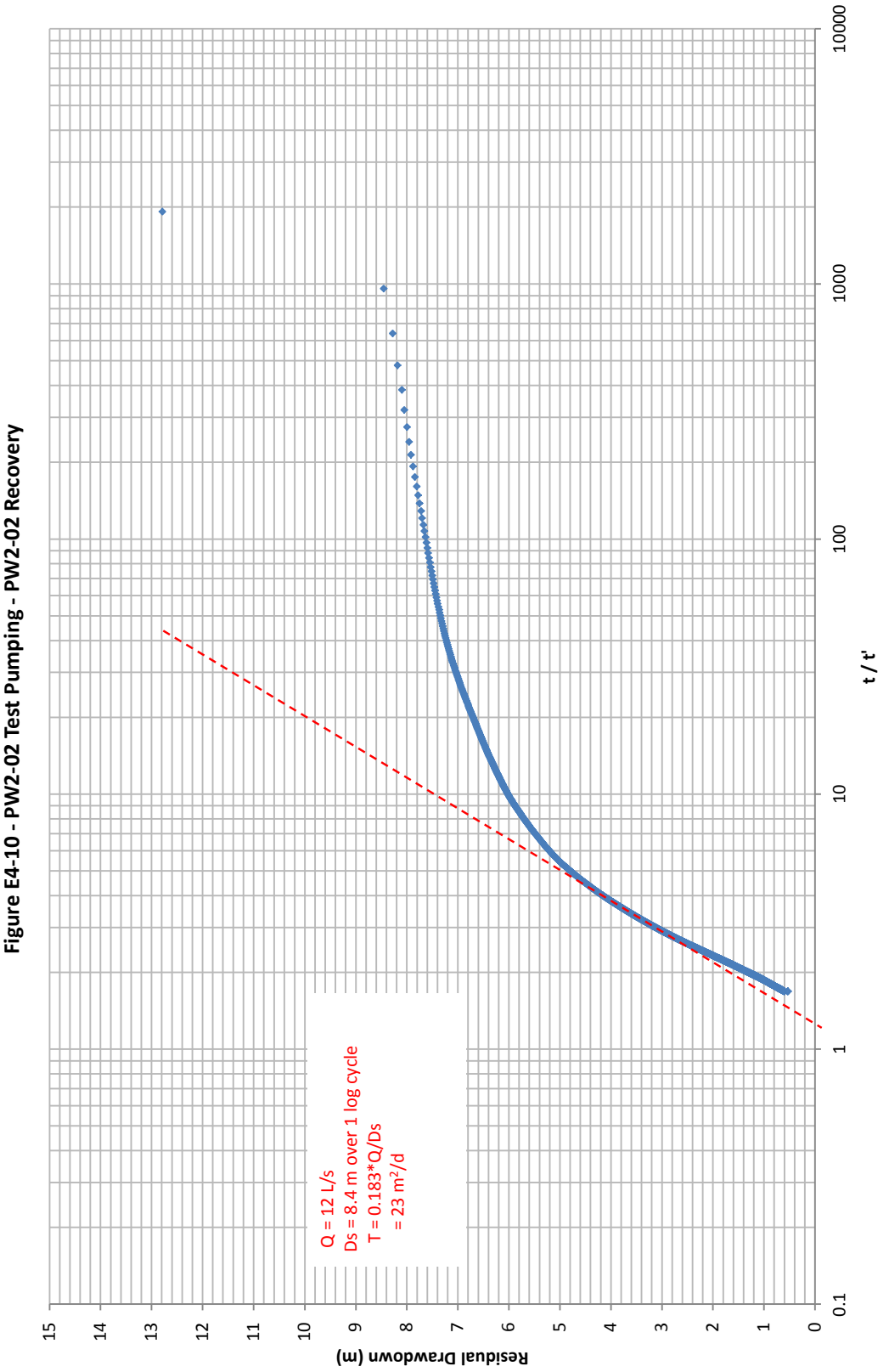


Figure E4-10 - PW2-02 Test Pumping - PW2-02 Recovery

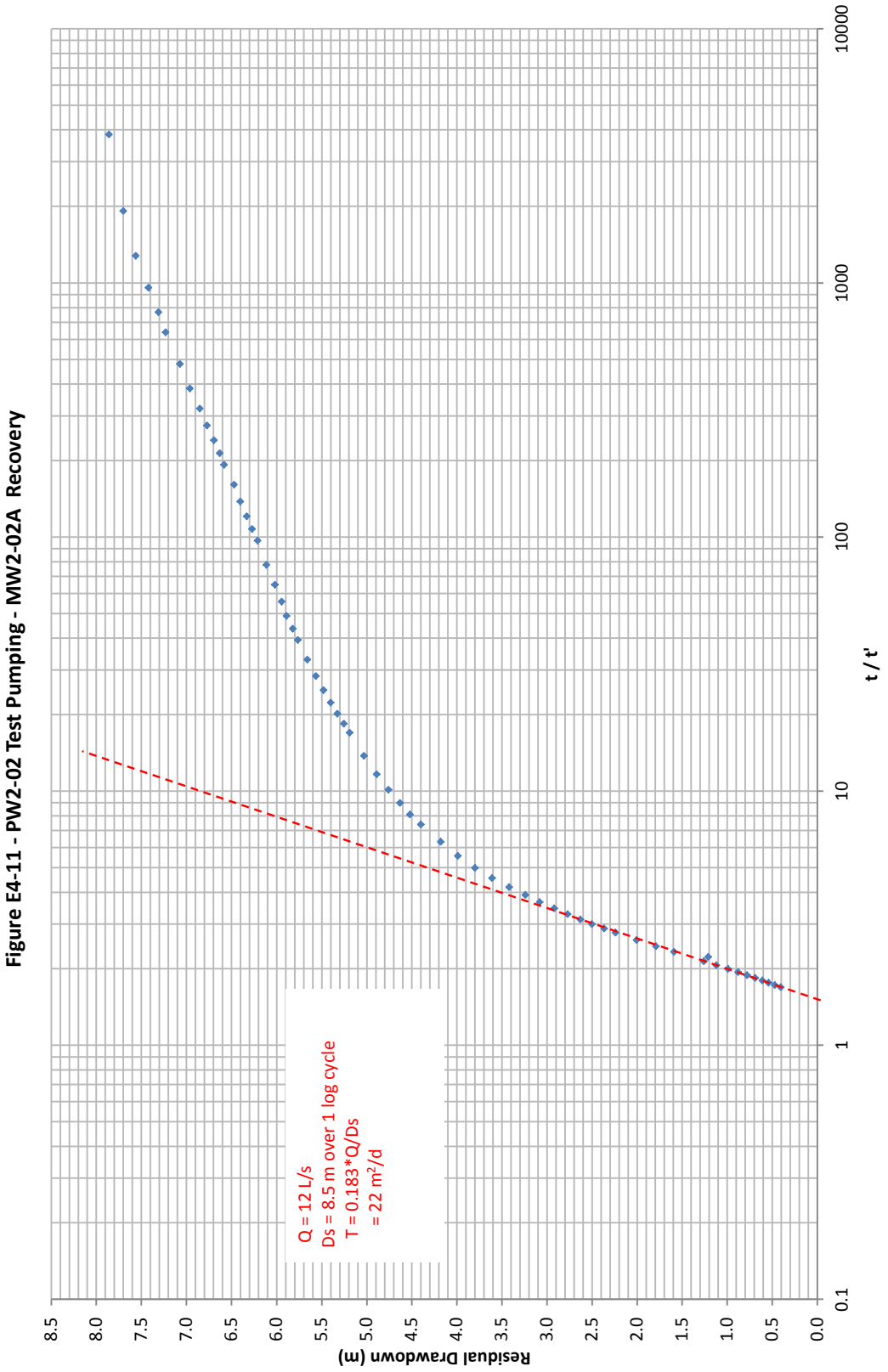


Figure E4-11 - PW2-02 Test Pumping - MW2-02A Recovery
Residual Drawdown (m)
t / t'
Q = 12 L/s
Ds = 8.5 m over 1 log cycle
T = 0.183*Q/Ds
= 22 m²/d
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
1
10
100
1000
10000

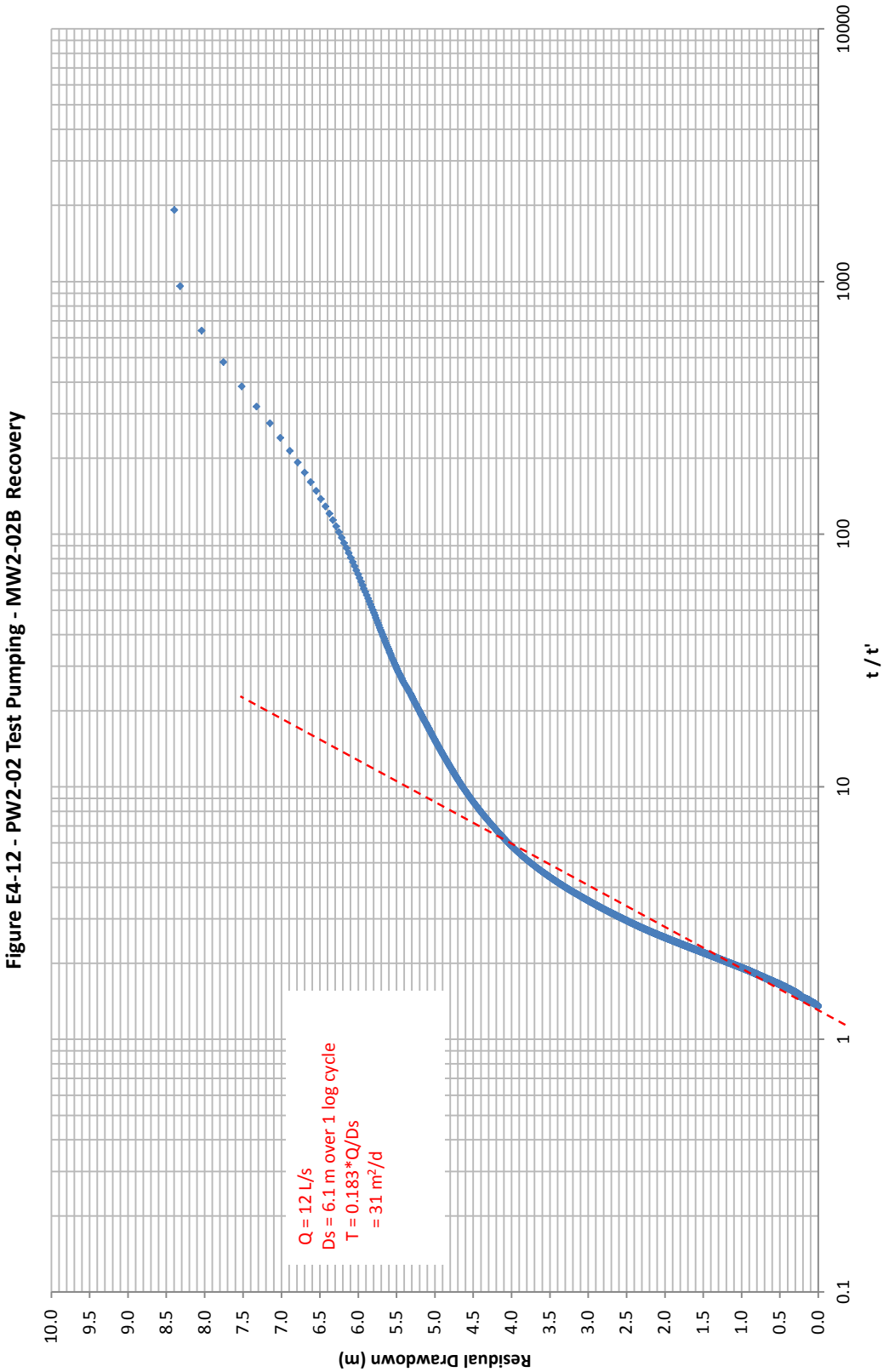


Figure E4-12 - PW2-02 Test Pumping - MW2-02B Recovery
Residual Drawdown (m)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
1
10
100
1000
10000
t / t'
Q = 12 L/s
Ds = 6.1 m over 1 log cycle
T = 0.183*Q/Ds
= 31 m²/d

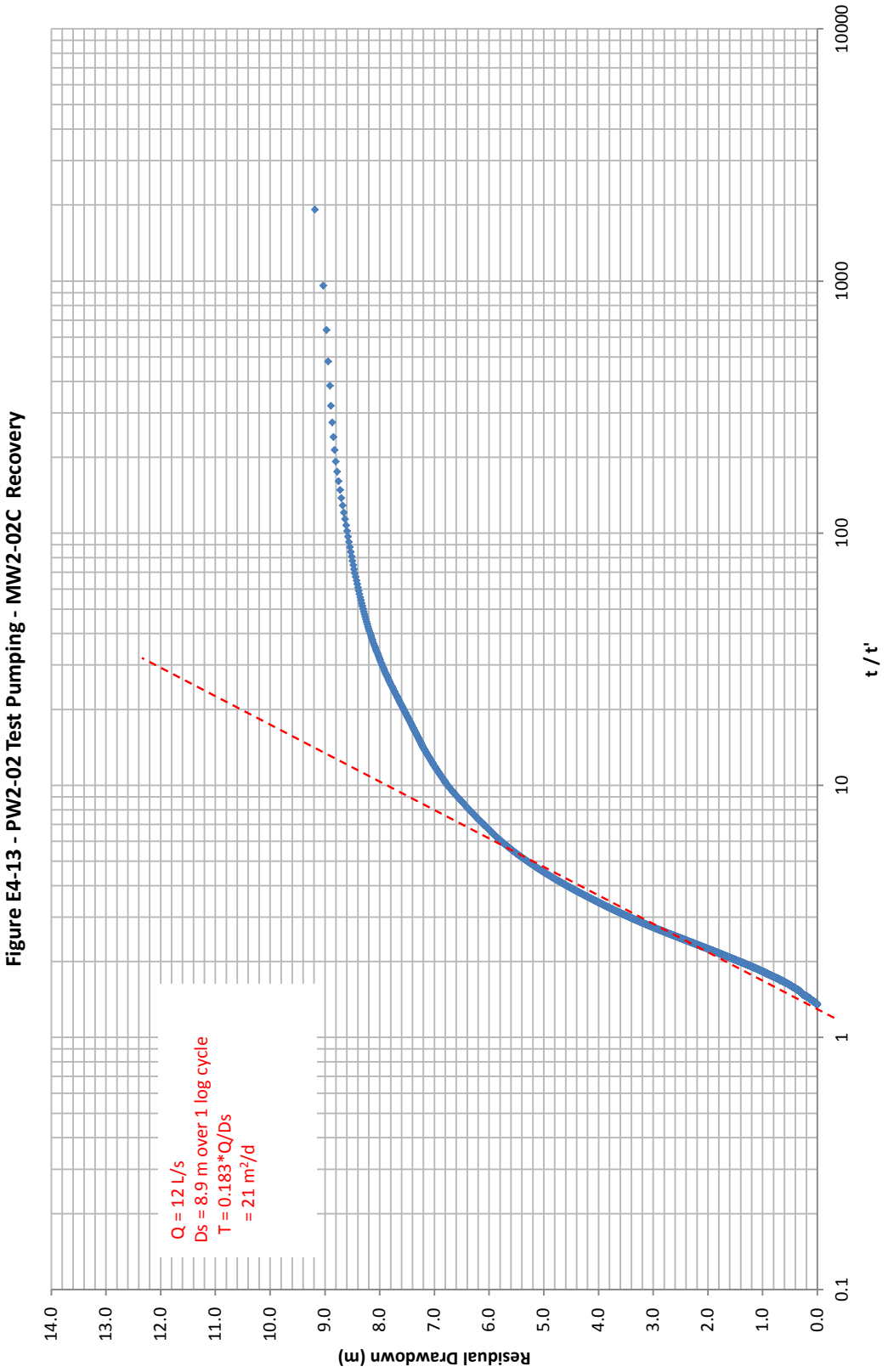


Figure E4-13 - PW2-02 Test Pumping - MW2-02C Recovery
Residual Drawdown (m)
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0.1
1
10
100
1000
10000
t / t'
Q = 12 L/s
Ds = 8.9 m over 1 log cycle
T = 0.183*Q/Ds
= 21 m²/d

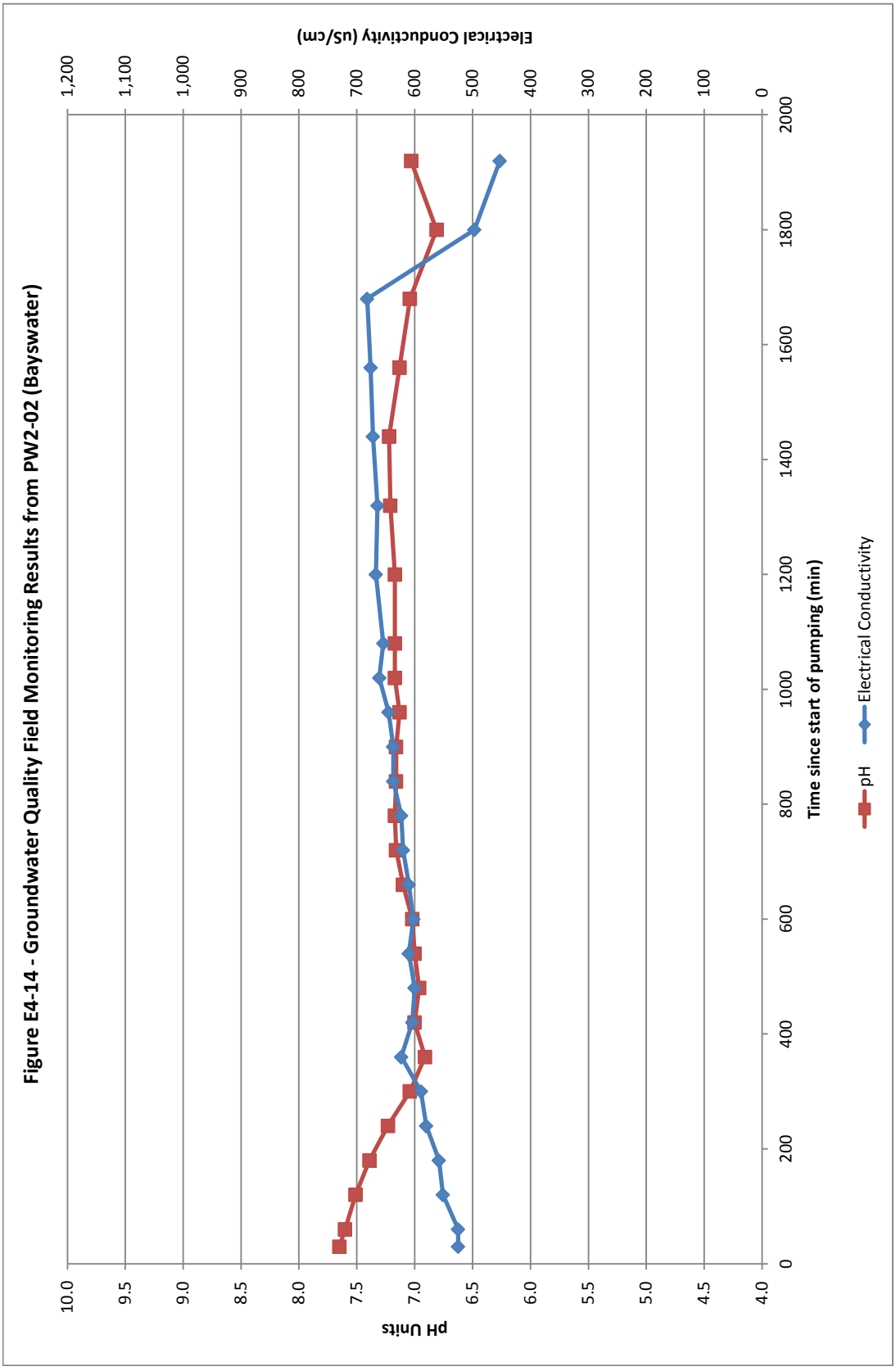


Figure E4-14 - Groundwater Quality Field Monitoring Results from PW2-02 (Bayswater)
pH Units
Electrical Conductivity (uS/cm)
Time since start of pumping (min)
pH
Electrical Conductivity
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
1,200
1,100
1,000
900
800
700
600
500
400
300
200
100
0
0
200
400
600
800
1000
1200
1400
1600
1800
2000

APPENDIX F

Description of Preliminary Groundwater Model Setup

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX F
Groundwater Model Construction and Calibration

1.0 INTRODUCTION

This appendix provides details on the construction and assumptions used in the 3D numerical groundwater model. Golder prepared a basic groundwater model in 2014 (Golder, 2014b). This model was used as the platform and updated to include more detailed information on the geology and hydrogeological properties.

The modelling development of the groundwater has been carried out in general accordance with the guiding principles in the Australian Groundwater Modelling Guidelines (NWC, 2012).

The objective of the preliminary groundwater model is to estimate the effect the construction dewatering could have on the groundwater level and piezometric levels and to provide preliminary dewatering rates by modelling conceptual dewatering designs. The model is still considered preliminary (Class 1 according to NWC, 2012) and further detailing of the groundwater model could be required during the dewatering design (Class 2 according to NWC, 2012).

2.0 MODEL ASSUMPTIONS

The following is a list of assumptions used in the design of the preliminary 3D groundwater model:

- Geology beneath the proposed alignment is representative of the geology to the north and south of the alignment and is relatively consistent across the modelled area. Publicly available information, where available, has been utilised to assist with creating the geological surfaces within the model domain outside of the tunnel alignment (e.g. extent and depth of the Mirrabooka Aquifer). However, this data can only be considered approximate.
- Each geological unit is homogenous and isotropic in terms of hydraulic properties with the exception of:
  - The ratio of vertical to horizontal hydraulic conductivity ranges between 1:1 and 1:10 (i.e. $k_v/k_h = 1$ to $k_v/k_h = 0.1$). These ratios were considered appropriate because:
    - The majority of the geological units represented in the model are sedimentary and have been deposited in fluvial environments.
    - Our professional experience on other projects in the Perth Metropolitan area indicates similar ratios.
    - Calibration of pumping test data indicated a best fit using such ratios.
  - Ascot Formation was divided into two areas of different hydraulic conductivity based on the results of the pumping tests which indicate higher hydraulic conductivity in the eastern part of this unit.
  - The Mirrabooka Member was divided into two layers of different hydraulic conductivity to simulate an interpreted decrease in hydraulic conductivity with depth based on bore logs and results from the pumping test.
- Diaphragm walls (D-walls) will be constructed around the perimeter (except for the entrance to the dive structures) of the excavations prior to the start of any excavation and dewatering. The proposed D-walls have been simulated using a low hydraulic conductivity, which allows for simulation of some seepage through the walls.
- Recharge and evapotranspiration are uniform spatially and temporally across the model. Seasonal variation has not been included in the model during transient model runs as this would mask the objective of assessing the changes in groundwater level and piezometric level caused by the construction dewatering. There is no recharge within the D-walls during the dewatering operation as it is assumed that any rainfall over the site will be pumped off site immediately.
- The model does not consider off-site influences, such as groundwater abstraction from other sites, which could affect the groundwater behaviour within the model.

2.1 Model Setup

The model setup and main input parameters are described in the sections below.

2.1.1 Extent and Grid Sizing

The model domain is 10.8 km long and 8.7 km wide (Figure F1). The model extent was selected so that the boundaries would be set at distances outside the influence of groundwater drawdown caused by the dewatering.

The grid sizing within the model ranges from 50 m by 50 m (mainly at the model peripheral boundaries) to 5 m by 5 m around the main structures where dewatering is required. This variation in grid size allows for greater resolution near the proposed structures. The model consists of a total of 438 rows and 493 columns.

2.1.2 Layers

The model has been divided into 13 layers to represent the main geological units. The geological surfaces have been obtained from the geological long section (Golder, 2015b) and geological control points away from the tunnel alignment. The reason for the additional model layers is to allow for the termination of the D-walls within a geological unit and to model variability of hydraulic conductivity with depth. Figure F2 shows a cross-section of the model along Row 111 (refer to Section A – A' on Figure F1 for alignment - given the orientation of the model the software does not allow for a cross-sectional representation along the alignment). Table F1 presents the different geological units per layer.

Table F1: Summary of Groundwater Model Layers.

Layers	West Zone			East Zone	
1	Bassendean Sand/Perth Formation Sand	Swan River Formation		Bassendean Sand, Guildford Formation, Gnangara Sand	Guildford Formation
2					
3	Perth Formation Clay				
4					
5					
6	Perth Formation Sand			Ascot Formation	
7					
8	Kardinya Shale (Osborne Formation)			Mirrabooka Member (Osborne Formation)	Kardinya Shale (Osborne Formation)
9					
10				Kardinya Shale (Osborne Formation)	
11					
12					
13					

The elevation of the Layer 1 is based on the current smoothed topography ranging from approximately RL 0 m AHD at the Swan River to RL 50 m AHD at the eastern boundary of the model towards the Darling Scarp. The bottom of Layer 13 is flat with an elevation of RL -100 m AHD, based on the inferred depth of the Kardinya Shale in Davidson (1995).

2.2 Steady-State Model Boundaries

Boundary conditions are assigned in the model to control how water will flow into and out of the model. It is preferable that natural boundaries are used where they exist, otherwise boundaries should be set far enough away that they do not influence what is occurring within the model. Figure F1 shows the assigned boundary locations.

2.2.1 General Head Boundaries

General head boundaries (GHB) were assigned at the north-western and south-eastern boundaries of the model in all layers. The boundaries were set using off-set groundwater level contours estimated from both the Perth Groundwater Atlas (WRC, 1997) and observed groundwater levels.

2.2.2 Constant Head Boundaries

A constant head boundary at an elevation of RL 0.2 m AHD was assigned to the portions of the model representing the Swan River.

2.2.3 Drain Boundaries

A river function was applied to represent the Bayswater Main Drain and the Belmont Main Drain; the locations of these drains are shown in Figure F1. River functions were used in preference to drain functions as the two drains modelled contain a phreatic surface for a significant proportion of the year. In the event that dewatering results in drawdown which extends to these drains, the use of a river function will allow water to flow from the drain into the aquifer (i.e. the drains could start giving water to the aquifer rather than draining it from the aquifer).

2.2.4 No-flow Boundaries

Inactive cells or no-flow boundaries were assigned to large portions of the north-eastern and south-western portions of the model because of the model geometry relative to the alignment. The areas within these inactive cells are unlikely to experience any influence from the proposed dewatering and are roughly perpendicular to groundwater flow direction. Assigning inactive flow to these large areas helps to improve accuracy and increase speed of computation. The bottom of the model was also considered a no flow boundary.

2.2.5 Recharge and Evapotranspiration

A spatially and temporally uniform recharge value of 100 mm/year (approximately 12% of Mean Annual Rainfall) was applied across the whole model. Evapotranspiration was applied to the model with an extinction depth of 1.5 m (i.e. the evapotranspiration function will only be active when the groundwater level comes within 1.5 m of the model surface, which mainly occurs in the Perth Airport area. Given that the whole of Perth Airport contains a large network of closely spaced drains to control the groundwater level at the airport estate (the Perth Airport area used to be a wetland area), the evapotranspiration function was used to simulate the dense network of drains rather than incorporating all the drains in the model using drain functions.

2.3 Transient Model (Non-steady State) Boundaries

The following input parameters are assigned for transient modelling components of the groundwater model.

2.3.1 Diaphragm Walls (D-walls)

D-walls were introduced into the model at varying depths for each dive structure and at the two stations and were based on the Rev F alignment plan and Reference Design Summary Table RevB:

- The D-walls will terminate 3 m into the Osborne Formation at the Airport West Station and Consolidated Airport Terminal Station.
- At the Bayswater and Forrestfield Dive Structures the D-Walls will extend to a depth that is about 1.9 and 2.3 times the depth of excavation, respectively.

The D-walls are introduced around the structures as cells with a low hydraulic conductivity. The hydraulic conductivity of the walls was varied between 0.001 m/d and 0.01 m/d to simulate some leakage through the D-walls.

2.3.2 Dewatering Method

Dewatering will likely be undertaken using a combination of dewatering wells, dewatering spears and sump pumping. The dewatering in the groundwater model was simulated by using drain cells assigned a particular drain elevation equivalent to 1m below the excavation base. For dive structures the drain elevation was inserted along a linear gradient from the surface (no dewatering) to the maximum dewatering elevation at the portal of the dive structure.

2.3.3 Recharge Wells

To simulate water being re-injected back into the aquifer, the water was introduced back into the groundwater model using the well function. The recharge wells were generally screened across the whole Superficial Aquifer to a similar depth as the D-walls. It was assumed in the model that the long-term recharge rate of a recharge well would not exceed 1 L/s.

3.0 MODEL CALIBRATION

3.1 Steady-State Calibration

The steady state model was run to simulate the estimated average seasonal groundwater level contours and groundwater flow direction based on the March 2015 groundwater level survey. The groundwater level results from the steady state calibration were then used as input into the transient (non-steady state) simulation.

The model boundaries and hydraulic conductivities were changed until groundwater and piezometric level contours across the model were approximately equal to the estimated average seasonal groundwater levels along the alignment (main focus was given to the groundwater and piezometric levels at the four main structures) and the observed hydraulic gradients and flow direction from the March 2015 groundwater level survey and Perth Groundwater Atlas (WRC, 1997).

Table F2 presents a comparison between estimated average seasonal groundwater and piezometric levels and modelled groundwater and piezometric levels at the four main structures along the alignment based on the March 2015 groundwater level survey and the 2014/2015 seasonal variation from the closest VWPs. The modelled groundwater levels were consistently 0.6 m to 0.7 m higher than the groundwater levels observed in March 2015, which is considered a reasonable representation of the average seasonal groundwater level, given that the March 2015 groundwater levels represent close to the seasonal low levels and that seasonal fluctuations have been observed to range between 0.7 m and 1.2 m over majority of the alignment and 3.0 m in the Forrestfield area.

Table F2: Comparison of Modelled Groundwater and Piezometric Levels (m AHD).

Structure	March 2015 Measure Groundwater and Piezometric Level	Estimated Average Seasonal Groundwater and Piezometric Level	Modelled Groundwater and Piezometric Level
Bayswater Structures – Upper Aquifer	5.3	5.9	6.0
Bayswater Structures – Lower Aquifer	1.6	2.0	2.0
Airport West Station and Structure	8.1	9.1	8.9
Consolidated Terminal Station	17.1	17.6	17.5
Forrestfield Structures	23.0	24.5	24.4

3.2 Transient Calibration

There is currently insufficient data to carry out a transient calibration on a regional scale. Instead the groundwater model was used to simulate the short-term pumping tests carried out at the four main structures by using the regional groundwater model structure and layers in four sub-models, one for each structure, with a greater discretisation (minimum cell size of 0.5 m) to better simulate the pumping tests. The results from modelling of the pumping tests are discussed in detail in Appendix E, but the main conclusions were:

- The hydrogeological units below Bassendean Sand were found to be semi-confined to confined, likely caused by the lower permeability layers in the Guildford Formation and Perth Formation.
- The vertical to horizontal hydraulic conductivity ratio (k_v/k_h) was found to range between 0.1 and 1 for all hydrogeological layers with the ratio of 0.1 generally providing the best fit with the monitoring data from the pumping tests.

Based on the favourable comparison between estimated average seasonal and modelled groundwater levels, similar hydraulic gradients and good comparison between the observed and modelled pumping test data (in the sub-models), Golder judge that the preliminary groundwater model is an adequate representation of actual groundwater conditions for the purposes of the study reported here. We therefore conclude that the preliminary groundwater model can be used for predictive purposes of dewatering design concepts to assess the effect the construction dewatering could have on the groundwater level and piezometric levels and to provide preliminary dewatering rates for the assumed design and construction methodology.

3.3 Hydraulic Parameters Used in the Calibrated Model

Table 1 presents the hydraulic input parameters of the calibrated preliminary groundwater steady-state model which was used for the transient scenario model runs.

Table 1: Summary of Hydraulic Parameters Used in the Groundwater Model

Unit	Horizontal Hydraulic Conductivity (k_h) (m/d)	Vertical Hydraulic Conductivity (k_v) (m/d)	Specific Yield	Specific Storage (1/m)
Swan River Formation	1	0.1	0.05	2×10^{-5}
Combined Bassendean Sand, Guildford Formation and Gnangara Sand	5	0.5	0.1	2×10^{-5}
Perth Formation – sandy unit	7.5	0.75 *	0.1	5×10^{-6}
Perth Formation – clay unit	0.0001	0.00001	0.02	2×10^{-5}
Guildford Formation	5	0.5 *	0.1	1×10^{-4}
Ascot Formation	8 – 14	0.8 – 1.4	0.15	2×10^{-5}
Mirrabooka Member (Osborne Formation)	1 – 3	0.1 – 0.3 *	0.1	2×10^{-5}
Kardinya Shale (Osborne Formation)	0.0005	0.00005	0.02	2×10^{-5}

* Sensitivity runs were carried out for the parameters by increasing the vertical hydraulic conductivity values 10 times (giving a K_v/K_h ratio of 1).

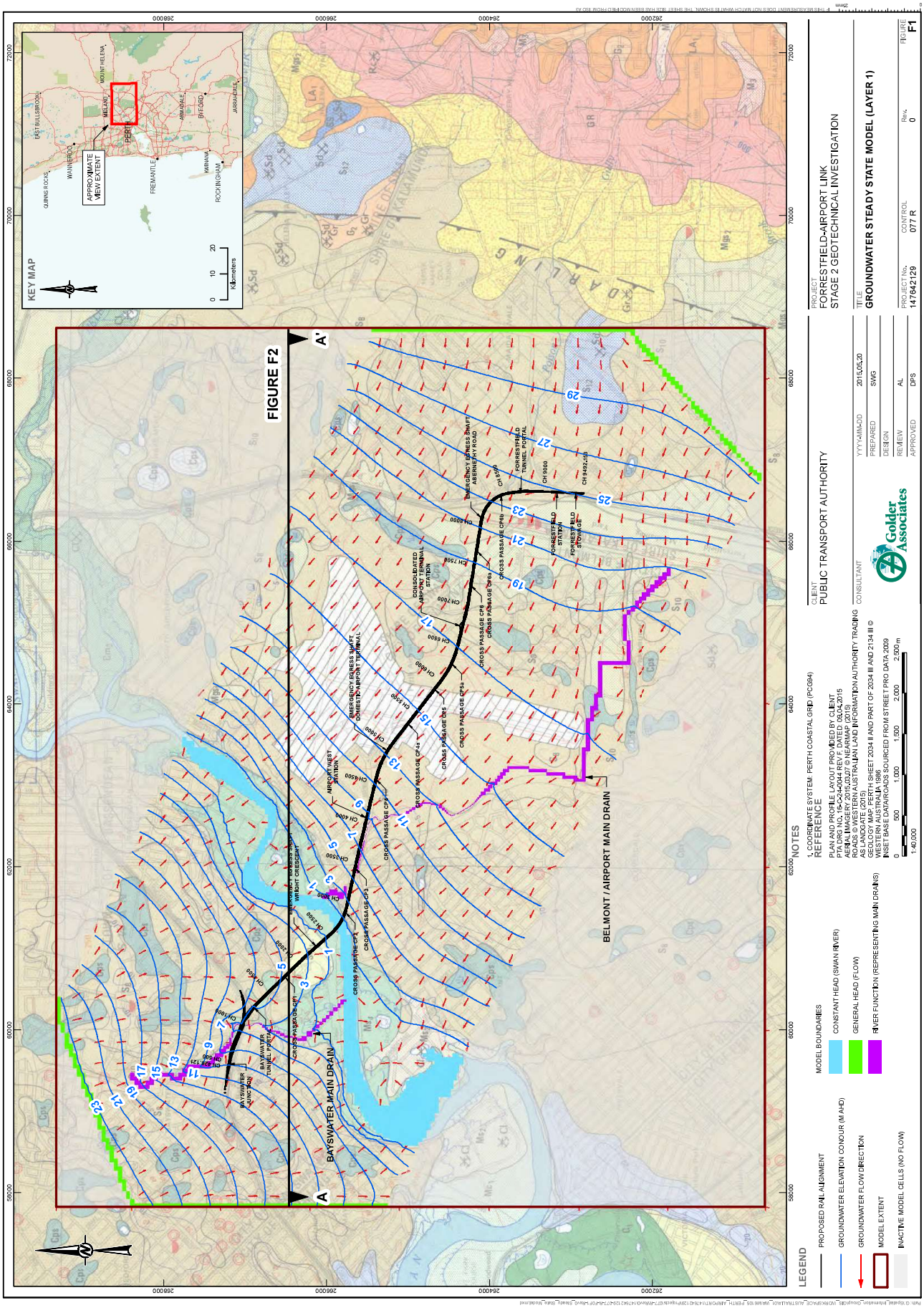
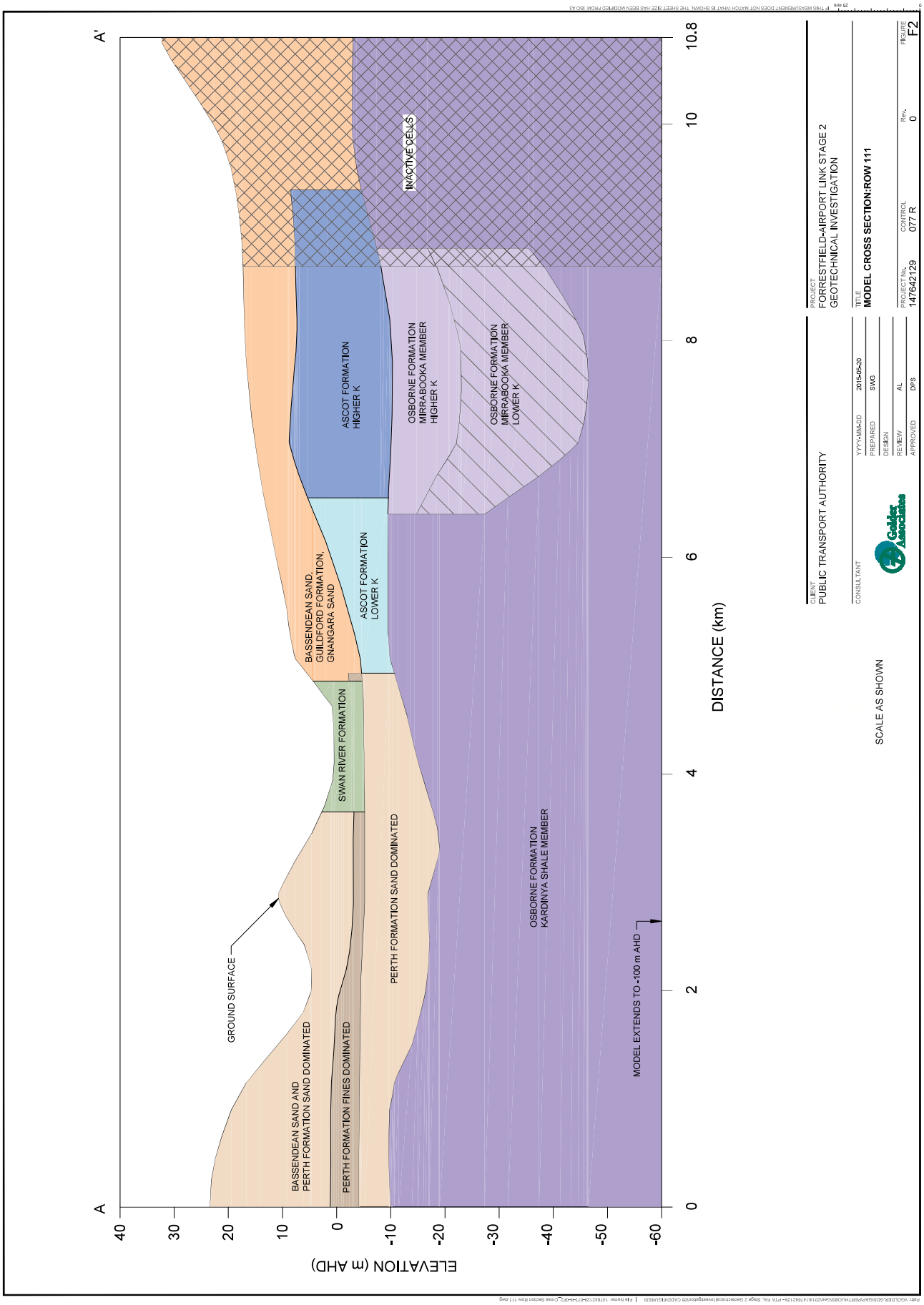


FIGURE F2
KEY MAP
APPROXIMATE VIEW EXTENT
PERTH
BAYSWATER MAIN DRAIN
BELMONT / AIRPORT MAIN DRAIN
A
A'
LEGEND
PROPOSED RAIL ALIGNMENT
GROUNDWATER ELEVATION CONOUR (M AHD)
GROUNDWATER FLOW DIRECTION
MODEL EXTENT
INACTIVE MODEL CELLS (NO FLOW)
MODEL BOUNDARIES
CONSTANT HEAD (SWAN RIVER)
GENERAL HEAD (FLOW)
RIVER FUNCTION (REPRESENTING MAIN DRAINS)
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-24-0044 REV F, DATED: 09.04.2015
AERIAL IMAGERY 2015.03.07 © NEARMAP (2015)
ROADS © WESTERN AUSTRALIAN LAND INFORMATION AUTHORITY TRADING AS LANDGATE (2015)
GEOLOGY MAP, PERTH SHEET 2034 II AND PART OF 2034 III AND 2134 III © WESTERN AUSTRALIA 1986
INSET BASE DATA/ROADS SOURCED FROM STREET PRO DATA 2009
1:40,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER STEADY STATE MODEL (LAYER 1)
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE F1


A
A'
ELEVATION (m AHD)
DISTANCE (km)
GROUND SURFACE
BASSENDEAN SAND AND
PERTH FORMATION SAND DOMINATED
PERTH FORMATION FINES DOMINATED
SWAN RIVER FORMATION
BASSENDEAN SAND,
GUILDFORD FORMATION,
GNANGARA SAND
ASCOT FORMATION
LOWER K
PERTH FORMATION SAND DOMINATED
ASCOT FORMATION
HIGHER K
OSBORNE FORMATION
MIRRABOOKA MEMBER
HIGHER K
OSBORNE FORMATION
MIRRABOOKA MEMBER
LOWER K
INACTIVE CELLS
OSBORNE FORMATION
KARDINYA SHALE MEMBER
MODEL EXTENDS TO -100 m AHD
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
MODEL CROSS SECTION:ROW 111
SCALE AS SHOWN
PROJECT No.
147642129
CONTROL
077 R
REV.
0
FIGURE
F2

APPENDIX G

Summary of Analysis of Inverse Auger Hole Testing

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX G
Inverse Auger Hole Testing Results

Shallow infiltration testing using the "inverse auger hole method" (Cocks, 2007) was carried out at six locations I1-01, I1-02, I1-03 (Forrestfield Area) and I1-04, I1-05 and I1-06 (Consolidated Airport Terminal Station). Attachment G1 shows the infiltration locations. All tests were carried out in Bassendean Sand except for I1-05 which was carried out in Fill.

The inverse auger hole method uses a machine slotted PVC casing sealed into an open auger hole followed by numerous falling head tests carried out by introducing water into the PVC casing. The decline in head inside the PVC casing is recorded at regular intervals and used to calculate the permeability as described by Cocks (2007).

The permeability values calculated from each test are summarised in Table 1 and the analysis sheets are presented in Attachment G2.

Table 1: Infiltration Tests Results

Test ID	Location	Soil Description*	Average Calculated Permeability (m/d)
I1–01	Forrestfield	Sand (*f-m*), some clay	12
I1–02		Sand (*f-m*), trace silt and clayey gravelly sand (*f-c*)	3
I1–03		Sand (*f-m*) and sandy gravel (*f-m*)	10
I1–04	Consolidated Airport	Sand (*f-m*)	10
I1–05		Fill/Topsoils: Sand (*f-m*)	5
I1–06		Sand (*f-m*)	4

*Material description based on upper 1 metre of soil profile encountered in closest test pit or borehole, *f*,- fine grained, *m* – medium grained, *c* – coarse grained,

ATTACHMENT G1
Infiltration Test Locations

May 2015
Reference No. 147642129-077-R-Rev0

APPENDIX G
Inverse Auger Hole Testing Results



Infiltration Test Locations at Consolidated Airport Terminal Station

APPENDIX G
Inverse Auger Hole Testing Results



Infiltration Test Locations in Forrestfield



APPENDIX G
Inverse Auger Hole Testing Results

ATTACHMENT G2
Inverse Auger Hole Permeability Testing Analysis

May 2015
Reference No. 147642129-077-R-Rev0

Golder Associates

Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

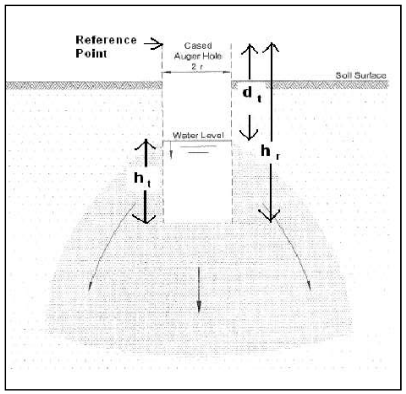
$$K = 1.15r \frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}(h_t)}{t - t_0}$$

Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-01

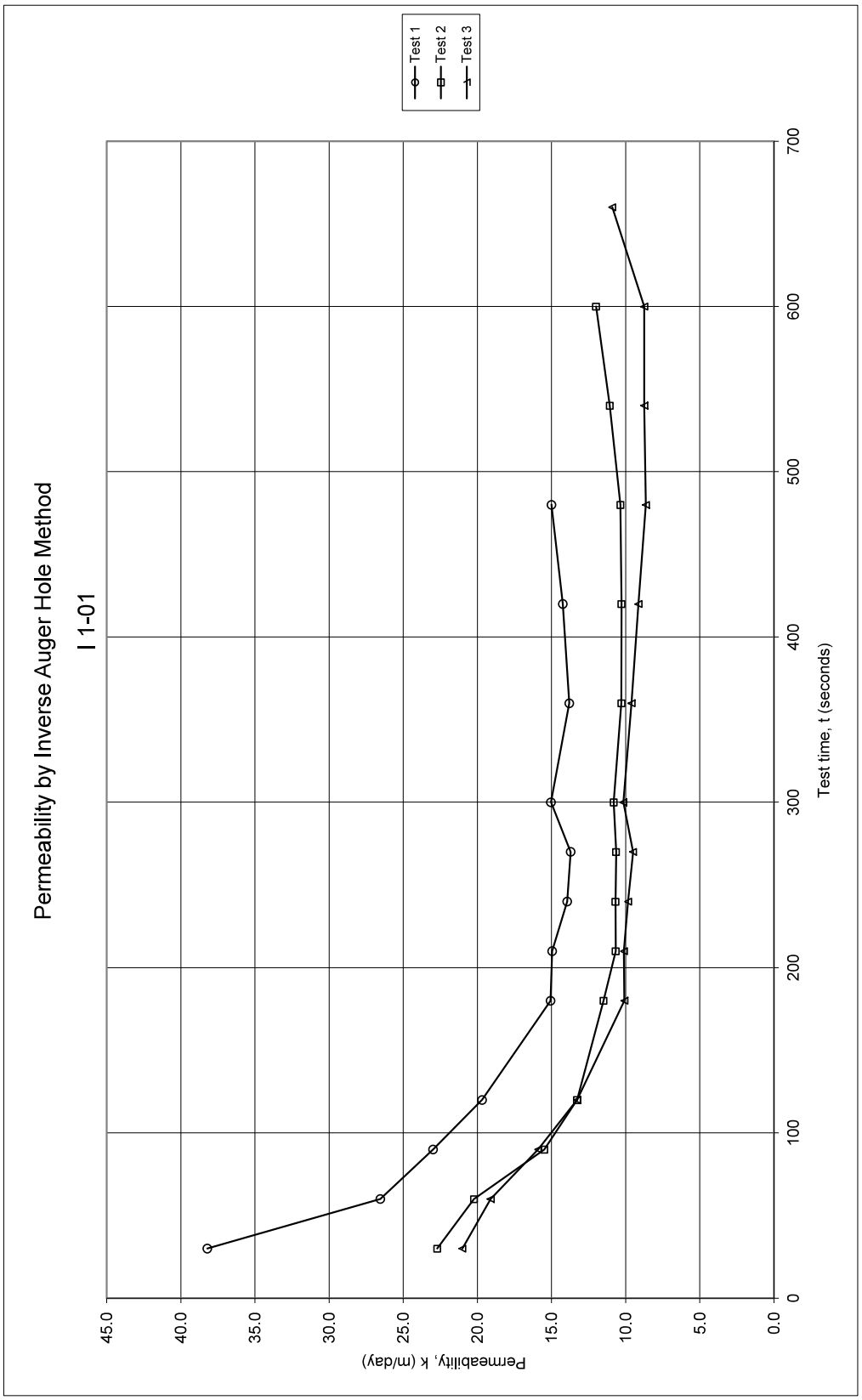
Spreadsheet Legend

	Required input
	Calculated field
	Comment field
X	Field not used
	Fixed field

Parameter	Description	Value	Units
K	Permeability	X	m/s
r	radius of test hole	0.043	m
t	time since start of measurement	X	s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t		m
h_t	Water column height at time t	X	m
h_0	h_t at t=0	X	m



Test 1					Test 2					Test 3				
t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02	X	X	0	0	1.02	X	X	0	0	1.02	X	X
30	0.48	0.54	4.4E-04	38.2	30	0.32	0.7	2.6E-04	22.7	30	0.3	0.72	2.4E-04	21.0
60	0.6	0.42	3.1E-04	26.5	60	0.5	0.52	2.3E-04	20.2	60	0.48	0.54	2.2E-04	19.1
90	0.7	0.32	2.7E-04	23.0	90	0.55	0.47	1.8E-04	15.5	90	0.56	0.46	1.8E-04	15.9
120	0.75	0.27	2.3E-04	19.7	120	0.6	0.42	1.5E-04	13.3	120	0.6	0.42	1.5E-04	13.3
180	0.8	0.22	1.7E-04	15.1	180	0.7	0.32	1.3E-04	11.5	180	0.65	0.37	1.2E-04	10.1
210	0.85	0.17	1.7E-04	15.0	210	0.73	0.29	1.2E-04	10.7	210	0.71	0.31	1.2E-04	10.1
240	0.87	0.15	1.6E-04	13.9	240	0.78	0.24	1.2E-04	10.7	240	0.75	0.27	1.1E-04	9.8
270	0.9	0.12	1.6E-04	13.7	270	0.82	0.2	1.2E-04	10.6	270	0.78	0.24	1.1E-04	9.5
300	0.95	0.07	1.7E-04	15.0	300	0.86	0.16	1.3E-04	10.8	300	0.84	0.18	1.2E-04	10.2
360	0.97	0.05	1.6E-04	13.8	360	0.9	0.12	1.2E-04	10.3	360	0.88	0.14	1.1E-04	9.6
420	1	0.02	1.6E-04	14.2	420	0.94	0.08	1.2E-04	10.3	420	0.91	0.11	1.1E-04	9.1
480	1.02	0	1.7E-04	15.0	480	0.97	0.05	1.2E-04	10.4	480	0.93	0.09	1.0E-04	8.6
					540	1	0.02	1.3E-04	11.1	540	0.96	0.06	1.0E-04	8.8
					600	1.02	0	1.4E-04	12.0	600	0.98	0.04	1.0E-04	8.7
										660	1.02	0	1.3E-04	10.9
		AVERAGE	2.2E-04	18.6			AVERAGE	1.5E-04	12.9			AVERAGE	1.3E-04	11.7



\\Golder.gds\gap\Perth\Jobs\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\08 Analysis\Permeability\Permeability Inverse Auger Hole Method.xls

Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

$$K = 1.15r\frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}(h_t)}{t - t_0}$$

Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-02

Spreadsheet Legend

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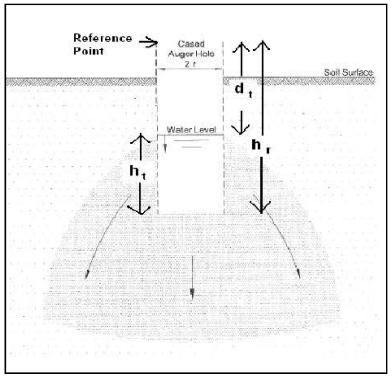
Parameter	Description	Value	Units
K	Permeability		m/s
r	radius of test hole	0.043	m
t	time since start of measurement		s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t		m
h_t	Water column height at time t		m
h_0	h_t at t=0		m



Test 1

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02		
30	0.3	0.72	2.4E-04	21.0
60	0.33	0.69	1.4E-04	11.8
90	0.37	0.65	1.0E-04	9.0
120	0.39	0.63	8.4E-05	7.3
180	0.41	0.61	6.0E-05	5.2
210	0.43	0.59	5.4E-05	4.7
240	0.45	0.57	5.1E-05	4.4
270	0.46	0.56	4.6E-05	4.0
300	0.47	0.55	4.3E-05	3.7
360	0.5	0.52	3.9E-05	3.4
420	0.52	0.5	3.5E-05	3.1
480	0.53	0.49	3.2E-05	2.7
540	0.535	0.485	2.9E-05	2.5
600	0.55	0.47	2.7E-05	2.3
660	0.56	0.46	2.5E-05	2.2
720	0.57	0.45	2.4E-05	2.0
780	0.59	0.43	2.3E-05	2.0
840	0.61	0.41	2.3E-05	1.9
900	0.62	0.4	2.2E-05	1.9
		AVERAGE	5.8E-05	5.0

Test 2

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02		
30	0.21	0.81	1.6E-04	13.9
60	0.24	0.78	9.4E-05	8.1
90	0.27	0.75	7.2E-05	6.2
120	0.3	0.72	6.1E-05	5.3
180	0.31	0.71	4.2E-05	3.6
210	0.33	0.69	3.9E-05	3.4
240	0.345	0.675	3.6E-05	3.1
270	0.36	0.66	3.4E-05	2.9
300	0.385	0.635	3.3E-05	2.9
360	0.42	0.6	3.1E-05	2.7
420	0.44	0.58	2.8E-05	2.4
480	0.46	0.56	2.6E-05	2.3
540	0.48	0.54	2.5E-05	2.1
600	0.5	0.52	2.3E-05	2.0
660	0.51	0.51	2.2E-05	1.9
720	0.52	0.5	2.1E-05	1.8
780	0.53	0.49	2.0E-05	1.7
840	0.545	0.475	1.9E-05	1.6
900	0.555	0.465	1.8E-05	1.6
		AVERAGE	4.2E-05	3.7

Test 3

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02		
30	0.2	0.82	1.5E-04	13.2
60	0.22	0.8	8.5E-05	7.3
90	0.25	0.77	6.5E-05	5.7
120	0.27	0.75	5.4E-05	4.6
180	0.29	0.73	3.9E-05	3.4
210	0.3	0.72	3.5E-05	3.0
240	0.32	0.7	3.3E-05	2.8
270	0.34	0.68	3.1E-05	2.7
300	0.36	0.66	3.0E-05	2.6
360	0.4	0.62	2.9E-05	2.5
420	0.42	0.6	2.6E-05	2.3
480	0.44	0.58	2.5E-05	2.1
540	0.46	0.56	2.3E-05	2.0
600	0.48	0.54	2.2E-05	1.9
660	0.5	0.52	2.1E-05	1.8
720	0.51	0.51	2.0E-05	1.7
780	0.52	0.5	1.9E-05	1.6
840	0.53	0.49	1.8E-05	1.6
900	0.54	0.48	1.7E-05	1.5
1200	0.6	0.42	1.5E-05	1.3
1500	0.65	0.37	1.4E-05	1.2
1800	0.7	0.32	1.3E-05	1.1
		AVERAGE	3.6E-05	3.1

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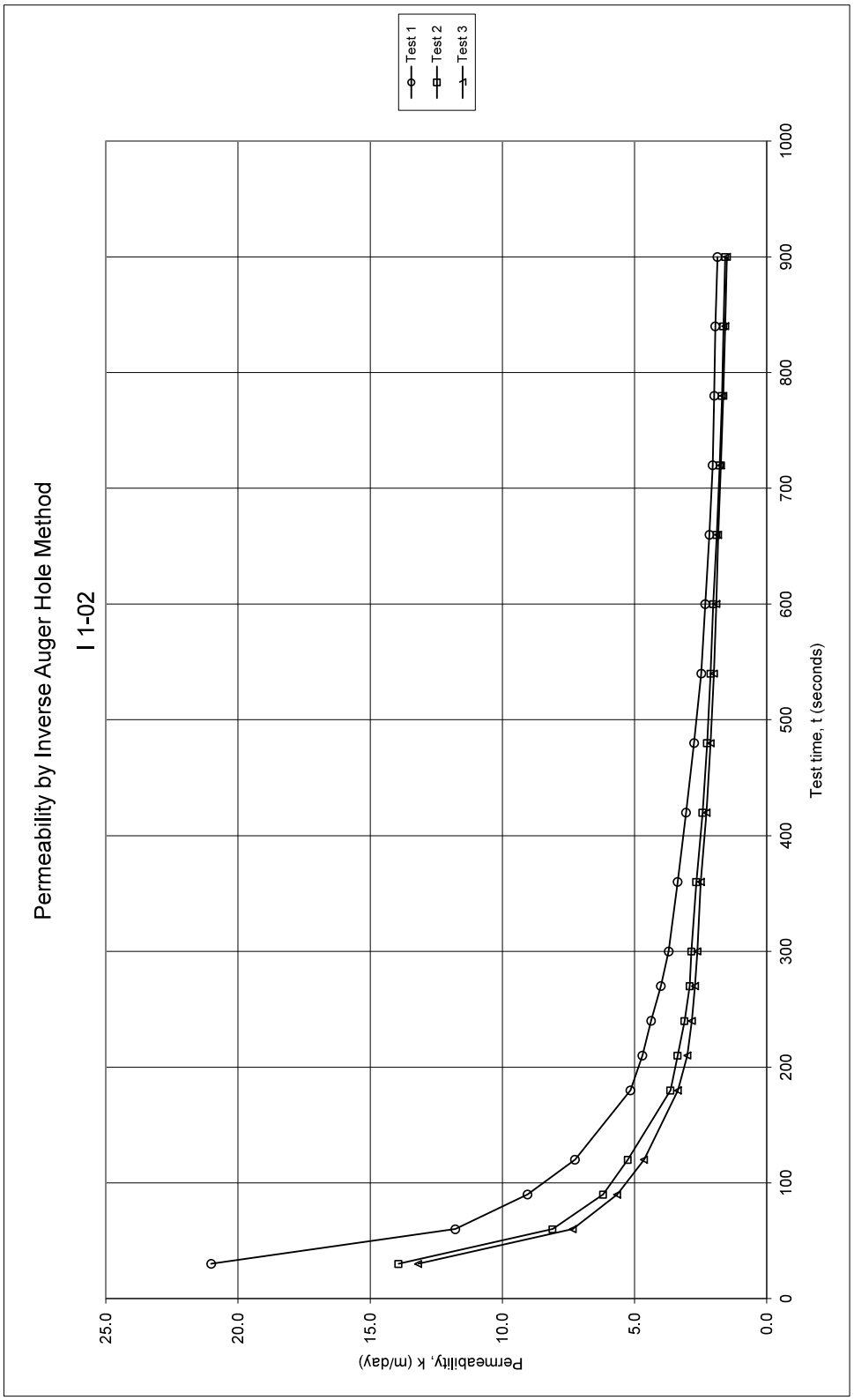

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Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

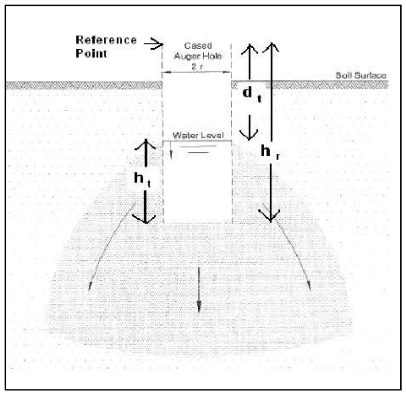
Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-03

$$K = 1.15r\frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}(h_t)}{t - t_0}$$

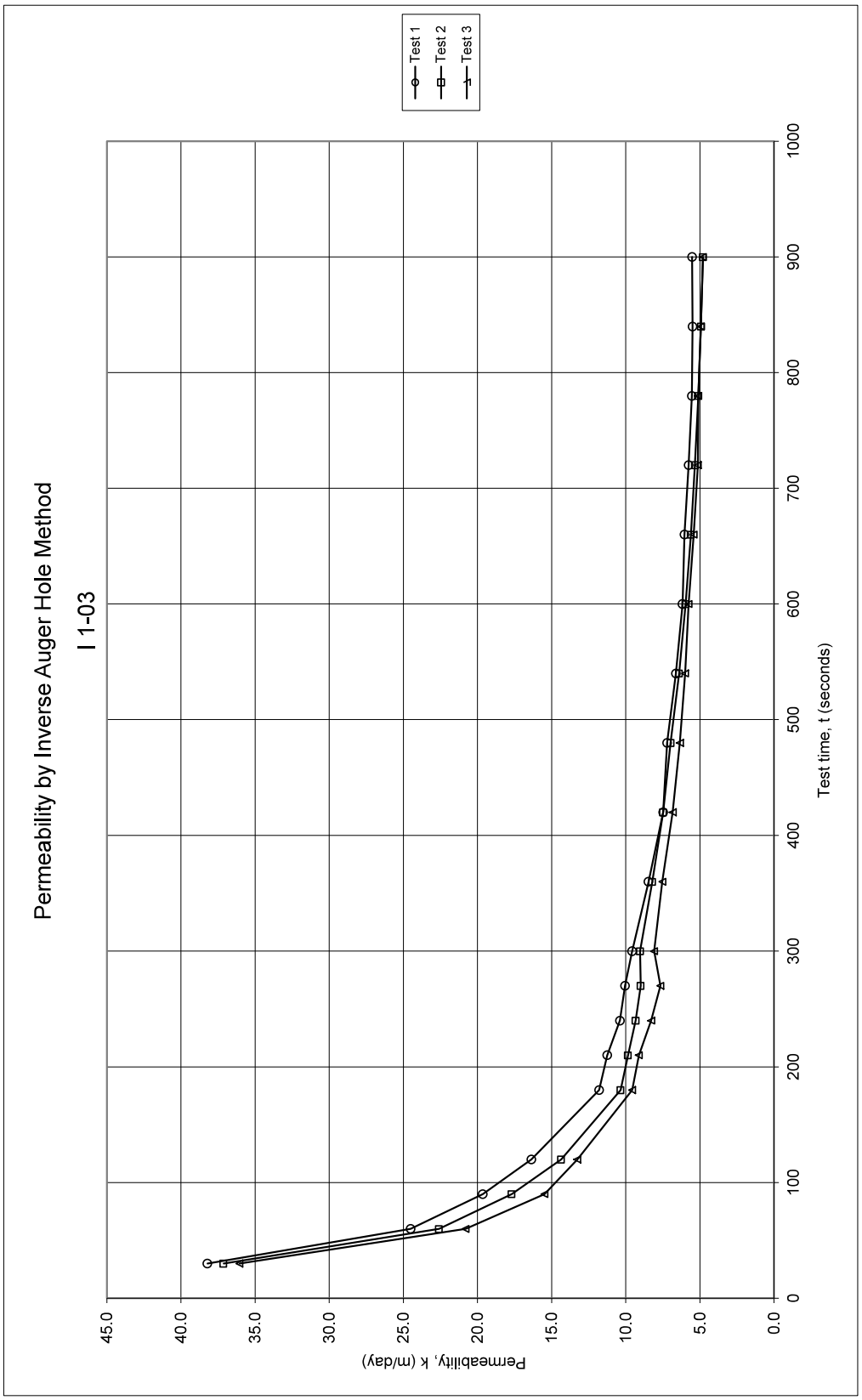
Spreadsheet Legend

	Required input
	Calculated field
	Comment field
X	Field not used
	Fixed field

Parameter	Description	Value	Units
K	Permeability		m/s
r	radius of test hole	0.043	m
t	time since start of measurement		s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t		m
h_t	Water column height at time t		m
h_0	h_t at t=0		m



Test 1					Test 2					Test 3				
t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02			0	0	1.02			0	0	1.02		
30	0.48	0.54	4.4E-04	38.2	30	0.47	0.55	4.3E-04	37.1	30	0.46	0.56	4.2E-04	36.0
60	0.57	0.45	2.8E-04	24.5	60	0.54	0.48	2.6E-04	22.6	60	0.51	0.51	2.4E-04	20.8
90	0.64	0.38	2.3E-04	19.7	90	0.6	0.42	2.0E-04	17.7	90	0.55	0.47	1.8E-04	15.5
120	0.68	0.34	1.9E-04	16.4	120	0.63	0.39	1.7E-04	14.4	120	0.6	0.42	1.5E-04	13.3
180	0.71	0.31	1.4E-04	11.8	180	0.66	0.36	1.2E-04	10.4	180	0.63	0.39	1.1E-04	9.6
210	0.75	0.27	1.3E-04	11.3	210	0.7	0.32	1.1E-04	9.9	210	0.67	0.35	1.1E-04	9.1
240	0.77	0.25	1.2E-04	10.4	240	0.73	0.29	1.1E-04	9.3	240	0.685	0.335	9.6E-05	8.3
270	0.8	0.22	1.2E-04	10.0	270	0.76	0.26	1.0E-04	9.0	270	0.7	0.32	8.9E-05	7.7
300	0.82	0.2	1.1E-04	9.6	300	0.8	0.22	1.0E-04	9.0	300	0.76	0.26	9.4E-05	8.1
360	0.84	0.18	9.8E-05	8.5	360	0.83	0.19	9.5E-05	8.2	360	0.8	0.22	8.7E-05	7.5
420	0.85	0.17	8.7E-05	7.5	420	0.85	0.17	8.7E-05	7.5	420	0.82	0.2	7.9E-05	6.8
480	0.88	0.14	8.3E-05	7.2	480	0.87	0.15	8.1E-05	7.0	480	0.84	0.18	7.3E-05	6.3
540	0.89	0.13	7.7E-05	6.6	540	0.88	0.14	7.4E-05	6.4	540	0.86	0.16	6.9E-05	6.0
600	0.9	0.12	7.1E-05	6.2	600	0.89	0.13	6.9E-05	6.0	600	0.88	0.14	6.7E-05	5.8
660	0.92	0.1	7.0E-05	6.0	660	0.9	0.12	6.5E-05	5.6	660	0.89	0.13	6.3E-05	5.4
720	0.93	0.09	6.7E-05	5.8	720	0.91	0.11	6.2E-05	5.3	720	0.9	0.12	6.0E-05	5.1
780	0.94	0.08	6.4E-05	5.5	780	0.92	0.1	5.9E-05	5.1	780	0.92	0.1	5.9E-05	5.1
840	0.955	0.065	6.4E-05	5.5	840	0.93	0.09	5.7E-05	4.9	840	0.93	0.09	5.7E-05	4.9
900	0.97	0.05	6.4E-05	5.5	900	0.94	0.08	5.6E-05	4.8	900	0.94	0.08	5.6E-05	4.8
		AVERAGE	1.3E-04	11.4			AVERAGE	1.2E-04	10.5			AVERAGE	1.1E-04	9.8



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Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

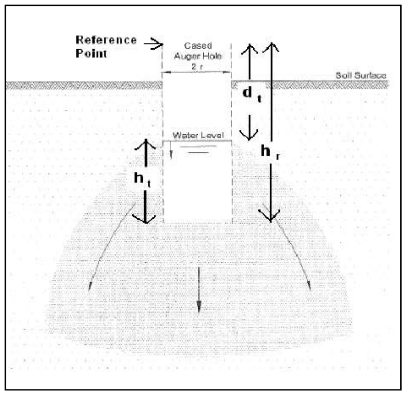
$$K = 1.15r\frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}(h_t)}{t - t_0}$$

Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-04

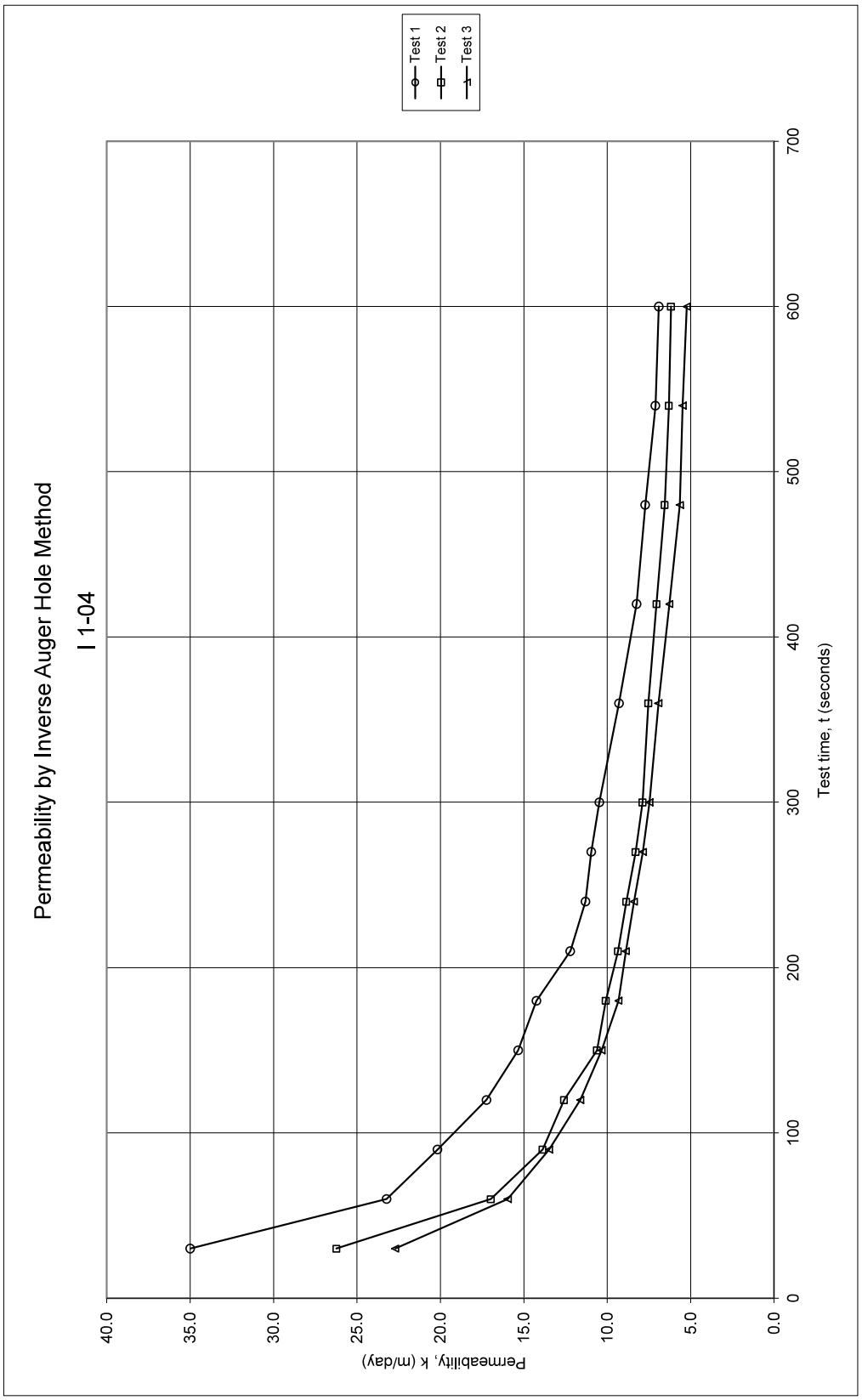
Spreadsheet Legend

	Required input
	Calculated field
	Comment field
X	Field not used
	Fixed field

Parameter	Description	Value	Units
K	Permeability	X	m/s
r	radius of test hole	0.043	m
t	time since start of measurement	X	s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t		m
h_t	Water column height at time t	X	m
h_0	h_t at t=0	X	m



Test 1					Test 2					Test 3				
t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02	X	X	0	0	1.02	X	X	0	0	1.02	X	X
30	0.45	0.57	4.1E-04	35.0	30	0.36	0.66	3.0E-04	26.2	30	0.32	0.7	2.6E-04	22.7
60	0.55	0.47	2.7E-04	23.2	60	0.44	0.58	2.0E-04	17.0	60	0.42	0.6	1.8E-04	16.0
90	0.65	0.37	2.3E-04	20.2	90	0.51	0.51	1.6E-04	13.9	90	0.5	0.52	1.6E-04	13.5
120	0.7	0.32	2.0E-04	17.2	120	0.58	0.44	1.5E-04	12.6	120	0.55	0.47	1.3E-04	11.6
150	0.74	0.28	1.8E-04	15.3	150	0.6	0.42	1.2E-04	10.6	150	0.59	0.43	1.2E-04	10.3
180	0.78	0.24	1.6E-04	14.2	180	0.65	0.37	1.2E-04	10.1	180	0.62	0.4	1.1E-04	9.3
210	0.8	0.24	1.4E-04	12.2	210	0.68	0.34	1.1E-04	9.3	210	0.66	0.36	1.0E-04	8.9
240	0.83	0.22	1.3E-04	11.3	240	0.71	0.31	1.0E-04	8.9	240	0.69	0.33	9.7E-05	8.4
270	0.85	0.19	1.3E-04	11.0	270	0.73	0.29	9.6E-05	8.3	270	0.71	0.31	9.1E-05	7.9
300	0.87	0.17	1.2E-04	10.5	300	0.75	0.27	9.1E-05	7.9	300	0.73	0.29	8.6E-05	7.5
360	0.88	0.15	1.1E-04	9.3	360	0.8	0.22	8.7E-05	7.5	360	0.77	0.25	8.0E-05	6.9
420	0.9	0.14	9.5E-05	8.2	420	0.83	0.19	8.2E-05	7.0	420	0.79	0.23	7.3E-05	6.3
480	0.91	0.12	8.9E-05	7.7	480	0.85	0.17	7.6E-05	6.5	480	0.8	0.22	6.5E-05	5.6
540	0.93	0.11	8.2E-05	7.1	540	0.875	0.145	7.3E-05	6.3	540	0.83	0.19	6.3E-05	5.5
600	0.95	0.09	8.0E-05	6.9	600	0.9	0.12	7.1E-05	6.2	600	0.85	0.17	6.1E-05	5.2
		AVERAGE	1.6E-04	14.0			AVERAGE	1.2E-04	10.6			AVERAGE	1.1E-04	9.7



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Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

$$K = 1.15r\frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}(h_t)}{t - t_0}$$

Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-05

Spreadsheet Legend

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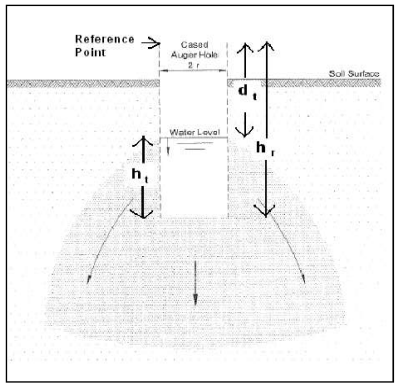
	Comment field
X	Field not used
	Fixed field

Parameter	Description	Value	Units
K	Permeability		m/s
r	radius of test hole	0.043	m
t	time since start of measurement		s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t		m
h_t	Water column height at time t		m
h_0	h_t at t=0		m



Test 1

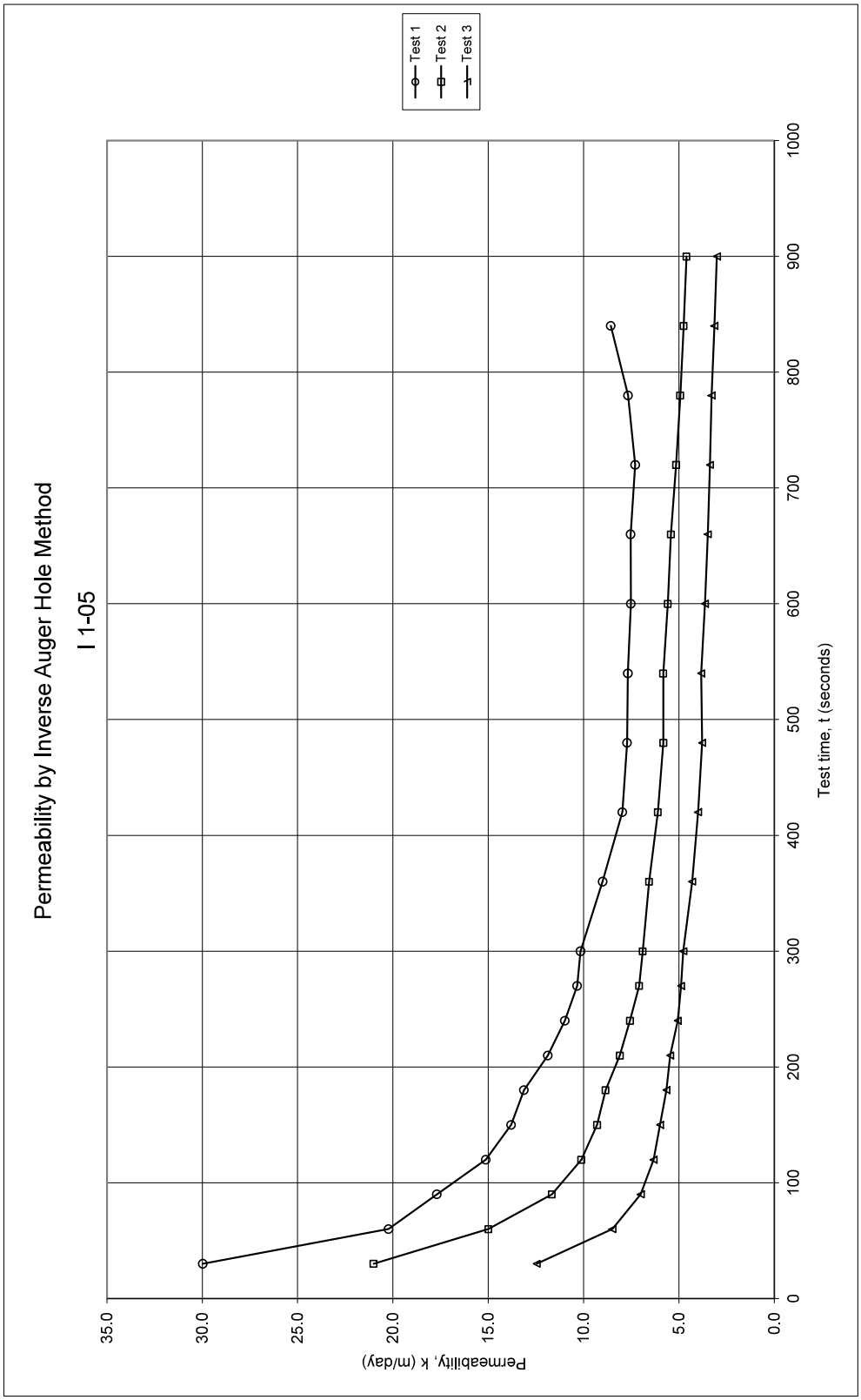
t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02		
30	0.4	0.62	3.5E-04	30.0
60	0.5	0.52	2.3E-04	20.2
90	0.6	0.42	2.0E-04	17.7
120	0.65	0.37	1.8E-04	15.1
150	0.7	0.32	1.6E-04	13.8
180	0.75	0.27	1.5E-04	13.1
210	0.77	0.25	1.4E-04	11.9
240	0.79	0.23	1.3E-04	11.0
270	0.81	0.21	1.2E-04	10.3
300	0.84	0.18	1.2E-04	10.2
360	0.86	0.16	1.0E-04	9.0
420	0.87	0.15	9.2E-05	8.0
480	0.9	0.12	8.9E-05	7.7
540	0.93	0.09	8.9E-05	7.7
600	0.95	0.07	8.7E-05	7.5
660	0.97	0.05	8.7E-05	7.5
720	0.98	0.04	8.4E-05	7.3
780	1	0.02	8.9E-05	7.7
840	1.02	0	9.9E-05	8.6
		AVERAGE	1.4E-04	11.8

Test 2

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02		
30	0.3	0.72	2.4E-04	21.0
60	0.4	0.62	1.7E-04	15.0
90	0.45	0.57	1.4E-04	11.7
120	0.5	0.52	1.2E-04	10.1
150	0.55	0.47	1.1E-04	9.3
180	0.6	0.42	1.0E-04	8.8
210	0.625	0.395	9.4E-05	8.1
240	0.65	0.37	8.8E-05	7.6
270	0.67	0.35	8.2E-05	7.1
300	0.7	0.32	8.0E-05	6.9
360	0.75	0.27	7.6E-05	6.6
420	0.78	0.24	7.1E-05	6.1
480	0.81	0.21	6.7E-05	5.8
540	0.85	0.17	6.7E-05	5.8
600	0.87	0.15	6.5E-05	5.6
660	0.89	0.13	6.3E-05	5.4
720	0.9	0.12	6.0E-05	5.1
780	0.91	0.11	5.7E-05	4.9
840	0.92	0.1	5.5E-05	4.7
900	0.93	0.09	5.3E-05	4.6
		AVERAGE	9.3E-05	8.0

Test 3

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02		
30	0.19	0.83	1.4E-04	12.5
60	0.25	0.77	9.8E-05	8.5
90	0.3	0.72	8.1E-05	7.0
120	0.35	0.67	7.3E-05	6.3
150	0.4	0.62	6.9E-05	6.0
180	0.44	0.58	6.6E-05	5.7
210	0.48	0.54	6.3E-05	5.5
240	0.5	0.52	5.9E-05	5.1
270	0.53	0.49	5.7E-05	4.9
300	0.56	0.46	5.5E-05	4.8
360	0.59	0.43	5.0E-05	4.3
420	0.62	0.4	4.6E-05	4.0
480	0.65	0.37	4.4E-05	3.8
540	0.7	0.32	4.4E-05	3.8
600	0.72	0.3	4.2E-05	3.6
660	0.74	0.28	4.0E-05	3.5
720	0.76	0.26	3.9E-05	3.4
780	0.78	0.24	3.8E-05	3.3
840	0.79	0.23	3.6E-05	3.1
900	0.8	0.22	3.5E-05	3.0
		AVERAGE	5.9E-05	5.1



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Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

$$K = 1.15r\frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}(h_t)}{t - t_0}$$

Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-06

Spreadsheet Legend

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	Calculated field
	Comment field
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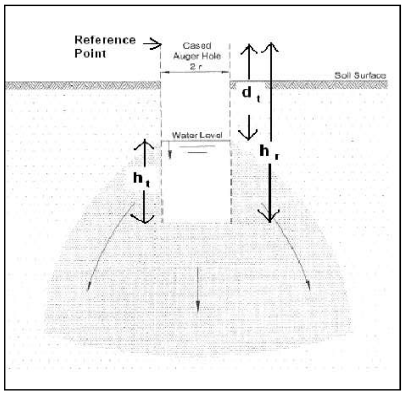
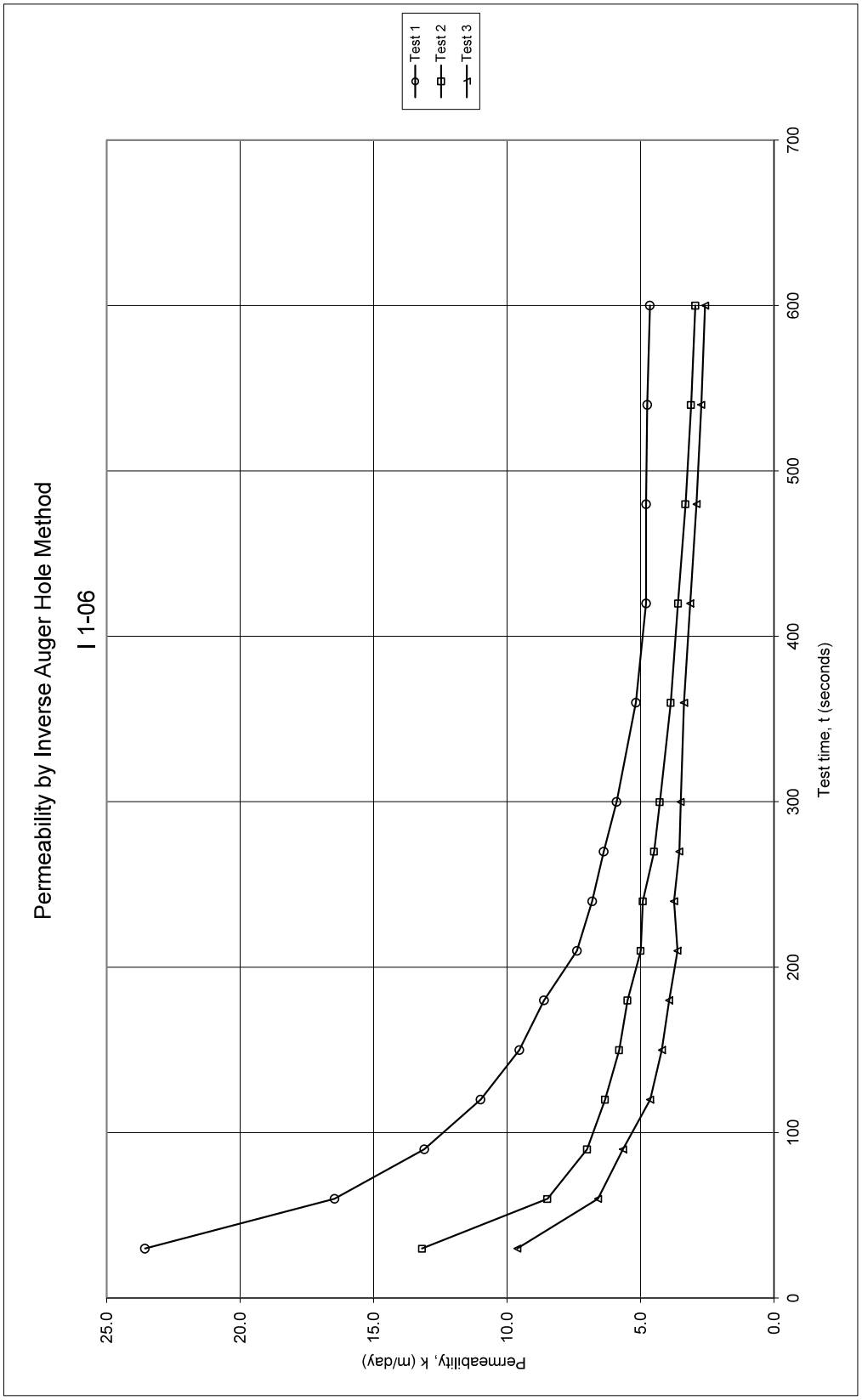
Parameter	Description	Value	Units
K	Permeability		m/s
r	radius of test hole	0.043	m
t	time since start of measurement		s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t		m
h_t	Water column height at time t		m
h_0	h_t at t=0		m



Test 1					Test 2					Test 3				
t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02			0	0	1.02			0	0	1.02		
30	0.33	0.69	2.7E-04	23.6	30	0.2	0.82	1.5E-04	13.2	30	0.15	0.87	1.1E-04	9.6
60	0.43	0.59	1.9E-04	16.5	60	0.25	0.77	9.8E-05	8.5	60	0.2	0.82	7.6E-05	6.6
90	0.49	0.53	1.5E-04	13.1	90	0.3	0.72	8.1E-05	7.0	90	0.25	0.77	6.5E-05	5.7
120	0.53	0.49	1.3E-04	11.0	120	0.35	0.67	7.3E-05	6.3	120	0.27	0.75	5.4E-05	4.6
150	0.56	0.46	1.1E-04	9.5	150	0.39	0.63	6.7E-05	5.8	150	0.3	0.72	4.9E-05	4.2
180	0.59	0.43	1.0E-04	8.6	180	0.43	0.59	6.4E-05	5.5	180	0.33	0.69	4.5E-05	3.9
210	0.61	0.43	8.5E-05	7.4	210	0.45	0.57	5.8E-05	5.0	210	0.35	0.67	4.2E-05	3.6
240	0.63	0.41	7.9E-05	6.8	240	0.49	0.53	5.7E-05	4.9	240	0.4	0.62	4.3E-05	3.7
270	0.64	0.39	7.4E-05	6.4	270	0.5	0.52	5.2E-05	4.5	270	0.42	0.6	4.1E-05	3.5
300	0.66	0.38	6.8E-05	5.9	300	0.52	0.5	5.0E-05	4.3	300	0.45	0.57	4.1E-05	3.5
360	0.69	0.36	6.0E-05	5.2	360	0.55	0.47	4.5E-05	3.9	360	0.5	0.52	3.9E-05	3.4
420	0.74	0.33	5.6E-05	4.8	420	0.58	0.44	4.2E-05	3.6	420	0.53	0.49	3.6E-05	3.1
480	0.78	0.28	5.5E-05	4.8	480	0.6	0.42	3.8E-05	3.3	480	0.55	0.47	3.4E-05	2.9
540	0.81	0.24	5.5E-05	4.7	540	0.62	0.4	3.6E-05	3.1	540	0.57	0.45	3.2E-05	2.7
600	0.85	0.21	5.4E-05	4.7	600	0.64	0.38	3.4E-05	2.9	600	0.59	0.43	3.0E-05	2.6
		AVERAGE	1.0E-04	8.9			AVERAGE	6.3E-05	5.5			AVERAGE	4.9E-05	4.3



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APPENDIX H

Summary of Analysis of Infiltration Testing

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX H
Infiltration Testing

1.0 INTRODUCTION

This appendix presents the results from the infiltration testing carried out during the development of the pumping test wells at Bayswater Dive Structure, Airport West Station, Consolidated Airport Terminal Station and Forrestfield Dive Structure. In addition the results from monitoring the infiltration basin at the Bayswater Dive Structure during the pumping test are described.

2.0 BAYSWATER DIVE STRUCTURE

2.1 Infiltration Test During Well Development

The pumping test well (PW2-02) at Bayswater Dive Structure was developed on 4 and 5 November 2014. The groundwater from the development was pumped into two different infiltration pits excavated with a backhoe. Attachment H1 shows the location of the infiltration basins while Figure H1 shows photos of the two infiltration pits.



Figure H1: Infiltration Testing at Bayswater Dive Structure (Pit 1 left, Pit 2 right)

The dimensions of the two infiltration pits were approximately 5 m long by 5 m wide and 1 m deep, giving a volume of 25 kL. Given the loose sandy conditions it was not possible to form steep batter slopes and the pits kept collapsing during excavation. The dimensions are therefore approximate only.

PW2-02 was developed at a rate of approximately 4 L/s for a total period of 12 hours (over 2 days). Generally, it was found that the infiltration pits were able to infiltrate all of the discharged water by alternating the pumping into the pits.

The infiltration test consisted of filling the infiltration pit with water from the well development process and then measuring the water level decline in the pit over time. Infiltration Pit 1 was tested three times while Infiltration Pit 2 was only tested once.

Figure H2 shows the resulting water level decline while Table H1 presents the estimated infiltration rate for each test.

APPENDIX H
Infiltration Testing

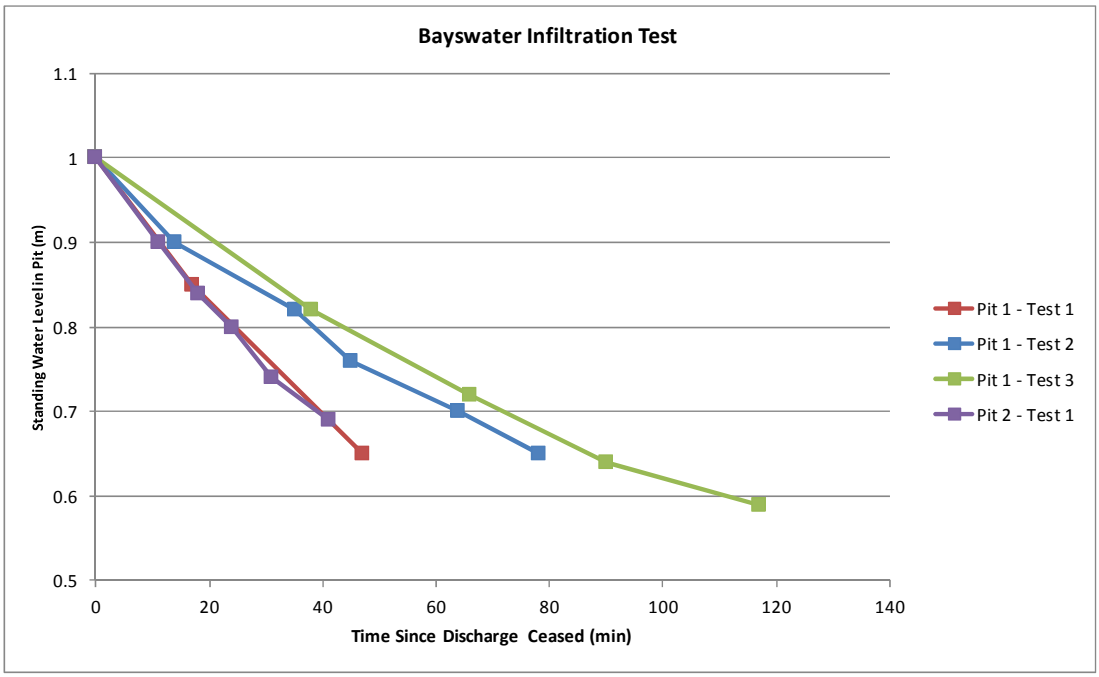

Figure H2: Infiltration Testing Data at Bayswater Dive Structure

Table H1: Estimated Infiltration Rate at Bayswater Dive Structure

Test	Approximate Infiltration Rate	
	L/s	m/d *
Pit 1 – Test 1	3.1	11
Pit 1 – Test 2	1.9	7
Pit 1 – Test 3	1.6	6
Pit 2 – Test 1	3.1	11

* Assuming all water is infiltrating through the base of the Infiltration Pit, which is a conservative assumption.

The results indicate:

- The derived infiltration rate ranges from 6 to 11 m/d assuming that all water infiltrated through the 25 m^2 base of the pits.
- Similar water level decline was observed in both pits for Test 1, resulting in similar infiltration rate at the two locations.
- The infiltration rates for Infiltration Pit 1 was found to decrease in Test 2 and Test 3, which is likely due to progressive saturation of the soil profile and fines being deposited at the bottom of the pit during the development (the development water contained some fines from the well).

2.2 Infiltration During Pumping Test

Due to restrictions on the groundwater discharge disposal, the pumped groundwater was also infiltrated on site during the pumping test carried out from 3 to 6 February 2015. Attachment H1 shows the location of the

main infiltration basin and a contingency infiltration basin. The footprints of the basins were approximately 250 m^2 and 25 m^2. The main basin was only excavated to approximately 1 m depth at the periphery of the basin to create the bund walls while the central area was not excavated. The estimated volume of the main basin is approximately 115 kL.

A monitoring well (unknown construction details but with a depth of 4.5 m) existed inside the main infiltration basin. The groundwater level was measured to be 1.8 m below ground surface prior to start of the infiltration. A groundwater level logger was installed in the well to measure the change in groundwater level during the infiltration. The results from the groundwater level logger indicate that the groundwater level rose to the surface, indicating that fully saturated conditions occurred below the infiltration basin.

The discharge rate from the pumping test was approximately 12 L/s, but after 6 hours the basin was full and the discharge was therefore diverted into the contingency basin. The total volume of water pumped to the main basin within the first 6 hours was approximately 260 kL. With a storage volume of 115 kL, a total of 145 kL would have infiltrated over the 6 hour period, which corresponds to an average infiltration rate of approximately 2 m/d.

3.0 AIRPORT WEST STATION

The pumping test well (PW2-01) at Airport West Station was developed on 25 November 2014. The groundwater from the development was pumped into an infiltration pit excavated with a backhoe. Attachment H2 shows the location of the infiltration pit while Figure H3 shows a photo of the infiltration pit.



Figure H3: Infiltration Pit at Airport West Station

The dimensions of the infiltration pit were measured to be 10.3 m long by 2.9 m wide and 1.1 m deep, giving a volume of 33 kL. The ground was fairly consolidated and steep batter slopes were therefore achievable.

PW2-01 was developed at a rate of approximately 4 L/s. Generally, it was found that the infiltration pit would fill up and that well development had to cease until the water had infiltrated in the infiltration pit. This indicates that the infiltration rate is less than 10 m/d.

The infiltration tests consisted of pumping water into the pits until the standing water level in the pit was approximately 0.8 m. The pumping was then ceased and the decline in water level in the infiltration pit was measured over time.

It took approximately 110 minutes of pumping at 4 L/s (total volume of 26 kL) to fill the infiltration pit to 0.8 m (total volume in pit of 20 kL), indicating that approximately 6 kL infiltrated during the 110 minutes, corresponding to an average infiltration rate of 0.9 L/s or approximately 2.6 m/d during the time of filling.

Figure H4 shows the resulting water level decline and indicates that the groundwater declined by approximately 0.13 m over a period of 55 minutes, corresponding to an infiltration rate of 1.2 L/s or approximately 3.4 m/d (assuming that all water is infiltrating through the base of the Infiltration Pit, which is a conservative assumption).

It is noted that the calculated infiltration rate for the emptying of the pit is more precise than for filling due to the accumulation of potential measurement errors.

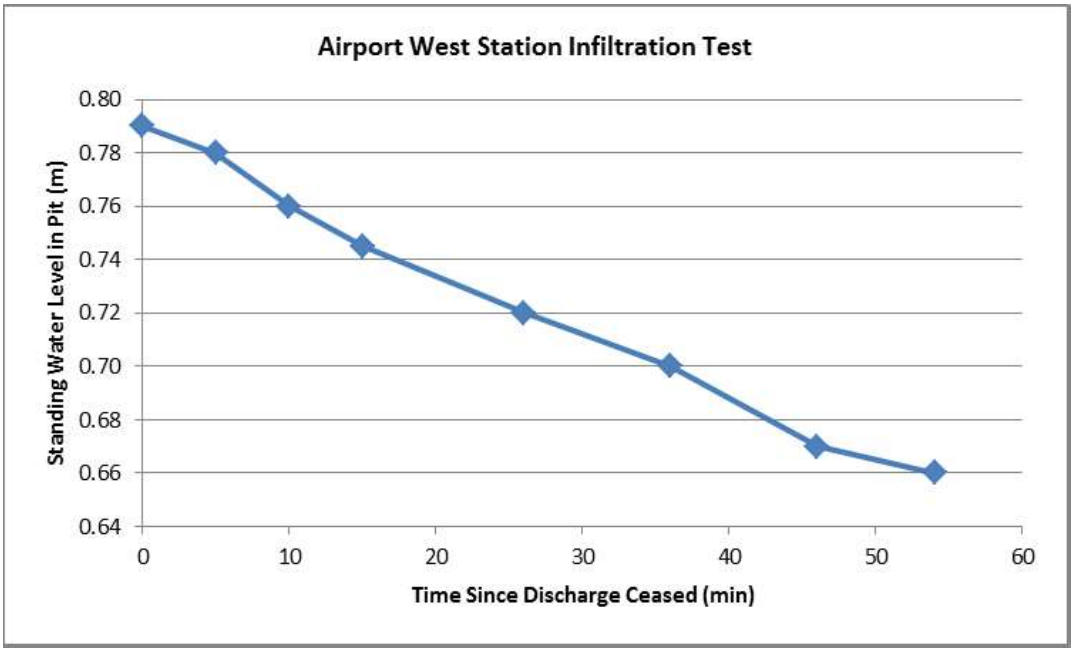

Figure H4: Infiltration Test Data at Airport West Station

4.0 CONSOLIDATED AIRPORT TERMINAL STATION

The pumping test well (PW1-02) at Consolidated Airport Terminal Station was developed on 25 November 2014. The groundwater from the development was pumped into an infiltration pit excavated with a backhoe. Attachment H3 shows the location of the infiltration pit while Figure H5 shows a photo of the infiltration pit.

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Infiltration Testing



Figure H5: Infiltration Pit at Consolidated Station (left before discharge, right after start of discharge)

The dimensions of the infiltration pit were approximately 6 m long by 2 m wide and 1 m deep, giving a volume of 12 kL. The ground was fairly consolidated and steep batter slopes were therefore achievable. The red colour on the left photo in Figure H5 indicates that coffee rock was present at approximately 1 m depth at the infiltration pit location.

PW1-02 was developed at a rate of approximately 8 L/s. The infiltration test consisted of filling the infiltration pit with water from the development process and then measuring the water level decline in the pit over time.

It took approximately 27 minutes of pumping at approximately 8 L/s (total volume of 13 kL) to fill the infiltration basin, indicating that approximately 1 kL infiltrated during the 27 minutes, corresponding to an average infiltration rate of 0.6 L/s or approximately 4 m/d during the time of filling.

Figure H6 shows the resulting water level decline and indicates that the groundwater declined by approximately 0.03 m over a period of 33 minutes, corresponding to an infiltration rate of 0.2 L/s or approximately 1 m/d (assuming that all water is infiltrating through the base of the Infiltration Pit, which is a conservative assumption).

It is noted that the calculated infiltration rate for the emptying of the pit is more precise than for filling due to the accumulation of potential measurement errors.

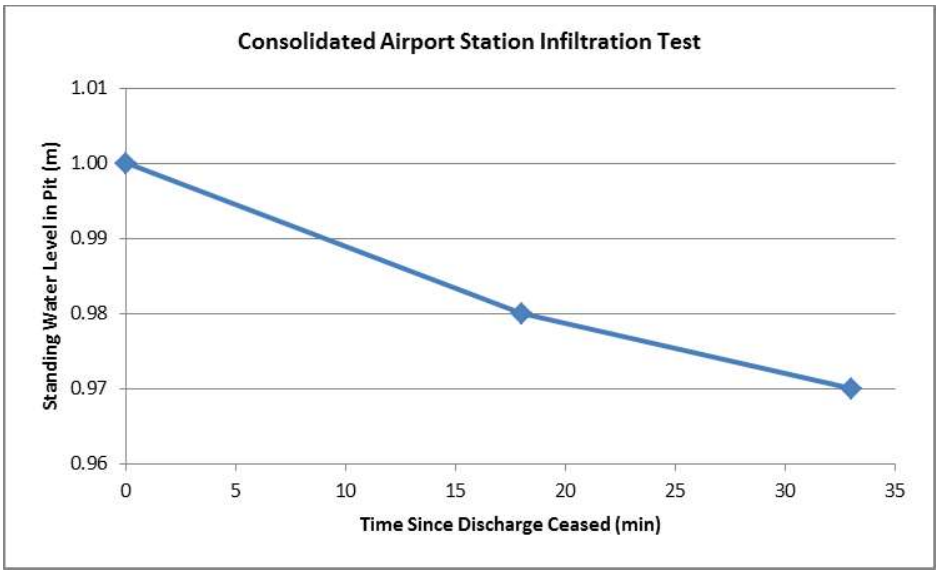

Figure H6: Infiltration Test Data at Consolidated Airport Terminal Station

5.0 FORRESTFIELD DIVE STRUCTURE

The pumping test well (PW1-01) at Forrestfield Dive Structure was developed on 7 November 2014. Some of the groundwater from the well development was pumped into an infiltration pit excavated with a backhoe. Attachment H4 shows the location of the infiltration pit while Figure H7 shows a photo of the infiltration pit.



Figure H7: Infiltration Pit at Forrestfield Dive Structure

The dimensions of the infiltration pit were approximately 3 m long by 1 m wide and 0.5 m deep, giving a volume of 1.5 kL. The ground was hard and steep batter slopes were therefore achievable.

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Infiltration Testing

PW1-01 was developed at a rate of approximately 6 to 10 L/s. The infiltration test consisted of filling the infiltration pit with water from the well development and then measuring the water level decline in the pit over time.

The capacity of the pit was reached within five minutes of the commencement of development of PW1-01. The change in water level in the pit was found to negligible between ceasing pumping into the pit and the end of the day (approximately 6 hours). The water level in the test pit was observed again the following day and the water level had not declined noticeably, indicating that the infiltration capacity of the soil at the infiltration location is negligible.

ATTACHMENT H1
Bayswater Infiltration Test Locations

May 2015
Reference No. 147642129-077-R-Rev0


Properties
Infiltration Pit 1
Infiltration Pit 2
PW2-02
Contingency Infiltration Basin
Main Infiltration Basin

ATTACHMENT H2
Airport West Infiltration Test Locations

May 2015
Reference No. 147642129-077-R-Rev0


Infiltration Pit
PW2-02

ATTACHMENT H3
Consolidated Infiltration Test Locations

May 2015
Reference No. 147642129-077-R-Rev0


I1-06
I1-04
PW1-02
Infiltration Pit
I1-05

ATTACHMENT H4
Forrestfield Infiltration Test Locations

May 2015
Reference No. 147642129-077-R-Rev0

Golder Associates


Infiltration Pit 1
PW1-01
I1-03
I1-02
I1-01
Maida Vale Rd
Ibis Pl
Palmer Crescent
Durant Way
Mack Pl
Dundas Rd
Imperial St
Eureka St
Milner Rd
Avonside Crescent
55

APPENDIX I

Limitations

May 2015
Report No. 147642129-077-R-Rev0

LIMITATIONS

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Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 44 1628 851851
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Pty Ltd
Level 3, 1 Havelock Street
West Perth, Western Australia 6005
Australia
T: +61 8 9213 7600

Golder Associates

APPENDIX 4 MONITORING DATA (STRATEGEN, 2020)

						Physical							Metals and Metaloids							Nutrients								
	Month	Date	Time	GWL (mBTOC)	GWL (mAHD)	pH	Change in pH	EC	Temp	Diss Oxy (mg/L)	Diss Oxy (%)	Redox (mV)	Arsenic	Cadmium	Copper	Chromium	Nickel	Lead	Zinc	Ammonia as N	Nitrite as N	Nitrate as N	Nitrite + Nitrate as N	TKN	TN	TP	FRP	Comments
Swan Canning WQIP (Short Term Limits)																								2.0	0.2		
Swan Canning WQIP (Long Term Limits)																								0.5	0.05		
Australian and NZ Guidelines for Fresh and Marine Water Quality						7.0-8.5							0.013	0.0002	0.003	0.0031	0.011	0.0034	0.008								
Australian Drinking Water Guidelines (2000)																						11.3					
MB01	September	28/09/2011	12:25	4.59	26.05	6.61	0.31		21.3	3.42	39	188															Bailed 30L, bore still not dry
	October	19/10/2011	11:40	4.21	26.43	6.62	0.01	0.45	21.5	2.76	31	188	0.009	<0.0001	0.024	0.0020	0.011	0.0620	0.170	0.65	0.27	1.20	1.47	1.00	2.40	0.61	0.003	Bailed 10L, bore still not dry, samples taken
	November	14/11/2011	8:45	5.23	25.41																						
	December	22/12/2011	9:25	5.86	24.78																						
	January	16/01/2012		6.41	24.23	5.87	-0.75	0.32	23.2	2.78	87	133	0.007	<0.0001	0.044	0.0070	0.022	0.1000	0.520	1.90	0.03	0.97	1.00	4.90	5.90	1.70	<0.002	Bailed 15L, Bore still not dry, samples taken
	February	22/02/2012		6.87	23.77																						
	March	27/03/2012		7.20	23.44																						
	April	23/04/2012		7.31	23.33	5.68	-0.19	0.25	22.2	2.72	32	67								0.07	<0.05	1.12	5.25	0.66	1.80	0.41	<0.002	Bailed 10L, Bore still not dry, samples taken
	May	24/05/2012		7.00	23.64																						
	June	27/06/2012		6.01	24.63																						
	July	26/07/2012		6.19	24.45	5.84	0.16	0.24	22.2	2.24	26	114								0.09	<0.05	0.63	0.66	0.45	1.10	0.21	0.008	Bailed 10L, Bore still not dry, samples taken
	August	29/08/2012		5.70	24.94																						
	September	26/09/2012		5.51	25.13																						
	October	24/10/2012		5.95	24.69																						
	November	21/11/2012		6.25	24.39	6.07	-0.23	0.23	22.3	3.10	32	145								0.05	0.20	0.25	0.45	0.66	0.93	0.68	<0.005	Bailed 10L, Bore still not dry, samples taken
	December	20/12/2012		6.32	24.32																						
	Minimum			4.21	23.33	5.68		0.23	21.3	2.24	26	67	0.007	0.0001	0.024	0.0020	0.011	0.0620	0.170	0.05	0.03	0.25	0.45	0.45	0.93	0.21	0.001	
	Maximum			7.31	26.43	6.62		0.45	23.2	3.42	87	188	0.009	0.0001	0.044	0.0070	0.022	0.1000	0.520	1.90	0.27	1.20	5.25	4.90	5.90	1.70	0.008	
	Median					5.97		0.25	22.2	2.77	32	139	0.008	0.0001	0.034	0.0045	0.017	0.0810	0.345	0.09	0.03	0.97	1.00	0.66	1.80	0.61	0.003	
MB07	September	28/09/2011		9.98	25.11	6.2	0.00	0.26	21.2	5.66	63	235																Bailed 30L, bore still not dry
	October	19/10/2011		10.21	24.88	6.15	-0.05	0.29	21.0	6.69	76	184	0.001	<0.0001	0.013	0.0020	0.004	0.0060	0.066	0.04	<0.005	7.50	7.50	0.43	8.00	0.38	<0.002	Bailed 10L, bore still not dry, samples taken
	November	14/11/2011		10.15	24.94																						
	December	22/12/2011		10.76	24.33																						
	January	16/01/2011		11.00	24.09	5.84	-0.31	0.25	21.8	15.18	134	188	0.001	<0.0001	0.018	0.0080	0.003	0.0070	0.058	0.10	<0.05	6.32	28.00	0.91	7.30	0.77	<0.002	Bailed 15L, Bore still not dry, samples taken
	February	22/02/2011		11.23	23.86																						
	March	27/03/2012		11.55	23.55																						
	April	23/04/2012		11.84	23.25	6.05	0.21	0.31	21.9	6.60	75	159								0.10	<0.05	7.67	34.03	0.25	8.00	0.20	<0.002	Bailed 8L, slow recharge, samples taken
	May	24/05/2012		11.45	23.64																						
	June	27/06/2012		10.99	24.11																						
	July	26/07/2012		11.14	23.95	6.34	0.29	0.14	20.8	4.52	50	164								0.08	<0.005	1.60	1.60	5.50	7.10	1.60	0.003	Bailed 15L, Bore still not dry, samples taken
	August	29/08/2012		10.74	24.35																						
	September	26/09/2012		10.54	24.55																						
	October	24/10/2012		10.86	24.23																						
	November	21/11/2012		10.96	24.13	6.25	0.09	0.20	21.4	4.53	49	156								<0.005	<0.005	6.10	6.10	4.10	10.00	1.10	<0.005	Bailed 10L, Bore still not dry, samples taken
	December	20/12/2012		11.12	23.97																						
	Minimum			9.98	23.25	5.84		0.14	20.8	4.52	49	156	0.001	0.0001	0.013	0.0020	0.003	0.0060	0.058	0.04	<0.005	1.60	1.60	0.25	7.10	0.20	0.001	
	Maximum			11.84	25.11	6.34		0.31	21.9	15.18	134	235	0.001	0.0001	0.018	0.0080	0.004	0.0070	0.066	0.10	0.03	7.67	34.03	5.50	10.00	1.60	0.003	
	Median					6.18		0.25	21.3	6.13	69	174	0.001	0.0001	0.016	0.0050	0.004	0.0065	0.062	0.09	0.00	6.32	7.50	0.91	8.00	0.77	0.001	
MB08	September	28/09/2011		3.95	28.49	6.15	0.00	0.69	20.4	6.88	77	229																Bailed 30L, bore still not dry
	October	19/10/2011		4.57	27.87	6.03	-0.12	0.92	20.1	6.65	76	202	0.001	<0.0001	0.006	<0.001	<0.001	0.0080	0.014	0.02	<0.005	22.00	22.00	0.81	23.00	0.18	<0.002	Bailed 10L, bore still not dry, samples taken
	November	14/11/2011		4.68	27.76																						
	December	22/12/2011		4.87	27.57																						
	January	16/01/2011		5.78	26.67	5.83	-0.20	0.71	22.8	24.73	271	204	0.002	<0.0001	0.047	0.0100	0.005	0.2400	0.170	0.19	<0.005	20.32	90.00	4.30	25.00	0.72	<0.002	Bailed 6L and bore ran dry, replenished in 15-20 mins, sample taken
	February	22/02/2012		6.55	25.89																						
	March	27/03/2012		dry																							dry
	April	23/04/2012		dry																							dry
	May	24/05/2012		dry																							dry
	June	27/06/2012		5.31	27.13																						
	July	26/07/2012		5.31	27.13															0.03	<0.005	18.00	18.00	2.50	20.00	0.52	0.017	Bailed 1L and bore ran dry, replenished in 25mins, samples taken
	August	29/08/2012		5.77	26.67																						
	September	26/09/2012		5.50	26.94																						
	October	24/10/2012		6.69	25.75																						
	November	21/11/2012		6.30	26.14	6.58	-0.75	0.01	23.0	6.23	73	115								0.15	<0.005	13.00	13.00	5.70	18.00	1.20	0.013	Bailed 10L, bore still not dry, samples taken
	December	20/12/2012		6.97	25.47																						
	Minimum			3.95	25.47	5.83		0.01	20.1	6.23	73	115	0.001	0.0001	0.006	0.0005	0.001	0.0080	0.014	0.02	<0.005	13.00	13.00	0.81	18.00	0.18	<0.002	
	Maximum			6.97	28.49	6.58		0.92	23.0	24.73	271	229	0.002	0.0001	0.047	0.0100	0.005	0.2400	0.170	0.19	<0.005	22.00	90.00	5.70	25.00	1.20	0.017	
	Median					6.09		0.70	21.6	6.77	76	203	0.001	0.0001	0.027	0.0053	0.003	0.1240	0.092	0.09	<0.005	19.16	20.00	3.40	21.50	0.62	0.007	

		Physical	Metals and Metaloids	Nutrients	

	Month	Date	Time	GWL (mBTOC)	GWL (mAHD)	pH	Change in pH	EC	Temp	Diss Oxy (mg/L)	Diss Oxy (%)	Redox (mV)	Arsenic	Cadmium	Copper	Chromium	Nickel	Lead	Zinc	Ammonia as N	Nitrite as N	Nitrate as N	Nitrite + Nitrate as N	TKN	TN	TP	FRP	Comments
Swan Canning WQIP (Short Term Limits)																								2.0	0.2			
Swan Canning WQIP (Long Term Limits)																								0.5	0.05			
Australian and NZ Guidelines for Fresh and Marine Water Quality						7.0-8.5							0.013	0.0002	0.003	0.0031	0.011	0.0034	0.008									
Australian Drinking Water Guidelines (2000)																						11.3						
MB09	September	28/09/2011		0.99	32.97	6.02	0.00	0.17	16.8	4.18	44	232																	Bailed 30L, bore still not dry
	October	19/10/2011		1.55	32.41	6.68	0.66	0.19	17.3	5.89	62	205	0.001	<0.0001	0.004	<0.001	<0.001	0.0030	0.020	0.03	0.00	0.54	0.54	0.76	1.30	0.08	0.001	Bailed 10L, bore still not dry, samples taken
	November	14/11/2011		2.08	31.89																							
	December	22/12/2011		dry																								Dipper muddy? May just be moisture at bottom of bore
	January	16/12/2011		dry																								Bore was dry, water level obtained by moist mud level, no sample taken
	February	22/02/2012		dry																								Bore was dry, water level obtained by moist mud level
	March	27/03/2012		dry																								dry
	April	23/04/2012		dry																								dry
	May	24/05/2012		dry																								dry
	June	27/06/2012		dry																								dry
	July	26/07/2012		dry																								dry
	August	29/08/2012		dry																								dry
	September	26/09/2012		dry																								Bore was dry, water level obtained by moist mud level, no sample taken
	October	24/10/2012		5.61	28.35																							Water level obtained by moist mud level, no sample taken
	November	21/11/2012		5.63	28.33																							Water level obtained by moist mud level, no sample taken
	December	20/12/2012																										
	Minimum			0.99	28.33	6.02		0.17	16.8	4.18	44	205	0.001	0.0001	0.004	0.0005	0.001	0.0030	0.020	0.03	0.00	0.54	0.54	0.76	1.30	0.08	0.001	
	Maximum			5.63	32.97	6.68		0.19	17.3	5.89	62	232	0.001	0.0001	0.004	0.0005	0.001	0.0030	0.020	0.03	0.00	0.54	0.54	0.76	1.30	0.08	0.001	
	Median					6.35		0.18	17.1	5.04	53	219	0.001	0.0001	0.004	0.0005	0.001	0.0030	0.020	0.03	0.00	0.54	0.54	0.76	1.30	0.08	0.001	
Poison Gully Creek (at Milner Road)	September	28/09/2011				7.21	0.64	14.80	9.0	87.30	181																	
	October	19/10/2011		<0.20cm		6.75	-0.46	0.70	16.6	8.70	89	209	0.001	<0.0001	<0.001	<0.001	<0.001	<0.001	0.022	0.02	0.00	0.15	0.15	0.28	0.42	0.01	0.001	Less flow than September
	November	14/11/2011		<0.20cm																								Similar flow to October
	December	22/12/2011																										Creek is dry
	January	16/01/2012																										Creek is dry
	February	22/02/2012																										Creek is dry
	March	27/03/2012																										dry
	April	23/04/2012																										dry
	May	24/05/2012																										dry
	June	27/06/2012		50 cm																								No samples taken
	July	26/07/2012		25 cm																								No samples taken
	August	29/08/2012		50 cm																								No samples taken
	September	26/09/2012		50 cm																								No samples taken
	October	24/10/2012		45 cm																0.00	0.00	0.00	0.00	0.36	0.36	0.01	0.010	Samples taken
	November	21/11/2012		Dry																								Creek
	December	20/12/2012																										
	Minimum			0.00	0.00	6.75		0.70	9.0	8.70	89	209	0.001	0.0001	0.001	0.0005	0.001	0.0005	0.022	0.00	0.00	0.00	0.00	0.28	0.36	0.01	0.001	
	Maximum			0.00	0.00	7.21		14.80	16.6	87.30	181	209	0.001	0.0001	0.001	0.0005	0.001	0.0005	0.022	0.02	0.00	0.15	0.15	0.36	0.42	0.01	0.010	
	Median					6.98		7.75	12.8	48.00	135	209	0.001	0.0001	0.001	0.0005	0.001	0.0005	0.022	0.01	0.00	0.08	0.08	0.32	0.39	0.01	0.006	



Forrestfield Groundwater Level Monitoring - Forrestfield -Airport Link

Desktop Review

Prepared for
Public Transport Authority
by Strategen

May 2016

strategen ENVIRONMENTAL

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Forrestfield Groundwater Level Monitoring - Forrestfield -Airport Link

Desktop Review

Strategen is a trading name of
Strategen Environmental Consultants Pty Ltd
Level 1, 50 Subiaco Square Road Subiaco WA 6008
ACN: 056 190 419

May 2016

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Limitations

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Environmental conclusions

Within the limitations imposed by the scope of services, the preparation of this report has been undertaken and performed in a professional manner, in accordance with generally accepted environmental consulting practices. No other warranty, whether express or implied, is made.

Client: Public Transport Authority

Report Version	Revision No.	Purpose	Strategen author/reviewer	Submitted to Client	
				Form	Date
Preliminary Draft Report	A	Client review	M Dunlop, D Jarvis/ P Bourgault	Electronic	3/3/2016
Draft Report	B	Client submission	M Dunlop, D Jarvis/ P Bourgault	Electronic	4/4/2016
Final Report	C	Client submission	M Dunlop, D Jarvis/ P Bourgault	Electronic	7/4/2016
Final for agency submission	0	Agency submission	M Dunlop, D Jarvis/ P Bourgault	Electronic	5/5/2016
Revised final for agency submission	1	Agency submission	M Dunlop, D Jarvis/ P Bourgault	Electronic	9/5/2016

Filename: PTA16057_01 R001 Rev 2 - 9 May 2016

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Appendix 1 Bore logs
Appendix 2 Test pit locations and logs

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1. Introduction

The Forrestfield-Airport Link Project (the Project) is a new rail line to the eastern suburbs of Perth. The project will be delivered by a lead contractor who will be appointed in early to mid-2016.

Preliminary groundwater modelling of the Forrestfield Development Envelope (FDE) identified that the cone of depression from dewatering activities at the FDE (Dive Structure) potentially extends into surrounding Threatened Ecological Community (TEC) vegetation within Poison Gully and Lot 12 Ibis Place (Figure 1-1, Figure 1-2). The drawdown shown on Figure 1-2 is considered to represent the current worst case scenario (Golder 2015b). The more likely scenario is shown in Figure 1-3.

Advice from Department of Parks and Wildlife (DPaW) suggests that a shallow perched water table is present within Ibis Place and potentially the Poison Gully TEC.

Following the Environmental Protection Authority (EPA) assessing and recommending approval of the Project, the Minster for Environment approved the project on 13 November 2015 with the issue of Ministerial Statement 1022.

Condition 6-2 of Ministerial Statement 1022 requires that a Flora and Vegetation Monitoring Plan be prepared that "shall…

- detail baseline groundwater levels as close as practicable to Poison Gully and Lot 12 Ibis Place
- attach the results of the groundwater level baseline survey, include a report on the extent of perched groundwater levels at Poison Gully Creek and Lot 12 Ibis Place…"

Groundwater level monitoring is recommended to determine the level of connectivity between the regional aquifer, which will be subject to temporary construction dewatering, and any perched groundwater in the immediate surrounds of the dive structure at Forrestfield. The objective of this report is to review the existing geological and hydrogeological information and develop a baseline groundwater level monitoring program. This report forms Phase 1 of the Scope of Services set out in RFQ160300.

1.1 Scope

This report presents a desktop review of the existing bore network and site investigation data for the FDE and surrounds, specifically surrounding the proposed dive structure (site of dewatering works) and the neighbouring TECs at Poison Gully and Ibis Place.

Based on the Scope of Services, this desktop review:

- assesses the existing information, including available geotechnical and hydrogeological data, to determine the existence or potential presence of perched groundwater within the Forrestfield Development Envelope and adjacent areas of TEC
- assess the suitability of the monitor bore network with regard to well distribution and depth
- where perched groundwater is identified, determine whether sufficient information is currently available to determine connectivity between the regional groundwater table and perched groundwater, and assess the potential for the dewatering to draw down the perched groundwater
- where insufficient information is available, develop a monitoring program to identify the existence/absence of shallow perched groundwater in the zone of influence of dewatering.

The Scope of Services requires that the proposed monitoring program includes:

- location of existing wells and access constraints
- location of new wells to ensure adequacy of data
- the specifications required for the installation of the wells
- post installation survey of the wells
- frequency and type of monitoring, e.g. bore loggers or frequency of dipping.



Figure 1-1: Site location and Threatened Ecological Communities

Scale 1:6,500 at A4

0 100 200 m

N

Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 26/02/2016
Author: DWhite
Source: Aerial image: ESRI online , approx. 2010.
All other data: Client 2015.
Path: Q:\Consult\2016\PTA\PTA16057\ArcMap_documents\R001\Rev A\PTA16057_01_R001_RevA_F001_1.mxd

Legend

- Poison Gully Creekline
- Development Envelope
- Stations
- Surface disturbance

Threatened Ecological Community

- Poison Gully TEC
- Ibis Place TEC

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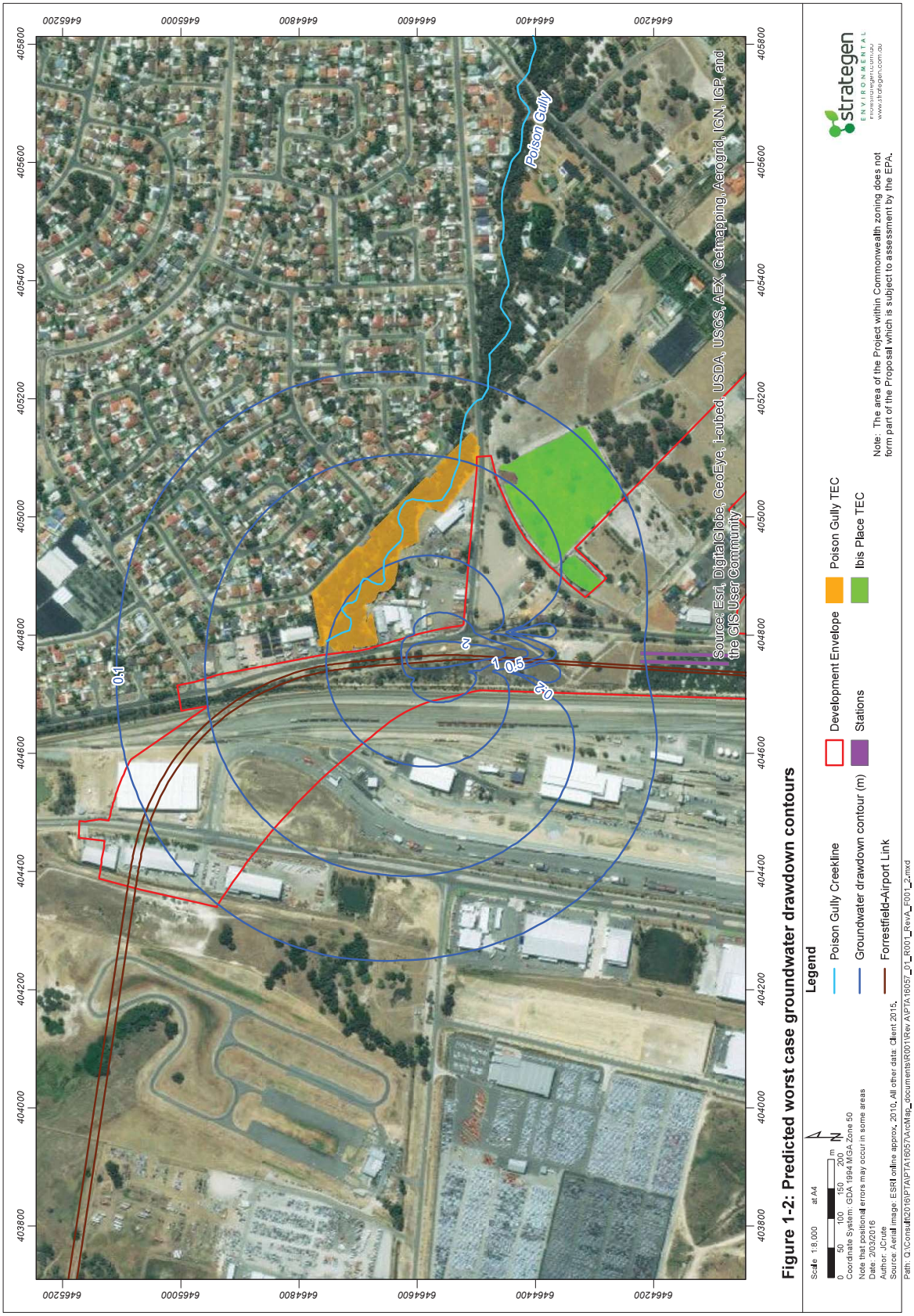
Poison Gully
Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, and the GIS User Community
0.1
1
0.5
0.2
Figure 1-2: Predicted worst case groundwater drawdown contours
Legend
Poison Gully Creekline
Groundwater drawdown contour (m)
Forrestfield-Airport Link
Development Envelope
Stations
Poison Gully TEC
Ibis Place TEC
Note: The area of the Project within Commonwealth zoning does not form part of the Proposal which is subject to assessment by the EPA.
Scale 1:8,000 at A4
0 50 100 150 200 m
Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 2/03/2016
Author: JCrute
Source: Aerial image - ESRI online approx. 2010, All other data - Client 2015.
Path: Q:\Consult\2016\PTA\PTA16057\ArcMap_documents\R001\Rev A\PTA16057_01_R001_RevA_F001_2.mxd
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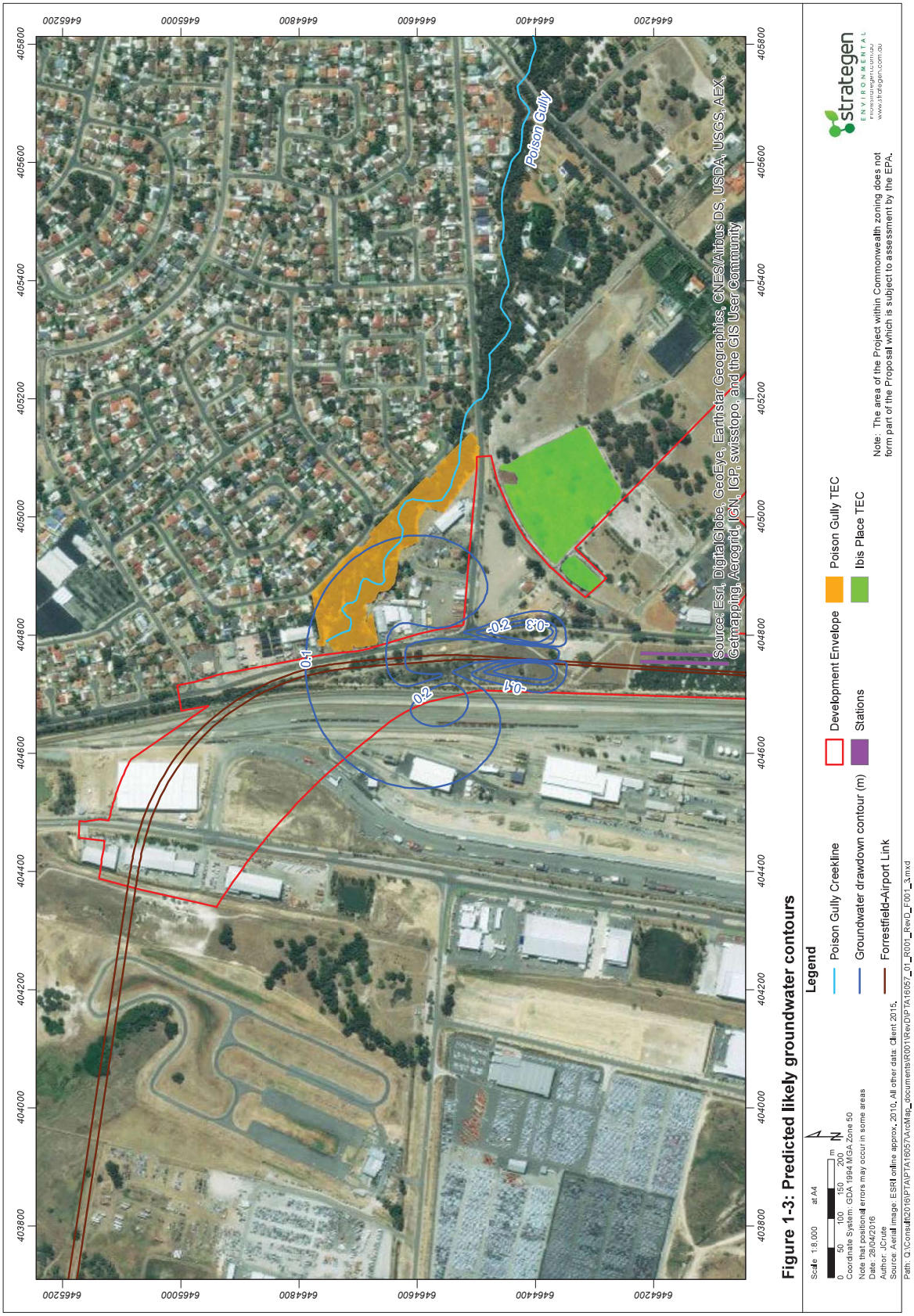
Poison Gully
0.1
0.2
0.3
0.4
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community
Figure 1-3: Predicted likely groundwater contours
Legend
Poison Gully Creekline
Groundwater drawdown contour (m)
Forrestfield-Airport Link
Development Envelope
Stations
Poison Gully TEC
Ibis Place TEC
Note: The area of the Project within Commonwealth zoning does not form part of the Proposal which is subject to assessment by the EPA.
Scale 1:8,000 at A4
0 50 100 150 200 m
Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 28/04/2016
Author: JCrute
Source: Aerial image - ESRI online approx. 2010, All other data - Client 2015.
Path: Q:\Consult\2016\PTA\PTA16057\ArcMap_documents\R001\Rev0\PTA16057_01_R001_Rev0_F001_3.mxd
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melbourne@strategen.com.au
www.strategen.com.au

1.2 Documents reviewed

Documents reviewed for this report include:

- *Contamination and Acid Sulphate Soils Investigation* (Golder 2015a)
- *Groundwater Conditions Report* (Golder 2015b)
- *Geotechnical Baseline Report* (PTA 2015)
- *Stage 2 Geotechnical Investigations - Factual Geotechnical Report* (Golder 2015c)
- *Stage 2 Geotechnical Investigations - Factual Geological and Hydrogeological Model Report* (Golder 2015d)
- *Stage 3 Geotechnical Investigation - Monthly Vibrating Wire Piezometer Monitoring Report no. 06* (Worley Parsons 2016)
- *FAL Project: Forrestfield Station Groundwater Levels Investigation* (Western 2015)
- *Summary of advice from other agencies* 25 February 2015 (OEPA 2015)
- *Forrestfield Airport Link Assessment on Proponent Information - Environmental Review Document* (Strategen 2015a)
- *Forrestfield Airport Link Surface Water Monitoring Report* (Strategen 2016)
- *Forrestfield North District Structure Plan – District Water Management Strategy* (Strategen 2015b).

DPaW has raised questions about the presence of perched groundwater at Ibis Place and in the surrounding areas to the Office of the Environmental Protection Authority (OEPA 2015). The evidence of perched groundwater is based on a report by Ecoscape (2010) titled *Threatened Ecological Community (TEC) Assessment of Lot 12 Ibis Place, High Wycombe.* This report included information obtained from installing five piezometers in the Ibis Place TEC. DPaW has provided PTA with a one page extract of this report but this does not include information regarding piezometer installation and geology.

2. Desktop review

2.1 Conditions required for perched groundwater

Rainfall infiltrating into a soil profile will percolate downwards with gravity at a rate governed by the permeability of the surface soils. In circumstances where the percolating rainfall encounters a soil layer of lower permeability, the downward movement of water is impeded and it is possible the soil above the layer of lower permeability becomes saturated. If a borehole or equivalent is placed into this zone of saturation, free water will flow into the void made by the hole. In many instances this free water is referred to as perched groundwater. Figure 2.1 illustrates this concept.

Perched groundwater can occur at any point in the soil profile where an impeding layer occurs above the permanent superficial aquifer. When present the perched groundwater reflects a localised zone of saturation within the vadose (unsaturated) zone above the permanent watertable. In some circumstances where there is sufficient recharge and sufficient thickness of permeable soils over less permeable soils the presence of perched groundwater can be semi-permanent to permanent. Soils beneath the impeding layer remain unsaturated and are not directly hydraulically connected to the underlying permanent superficial aquifer.

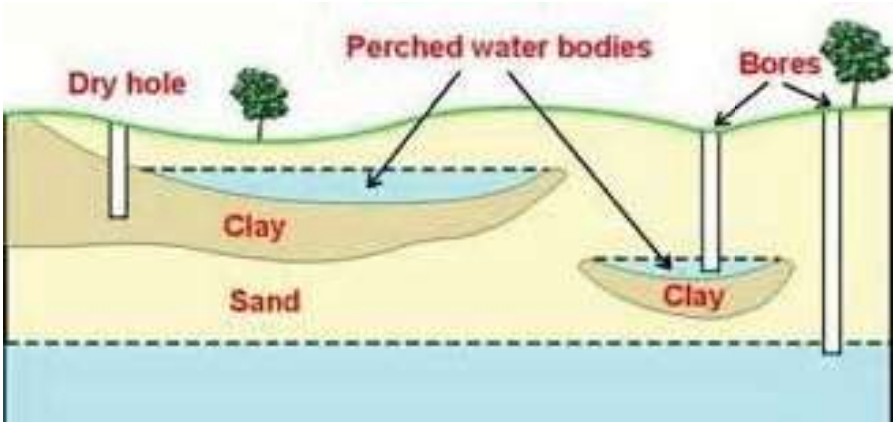

Figure 2-1: Conceptual cross section of perched water body

The presence of perched water can be seasonal. In this case, perched water may be present during the wetter months but dried out during summer as a result of evapotranspiration by vegetation and slow percolation of water through the impeding layer into the regional aquifer.

Understanding the potential for perched water requires an understanding of the soil type, soil texture, underlying geology and hydrology of a site. For the purposes of this report, the term 'impeding layer' has been used to describe less permeable layers in the soil profile above which perched groundwater may occur. Until information regarding the relative hydraulic conductivities of soil strata has been gathered, it is difficult to determine if any soil layers are sufficiently impermeable to impede the vertical percolation of infiltrating rainwater to allow saturation above the impeding layer to occur and perched groundwater to be present.

2.2 Soil investigations

2.2.1 Surface environmental geology mapping

Surface environmental geology mapping for the Perth Metropolitan Region (Gozzard 1986) indicates that the soils at the site are from the S_{10} soil unit, being thin Bassendean Sand over Guildford Formation (Qpb/Qpa) (Figure 2-2). Bassendean Sand is defined as the S_8 soil unit, very light grey at surface, yellow at depth, fine to medium grained, sub-rounded quartz, moderately well sorted of eolian origin.

The upper profile of soils from the S_{10} soil unit are typically layers of Bassendean Sand with interbedded lenses of clay derived from the Guildford Formation. The sands from the Guildford Formation may also be expressed at the top surface of the soil profile. Seasonal perched groundwater is common in this soil type.

2.2.2 Soil investigations

Geological investigations have been undertaken within the area of surface disturbance and have included test pitting and the construction of bores at the locations shown on Figure 2-3. The following represents a summary of the geological conditions based on the bore logs provided to Strategen for the MW 3 series of bores shown in Figure 2-3 and test pits as presented in Appendix 2. Information was available for 22 bores installed between 20 October 2014 and 11 May 2015. Bore logs are presented in Appendix 1.

Surficial fill

At MW3-001, MW3-009, MW3-034 and MW3-038 a layer of sand to sandy gravel fill has been placed on the site. The Golder logs identify that the fill depth ranges from 0.8 m at MW3-034 to 4 m at MW3-009. On review of the logs following observations of soils in the TEC, Strategen considers that the brown orange and yellow orange sands to sandy gravel in the Golder logs are natural soils and represent the surficial layer of the Guildford Formation. On this basis, Strategen considers the actual fill depth at these bores to be more likely 0.2 m (MW 3-001) to 0.9 m (MW 3-009) with the possibility that some of the grey sands above these layers being misidentified Bassendean Sand.

Bassendean Sand

A layer of sand is present at the top of the profile at most locations. This sand was confirmed as being from the S_{10} Formation, being “light grey, yellow, dark brown, fine to medium grained sand, loose to dense” (Golder 2015d). Some of this sand layer contains gravel or had orange to red colouring, indicating that these gravels may actually be derived from the reworked Guildford Formation that occurs below the sand. For the purposes of this report, this sand layer with intermittent gravels is referred to as Bassendean Sand. Where present, the material ranges from 0.3 to 3.5 m in thickness, with base levels between 24.18 mAHD (MW 3-001) and 29.67 mAHD (MW 1-01 A).

This top layer of sand was not present at MW 3-013, MW3 -038, MW 3-044, MW 3-051, MW 3-052 or MW 3-055 (Table 2-1). Apart from MW 3-013, these are all located in the north of the Site near Poison Gully (Figure 2-3). MW 3-013 is located near Milner Road, to the south of Ibis Place.

Guildford Formation

The gravelly and clayey soils of the Site are primarily associated with the Guildford Formation. Golder 2015 (d) described the Guildford Formation in the Forrestfield portion of the Project as consisting of “pale grey and brown, fine to coarse grained, rounded to sub-angular, medium dense to very dense sand, silty sand and clayey sand with trace quartz gravel generally less than 5 mm of size. The Guildford Formation also contains a sequence of mottled orange, low plasticity, stiff to hard clays in the Forrestfield area that are generally present within 10 m of ground surface and being up to 6 m thick”.

Table 2-1: Summary of geology and potential for perched groundwater

Name	Ground level (mAHD)	Bottom of sand and fill (mbgl)	Bottom of sand (mAHD)	Top of potentially impeding layer (mbgl)	Top of potentially impeding layer (mAHD)	Type of impeding material	Moisture present over dry soil?	Top of screen (mbgl)
MW 3-001	27.78	3.60	24.18	Not present	Not present	Not present, clayey sand only	No	3.00
MW 3-004	30.00	2.10	27.90	Not present	Not present	Not present, silty clayey sand only	No	5.00
MW 3-009	30.20	5.50	24.70	5.50	24.70	sandy clay	No	3.90
MW 3-012	29.24	0.30	28.94	0.30	28.94	silt then silty clay at 4.8 mbgl	No	7.50
MW 3-013	34.68	Not present	Not present	2.00	32.68	clayey silt	No	16.30
MW 3-019	30.61	1.00	29.61	Not present	Not present	Not present, clayey sand only	No	3.50
MW 3-026	28.39	0.60	27.79	3.00	25.39	sandy gravelly clay, then sandy clay	No	5.00
MW 3-034	31.23	2.20	29.03	2.20	29.03	clayey silt	No	4.00
MW 3-038	29.33	1.50	27.83	1.50	27.83	sandy clay	No	4.50
MW 3-044 D	29.54	Not present	Not present	1.40	28.14	clayey silt	No	11.00
MW 3-044 S	29.50	Not present	Not present	1.40	28.14	clayey silt	No	3.30
MW 3-047	30.36	1.50	28.86	1.50	28.86	clayey silt	No	9.00
MW 3-051 D	29.41	Not present	Not present	2.40	27.01	clayey silt	No	15.50
MW 3-051 S	29.39	Not present	Not present	2.40	28.19	clayey silt	No	4.00
MW 3-052	28.35	Not present	Not present	2.00	26.35	clayey silt	No	7.00
MW 3-055	28.81	Not present	Not present	1.50	27.31	silty clay	No	4.00
MW 1-01 A	30.47	3.50	26.97	5.30	25.17	Clay	Moisture not logged	8.00
MW 1-01B	29.33	1.00	28.33	Not present	Not present	Not present	Moisture not logged	8.50
MW 1-01 C	30.47	2.15	28.32	Not present above 0 mAHD (i.e. above sea level)	Not present above 0 mAHD	Not present above 0 mAHD	Moisture not logged	7.00
MW 1-01 D	30.19	5.00	25.19	Not present above 0 mAHD (i.e. above sea level)	Not present above 0 mAHD	Not present above 0 mAHD	Moisture not logged	7.00
MW 1-05	28.42	1.00	27.42	Not present	Not present	Not present	Moisture not logged	7.00
PW 1-01	30.32	2.00	28.32	Not present	Not present	Not present	Moisture not logged	7.00

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Ordinary Council Meeting 12 October 2021 Attachments
Attachment 10.5.2.3
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The geology of the Guildford Formation as present in the bore logs is highly variable and includes units from sandy gravel to clays. From a perched groundwater perspective, the key factors are the presence of fine grained layers which may impede downward percolation of water. Where the majority component of a soil unit is described as silt or clay and the material above is predominantly sand, the difference in permeability may be sufficient for seasonal perched groundwater to occur. A silty or clayey sand was not considered to offer adequate difference in permeability to significantly impede the downwards movement of water and thus cause perched groundwater on top of the layer.

Test pits

The test pits along the south-western edge of the Ibis Place TEC indicate:

- TP 1-46: sand then sandy clayey gravel 'weakly to very weakly iron cemented mass' at 0.4 m then refusal at 1.3 m (no groundwater encountered, excavated on 2/12/14)
- TP1-49: sand over moist gravelly clayey sand 'weakly to well cemented' at 0.7 m then refusal at 2.4 m (no groundwater encountered, excavated on 2/12/14)
- TP1-20: sand then moist interbedded sand and clayey sand 'weakly to well cemented lateritic gravel at 1.3m, no refusal (no groundwater encountered, excavated on 12/12/14) (Appendix 2).

The material causing refusal at TP1-46 and TP1-49 could possibly be a cemented perching layer, but there no evidence of actual perching (i.e. no saturation was encountered).

TP1-28 and TP1-29 are located in the vicinity of Poison Gully

- TP1-28: gravelly sand, then sand over clayey sand and then sandy clay at 1.5 mbgl. No evidence of groundwater to 2.1 mbgl during excavation on 15/10/14. Base of hole at approximately 27.15 mAHD
- TP1-29: sandy clay over sandy gravel then clayey sand to 2.6 m. No evidence of groundwater to this depth during excavation on 27/10/14. Base of hole at approximately 26.62 mAHD (Appendix 2).

The bases of the holes appear to be above the invert of the creek at this location. As such, this does not provide any indication of whether the creek is intersecting perched groundwater, which could be present below the base of the holes.

Data gaps for the assessment of perched groundwater

Based on the observed soil profiles, layers with the potential to impede flow were considered to occur in 14 of the 22 bores. Where impeding layers were present, the top of the impeding layer varied from 0.3 m (MW 3-012) to 5.5 mbgl (MW 3-009), with the majority being less than 2.5 m below the surface. The elevation (in mAHD) of these impeding layers varied from 24.7mAHD (MW 3-009) to 32.68 mAHD (MW 3-013). In all cases, the bores were only screened below the impeding layer (Table 2-1).

Logs from four bores installed by Strategen in September 2011 for the Shire of Kalamunda (SoK) in the broader Forrestfield area (MB01, MB07, MB08 and MB09) did not record any potential impeding layers, with sand or gravelly sand/sandy gravel at the surface and then clayey sand at depths between 2.25 and 3.5 mbgl to the end of the hole (Strategen 2015b). Locations of these bores are shown on Figure 2-3.

The geology quoted by DPaW for Ibis Place is consistent with very shallow Bassendean Sand over the Guildford Formation, however the bore logs for these bores have not been provided to Strategen.

The bores and test pits are focussed on investigating the upper soil profile of the regional superficial aquifer along the proposed alignment. One bore (MW 3-034) has been located adjacent to Ibis Place and one on Milner Road (MW 3-013) (Figure 2-3). Only MW 3-055 is located to the north of Poison Gully. Given the complex geology of the subject area, the current geological information is considered limited within and surrounding the TECs. Information in bore logs is variable and determining the presence or otherwise of impeding layers is difficult. Additional geological information would be beneficial.

2.3 Hydrogeology

Ecoscape (2010) report

Ecoscape (2010) reported a depth to groundwater in August 2008 of less than 0.4 m in five piezometers installed within the Ibis Place TEC. The soil samples taken from the installation of the bores showed a "shallow layer of wet sand approximately 10 cm thick over a wet clayey sand, to wet sandy clay, with dry mottled clay at a depth of approximately 1 m" (Ecoscape 2010, quoted in DPaW undated). These readings were taken in winter and it is possible that the perched water was ephemeral and resulted from a period of rain that occurred leading up to the monitoring event. The exact date of the monitoring event has not been provided to allow comparison to rainfall records.

These water levels are in contrast to the groundwater levels at the adjacent PTA bore MW 3-034, which was dry during monitoring by Western Environmental in July and October 2015 (Western 2015). MW 3-034 was installed to a depth of 6.3 metres below ground level (mbgl) and was screened between 3.3 and 6.3 mbgl. Water may have been present as a perched layer above this screened depth.

Evidence from bore installation and test pits

If perched groundwater was present during bore installation or test pitting, a layer of wet soil would have been found over a dry impeding layer. Such perched groundwater conditions were not encountered during drilling or test pitting, indicating that permanent perched groundwater is unlikely to be present. Bore logs were available to Strategen for the bores listed in Table 2-1 and four bores installed for SoK by Strategen (Figure 2-3). Test pit logs were available for locations in the vicinity of both the Ibis Place TEC and Poison Gully (Appendix 2).

These bore logs show that perched groundwater was not present during bore or test pit installation. Groundwater levels on the Swan Coastal Plain generally peaks in October, and as such perched groundwater would be more likely to be present at this time of year. In terms of timing of installation:

1. Installation of the Golder bores for which soil moisture was logged occurred during January and February 2015, when ephemeral perching would be unlikely to be present.
2. Strategen bores were installed in September 2011, close to peak groundwater when ephemeral perching would be more likely to be expressed.
3. Of the Golder test pits, two were installed in October 2014 (around peak groundwater) and three in December 2014. Perched groundwater is likely to be found if present during October but is less likely to be present in December.

The lack of perching in the pits and bores constructed in September and October indicates that perching is not present at these locations.

Screened depth

Of the bores installed to date, none have been screened in the top 3 m of the profile. Top of screen levels vary from 3.0 mbgl at MW 3-001 to 16.3 mbgl at MW 3-013 with bottoms of screens located from 5.9 to 32.5 mbgl. All of the bores that had intersected impeding layers were screened below the top of the impeding material.

Groundwater levels encountered during the summer installation period were below this level and thus the shallowest bores were screened across the groundwater table, as is standard practice. A minimum of 0.5 m of bentonite was placed above the screens to ensure that water from aquifers above the aquifer being monitored did not affect water levels in the bore. Assuming this process was effective and the top of all screens are located below the top of potentially impeding layers, groundwater levels in these bores will not accurately reflect water levels occurring above the impeding layer.

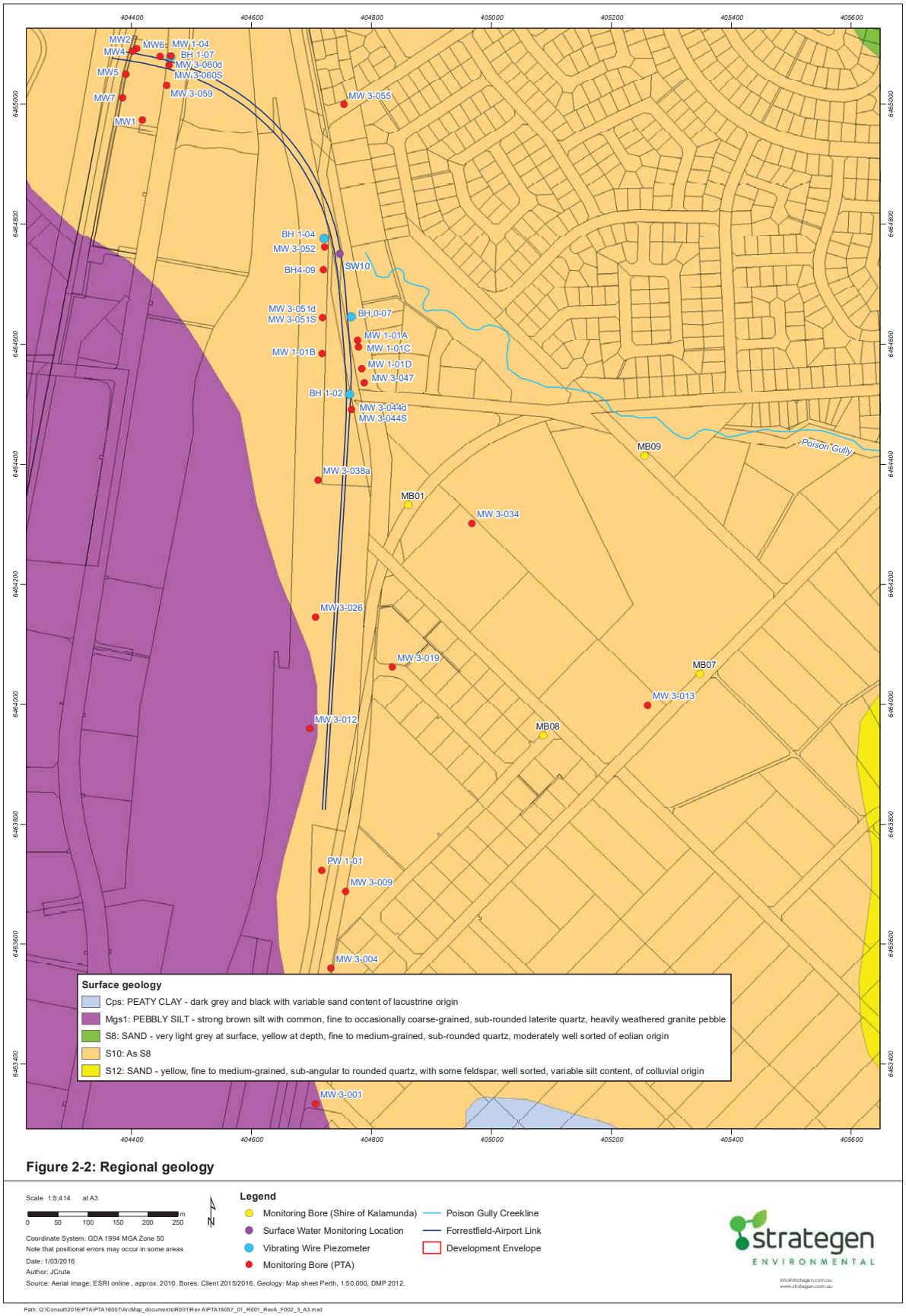

Figure 2-2: Regional geology

Scale 1:5,414 at A3

0 50 100 150 200 250 m

Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 1/03/2016
Author: JCrute

Source: Aerial image: ESRI online , approx. 2010. Bores: Client 2015/2016. Geology: Map sheet Perth, 1:50,000, DMP 2012.

Legend

- Monitoring Bore (Shire of Kalamunda)
- Surface Water Monitoring Location
- Vibrating Wire Piezometer
- Monitoring Bore (PTA)
- Poison Gully Creekline
- Forrestfield-Airport Link
- Development Envelope

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Figure 2-3: Groundwater and surface water monitoring locations

Scale 1:5,414 at A3

0 50 100 150 200 250 m

Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 31/03/2016
Author: JCrute
Source: Aerial image: ESRI online , approx. 2010. All other data: Client 2015/2016.

Legend

- Monitoring Bore (Shire of Kalamunda)
- Surface Water Monitoring Location
- Vibrating Wire Piezometer
- Monitoring Bore (PTA)
- Open Pit Observations
- Poison Gully Creekline
- Forrestfield-Airport Link
- Development Envelope
- Poison Gully TEC
- Ibis Place TEC

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Vibrating wire piezometers

Vibrating wire piezometers (VWPs) have been installed at three locations to monitor groundwater pressure/levels at depth (Figure 2-3). The VWPs record water levels on a minimum of daily frequency. VWPs are located at:

- BH0-07 contains at 12.5 mbgl, monitored since April 2014
- BH1-02 at 12.5 mbgl and 24.5 mbgl, monitored since October 2014
- BH1-04 at 9.8 mbgl and 13.7 mbgl, monitored since October 2014.

Strategen understands that the VWP monitoring program will continue until May/June 2016. The VWPs have been installed too deep to represent any perched groundwater but will provide useful historical water level data.

Groundwater levels

The PTA bores were monitored for groundwater levels on:

- 10 March 2015 by Golder
- 23 July 2015 by Western Environmental
- 20 October 2015 by Western Environmental
- 26 February 2016 by Strategen.

The SoK bores were monitored on 12 October 2015 by Gecko Environmental. The only SoK bore located by Gecko (MB09) was dry. Strategen attempted to locate the SoK bores on 26 February 2016, but only MB09 could be located. The SoK bores were installed in the road reserve, so it is likely that these have been either destroyed or covered over with material (e.g. concrete) where flush covers were used.

The only bore showing water levels clearly above an impeding layer is MW 3-009. The water level on 20 October 2015 was 24.86 mAHD, with an impeding layer logged as being present at 24.70 mAHD. Groundwater levels at this bore varied from 24.54 mAHD to 24.86 mAHD during the monitoring events, and thus has been both above and below the top of the impeding layer. This is consequently not considered to be evidence of consistent perching.

Data gaps for assessment of perched groundwater

The bores installed by Golder have been designed to monitor the deeper parts of the superficial aquifer that will be directly affected by dewatering, close to the proposed alignment. Additional monitoring is recommended in the vicinity of the TECs to better understand the hydrology of these areas.

2.4 Surface water

Poison Gully (Plate 1 to Plate 3) is an ephemeral creek which collects water from elevated areas to the east of the Site including parts of Kalamunda and flows in a westerly direction via the Perth Airport Northern Main Drain and Limestone Creek into the Swan River. Between Maida Vale Road and the location to the west of Dundas Road, where Poison Gully becomes a piped drain, the creek is incised up to approximately 2 m below the normal ground surface. No surface water was observed in this section of Poison Gully during a site visit on 26 February 2016. During 2015, the creek was dry at SW10 in the vicinity of the Site during two of four surface water monitoring events (March and December) (Strategen 2016). The location of SW10 is shown on Figure 2-3.

Groundwater monitoring undertaken by Strategen for Shire of Kalamunda in the Forrestfield area indicated a southerly regional groundwater flow direction in the area to the south of Poison Gully in October 2011 (Strategen 2014). The study area and bores for this project were predominantly located to the east of FDE, with the easternmost bore located in Ibis Place. The implication of this flow direction is that Poison Gully may be a losing stream (i.e. one that recharges groundwater) during the period over which it flows. In saying this, the study was undertaken at a regional level and the majority of the project area was located to the east of the area of interest for this study, within soils mapped as deep Yoganup Sands rather than the Guildford Formation.



Plate 1: Poison Gully, February 2016



Plate 2: Poison Gully, February 2016



Plate 3: Poison Gully, February 2016 showing erosion

Surface water levels recorded by Strategen (2016) at SW10 were approximately 26.5 mAHD on 3 June and 3 September 2015. The maximum groundwater level recorded by Western Environmental during their July and October 2015 groundwater monitoring events was 25.31 mAHD in MW 1-01A on 23 June. This implies that the water level in Poison Gully when flowing is generally above the regional groundwater level. This implies that the general direction of water flow when Poison Gully is flowing would be from Poison Gully to the regional groundwater (i.e. this section of Poison Gully is a losing stream).

Poison Gully has previously been surveyed by Water Corporation (2010) as part of a drainage study of the area. Strategen has been provided with this information for another job under the condition that it is used for that purpose. Permission to use this information should be obtained by PTA to assist in determining the interaction between Poison Gully and any perched groundwater.

2.5 Observation of open pits

On the 24 March, Strategen inspected several open pits ranging in depth from 1-2.5m to assess the textural differences within the soil profile. Two pits to approximately one metre depth where inspected along Raven Road on the north-eastern side of the Ibis Road TEC (Plate 4). Another large excavation along the alignment of the gas pipeline running parallel with Dundas Road on the western side was inspected (Plate 5). This excavation was close to the culvert on Dundas Road, through which Poison Gully flows. The approximate locations of the observation pits are shown on Figure 2-3.

The soil profile along Raven Road was found to be grey fine-medium sand to about 0.3m depth over yellow to orange clayey sand (~5% clay) with weathered coffee rock. This soil, which also contains a large percentage of gravel, is unlikely to retard downward percolation of rainwater sufficient to result in perching during normal flows. High intensity rainfall may be sufficiently retarded above the clayey sand for short periods to cause isolated pockets of water saturation, but this would be of very short duration.

The soil profile along the gas pipeline alignment had three distinct horizons. The surface 0.3-0.4m is grey fine-medium sand, typical of Bassendean sand. The underlying 0.4-1.0m of soil profile textured as a clayey sand and was similar to the soil encountered in Raven Road. At a depth of ~0.8m there is a distinct textural change from the clayey sand to sandy clay. The sandy clay is quite plastic but the sand grains are obvious. Based on the bolus formed by wetting some of the sandy clay material and manipulating it in the hand and forming a ribbon by squeezing between the thumb and forefinger (The National Committee on Soil and Terrain, 2009), it is estimated there is about 25% clay. During normal rainfall events, this textural difference at about 1m is possibly sufficient to retard downward percolation of rainfall for long enough to result in minor amounts of water saturation within the soil profile (perched groundwater). However it was noticed there were regular macropores (cracks and old root channels greater than 2 mm in diameter) that would conduct any perched groundwater deeper into the soil profile (Plate 6). Macropores can rapidly convey water through the unsaturated zone of the soil profile, draining areas of temporary saturation (Beven and Germann 1982; Bourgault Du Coudray 1996).

At all inspected locations, the soil profile was dry and showed no evidence of perching. It is highly unlikely there is any connectivity between potential perched groundwater layers and the regional superficial groundwater table.

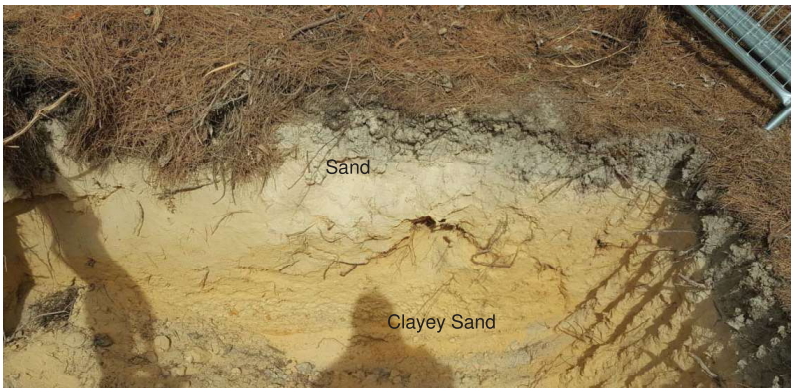

Plate 4: Soil profile at OP2

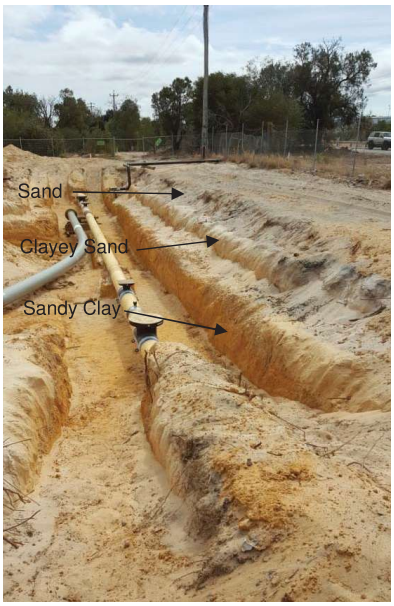

Plate 5: Soil observations at OP1

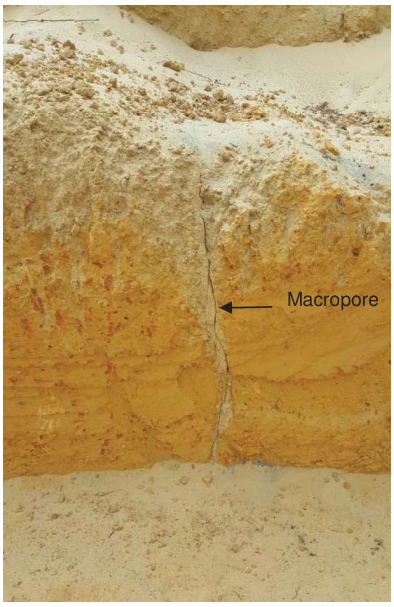

Plate 6: Macropore through sandy clay layer at OP1 and natural gravel within the profile

2.6 Potential for perched groundwater

Apart from the Ecoscape (2010) report, there is no conclusive evidence that perched water occurs near the proposed Forrestfield area. It is possible that the perched groundwater seen by Ecoscape is temporary perched groundwater following the retardation of percolating rainfall by a impeding soil layer of lower hydraulic conductivity, following a short period of relatively high intensity rainfall. Based on the inspection of the soil profile near Raven Road and adjacent to Dundas Road, there is sufficient textural difference between overlying sands and underlying sandy clay to give rise to the possibility of localised temporary perched groundwater following rainfall. The refusal in test pits adjacent to Ibis Place TEC is also indicative of a cemented layer that may cause perching.

Soil logs show potential impeding layers that may cause perched groundwater after rainfall are present at 13 out of 16 Golder bores and at TP 1-46 and TP 1-49 on the southwestern boundary of the Ibis Place TEC. However, perched water was not encountered during bore installation in January/February 2015 or test pitting. This suggests that if there is any perched groundwater, it is not permanent. A potentially impeding layer was noted at a depth of 2.2 mbgl at the bore closest to Ibis Place and in the form of refusal during test pitting at 1.3 m and 2.4 mbgl at TP1-46 and TP 1-49 respectively. There is no evidence of hydraulic connection between the perching layers and the underlying superficial aquifer.

Where potential impeding layers were found, they were of variable texture, depth and thickness. This implies that temporary perched groundwater may exist in some areas but not others.

If permanent perched groundwater occurred and was above the level of Poison Gully, springs or seeps where lateral flow of the perched water intercepted the creek would be expected. If significant volumes of perched water were present and seeping into Poison Gully permanent to semi-permanent pools would be expected along the creek line. There is no evidence of subsurface seepage into Poison Gully from perched groundwater, even in areas where the creek is deeply incised.

2.7 Conclusions

The groundwater level monitoring to date has been focussed on the regional superficial aquifer. While more information regarding the hydrology of the TECs would be beneficial, the evidence available indicates that any perched groundwater that may occur is likely to be ephemeral and limited to isolated areas. Hence temporary lowering of the regional groundwater table is unlikely to impact the TECs adjacent to the FDE.

3. Proposed future works program

3.1 Groundwater monitoring

3.1.1 Bore specifications

The proposed additional monitoring bore locations are shown in Figure 3-1. Bore depth will depend on the geology and presence of groundwater at the drilling location.

Monitoring bores will be installed as a pair or shallow bore only on the following basis:

1. As a pair of monitoring bores, typically 2 to 5 metres apart, where a deep bore is not currently present and a better understanding of the interaction between the shallower and regional (deeper) superficial aquifer is required. The shallow bore will be screened so that it is above the first impeding layer that is encountered in the profile (typically from 0 to 3 mbgl) to intersect any seasonal perched groundwater that may occur in that location. The deepest of the pair will be screened below any confining layer (typically at depths of more than 4 mbgl, but not more than 8 mbgl) so as to intersect the regional groundwater table. If no impeding layer is identified, the shallow bore will still be installed to a depth of 2mbgl.
2. As a shallow bore on top of the impeding layer at a location where a deeper bore already exists or as a supplementary shallow bore located adjacent to the TEC. Where this is not located adjacent to a deeper bore, the hole will extend a minimum of 300 mm into the potential impeding layer to confirm that perched groundwater is likely to be present and then the hole refilled to the top of the impeding layer. If no impeding layer is identified, then the bore will be installed to depth of 2mbgl.

Bores will be constructed in accordance with the requirements outlined in the Land and Water Biodiversity Committee (2003) guidance document *Minimum Construction Requirements for Water Bores in Australia.* Minimum bore internal diameter shall be 50 mm based on this guidance and to allow installation of 41 mm diameter data loggers in any bore.

The screening intervals for all bores will be carefully selected based on the soil profiles encountered at each location. A screened interval of 2-3 m will be installed either completely above (shallow bores) or completely below (deep bores) any impeding layer that is identified.

The soil lithology will be logged for all new bores proposed for installation at the locations shown in Figure 3-1 and Table 3-1 by a suitably qualified environmental scientist. Where bores are paired as a deep and shallow set, logging is required for both bores. The deep bore shall be installed first and this geological information will be used to determine the depth of the shallow well. This will include logging of soil type, texture and moisture status at a minimum of every 20 cm down the hole. Bores will be installed using a push core drilling rig to ensure a clean core is obtained for accurate geological logging.

The proposed locations for the additional bores are provided in Table 3-1. A deep bore screened below the perching layer to access the regional groundwater table will be paired with MW3-009 as a potential control bore set (Figure 3-1). This location has been selected as a control bore as:

- a perched groundwater layer is known to occur in this area
- the bores are located outside the drawdown area shown on Figure 1-2.

Table 3-1: Additional monitoring bore locations

Label	Type	E (MGA94z50)	N (MGA94z50)
MW 8-001	A, D, K	404846.2	6464670
MW 8-002	B	404918.1	6464733.1
MW 8-003	A	404929.3	6464604.2
MW 8-004	B, K	405114.4	6464477.6
MW 8-005	A	405021.3	6464444.5
MW 8-006	A, D, K	404874.8	6464295.5
MW 8-007	A, D	405144.3	6464305.7
MW 3-009 (D)	R	404846.2	6464670

A: New Bore Pair
B: Shallow Bore Only
D: Data Logger Pair
K: Permeability Test
R: Deeper bore to Access Regional Groundwater Table

All new monitoring bores will be surveyed for Easting, Northing and Elevation (surface and top of casing in mAHD) to allow accurate assessment of the groundwater at the site.

3.1.2 Proposed monitoring program

Groundwater water levels will be measured on a monthly basis for all bores shown on Figure 3-1. It is anticipated that monitoring will commence in May 2016 and be ongoing. The amount of data collected should be maximised.

In addition to the monthly measurements, opportunistic monitoring immediately following daily rainfall greater than 10 mm, or following high-intensity rainfall, such as following a thunderstorm, even though the duration and total amount of rainfall is relatively modest. It may be necessary to visit the shallow bores several times following a rain event to ensure any temporary perched groundwater is captured.

The period from March to July is associated with seasonal low groundwater levels (typically March/April) followed by the subsequent rise of the watertable as the winter months approach. Heavy falls are likely to occur in the proposed monitor period, and the monthly and opportunistic monitoring of the shallow bores during these periods should provide sufficient data to help assess any potential impact to the TECs from drawdown in the regional water table.

Data loggers

Data loggers will be installed in three paired sets (i.e. six loggers in total) at the locations shown on Figure 3-1 (two associated with the Ibis Place TEC and one with the Poison Gully TEC). If possible, a data logger should be placed in one of the Ecoscape piezometers. These data loggers will log water levels on an hourly basis to observe the behaviour of water levels following rain. At this logging frequency, a battery life of at least four months is anticipated.

Ecoscape bores

Access to the Ibis Place piezometers installed by Ecoscape should be obtained if possible for monitoring purposes. These piezometers would need to be assessed prior to use to determine that they are still present and in a suitable condition for monitoring. If these bores are accessible and suitable, they should be included in the monitoring program and a data logger placed in one of the bores.

Vibrating wire piezometers

Strategen understands that the VWPs are intended to be monitored until May/June 2016. If possible, this program should be continued as long as practical up until the period of maximum groundwater levels (typically September). Data from the data loggers and dipping program should be compared to data from the VWPs to assist in investigating interactions between any perched and deeper superficial groundwater.

3.2 Soil investigations

3.2.1 Permeability testing

Permeability testing will be undertaken adjacent in hand augered holes adjacent to bores where impeding layers are identified using a falling head (Talsma-Hallam) permeameter based on methodology described in Appendix G of Australian Standard AS/NZS 1547:2012. The permeability tests will be undertaken in hand augered holes:

- within the permeable layer
- within the potentially impeding layer.

3.2.2 Poison Gully creekline

Obtaining soil logs from the Poison Gully creekline could be useful to determine the interaction between surface water in Poison Gully and groundwater. The creek is highly incised (approximately 2 m) and provides a useful geological cross section. During high flow periods, the sides of the creek are covered in clayey sediment, which sticks to the soil and can make such observations difficult. This section of Poison Gully is a significant site to Aboriginal people, and as such the sides of the creek should not be altered or damaged. Visual inspection should be undertaken and logging undertaken where it is considered that this will provide useful data (e.g. freshly eroded areas). This should ideally be done at a minimum of six locations between where the creek becomes a drain to the west of Dundas Road and Maida Vale Road, with the locations recorded by GPS and surveyed.

For safety reasons, any work in Poison Gully creek line should be done when the creek is dry.

3.3 Surface water

Monitoring of surface water levels in Poison Gully should continue at SW10 on the same frequency as the groundwater monitoring (Section 3.1.2). Data loggers are not proposed for Poison Gully.

As discussed in Section 2.4, survey information for Poison Gully should be obtained from Water Corporation to assist in determining the interaction between Poison Gully and any perched groundwater.

3.4 Potential ethnographic constraints

Investigations should be undertaken at the locations shown on Figure 3-1. Strategen notes that cultural reasons may prevent ground disturbing activities within vegetation areas adjacent to Poison Gully as well as the creek line itself. The cultural sensitivity of these areas should be confirmed with PTA prior to work commencing.

Should this be the case, locations shall be moved to a nearby location without such sensitivities. The movement of locations may impact on the relevance of the data obtained to the Poison Gully TEC. Where such movement of locations is required, this shall be identified within subsequent reporting and the reason for such movements identified.



Figure 3-1: Proposed monitoring locations

Scale 1:5,500 at A3

0 50 100 150 200 250 m

Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 6/05/2016
Author: JCrute
Source: Aerial image: ESRI online , approx. 2010.
All other data: Client 2015/2016.

N

Legend

- New Monitoring Location
- Monitoring Bore (Shire of Kalamunda)
- Existing Surface Water Monitoring Location
- Vibrating Wire Piezometer monitored by Worley Parsons
- Existing Monitoring Bore (PTA) to be monitored
- Poison Gully Creekline
- Forrestfield-Airport Link
- Development Envelope
- Poison Gully TEC
- Ibis Place TEC

A: New Bore Pair
B: Shallow Bore Only
D: Data Logger Pair
K: Hydraulic Conductivity Testing
R: Deeper Bore to Access Regional Groundwater Table

strategen ENVIRONMENTAL
info@strategen.com.au
www.strategen.com.au

Path: Q:\Consult\2016\PTA\PTA16057\ArcMap_documents\R001\RevD\PTA16057_01_R001_RevD_F003_1_A3.mxd

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4. Conclusion

The groundwater level monitoring to date has been focussed on the regional superficial aquifer. While more information regarding the hydrology of the TECs would be beneficial, the evidence available indicates that any perched groundwater that may occur is likely to be ephemeral and limited to isolated areas. Hence temporary lowering of the regional groundwater table is unlikely to impact the TECs adjacent to the FDE.

The data gathered from the additional groundwater level monitoring program outlined in this report will form the basis of further assessment of the seasonal behaviour of the Superficial Aquifer and localised areas of perched groundwater. This assessment will assist in the investigation of potential impacts to TECs in the area due to temporary drawdown of the regional watertable due to dewatering.

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APPENDIX 5 WATER CORPORATION CATCHMENTS (2010)

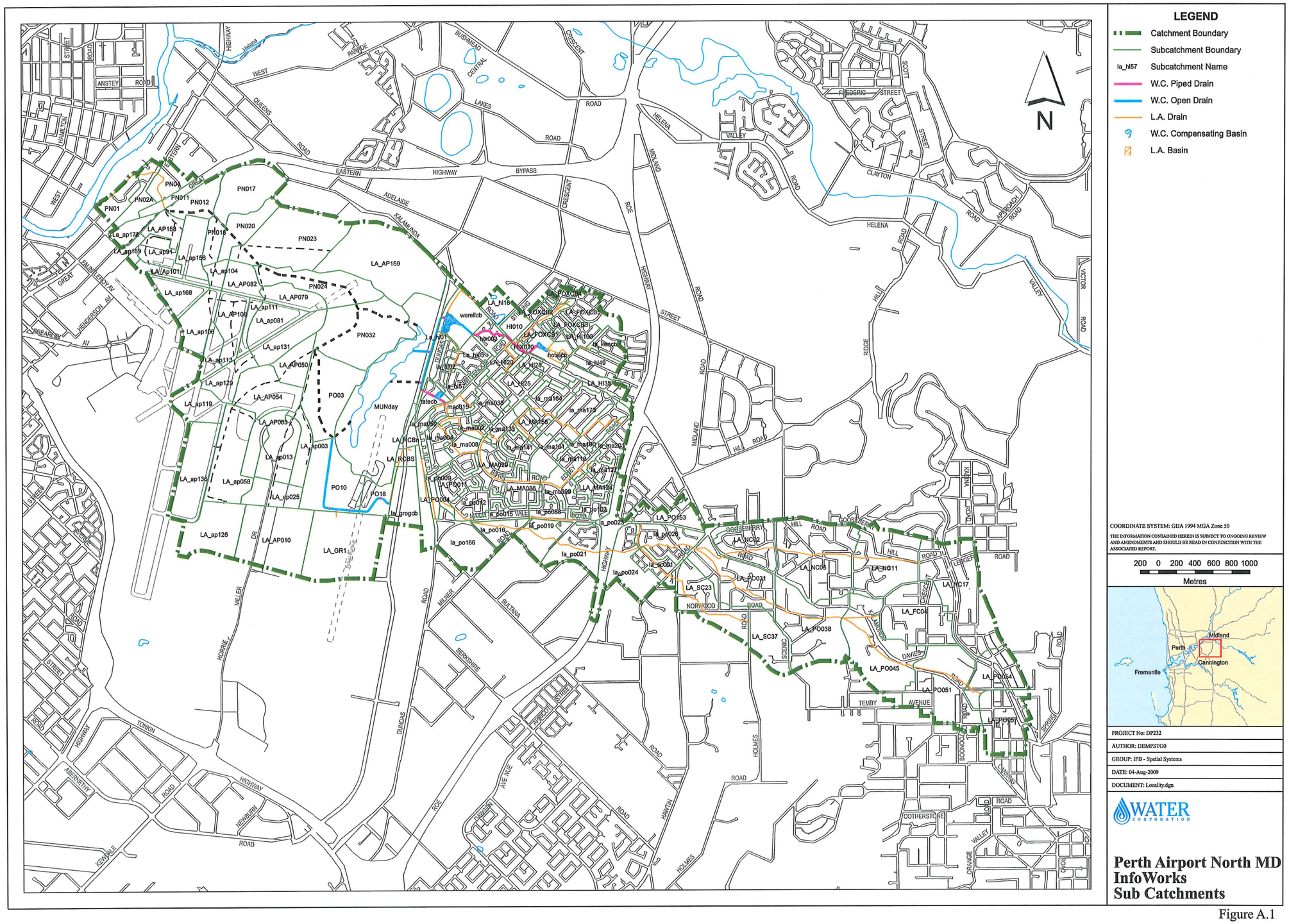

LEGEND
Catchment Boundary
Subcatchment Boundary
la_hl57 Subcatchment Name
W.C. Piped Drain
W.C. Open Drain
L.A. Drain
W.C. Compensating Basin
L.A. Basin
N
COORDINATE SYSTEM: GDA 1994 MGA Zone 50
THE INFORMATION CONTAINED HEREIN IS SUBJECT TO ONGOING REVIEW AND AMENDMENTS AND SHOULD BE READ IN CONJUNCTION WITH THE ASSOCIATED REPORT.
200 0 200 400 600 800 1000
Metres
Perth
Midland
Cannington
Fremantle
PROJECT No: DP232
AUTHOR: DEMPSTG0
GROUP: IPB - Spatial Systems
DATE: 04-Aug-2009
DOCUMENT: Locality.dgn
WATER CORPORATION
Perth Airport North MD
InfoWorks
Sub Catchments

Figure A.1

APPENDIX 6 UNDO RESULTS PRE-DEVELOPMENT

Government of Western Australia
Department of Water

Project: HW Pre-Dev

Date: 12-Dec-20

Version: Version 1.1.0.16333

Subregion name: Subregion 1

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	0	0.00	0.00	0.00
Industrial, commercial & schools	41	22.92	327.74	66.47
Rural living	31	17.33	852.59	58.92
Public open space	7	3.91	13.30	0.00
Road reserve	21	11.74	65.74	8.80

Total area (ha)	Total percent (%)
55.90	100

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
1551.73	142.57

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
91.47	2.01

Commercial, Industry and Schools

Landuse	Percent (%)	Area (ha)
Light industrial	100	22.92
Heavy industrial	0	0.00
Commercial / Offices	0	0.00
Schools	0	0.00
Public buildings	0	0.00

Total area (ha)	Total percent (%)
22.92	41

Nitrogen input (kg)	Phosphorus input (kg)
327.74	66.47

Rural living

Landuse	Percent (%)	Area (ha)
Unrestricted	0	0.00
No livestock	100	17.33
No clearing apart from the housing pad	0	0.00

Total area (ha)	Total percent (%)
17.33	31

Nitrogen input (kg)	Phosphorus input (kg)
852.59	58.92

Note: Commercial horticultre is not permitted in the rural living zone, due to spray drift buffers.

Public Open Space (POS)

Landuse	Percent (%)	Area (ha)
Native gardens	0	0.00
Non-native gardens	0	0.00
Not fertilised	15	0.59
Nature	85	3.33
Sport	0	0.00
Recreation	0	0.00
Golf course	0	0.00
Bowling green	0	0.00
Impervious	0	0.00
Water body	0	0.00

Total area (ha)	Total percent (%)
3.91	7

Nitrogen input (kg)	Phosphorus input (kg)
13.30	0.00

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	8.22
Road reserve - impervious	0	0.00
Road reserve - native garden	0	0.00
Road reserve - non-native garden	0	0.00
Road reserve - turf	5	0.59
Road reserve - not fertilised	25	2.93

Total area (ha)	Total percent (%)
11.739	21

Nitrogen input (kg)	Phosphorus input (kg)
65.74	65.74

Soil and drainage information

Type of drainage	**Open channel drains**	**Does it contain imported fill?**	**No**
Soil type	**Pinjarra**	**Does subregion contain onsite sewage diposal system?**	**N**
Depth to groundwater (m)	**2**		
Groundwater slope (%)	**1**		
Soil PRI	**19.3**		

Note: Please attach the results of soil tests to this report when submitting.

Summary: Nutrient stripping devices

Treatment Name	Size (m²)	Treated area (ha)	Treating	N removed (kg/yr)	P removed (kg/yr)
Load removed				**0.00**	**0.00**
Net export				**91.47**	**2.01**

Summary: Nutrient load exports

Region	Area (ha)	P export (kg/yr)	N export (kg/yr)
Subregion 1	55.90	2.01	91.47

PRE-TREATMENT LOAD (kg/yr)		LOAD REMOVED (kg/yr)		NET LOAD EXPORT (kg/yr)	
NITROGEN	PHOSPHORUS	NITROGEN	PHOSPHORUS	NITROGEN	PHOSPHORUS
91.47	2.01	0.00	0.00	91.47	2.01

APPENDIX 7 UNDO RESULTS POST-DEVELOPMENT

Government of Western Australia
Department of Water

Project: High Wycombe

Date: 12-Dec-20

Version: Version 1.1.0.16333

Subregion name: Catchment 1

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	0	0.00	0.00	0.00
Industrial, commercial & schools	65	2.36	33.78	6.85
Rural living	0	0.00	0.00	0.00
Public open space	0	0.00	0.00	0.00
Road reserve	35	1.27	26.71	3.03

Total area (ha)	Total percent (%)
3.63	7

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
79.49	10.43

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
5.08	0.49

Commercial, Industry and Schools

Landuse	Percent (%)	Area (ha)
Light industrial	100	2.36
Heavy industrial	0	0.00
Commercial / Offices	0	0.00
Schools	0	0.00
Public buildings	0	0.00

Total area (ha)	Total percent (%)
2.36	65

Nitrogen input (kg)	Phosphorus input (kg)
33.78	6.85

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	0.89
Road reserve - impervious	0	0.00
Road reserve - native garden	15	0.19
Road reserve - non-native garden	0	0.00
Road reserve - turf	15	0.19
Road reserve - not fertilised	0	0.00

Total area (ha)	Total percent (%)
1.271795	35

Nitrogen input (kg)	Phosphorus input (kg)
26.71	26.71

Soil and drainage information

Type of drainage	**Piped drainage**	**Does it contain imported fill?**	**Yes**
Soil type	**Pinjarra**	Type of fill imported	**White sands (Bassendean)**
Depth to groundwater (m)	**3**	Fill depth (m)	**1**
Groundwater slope (%)	**1**	Approximate PRI of imported fill	**5**
Soil PRI	**14.6**	**Does subregion contain onsite sewage diposal system?**	**No**

Note: Please attach the results of soil tests to this report when submitting.

Subregion name: Catchment 2

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	37	1.54	32.08	9.75
Industrial, commercial & schools	13	0.54	14.25	3.46
Rural living	0	0.00	0.00	0.00
Public open space	36	1.50	5.98	0.00
Road reserve	14	0.58	12.21	1.39

Total area (ha)	Total percent (%)
4.15	8

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
86.24	15.21

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
5.07	0.21

Residential

Size (m²)	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
<400	0	0.00	0.00	0.00
400-500 m²	0	0.00	0.00	0.00
501-600 m²	0	0.00	0.00	0.00
601-730 m²	0	0.00	0.00	0.00
>730 m²	0	0.00	0.00	0.00
Multiple dwellings	100	1.54	32.08	9.75

Total area (ha)	Total percent (%)
1.536536	37

Nitrogen input (kg)	Phosphorus input (kg)
32.08	9.75

Commercial, Industry and Schools

Landuse	Percent (%)	Area (ha)
Light industrial	0	0.00
Heavy industrial	0	0.00
Commercial / Offices	0	0.00
Schools	0	0.00
Public buildings	100	0.54

Total area (ha)	Total percent (%)
0.54	13

Nitrogen input (kg)	Phosphorus input (kg)
14.25	3.46

Public Open Space (POS)

Landuse	Percent (%)	Area (ha)
Native gardens	0	0.00
Non-native gardens	0	0.00
Not fertilised	0	0.00
Nature	100	1.50
Sport	0	0.00
Recreation	0	0.00
Golf course	0	0.00
Bowling green	0	0.00
Impervious	0	0.00
Water body	0	0.00

Total area (ha)	Total percent (%)
1.50	36

Nitrogen input (kg)	Phosphorus input (kg)
5.98	0.00

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	0.41
Road reserve - impervious	0	0.00
Road reserve - native garden	15	0.09
Road reserve - non-native garden	0	0.00
Road reserve - turf	15	0.09
Road reserve - not fertilised	0	0.00

Total area (ha)	Total percent (%)
0.581392	14

Nitrogen input (kg)	Phosphorus input (kg)
12.21	12.21

Soil and drainage information

Type of drainage	**Open channel drains**	**Does it contain imported fill?**	**Yes**
Soil type	**Pinjarra**	Type of fill imported	**White sands (Bassendean)**
Depth to groundwater (m)	**2**	Fill depth (m)	**1**
Groundwater slope (%)	**1**	Approximate PRI of imported fill	**5**
Soil PRI	**12.2**	**Does subregion contain onsite sewage diposal system?**	**N**

Note: Please attach the results of soil tests to this report when submitting.

Subregion name: Catchment 3

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	83	3.45	72.36	22.00
Industrial, commercial & schools	0	0.00	0.00	0.00
Rural living	0	0.00	0.00	0.00
Public open space	0	0.00	0.00	0.00
Road reserve	17	0.71	14.83	1.68

Total area (ha)	Total percent (%)
4.15	8

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
108.90	24.31

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
6.95	1.14

Residential

Size (m²)	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
<400	0	0.00	0.00	0.00
400-500 m²	0	0.00	0.00	0.00
501-600 m²	0	0.00	0.00	0.00
601-730 m²	0	0.00	0.00	0.00
>730 m²	0	0.00	0.00	0.00
Multiple dwellings	100	3.45	72.36	22.00

Total area (ha)	Total percent (%)
3.446824	83

Nitrogen input (kg)	Phosphorus input (kg)
72.36	22.00

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	0.49
Road reserve - impervious	0	0.00
Road reserve - native garden	15	0.11
Road reserve - non-native garden	0	0.00
Road reserve - turf	15	0.11
Road reserve - not fertilised	0	0.00

Total area (ha)	Total percent (%)
0.705976	17

Nitrogen input (kg)	Phosphorus input (kg)
14.83	14.83

Soil and drainage information

Type of drainage	**Piped drainage**
Soil type	**Pinjarra**
Depth to groundwater (m)	**3**
Groundwater slope (%)	**1**
Soil PRI	**19.3**

Does it contain imported fill? **No**

Does subregion contain onsite sewage diposal system? **No**

Note: Please attach the results of soil tests to this report when submitting.

Subregion name: Catchment 4

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	0	0.00	0.00	0.00
Industrial, commercial & schools	77	2.80	40.01	8.11
Rural living	0	0.00	0.00	0.00
Public open space	7	0.25	16.79	0.51
Road reserve	16	0.58	12.21	1.39

Total area (ha)	Total percent (%)
3.63	7

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
88.01	10.55

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
5.53	0.49

Commercial, Industry and Schools

Landuse	Percent (%)	Area (ha)
Light industrial	100	2.80
Heavy industrial	0	0.00
Commercial / Offices	0	0.00
Schools	0	0.00
Public buildings	0	0.00

Total area (ha)	Total percent (%)
2.80	77

Nitrogen input (kg)	Phosphorus input (kg)
40.01	8.11

Public Open Space (POS)

Landuse	Percent (%)	Area (ha)
Native gardens	0	0.00
Non-native gardens	0	0.00
Not fertilised	0	0.00
Nature	0	0.00
Sport	0	0.00
Recreation	100	0.25
Golf course	0	0.00
Bowling green	0	0.00
Impervious	0	0.00
Water body	0	0.00

Total area (ha)	Total percent (%)
0.25	7

Nitrogen input (kg)	Phosphorus input (kg)
16.79	0.51

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	0.41
Road reserve - impervious	0	0.00
Road reserve - native garden	15	0.09
Road reserve - non-native garden	0	0.00
Road reserve - turf	15	0.09
Road reserve - not fertilised	0	0.00

Total area (ha)	Total percent (%)
0.581392	16

Nitrogen input (kg)	Phosphorus input (kg)
12.21	12.21

Soil and drainage information

Type of drainage	**Piped drainage**	**Does it contain imported fill?**	**No**
Soil type	**Pinjarra**	**Does subregion contain onsite sewage diposal system?**	**No**
Depth to groundwater (m)	**3**		
Groundwater slope (%)	**0.5**		
Soil PRI	**19.3**		

Note: Please attach the results of soil tests to this report when submitting.

Subregion name: **Catchment 5**

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	0	0.00	0.00	0.00
Industrial, commercial & schools	74	26.89	384.52	77.98
Rural living	0	0.00	0.00	0.00
Public open space	5	1.82	119.91	3.63
Road reserve	21	7.63	160.25	18.20

Total area (ha)	Total percent (%)
36.34	70

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
854.72	105.26

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
47.26	1.43

Commercial, Industry and Schools

Landuse	Percent (%)	Area (ha)
Light industrial	100	26.89
Heavy industrial	0	0.00
Commercial / Offices	0	0.00
Schools	0	0.00
Public buildings	0	0.00

Total area (ha)	Total percent (%)
26.89	74

Nitrogen input (kg)	Phosphorus input (kg)
384.52	77.98

Public Open Space (POS)

Landuse	Percent (%)	Area (ha)
Native gardens	0	0.00
Non-native gardens	0	0.00
Not fertilised	0	0.00
Nature	0	0.00
Sport	0	0.00
Recreation	100	1.82
Golf course	0	0.00
Bowling green	0	0.00
Impervious	0	0.00
Water body	0	0.00

Total area (ha)	Total percent (%)
1.82	5

Nitrogen input (kg)	Phosphorus input (kg)
119.91	3.63

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	5.34
Road reserve - impervious	0	0.00
Road reserve - native garden	15	1.14
Road reserve - non-native garden	0	0.00
Road reserve - turf	15	1.14
Road reserve - not fertilised	0	0.00

Total area (ha)	Total percent (%)
7.63077	21

Nitrogen input (kg)	Phosphorus input (kg)
160.25	160.25

Soil and drainage information

Type of drainage	**Open channel drains**	**Does it contain imported fill?**	**No**
Soil type	**Pinjarra**	**Does subregion contain onsite sewage diposal system?**	**N**
Depth to groundwater (m)	**1.5**		
Groundwater slope (%)	**1**		
Soil PRI	**19.3**		

Note: Please attach the results of soil tests to this report when submitting.

Summary: Nutrient stripping devices

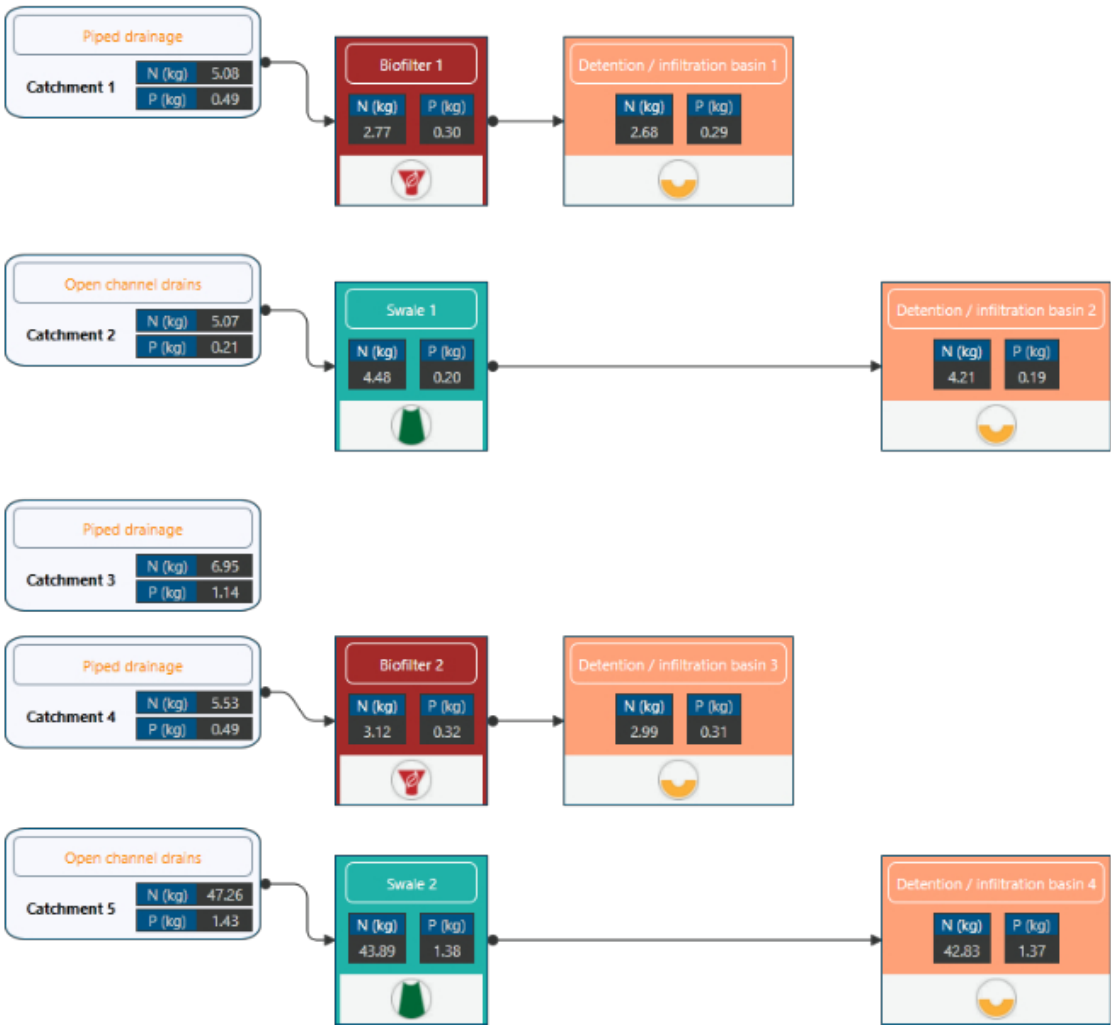
Treatment	Name	Size (m²)	Treated area (ha)	Treating	N removed (kg/yr)	P removed (kg/yr)
Detention / infiltration basin	Detention / infiltration basin 1	1260.00	3.63	Sandy soils – Runoff only (infiltration on lots)	0.09	0.01
Biofilter	Biofilter 1	325.00	3.63	Sandy soils – Runoff and subsoil drains	2.31	0.19
Swale	Swale 1	2750.00	4.15	Sandy soils – Runoff only (infiltration on lots)	0.59	0.01
Detention / infiltration basin	Detention / infiltration basin 2	1200.00	4.15	Sandy soils – Runoff and subsoil drains	0.27	0.01
Biofilter	Biofilter 2	152.00	3.63	Sandy soils – Runoff and subsoil drains	2.40	0.18
Detention / infiltration basin	Detention / infiltration basin 3	2000.00	3.63	Sandy soils – Runoff only (infiltration on lots)	0.13	0.01
Detention / infiltration basin	Detention / infiltration basin 4	4500.00	36.34	Sandy soils – Runoff only (infiltration on lots)	1.06	0.02
Swale	Swale 2	2750.00	36.34	Sandy soils – Runoff only (infiltration on lots)	3.37	0.05
Load removed					**10.24**	**0.47**
Net export					**59.66**	**3.30**

Summary: Nutrient load exports

Region	Area (ha)	P export (kg/yr)	N export (kg/yr)
Catchment 1	3.63	0.49	5.08
Catchment 2	4.15	0.21	5.07
Catchment 3	4.15	1.14	6.95
Catchment 4	3.63	0.49	5.53
Catchment 5	36.34	1.43	47.26

PRE-TREATMENT LOAD (kg/yr)		LOAD REMOVED (kg/yr)		NET LOAD EXPORT (kg/yr)	
NITROGEN	PHOSPHORUS	NITROGEN	PHOSPHORUS	NITROGEN	PHOSPHORUS
69.90	3.76	10.24	0.47	59.66	3.30

Treatment diagram



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Urbaqua
land & water solutions
Suite 4/226 Carr Place
p: 08 9328 4663 | f: 08 6316 1431
e: info@urbaqua.org.au
www.urbaqua.org.au

element.

Technical Appendix E - KCTT Traffic Impact Assessment

TRANSPORT IMPACT ASSESSMENT

Forrestfield North

TOD Precinct

September 2021

Final

kctt

Transport Impact Assessment
KC00604.000 Forrestfield North - TOD Precinct

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Prepared by: **KCTT (Trading as KC Traffic and Transport Pty Ltd)**
ABN 35 148 970 727 |
Postal address: **PERTH:** Unit 7, No 10 Whipple Street Balcatta WA 6021 | **BELGRADE**: 23 Hilandarska, Beograd 11000
Phone: 08 9441 2700 |
Website: www.kctt.com.au |

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1. Executive Summary

Site Context

- The Activity Centre Structure Plan Area is located within the City of Kalamunda, approximately 12km from the Perth CBD. Roe Highway binds it to the east, Poison Gully Creek and Maida Vale Road to the north, Dundas Road, the existing freight railway line to the west, and Berkshire Road to the south. It encompasses approximately 252 hectares. While the site is located within the City of Kalamunda, Develoment WA is managing planning processes.

- While some of the roads in the ACSP area are under construction, the most notable change to the existing road network is the realignment of Dundas Road to construct the proposed High Wycombe Train Station and the construction of the TOD Connector. These are discussed in depth within Section 2.20, which also details the proposed internal road network cross-sections.

Technical Findings

- Proposed Land Uses within the subject Activity Centre Structure Plan Area are Commercial, F High Wycombe Train Station Parking and Kiss'n'Ride parking. Please refer to section 2.3. Land Uses for further details.
- Significant changes are proposed within this ACSP, leading to an increase in residential density and improvements in transport infrastructure. Since the surrounding area will go through significant changes, it is expected that public transport services will intensify. At this stage of planning, it is expected that buses will operate along TOD Connector, Berkshire Road, Dundas Road and Maida Vale Road.
- Service and delivery vehicles for commercial land use may require appropriate parking allocated on site. The crossovers will be designed to accommodate the movement of dedicated service vehicles as a minimum. These requirements should be reviewed as part of the respective Development Applications for each of the sites.
- The total expected additional traffic generated by the proposed development is likely to be 17,185 vehicular trips per day, 2,696 vehicular trips in AM peak hour, and 2,955 vehicular trips in PM peak hour. This is the total theoretical maximum traffic generation from Forrestfield North TOD Precinct.
  The traffic discharged on the external network is somewhat lower. Traffic modelling includes the reciprocity between future Forrestfield North Residential Precinct residents and the subject precinct's land uses to prevent overestimating the traffic on the surrounding network.

- The Forrestfield North and surrounding areas are going through significant changes in the next couple of decades. The road network will expand significantly through the construction of new roads and upgrade existing roads. We believe that the planned network will accommodate the expected growth in population and activities in the surrounding area and adequately manage the increased traffic demand within the ACSP.

Relationship with Policies

- While not directly referencing, the structure plan is following the principles of Liveable Neighbourhoods of promotion of pedestrian and cyclist traffic:
    - Every major road within the structure plan area will have either a shared path or a separate pedestrian path. High Wycombe Train Station could be very important and attractive for the future residents of Forrestfield North, Maida Vale South and other surrounding suburbs. Adequate bicycle parking facilities will be available at all “end of trip” destinations for all non-residential uses to further encourage cycling in the area and reduce dependency on personal vehicles.
- The City’s Local Planning Scheme No 3 provides guidance on future individual developments' general parking provision requirements. However, the Forrestfield North ACSP has the site-specific consideration of being near a future railway station which is also expected to have 50% of its patronage arriving by bus. Given this consideration, the subject area should adopt reduced parking requirements for non-residential land uses. This is consistent with the general TOD principles and promotion of public transport usage in areas with higher public transport provision.
- Further to the principles of TOD Precinct on parking provision and design, it would be beneficial for the City of Kalamunda to consider implementing public car parking to be managed by the City. Therefore, the City of Kalamunda's can control the urban design response and ensure that public car parking is well integrated into the TOD precinct.
- Separate parking requirements are to be provided for the area as discussed in Section 2.10, which will prevent overprovision of parking.

Key Changes in This Revision

- Following a detailed economic analysis, the quantum of commercial land use was significantly decreased when compared to previous iterations of the report. This approach was taken to ensure the short and long term economic viability of the commercial precinct. The reduction in the quantum of available commercial floorspace by 2050 resulted in lower traffic volumes on the internal and external networks. The possibility of intensifying commercial land use still exists; however, it may eventuate beyond the planning timeframe for this structure plan (2050).
- Reduced projected demand in Forrestfield North Project Area removed the requirement for an overpass connecting Forrestfield North and Maida Vale South. As the requirement for this infrastructure may arise beyond the current planning timeframe (2050), the land will be reserved for it. This reservation only impacts the adjacent, Residential Precinct; however, the absence of this connection directly influenced the distribution of traffic within the TOD Precinct.

2. Transport Impact Assessment

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2.1. Location

Structure Plan	Forrestfield North Activity Centre and TOD Precinct – Activity Centre Plan (Local Structure Plan)
Street Number	n/a
Road Name/s	n/a
Suburb	High Wycombe / Forrestfield North

Description of Site

The Local Structure Plan Area is located within the City of Kalamunda, approximately 12km from the Perth CBD. Roe Highway bounds it to the east, Poison Gully Creek and Maida Vale Road to the north, Dundas Road, the existing freight railway line to the west, and Berkshire Road to the south. It encompasses approximately 252 hectares.

Within the District Structure Plan Area are allocated 4 precincts as follows:

- Forrestfield Station TOD Precinct (P1),
- Forrestfield North Activity Centre Precinct (P2),
- Forrestfield North Residential Precinct (P3) and
- Forrestfield / High Wycombe Light Industrial Area (P4).

Precincts P1 and P2 are subject to this TIA report, while Precinct P3 is assessed in a separate TIA report. Precinct P4 is already approved and partially completed; however, it is included in reporting calculations and traffic modelling. Please refer to Plan S01a in Appendix 2 for clarification of the Area Boundary analysed in this report.

This report reflects findings for three horizon years – 2031, 2041 and 2050.

2.2. Technical Literature Used

Local Government Authority	City of Kalamunda
Type of Development	Residential
	Retail / Showroom
	Commercial
	Medical Centre
	Childcare Centre
	High Wycombe Train Station - Parking

	High Wycombe Train Station - Kiss'N'Ride Parking
Are other State Planning Policies referenced?	YES
If YES, nominate which:	SPP 4.2 – Activity Centres SPP 7.2 – Precinct Design SPP 7.3 – Residential Design Codes
Is the NSW RTA Guide to Traffic Generating Developments Version 2.2 October 2002 (referenced to determine trip generation/attraction rates for various land uses) referenced?	YES
Which WAPC Transport Impact Assessment Guideline should be referenced?	Volume 2 – Planning Schemes, Structure Plans & Activity Centre Plans Volume 5 – Technical Guidance
Are there applicable LGA schemes for this type of development?	YES
Number of Scheme	No 3
Name of Scheme	City of Kalamunda Local Planning Scheme
Are Austroads documents referenced?	YES
If YES, nominate which:	Guide to Traffic Management – Part 3: Traffic Studies and Analysis, Austroads, 2008 Guide to Traffic Management – Part 11: Parking, Austroads, 2008 Guide to Traffic Management – Part 12: Traffic Impacts of Developments, Austroads, 2008
Is the Perth Transport Plan for 3.5 million and beyond referenced?	YES
List of other documents:	Restricted Access Vehicles: Prime Mover, Trailer Combinations – Operating Conditions, Main Roads WA, 2012 Forrestfield Station Access Strategy and Concept Design of Dundas Road, Worley Parsons, 2014 Disability Access and Inclusion Plan 2017-2022, City of Kalamunda Forrestfield North – District Structure Plan High Wycombe Railway Station Access Strategy GHD, 2020

2.3. Land Uses

Are there any existing Land Uses within the Structure Plan Area?	YES
If YES, nominate:	Multiple land uses on separate lots, including residential, retail and commercial.
What zone is the Structure Plan Area included in according to the Metropolitan Region Scheme and LPS / TPS?	According to the City of Kalamunda Local Planning Scheme No.3, the Activity Centre Structure Plan (ACSP) area encompasses land that is currently zoned as Urban Development.
Density / Yield / Area	The subject site currently features approximately 35 existing residences and a 32ha total area of light industry.

Proposed Land Uses

How many types of land uses are proposed?	Eight (8) proposed land-uses

Nominate land use type and yield

Development Type	Yields (2050)
Residential	508 dwellings
Commercial	1,200m^2
Retail	5,160m^2
Showroom Retail	2,500m^2
Medical Centre	1,200m^2
Childcare Centre	1,250m^2
Community Facilities	Potential Aquatic Centre for 1,256 persons per day
High Wycombe Train Station - Parking	1,200 parking bays
High Wycombe Train Station - Kiss'N'Ride Parking	100 parking bays

N/A

Is there any proposed staging of the Structure Plan Area?

Are the proposed land uses complementary with the surrounding land-uses?

The ACSP area is surrounded by complementary residential use to the north and industrial and commercial uses to the south and west.

2.4. Local Road Network Information

How many existing roads are there within the Structure Plan Area?	12 roads Parts of some of the roads are still under construction. Some minor roads in the immediate vicinity are mentioned in this section as they are likely to be extended into the ASCP area.

Name of Roads within the Structure Plan Area / Road Classification and Description:

Road 1

Road Name	**Berkshire Road**
Number of Lanes	one lane per direction
Road Reservation Width	approximately 20m
Road Pavement Width	9m (4.5m per lane)
Classification	Distributor B
Speed Limit	70 km/h
Bus Route	286 - Perth - Maida Vale via Belmont Forum 287 - Perth – Forrestfield via Belmont Forum 288 - Perth - Maida Vale via Belmont Forum & Forrestfield 294 - Midland Station - Westfield Carousel via Forrestfield & High Wycombe These bus routes operate east of Roe Highway
On-street parking	NO
RAV Network	RAV 7

Road 2

Road Name	**Dundas Road**

Number of Lanes	one lane per direction
Road Reservation Width	approximately 20m
Road Pavement Width	7m (3.5m per lane)
Classification	Distributor B
Speed Limit	70 km/h
Bus Route	NO
On-street parking	NO
RAV Network	RAV 6 north of Berkshire, RAV 7 south of Berkshiure

Road 3

Road Name	**Milner Road**
Number of Lanes	two-way one lane
Road Reservation Width	approximately 20m
Road Pavement Width	7.5m
Classification	Local Distributor
Speed Limit	70 km/h
Bus Route	NO
On-street parking	NO
RAV Network	RAV 2; South of Nardine Close only

Road 4

Road Name	**Sultana Road West**
Number of Lanes	two-way one lane
Road Reservation Width	approximately 20m
Road Pavement Width	6m
Classification	Access Road
Speed Limit	50 km/h
Bus Route	NO
On-street parking	NO
RAV Network	NO

Road 5

Road Name	**Brae Road**
Number of Lanes	two-way one lane
Road Reservation Width	approximately 20m
Road Pavement Width	6m
Classification	Access Road
Speed Limit	50 km/h
Bus Route	NO
On-street parking	NO
RAV Network	NO

Road 6

Road Name	**Stewart Road**
Number of Lanes	two-way one lane
Road Reservation Width	approximately 20m

Road Pavement Width	6.5m
Classification	Access Road
Speed Limit	50 km/h
Bus Route	NO
On-street parking	NO
RAV Network	NO

Road 7

Road Name	**Eureka Street**
Number of Lanes	two-way one lane
Road Reservation Width	approximately 20m
Road Pavement Width	9m
Classification	Access Road
Speed Limit	50 km/h
Bus Route	NO
On-street parking	NO
RAV Network	NO

Road 8

Road Name	**Imperial Street**
Number of Lanes	two-way one lane
Road Reservation Width	approximately 20m
Road Pavement Width	6m
Classification	Access Road
Speed Limit	50 km/h
Bus Route	NO
On-street parking	NO
RAV Network	NO

Road 9

Road Name	**Nardine Close**
Number of Lanes	two-way one lane
Road Reservation Width	approximately 20m
Road Pavement Width	10m
Classification	Access Road
Speed Limit	50 km/h
Bus Route	NO
On-street parking	NO
RAV Network	RAV 7

Road 10

Road Name	**Maida Vale Road**
Number of Lanes	one lane per direction
Road Reservation Width	approximately 20m
Road Pavement Width	9m (4.5m per lane)
Classification	Distributor B

Speed Limit	60 km/h
Bus Route	YES 294 - Midland Station - Westfield Carousel via Forrestfield & High Wycombe 296 - Perth – Kalamunda via Gooseberry Hill Road 298 - Perth - Maida Vale via Belmont Forum & Abernethy Road
On-street parking	NO
RAV Network	NO

Road 11

Road Name	**Ibis Place / Enterprise Boulevard**
Number of Lanes	two-way one lane
Road Reservation Width	approximately 20m
Road Pavement Width	6.5m
Classification	Access Road
Speed Limit	50 km/h
Bus Route	NO
On-street parking	NO
Note*	Road still in construction
RAV Network	NO

Road 12

Road Name	**Raven Street**
Number of Lanes	two-way one lane
Road Reservation Width	approximately 20m
Road Pavement Width	6m
Classification	Access Road
Speed Limit	50 km/h
Bus Route	NO
On-street parking	NO
Note*	Road still in construction
RAV Network	NO

Name of Other Roads within a 2km radius of the site, or roads likely to take increased traffic due to the development:

Road 1

Road Name	**Roe Highway**
Number of Lanes	two lanes per direction, with central median
Road Reservation Width	approximately 100 m
Road Pavement Width	7m pavement each direction (3.5m lanes) with a 12.5m median
Classification	Primary Distributor
Speed Limit	100 km/h - SLK [28.81 - 34.50]
Bus Route	NO
On-street parking	NO
RAV Network	RAV 7

2.5. Traffic Volumes

Road Name	Location of Traffic Count	Vehicles Per Day (VPD)	Vehicles per Peak Hour (VPH)				Heavy Vehicle %	Year	
			AM Peak VPH	AM Peak Time	PM Peak VPH	PM Peak Time	*If HV count is Not Available, are HV likely to be in higher volumes than generally expected?*	Date of Traffic Count	*If older than 3 years multiply with a growth rate (3% growth rate per annuum)*
Berkshire Road	West of Roe Highway (SLK 0.75)	6,531	552	06:45	564	15:15	26.8%	2019/ 2020	5,035
	East of Roe Highway (SLK 1.58)	5,829	460	07:45	468	14:45	11.2%	Dec 2015	6,757
	40m East of Milner Road *	5,054	383	07:00	463	16:00	15.9%	Aug 2016	5,688
	West of Roe Highway Ramp (Northbound) **	7,919	511	08:00	748	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
	East of Roe Highway Ramp (Northbound) **	7,101	522	08:00	640	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
	Ramp on to Roe Highway (Northbound) North of Berkshire Road **	2,487	250	08:00	183	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
	West of Roe Highway Ramp (Southbound) **	11,569	849	08:00	1,076	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
	East of Roe Highway Ramp (Southbound) **	11,292	928	08:00	1,047	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
	Ramp on to Roe Highway (Southbound) South of Berkshire Road **	2,551	142	08:00	260	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
Roe Highway	South of Kalamunda Road (SLK 33.94)	44,657	3,624	07:00	3,823	16:15	12.7%	2018/ 2019	-
	North of Berkshire Road	53,578	4,285	07:15	4,848	16:00	17.1%	2019/ 2020	-
	Ramp off to Maida Vale Road (Northbound) South of Maida Vale Road (SLK 0.26)	4,946	293	11:30	480	16:00	11.7%	Dec 2015	5,733
	Ramp off to Berkshire Road (Northbound) South of	6,180	457	08:00	527	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-

	Berkshire Road**								
	Ramp off to Berkshire Road (Southbound) North of Berkshire Road**	2,940	248	08:00	241	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
Newburn Road	South of Kalamunda Road (SLK 0.08)	5,882	462	08:15	555	16:00	4.8%	2018/ 2019	-
Hawtin Road	South of Maud Road (SLK 1.98)	7,025	592	08:00	668	15:15	8.3%	2019/ 2020	-
	South of Kalamunda Road**	8,944	828	08:00	868	17:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
Maida Vale Road	30m West of Myerson Crescent*	5,968	582	08:00	625	17:00	6.1%	Sep 2016	6,717
	166 m West of Butcher Road*	1,994	137	07:00	220	16:00	9.1%	Jun 2019	-
	67m East of Milner Road*	3,711	278	07:00	359	16:00	8.0%	Mar 2018	-
	20m West of Littlefield Street*	6,162	675	08:00	778	16:00	7.3%	Nov 2018	-
	170m East of Dundas Road*	2,430	187	07:00	254	17:00	7.0%	Mar 2018	-
	78m West of Milner Road*	3,062	211	08:00	299	16:00	7.3%	Apr 2018	-
	65m East of Plover Road*	8,851	659	08:00	971	16:00	7.6%	Sep 2019	-
	100m West of Jaeger Court*	3,870	276	08:00	411	16:00	8.3%	Nov 2020	-
	West of Hawtin Road (SLK 0.10)	7,106	596	08:00	683	17:00	9.2%	Dec 2015	8,237
	Ramp on to Roe Highway (Southbound) South of Maida Vale Road (SLK 0.23)	5,720	511	07:15	436	16:15	17.0%	Dec 2015	6,631
Abernethy Road	South of Dundas Road (SLK 4.96)	21,232	1,663	07:15	1,720	15:45	23.6%	Dec 2015	24,613
	South of Kalamunda Rd (SLK 6.54)	17,958	1,335	07:30	1,472	15:45	21.7%	2018/ 2019	-
Apricot Street	57 M South West of Fruit Tree Crescent*	1,538	109	08:00	142	17:00	5.6%	Apr 2016	1,731
Dundas Road	South of Maida Vale Road (SLK 2.04)	4,770	372	07:30	424	16:00	19.4%	2016/ 2017	5,368
	40m South of Sultana Road West*	2,708	194	06:00	264	16:00	22.3%	Jul 2016	3,047
	25m North of Onyx Court*	6,091	421	08:00	603	16:00	13.7%	Jun 2016	6,855
	41m South of Carolyn Way*	4,564	434	07:00	387	16:00	20.3%	Oct 2016	5,136

	205m North of Maida Vale Road*	5,687	373	08:00	506	16:00	12.3%	Mar 2018	-
	165m North of Berkshire Road*	4,267	311	07:00	347	15:00	19.4%	Mar 2018	-
	25m South of Kapok Court	5,953	422	07:00	615	16:00	11.5%	Nov 2020	
Dawson Avenue	60m South West of Berkshire Road *	3,100	267	08:00	293	17:00	5.4%	Jul 2016	3,489
Sultana Road West	60m North of Milner Road *	260	20	11:00	28	15:00	18.9%	Oct 2016	292
Harrison Road	35m East of Dundas Road *	971	80	11:00	101	14:00	24.1%	Aug 2016	1,092
Milner Road	32m South West of Sultana Road West *	2,397	228	07:00	236	16:00	14.1%	Mar 2018	-
	120m South of Raven Street *	1,537	163	07:00	151	16:00	9.2%	Mar 2018	-
	45m North East of Stewart Road *	1,807	182	07:00	173	16:00	9.4%	Mar 2018	-
	150m South of Eureka Street *	3,864	360	07:00	346	16:00	19.3%	Sep 2019	3,003
Brae Road	295m North of Sultana Road *	149	13	07:00	14	14:00	17.1%	Oct 2016	167

Note - These traffic counts have been received from the City of Kalamunda*
*Note**- These traffic volumes have been derived from SCATS data obtained through Main Roads. Although SCATS should not be used as a sole source of data, it is a good tool to verify fluctuations in flow.*

DELETED:
Kalamunda Rd; Midland Rd; Grogan Rd

2.6. Vehicular Crash Information

Is Crash Data Available on Main Roads WA website?	YES					
Analysis period	01/01/2016 – 31/12/2020					
Road Name	**Road Hierarchy**	**Speed Limit**	**Crash Statistics**			
			No of KSI Crashes (Fatal + Hospital)	No of Medical Attention Crashes	No of PDO Major Crashes	No of PDO Minor Crashes
Berkshire Road & Roe Highway (Northbound) off to Berkshire Road & Berkshire Road on to Roe Highway (Northbound)*	Distributor B / Primary Distributor / Primary Distributor	70kph / 70kph / 70kph	0	0	10	3
Berkshire Road & Roe Highway (Southbound) off to Berkshire Road & Berkshire Road on to Roe Highway (Southbound)*	Distributor B / Primary Distributor / Primary Distributor	70kph / 70kph / 70kph	0	0	1	2
Berkshire Road & Dundas Road	Distributor B / Distributor B	70kph/ 70kph	1	0	0	1
No of MVKT Travelled at Location	approximately 7,000*365*5years*0.3km=3.83 MVKT					
KSI Crash Rate	1 per 3.83= 0.26 KSI crashes / MVKT					

Comparison with Crash Density and Crash Rate Statistics		0.26 KSI crashes / MVKT is higher than the network average of 0.07 KSI crashes / MVKT.				
Other Crash Rate		2 per 3.83= 0.52 other crashes / MVKT				
Comparison with Crash Density and Crash Rate Statistics		0.52 crashes / MVKT crash rate is lower than the network average of 1.83 crashes / MVKT.				
Maida Vale Road & Maida Vale Road on to Roe Highway (Southbound)	Distributor B / Primary Distributor	60kph / 60kph	1	1	5	1
No of MVKT Travelled at Location		approximately 10,000*365*5years*0.3km=5.47 MVKT				
KSI Crash Rate		1 per 5.47 = 0.18 KSI crashes / MVKT				
Other Crash Rate		7 per 5.47= 1.28 other crashes / MVKT				
Comparison with Crash Density and Crash Rate Statistics		0.18 KSI crashes / MVKT is higher than the network average of 0.07 KSI crashes / MVKT. 1.28 crashes / MVKT crash rate is lower than network average 1.83 crashes / MVKT.				
Maida Vale Road & Roe Highway (Northbound) Off to Maida Vale Road	Distributor B / Primary Distributor	60kph / 60kph	1	8	13	13
No of MVKT Travelled at Location		approximately 9,000*365*5years*0.3km=4.93 MVKT				
KSI Crash Rate		1 per 4.93= 0.2 KSI crashes / MVKT				
		0.2 KSI crashes / MVKT is higher than the network average of 0.07 KSI crashes / MVKT.				
Other Crash Rate		35 per 4.93= 7.09 other crashes / MVKT				
Comparison with Crash Density and Crash Rate Statistics		7.09 crashes / MVKT crash rate is significantly higher than network average 1.83 crashes / MVKT.				
Maida Vale Road & Milner Road	Distributor B / Local Distributor	60kph / 70kph	0	0	2	0

*Note * - Even though the MRWA crash reports provide results for the past 5 years, Berkshire Road and Roe Highway ramps were constructed in early 2016, and therefore the results cannot be considered complete.*

The following tables shows the Crash Density and Crash Rates on Metropolitan Local and Regional Roads as obtained from Main Roads WA on the 13th May 2020 by email request:

Crash Density and Crash Rate on Metropolitan Local Roads Network only

	All Crashes		Serious Injury Crashes (Fatal+Hospital)	
	Average Annual Crash Density (All Crashes/KM)	Average Annual Crash Rate (All Crashes/MVKT)	Average Annual Crash Density (Ser. Inj. Crashes/KM)	Average Annual Crash Rate (Ser. Inj. Crashes/MVKT)
Metro Local Road - Midblock	2.67	0.86	0.11	0.04
Metro Local Road - All	5.70	1.83	0.22	0.07

Note: Based on 5-years data for the period 2015 to 2019.

Crash Density and Crash Rate on Metropolitan State Roads Network only

	All Crashes		Serious Injury Crashes (Fatal+Hospital)	
	Average Annual Crash Density (All Crashes/KM)	Average Annual Crash Rate (All Crashes/MVKT)	Average Annual Crash Density (Ser. Inj. Crashes/KM)	Average Annual Crash Rate (Ser. Inj. Crashes/MVKT)
Metro State Road - Midblock	22.39	0.42	0.87	0.02
Metro State Road - All	52.67	0.98	1.68	0.03

Note: Based on 5-years data for the period 2015 to 2019.

2.7. Public Transport Accessibility

How many bus routes are within 400 metres of the subject site?	3 bus routes
How many rail routes are within 800 metres of the subject site?	Currently no rail routes – Future Forrestfield Airport Link Route

Bus / Rail Route	Description	Peak Frequency	Off-Peak Frequency
294	Midland Station - Westfield Carousel via Forrestfield & High Wycombe	20 minutes	120 minutes
296	Perth - Kalamunda via Gooseberry Hill Road	60 minutes	120 minutes
298	Perth - Maida Vale via Belmont Forum & Abernethy Road	3 times a day	no service

Are high frequency bus routes required to justify a reduction in parking? NO

Walk Score Rating for Accessibility to Public Transport.

10 – 30 Some Transit. A few nearby public transportation options.

Is the development in a Greenfields area? Partially

Forrestfield-Airport Link METRONET

" Completing the Forrestfield-Airport Link - an 8.5km three-station railway spur connected to the Midland Line near Bayswater Station – is part of the first stage of METRONET.

As well as creating a 20-minute train trip between the city and the eastern suburbs, once complete, the rail link will make access to Perth Airport quicker, easier and more affordable for Perth residents and visitors alike.

The project is an important step in improving public transport options for our eastern suburbs. By 2022 the Forrestfield-Airport Link is expected to generate 20,000 passenger trips every day – increasing to 29,000 daily by 2031. With three new stations at Redcliffe, Airport Central and Forrestfield, the new train line will

- *Provide a viable alternative to car travel between the eastern suburbs and Perth.*
- *Enable Perth Airport to continue growing as a nationally important centre of employment, commerce and international trade.*
- *Encourage domestic and international tourism with improved access between Perth Airport and the city. Boost employment, residential and economic growth by encouraging transit oriented development around the new suburban train stations at Redcliffe and Forrestfield.*

Each new station presents different opportunities for their local communities.

High Wycombe Train Station

The catchment area for Forrestfield Station includes the suburbs of High Wycombe, Forrestfield, Maida Vale, Gooseberry Hill and Kalamunda. To make connecting with train services from these suburbs easier, the station precinct is being designed as an intermodal transport hub. This means there will be facilities for passengers arriving at the station by bus, car, bicycle or foot.

To meet demand for parking, a multi-level car park accommodating 1200 cars will be built on land bordered by Maida Vale Road and Ibis Place. While the original plan was for an at-grade car park, the new design will improve access as passengers will now be able to park within 300m of the station's entrance. This new design frees up approximately eight hectares of land for future development. The solution not only meets passenger needs, but also supports METRONET's vision to create connected communities within walking distance of the station and the City of Kalamunda's Forrestfield North District Structure Plan.

Construction of the car park is scheduled to being as soon as tunnelling is finished and will be ready for when first trains run on the Forrestfield Line."

Public Transport Authority website accessed on 30.07.2021.:
http://www.pta.wa.gov.au/forrestfieldairportlink/rail-map :

" High Wycombe Station

The above-ground High Wycombe Station, positioned alongside Dundas Road, will be a terminus station serving a large catchment area in Perth's eastern suburbs and foothills.

- *20-minute journey to CBD*
- *2-zone fare to Perth Station*
- *8-bay bus interchange*
- *1200 parking bays*

Once operational Airport Line trains will be integrated into the existing network, travelling between Claremont Station and Perth Station on the Fremantle Line, along the Midland Line east of the Perth CBD to Bayswater and then spur off towards High Wycombe utilising the new underground track."

Forrestfield Airport Link Project Transport Assessment – Forrestfield Station:

" The station will comprise an at grade passenger rail station, an adjacent bus station with 8 active and 4 layover bays, and a small kiosk to serve intermodal passengers. Car parking with up to 2,500 bays will be located at the development to provide park and ride opportunities.

Active transport will be well catered for with the provision of an integrated network of paths that will bring passengers directly to the Station. There will be secure bike parking with space for 180 bikes and U-rails in the Station forecourt.

The development of the Station will include upgrades to Sultana Road West, Ibis Place and Imperial Street, along with a reconfiguration of the Maida Vale Road / Ibis Place intersection to allow ease of access for buses into the bus station. Relocation of Dundas Road is required for the construction of the Station and this will assist the Shire in its aim of eliminating heavy vehicles from residential areas and restricting it to the light industrial areas to the south of the Station."

It should be noted that the early documents indicate a parking provision of 2,500 bays; however, the total number has been reduced to 1,800 bays. Initially, 1,200 bays will be developed as a multi-story parking facility opposite the train station, with the remainder to be built out at a later stage, if required.
This is in line with the general vision for the station as an alternative transport option which promotes the reduction of car dependence.

2.8. Pedestrian Infrastructure

Describe existing local pedestrian infrastructure within a 400m radius of the site:

Existing pedestrian access through the Forrestfield North is very limited due to the low intensity of the current land-uses. Almost all errands require a vehicle.

Classification	Road Name
"Principal Shared Path (PSP)"	Path along Roe Highway (south of Berkshire Road)
"Other Shared Path (Shared by Pedestrians & Cyclists)"	Maida Vale Road (east of Littlefield Road), Milner Road, Berkshire Road, Palmer Crescent (west of Mack Place), Mack Place, Madderson Road, Golden Crescent (east of Jade Court), Jade Court, Edney Road, Feeney Gardens, Roe Highway (400m north of Berkshire Road)
Does the site have existing pedestrian facilities	YES, on Milner Road and Berkshire Road
Does the site propose to improve pedestrian facilities?	YES

If YES, describe the measures proposed.

Pedestrian Paths

The extent of development for the area envisaged under the ACSP will have a significantly higher intensity of pedestrian activity. Therefore, the provision and design of the pedestrian network will need to match the envisioned demand. To this extent, one of the major objectives of the Forrestfield North ASSP TOD Precinct is to identify key

linkages within the proposed ASSP area. A plan depicting all of the linkages described below is shown in Appendix 2. The pedestrian aspects of Appendix 2 will now be outlined.

It is essential to develop a high quality, universally accessible and permeable network of pedestrian paths to encourage pedestrian movement. Every major road within the structure plan area will have either a shared path or a separate pedestrian path. Please refer to Section 2.22 for details on the Proposed Internal Road Network cross-sections of existing and proposed roads.

Furthermore, in accordance with the City of Kalamunda's Disability Access and Inclusion Plan 2017-2022 or any other subsequent document of this nature, all pedestrian and shared paths should be designed to be accessible by all members of the community. The exact location of pram ramps and other elements is to be determined at a later stage in the project.

What is the Current Walk Score Rating?

0-20 Car-Dependent. Almost all errands require a car.

2.9. Cyclist Infrastructure

Are there any PBN Routes within an 800m radius of the subject site? YES

If YES, describe:

Existing cyclist access through the Forrestfield North LSP area is very limited due to the low intensity of current land-uses.

Classification	Road Name
"Bicycle Lanes or Sealed Shoulder Either Side"	Abernethy Road, Roe Highway (north of Berkshire Road), Newburn Road
"Principal Shared Path (PSP)"	Path along Roe Highway (south of Berkshire Road)
"Good Road Riding Environment"	Maida Vale Road (west of Priory Road), Priory Road, Newburn Road, Milner Road, Berkshire road (west of Ashby Close), Gilba Road, Bruce Road, Apricot Street, Sultana Road East (east of Bruce Road), Myerson Crescent, Jacks Street
"Other Shared Path (Shared by Pedestrians & Cyclists)"	Maida Vale Road (east of Littlefield Road), Milner Road, Berkshire Road, Palmer Crescent (west of Mack Place), Mack Place, Madderson Road, Golden Crescent (east of Jade Court), Jade Court, Edney Road, Feeney Gardens, Roe Highway (400m north of Berkshire Road)

Are there any PBN Routes within a 400m radius of the subject site? YES

If YES, describe:

Classification	Road Name
"Bicycle Lanes or Sealed Shoulder Either Side"	Abernethy Road, Roe Highway (north of Berkshire Road), Newburn Road
"Principal Shared Path (PSP)"	Path along Roe Highway (south of Berkshire Road)
"Good Road Riding Environment"	Maida Vale Road (west of Priory Road), Priory Road, Newburn Road, Milner Road, Berkshire road (west of Ashby Close)
"Other Shared Path (Shared by Pedestrians & Cyclists)"	Maida Vale Road (east of Littlefield Road), Milner Road, Berkshire Road, Palmer Crescent (west of Mack Place), Mack Place, Madderson Road, Golden Crescent (east of Jade Court),

	Jade Court, Edney Road, Feeney Gardens, Roe Highway (400m north of Berkshire Road)
Does the site have existing cyclist facilities?	NO
Does the site propose to improve cyclist facilities?	YES

If YES, describe the measures proposed.

The extent of development for the area envisaged under the ACSP is expected to have a significantly higher intensity of activity; therefore, the provision and design of the cycling network will need to match the envisioned demand. One of the major objectives of the Forrestfield North DSP is to identify key linkages within the proposed ACSP area. A plan depicting all of the linkages described below is shown in Appendix 2. The cycling aspects of Appendix 2 will now be outlined.

To encourage cycling within the ACSP, every major road within the activity centre plan area will have either a shared path or a dedicated cycle lane. Please refer to Section 2.22 for details on the Proposed Internal Road Network cross-sections of existing and proposed roads.

The City of Kalamunda has also adopted a Bicycle Plan, dated December 2017, prepared by Cardno. The routes in Bicycle Plan which form part of the LSP are outlined below :

Classification	Road Name
" Principal Route"	Roe Highway, path along the railway
" Strategic Route"	Milner Road, Maida Vale Road
" Local Route"	Dundas Road, Berkshire Road

2.10. Vehicular Parking

Local Government	City of Kalamunda
Local Government Document Utilised	Local Planning Scheme No. 3 State Planning Policy 3.1 - R-Codes

Description of Parking Requirements in accordance with Utilised Documents:

State Planning Policy 7.3

- 1-bedroom dwellings:
  - Location A – 0.75 bay per dwelling
  - Location B – 1 bay per dwelling
- 2+ bedroom dwellings:
  - Location A – 1 bay per dwelling
  - Location B – 1.25 bay per dwelling
- Visitor Parking – 1 bay per 4 dwellings, up to 12 dwellings. 1 bay per 8 dwellings for the 13th dwelling and above.

Local Planning Scheme No. 3

- **Office** - 4 bays per 100m^2 of NLA.
- **Community Purpose** - 1 bay for every 5 persons the facility is designed to accommodate
- **Child Care Centre/Day Care** - 1 bay per staff member, plus 1 bay for every 10 children allowed under the approval.
- **Medical Centre -** 6 bays per practitioner and 1 per staff
- **Shop** - 5 bays per 100m2 of NLA.
- **Showroom** - 4 bays per 100m2 of NLA

Calculation of Parking

The calculations below are an approximation and are conducted only to give insight into the potential parking requirements for the entire area.

Development Type	Requirements	Yield	Total Parking
Residential	As per R Codes; Note: WAPC Apartment Design Policy (currently a DRAFT version) might be applicable for apartments once the finalised version is published.	508 dwellings	Dependent on type of dwellings
Commercial	App 4 bays per 100m^2 of NLA.	1,200m^2	48
Retail	5 bays per 100m2 of NLA.	5,160m^2	258
Showroom Retail	4 bays per 100m2 of NLA	2,500m^2	100
Medical Centre	6 bays per practitioner and 1 per staff	1,200m^2; Assumed 1 practitioner per 100m^2 (General practitioners 1/3; Specialists 2/3 of total) = 4 General practitioners; 8 specialists; 5 additional staff members	77
Childcare Centre	1 bay per staff member, plus 1 bay for every 10 children allowed under the approval.	1,250m^2; Assumed 125 children (1 per 10m^2) and 25 staff members (1/5 children)	38
Community Facilities	1 bay for every 5 persons the facility is designed to accommodate	Potential Aquatic Centre for 1,256 persons per day	251
		Total parking requirements:	**App 772**

PTA Parking

The PTA intends to provide approximately 1,800 parking bays to support long term parking demand for the proposed High Wycombe Train Station. The latest information available suggests that initially, there will be 1,200 bays provided. The bays will be located in a multi-story car parking facility opposite the railway station. An additional 80 Kiss and Ride drop off bays will be split into two sections in the vicinity of the station entrance. Parking facilities (inclusive of Kiss and Ride facilities) are likely to add significant traffic to the surrounding road network. Therefore, the access design around the station is an important consideration.

Ideally, in the long-term, additional parking areas will be distributed within the walkable catchment from the Proposed High Wycombe Railway Station along the western edge of the Forrestfield North ACSP area. Dispersion of parking areas and multiple access/egress points will also spread traffic impact and help avoid putting significant pressure on any one street. It is understood that these bays will be released in stages as the demand grows.

On-Street Parking

On-street parking will be provided through each of the main linkages in the Forrestfield North ACSP. It is considered that the following streets, as a minimum, will have some form of on-street parking: -

- Milner Street between Sultana Road West and Stewart Road;
- Imperial Street;
- Ibis Place (in the vicinity of the railway station forecourt, mainly for kiss and ride parking);

- TOD Connector;
- Some subdivision roads of lower order.

Other Public Car Parking Areas
A concentration of retail and business is likely to require additional parking, besides on-street parking. It would be beneficial for the City of Kalamunda to consider implementing Public Car Parking to be managed by the City. The most convenient locations for consideration would be within the Mixed-Use and town centre precincts. In order to avoid a major impact on the overall urban amenity, some basic principles are to be followed: -

- A maximum of 1 (one) individual car parking area at-grade of up to 100 car bays in aggregate can be provided;
- All additional public parking areas at-grade should have no more than 50 car bays each;
- Public parking areas at grade must not be adjacent, a minimum distance between at grade parking areas should be around 500 metres apart;
- Any car parking structures or multi-storey car parks should be designed to support the amenity of the urban surroundings and should consider activation of street frontages with parking provided behind.

With these principles in mind, we believe that a set of urban design guidelines should also be established at a later stage of planning for the integrated provision and design of public parking. The guidelines could be used to specify how parking will be integrated with the urban form and function of the Forrestfield North project area.

Parking Provisions for the Proposed Uses
The City's Local Planning Scheme No 3 prescribes general parking requirements for new developments. However, the Forrestfield North ACSP has the site-specific consideration of being near a future railway station which is also expected to have 50% of its patronage arriving by bus. Given these considerations, the subject area should adopt the parking-related principles of TOD to promote public transport usage in areas with higher public transport provision. This could take the form of modified parking ratios, similar to other examples in the Perth Metro Area. Therefore, to promote the beneficial usage of the railway station, establishing a site-specific set of parking provision ratios for future planned land uses is recommended.

Proposed parking requirements will be as follows:

- Residential – Maximum as per Design WA guide; minimum 0.75 per dwelling
- Non-residential – proposed 25% reduction on the requirements listed in the Planning Scheme with the possibility of another 25% reduction if there is a clear incentive for the use of sustainable transportation methods.
- Electric vehicles – minimum 1 parking bay per 10 required bays. This requirement is included in the total parking requirement.
- ACROD – minimum as per the National Construction Code Australia. Benefits should be considered for provisions above the minimum.
- Provision of motorcycle and scooter parking are encouraged as these types of micromobility vehicles are highly suited to short, local trips.
- Reciprocal arrangements and flexible use of parking should be encouraged.

Have Vehicle Swept Paths been checked for Parking? NO

If YES, provide a description of performance.

Not applicable for this stage of development.

2.11. Bicycle Parking

Local Government	City of Kalamunda
Reference Document Utilised	City of Kalamunda's Local Planning Scheme No 3
Other Document Utilised	Austroads – Guide to Traffic Management Part 11: Parking

Description of Parking Requirements in accordance with Utilised Documents:

The City of Kalamunda does not provide requirements for the provision of bicycle parking. Therefore, the following table shows Bicycle Parking Provision Rates based on requirements from Guide to Traffic Management Part 11: Parking, Table H 1: Bicycle parking provision, page 141. Rates are shown only to give insight into potential bicycle parking requirements for the area. They should be reviewed as part of the respective Development Applications for each of the sites.

Development Type	Yields	Employee/resident parking spaces	Class	Visitor/shopper parking spaces	Class
Commercial	app 7,000m^2	1 per 200 m^2 GFA	1or2	1 per 750 m2 over 1000 m2	3

Justification

To support cycling within the TOD and Residential Precincts, appropriate bicycle parking facilities will be required at all "end of trip" destinations for all non-residential uses. For the future residents of Forrestfield North, Maida Vale South and other surrounding suburbs, well designed and located bicycle infrastructure in the Forrestfield station area could be very attractive.

Therefore it would be reasonable to increase nominal Austroads requirements as stipulated in the table above. The Cycling Strategy recommended amendment of the Local Planning Scheme No 3 to include the requirements for bicycle parking.

The latest information taken from the fact sheet for the proposed Forrestfield (High Wycombe) Station noted that the PTA intends to provide 180 secure bicycle parking bays, which will help promote cycling within the TOD precinct.

2.12. ACROD Parking

Class of Building: Class 5

Does this building class require a specific provision of ACROD Parking?: YES

Reference Document Utilised: Building Code of Australia

Description of Parking Requirements:

Class 5 (An office building used for professional or commercial purposes):

- Minimum of 1 ACROD bay for every 100 car parking spaces.

Justification

Every land use should have adequate provision of ACROD parking, according to the above requirements.
All crossings and driveways should have flush kerbs.

2.13. Delivery and Service Vehicles

Guideline Document used as reference: NSW RTA Guide to Traffic Generating Developments

Requirements

Commercial premises *(50% of spaces adequate for trucks): 1 space per 4,000m^2 GFA (if GFA < 20,000m^2);*

Justification

It is expected that delivery and service vehicles (such as waste removal vehicles) servicing the residential area will not require designated parking spaces, given that they can operate safely within the road reserve.

Service and delivery vehicles for other uses will require appropriate parking allocated on site. The crossover are to be designed to accommodate the movement of dedicated service vehicles as a minimum.

These requirements should be reviewed as part of the respective Development Applications for each of the sites.

2.14. Calculation of Development Generated / Attracted Trips

What are the likely hours of operation?	Commercial: 09:00h to 21:00h Childcare Centre: 06:30 - 18:30 Medical Centre: 07:00 - 20:00 Retail: 09:00 - 17:00 Retail Showroom: 09:00 -17:00 Community facilities: 08:00 - 21:00
What are the likely peak hours of operation?	AM 08:00 to 09:00 PM 16:30 to 17:30 Train Station peak hours generally occur earlier than the general road traffic peak
Do the development generated peaks coincide with existing road network peaks?	YES
If YES, Which:	Partially AM and PM
Guideline Document Used	WAPC Transport Assessment Guidelines for Developments
Rates from the above document:	**Office and Commercial Area** - 2 per 100m^2 of GFA in the PM Peak hour. The same rate is assumed for the AM peak. An 80% IN / 20% OUT split has been assumed for the AM peak and the reverse for the PM peak. ***Residential*** *– 0.8 vehicle trips per dwelling for the AM and PM peak hours. A 25% IN / 75% OUT split has been adopted for the AM peak and a 67% IN / 33% OUT split for the PM peak hour;*
Guideline Document Used	NSW RTA Guide to Traffic Generating Developments
Rates from the above document:	**Office / Commercial -** 10 vehicular trips per 100m^2 GFA; ***Residential*** *- The NSW RTA Guide to Traffic Generating Developments suggests developments of this type in Sydney tend to generate between 4 and 5 vehicular trips per dwelling for medium to high density developments. In Perth, the Department of Planning and Infrastructure conducted a series of studies in the late 1990's / early 2000's which showed that higher density dwellings tended to average closer to 5.5 vehicle trips per day. These studies assumed that anywhere between 50% and 70% of commuters were travelling to the work by car as a driver.* The following rates were agreed upon with the City of Kalamunda: 8 vehicular trips per day for R30 and R40; 6.5 vehicular trips per day per residence for R60 houses, 5.5 vehicular trips per day per residents for R80 houses and apartments, and 5 vehicular trips per day for R100 Apartments. **Retail with Food -** 121 vehicular trips per day; 2.5 vehicular trips per hour in the AM peak hour; 10 vehicular trips per hour in the PM peak hour; **Retail without Food -** 17 vehicular trips per day; 2.7 vehicular trips per hour in the AM peak hour; 10 vehicular trips per hour in the PM peak hour;

KCTT calculation / summary:

Medical Centres:

Based on previous experience, KCTT has assumed 1 practitioner per 100m^2. This equates to 12 medical practitioners. General Practitioners are assumed to be 1/3, while specialists are likely to comprise 2/3 of all medical practitioners (4 and 8, respectively).

Medical Centre / Consulting rooms are likely to be used by people who live in the vicinity of the subject development. We have based our conclusions on the expected number of practitioners, which is a total of 12 for all medical centre tenancies. It is standard that consultations with general practitioners are booked in 10-15 minutes slots (a maximum of 36 clients per day) for the main Medical Centre (GP) and 30 minutes for all other medical centre tenancies (a maximum of 15 clients per day). Based on these assumptions, up to 144 clients per day in the GP practice and 120 clients for the specialists' consulting rooms can be expected on a typical working day. Using a 100% vehicular attraction rate equates to a total of 288 (General Practitioners) and 240 (Specialists) vehicular movements per day for the potential maximum number of clients this site can generate. Peak hour rates are estimated as 15% of daily traffic.

Childcare centre

Based on previous experience, KCTT has assumed 1 child per 10m^2. Therefore, the assumed number of children is 125. A total of 25 staff members were assumed based on a 1/5 children ratio.

Vehicular daily trips can be assumed to be 4 VPD per child and 2 VPD per employee. Each parent will make 2 vehicular trips when dropping off the child at the daycare centre and 2 vehicular trips when picking the child up. Employees will make 1 vehicular trip arriving at work, and another vehicular trip when leaving work.
In our experience, childcare centres tend to operate with an 85% utilisation rate of the licenced capacity over the year due to the number of days that children attend (this ranges from 2 to 5 days a week) and seasonal adjustments (end of the year and when people return to work from maternity leave). Therefore, the expected average daily operative maximum of this childcare facility can be estimated as 106 children. Market information indicates that between 10-20% of parents tend to have more than one child at a childcare centre, so those families only account for one vehicular trip. A further percentage of parents will have older siblings attending one of the nearby schools. However, in the calculations below, a conservative approach has been applied, showing the theoretical maximum number of children, assuming that all children are driven to school, and there are no siblings in the centre.

Development Type	Rate Above	Yield	Total VPD	PM Peak VPH	AM Peak VPH
Residential *25% expected to be attracted by the TOD precinct cells	5.5 vehicular trips per dwelling and 5 vehicular trips per dwelling Peak – 0.55 and 0.5 vehicular trips per 100m^2 per hour	207 - 5.5 vpd dwellings; 301 - 5 vpd dwellings	2,644* (1,983)**	265* (199)**	265* (199)**

Commercial *5% expected to be generated by the future residents of FFN area	10 vehicular trips per 100m^2 Peak – 2 vehicular trips per 100m^2 per hour	1,200m^2	120* (114)**	24* (23)**	24* (23)**
Retail with Food Component *45% expected to be generated by the future residents of FFN area	121 vehicular trips per 100m^2 AM Peak – 2.5 vehicular trips per 100m^2 per hour PM Peak – 10 vehicular trips per 100m^2 per hour	Assumed 70% of total retail = 3,612m^2	4,371* (2,404)**	90* (50)**	361* (199)**
Retail without Food Component (including Showroom Retail) *20% expected to be generated by the future residents of FFN area	17 vehicular trips per 100m^2 AM Peak – 2.7 vehicular trips per 100m^2 per hour PM Peak – 2.7 vehicular trips per 100m^2 per hour	Assumed 30% of total retail = 1,548m^2 + 2,500 Showroom = 4,048m^2	688* (551)**	110* (96)**	110* (96)**
Medical Centre *25% expected to be generated by the future residents of FFN area	72 vehicular trips per GP; 30 vehicular trips per day per specialist 15% VPD = VPH	4 General Practitioners; 8 Specialists	528* (396)**	79* (59)**	79* (59)**

Childcare Centre *25% expected to be generated by the future residents of FFN area	4 VPD per child and 2 VPD per staff member AM Peak - 0.8 VPH per child PM Peak - 0.7 VPH per child	125 children; 25 staff members	550* (413)**	100* (75)**	88* (66)**
Community Facilities (Potential Aquatic Centre) *20% expected to be generated by future residents of the FFN area	1.5 vehicular trips per day per person; 0.15 vehicular trips per person per hour	Assumed 1,256 persons per day as per information received from the planner	1,884* (1,507)**	188* (150)**	188* (150)**
High Wycombe Train Station Parking *20% expected to be generated by future residents of the FFN area	Peak Traffic obtained from 'Forrestfield Station Multi-Storey Car Park Traffic Impact Assessment'; Daily Traffic assumed 2 VPD per parking space ; *Train Station peak hours generally occur earlier than the general road traffic peak*	1,200 spaces	2,400* (1,920)**	840* (672)**	840* (672)**
High Wycombe Train Station Kiss'n'Ride Parking *20% expected to be generated by future residents of the FFN area	Peak Traffic obtained from 'Forrestfield Station - Multi-Storey Car Park Traffic Impact Assessment.'	N/A	4,000* (3,200)**	1,000* (800)**	1,000* (800)**
Total - The Proposed Development			**A: 17,185* (12,488)****	**2,696* (2,124**)**	**2,955* (2,264)****

*Note: This is the total theoretical maximum traffic generation from Forrestfield North TOD Precinct.

**Note: This is the traffic to be discharged on the external network. Traffic modelling includes the reciprocity between future Forrestfield North Residential Precinct residents and the subject precinct's land uses to prevent overestimating the traffic on the surrounding network.

Does the site have existing trip generation/attraction?	YES

Development Type	Rate Above	Yield (Area, Number of Dwellings etc.)	Equivalent Area	Total VPD	Total VPH
Residential	9 vehicular trips per day per dwelling 0.8 vehicular trips per day per dwelling	Approximately 35 single residences	N/A	315 VPD	28 VPH
Light Industry	4.6 vehicular trips per day per 100m^2 0.7 vehicular trips per hour per 100m^2	Approximately 320,000m^2	20% of total area 64,000m^2	2,944 VPD	448 VPH
			Total:	**B:** 3,259 VPD	476 VPH

What is the total impact of the new proposed development?	High impact The total expected additional traffic generated by the proposed development would be equal to **A-B**: 17,188 - 3,259 = 13,926 VPD; AM: 2,696 - 476 = 2,220 VPH; PM: 2,955 - 476 = 2,479 VPH

2.15. Trip Purposes

Determine the likely percentage share for different trip purposes based on the land usage.

Land Use Type	Employment	Shopping	Education	Social / Recreational
Residential	40%	17.5%	25%	17.5%
Industrial	100%	n.a.	n.a.	n.a.
Mixed Use	60%	20%	5%	15%

2.16. Expected Origin / Destination

Name the closest existing major residential generators and non-residential attractors of traffic and the distance from the boundaries of the Structure Plan Area.

Residential	Employment (profile.id)	• The proposed Forrestfield North ACSP area is likely to be a significant generator of localised employment opportunities a. West (City of Perth) – 32.5% b. Local (Perth Airport area) – 10% c. Local (LSP area) – 20% d. South (City of Gosnells / City of Canning) - 10%

		e. East (City of Kalamunda) - 10% f. West (City of Belmont) – 7.5% g. North (City of Swan) – 5% h. WA Undefined / Other Areas – 5% • Excluding work from home, likely sources of employment within the Forrestfield North ACSP area include industrial land-uses in the southeast in the vicinity of Berkshire Road and around the future proposed High Wycombe Train Station. • To the immediate west of the site is a significant employment catchment around the Perth Airport. • The High Wycombe Train Station provides excellent opportunities for commuting to the Perth CBD.
	Shopping	• Strong opportunities for local shopping within the Forrestfield North ACSP area. • Local shopping – High Wycombe Shopping Centre / Forrestfield Shopping Centre • Regional Shopping – Belmont Forum / Carousel Cannington
	Education	• Edney Primary School (corner Edney Road / Newburn Road, High Wycombe), Dawson Park Primary School, High Wycombe Primary School, Woodlupine Primary School; Maida Vale Primary School, Forrestfield Primary School (all within 3km from the LSP area). • Potential Primary School to be allocated within the Residential LSP area (pending future development yields). • Kalamunda Senior High School, Darling Range Sports College Forrestfield, Mazenod College Lesmurdie, St Brigid's College Lesmurdie, Belmont City College (High School)
	Social / Recreational	• 17.5% of all trips are deemed to be local for social/recreational purposes. These types of trip purposes are expected to include sporting / local social trips etc. • In an area such as Forrestfield North LSP Area, this could be as high as 25%; however, for this Transport Impact Assessment purposes, we will utilise 17.5% as it places a more conservative volume of traffic onto the key intersections for analysis.
	Employment	• The mixed-use is likely to attract employment trips from the Forrestfield North LSP area, plus generally from the: - o City of Kalamunda to the east (35%) o City of Belmont to the west (25%) o City of Swan / Shire of Mundaring to the north (15%) o City of Gosnells / City of Canning to the south (15%) o Other areas (10%)
Mixed Use	Shopping / Social	At this point, the full range of uses that will be located within the shopping centre is not determined. Therefore, the district centre is treated as commercial premises for the purposes of this report.

		The mixed-use is likely to attract employment trips from the Forrestfield North Residential LSP area, plus generally from the: -
Industrial	Employment	• City of Kalamunda to the east (35%) • City of Belmont to the west (25%) • City of Swan / Shire of Mundaring to the north (15%) • City of Gosnells / City of Canning to the south (15%) • Other areas (10%)

2.17. Traffic Flow Distribution onto External Road Networks

Nominate the external zones for traffic distribution

- Zone 1 Roe Highway (South of Berkshire Road)
- Zone 2 Hawtin Road (South of Berkshire Road)
- Zone 3 Kalamunda Road (East of Hawtin Road)
- Zone 4 Roe Highway (North of Maida Vale Road)
- Zone 5 Kalamunda Road (North of Abernethy Road)
- Zone 6 Abernethy Road (South of Dundas Road)
- Zone 7 Dundas Road (South of Berkshire Road)
- Zone 8 Gooseberry Hill Road

ZONE	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Zone 6	Zone 7	Zone 8	FFN Industrial	Internal Traffic
Forrestfield North Residential Zones	20%	5%	7.5%	15%	10%	8%	6%	2%	1.5%	25%

**Note: It was assumed that internal trips would make approximately 25% of the total generated trips because of the large attractors such as Forrestfield North Station, large Activity Centre and other education, shopping and employment opportunities within the Forrestfield North area.*

ZONE	Zone 17 - Retail	Zone 18 - Mixed Use Cell	Zone 19 - Conservation Sub-Precinct	Zone 20 - Forrestfield Station	Zone 21 - Showroom Retail	Primary School
Residential Zones	0.5% (20%)	13.5% (35%)	0% (0%)	6% (20%)	0.5% (15%)	4.5% (80%)

**Note: It was assumed that internal trips would make 25% of the total generated trips because of the large attractors such as High Wycombe Station, large Activity Centre and other education, shopping and employment opportunities within the Forrestfield North area.*

***Note: Percentage in the brackets represent the share of FFN residential zones in the total traffic generated by the above land uses.*

2.18. Road Safety

Are sight distances adequate at proposed intersections? YES

Justification

All distances between intersections will be following Liveable Neighbourhoods and relevant Austroads Guides. The design of road corridors wiil be constrained to ensure that vehicular operating speeds are lower than posted speed limits. To be reviewed further in next planning stages.

2.19. Proposed Internal Road Network

Guideline Document used as reference	Cross-sections for main transit routes were developed in discussion with the City of Kalamunda; however, comparable Liveable Neighbourhood classification is provided.
How many proposed roads are there within the Structure Plan Area?	Several road types as listed below Please note that the TOD Precinct is a portion of the wider Forrestfield North Area, and the following list of roads shows only the roads within the subject area. *Note: The internal road network within the LSP area shown in Appendix 1 and 2 is only an indicative road network. **Note: Cross-sections shown below are indicative in nature and are intended to show main principles and treatments for various traffic modes. In developing the lower order streets and public open spaces, Public Realm Guidelines should be referred to.

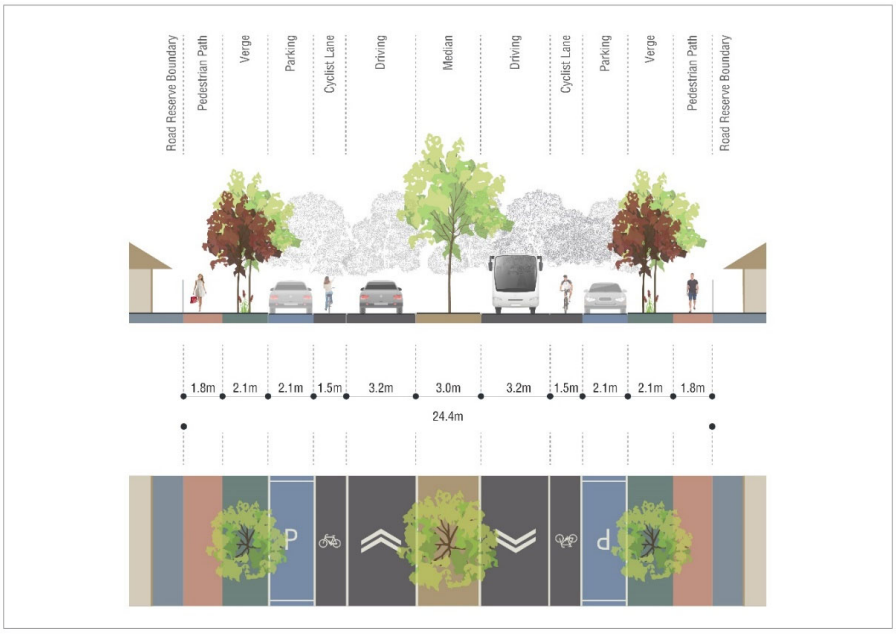
Name of Roads within the Structure Plan Area / Road Classification and Description:

Road 01

Name	TOD Connector
Projected Traffic Volumes	1,428 (2031) 6,432 (2050)
Proposed Number of Lanes	One lane per direction
Proposed Road Reservation Width	24.4m
Proposed Road Pavement Width	3.2m per traffic lane
Proposed Median Width	3.0m
Proposed Pedestrian / Cyclist / Shared Path Width	1.8 pedestrian path on both sides of the road reservation 1.5m cycling lane on both sides of the road reservation
Equivalent LN Classification	Neighbourhood Connector A
Proposed Speed Limit	50kph
Proposed Bus Route Extension / Introduction	YES
If YES Nominate Bus Routes	It is expected that new Bus Routes will be introduced to/from the High Wycombe Train Station along with an Autonomous Bus Circular Route
Proposed On-street parking	YES (2.1 parallel parking on both sides)

Provide graphics of the proposed internal road cross-section within the Structure Plan Area

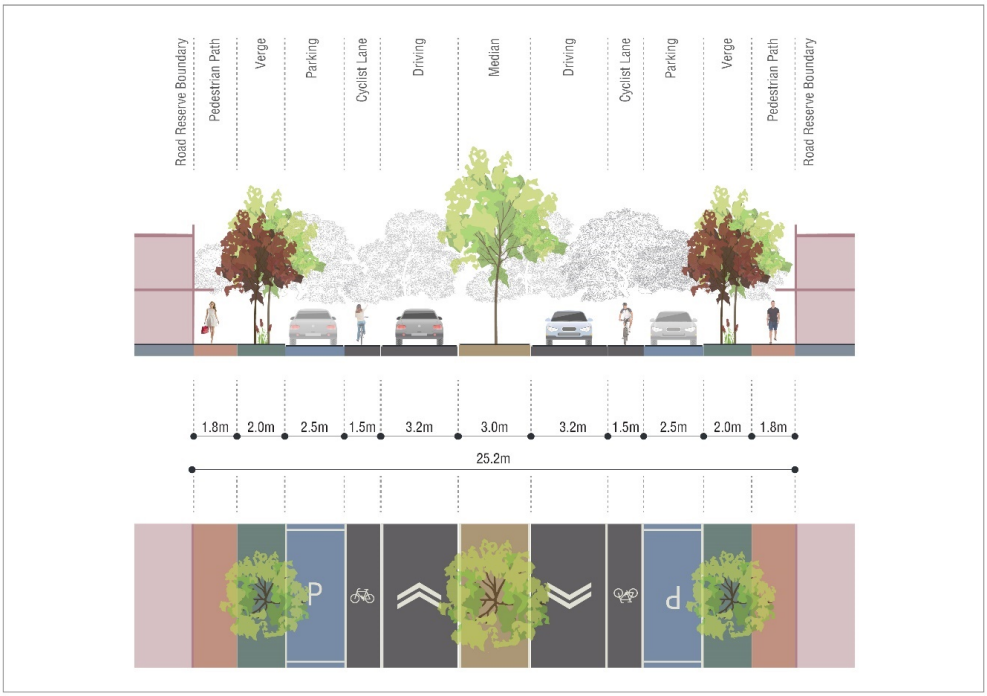
Transport Impact Assessment
KC00604.000 Forrestfield North TOD Precinct



Road 2a

Name	Milner Road (Between Sultana Road West and Stewart Road)
Projected Traffic Volumes	4,045 (2031) 8,110 (2050)
Proposed Number of Lanes	One lane per direction
Proposed Road Reservation Width	25.2
Proposed Road Pavement Width	3.2m per traffic lane
Proposed Median Width	3.0m
Proposed Pedestrian / Cyclist / Shared Path Width	1.8 pedestrian path on both sides of the road reservation 1.5m cycling lane on both sides of the road reservation
Equivalent LN Classification	Integrator B
Proposed Speed Limit	60kph
Proposed Bus Route Extension / Introduction	YES
If YES Nominate Bus Routes	Autonomous Bus Circular Route
Proposed On-street parking	YES (2.5m parallel parking on both sides)

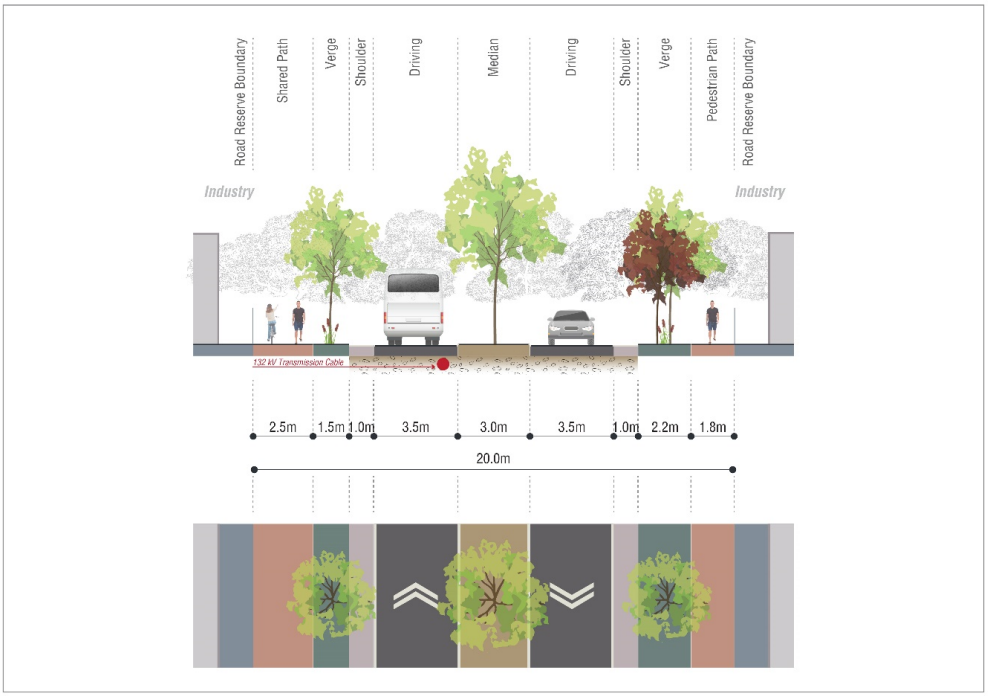
Provide graphics of the proposed internal road cross-section within the Structure Plan Area


Road Reserve Boundary
Pedestrian Path
Verge
Parking
Cyclist Lane
Driving
Median
Driving
Cyclist Lane
Parking
Verge
Pedestrian Path
Road Reserve Boundary
1.8m
2.0m
2.5m
1.5m
3.2m
3.0m
3.2m
1.5m
2.5m
2.0m
1.8m
25.2m

Road 2b

Name	Milner Road (Between Sultana Road West and Berkshire Road)
Projected Traffic Volumes	6,947 (2031) 12,928 (2050)
Proposed Number of Lanes	One lane per direction
Proposed Road Reservation Width	20.0m
Proposed Road Pavement Width	3.5m per traffic lane + 1.0m shoulder
Proposed Median Width	3.0m
Proposed Pedestrian / Cyclist / Shared Path Width	1.8 pedestrian path on the eastern side of the road reservation 2.5m shared path on the western side of the road reservation
Proposed Classification	Integrator B
Proposed Speed Limit	60kph
Proposed Bus Route Extension / Introduction	NO
If YES Nominate Bus Routes	-
Proposed On-street parking	NO

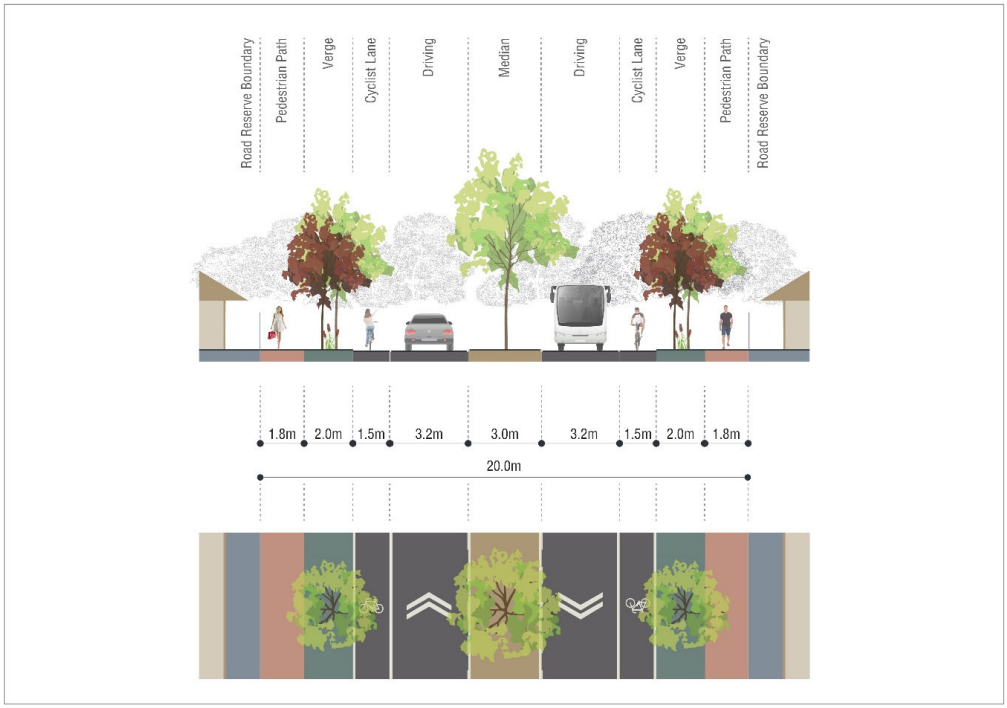
Provide graphics of the proposed internal road cross-section within the Structure Plan Area



Road 3

Name	Maida Vale Road
Projected Traffic Volumes	6,522 (2031) 13,072VPD (2050)
Proposed Number of Lanes	One lane per direction
Proposed Road Reservation Width	20.0m
Proposed Road Pavement Width	3.2m per traffic lane
Proposed Median Width	3.0 m
Proposed Pedestrian / Cyclist / Shared Path Width	1.8 pedestrian path on both sides of the road reservation 1.5m cycling lane on both sides of the road reservation
Equivalent LN Classification	Integrator B
Proposed Speed Limit	60kph
Proposed Bus Route Extension / Introduction	YES
If YES Nominate Bus Routes	It is expected that new Bus Routes will be introduced to/from the future High Wycombe Train Station.
Proposed On-street parking	NO

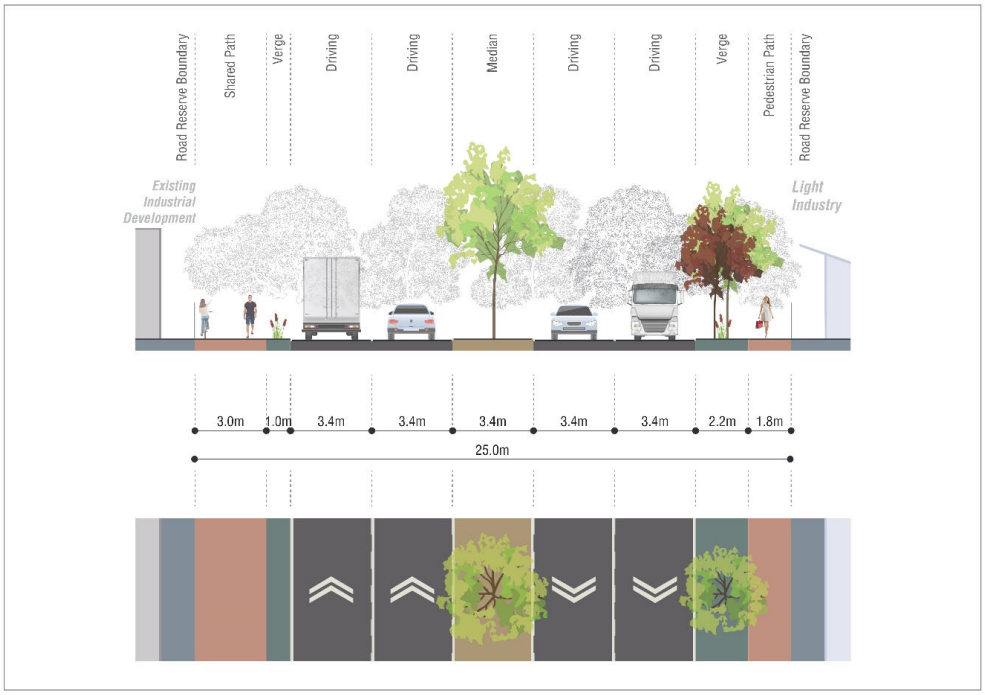
Provide graphics of the proposed internal road cross-section within the Structure Plan Area



Road 4

Name	Berkshire Road
Projected Traffic Volumes	15,659 (2031) 24,915 (2050)
Proposed Number of Lanes	Two lanes per direction
Proposed Road Reservation Width	25.0m
Proposed Road Pavement Width	3.4m per traffic lane
Proposed Median Width	3.4m
Proposed Pedestrian / Cyclist / Shared Path Width	1.8 pedestrian path on the northern side of the road reservation 3.0m shared path on the southern side of the road reservation
Proposed Classification	Integrator B
Proposed Speed Limit	70kph
Proposed Bus Route Extension / Introduction	YES
If YES Nominate Bus Routes	After completion of the High Wycombe Train Station, it is expected that new routes will be introduced connecting suburbs east of Roe Highway to the station via Berkshire Road.
Proposed On-street parking	NO

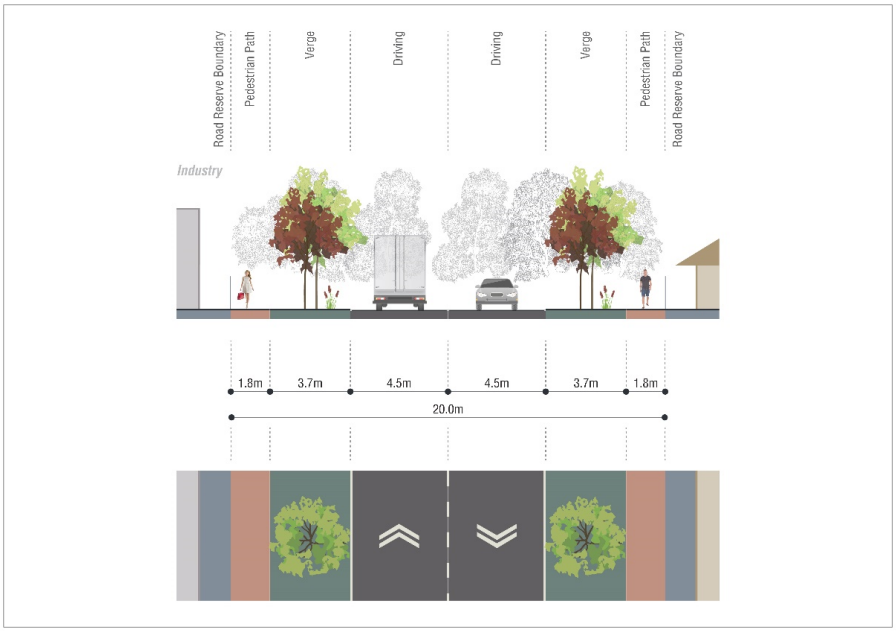
Provide graphics of the proposed internal road cross-section within the Structure Plan Area



Road 5a

Name	Sultana Road West (east of Milner Road)
Projected Traffic Volumes	786 (2031) 1,872 (2050)
Proposed Number of Lanes	One lane per direction
Proposed Road Reservation Width	20.0m
Proposed Road Pavement Width	4.5m per traffic lane
Proposed Median Width	no median
Proposed Pedestrian / Cyclist / Shared Path Width	1.8 pedestrian path on both sides of the road reservation
Equivalent LN Classification	Access Street
Proposed Speed Limit	50kph
Proposed Bus Route Extension / Introduction	NO
If YES Nominate Bus Routes	-
Proposed On-street parking	NO

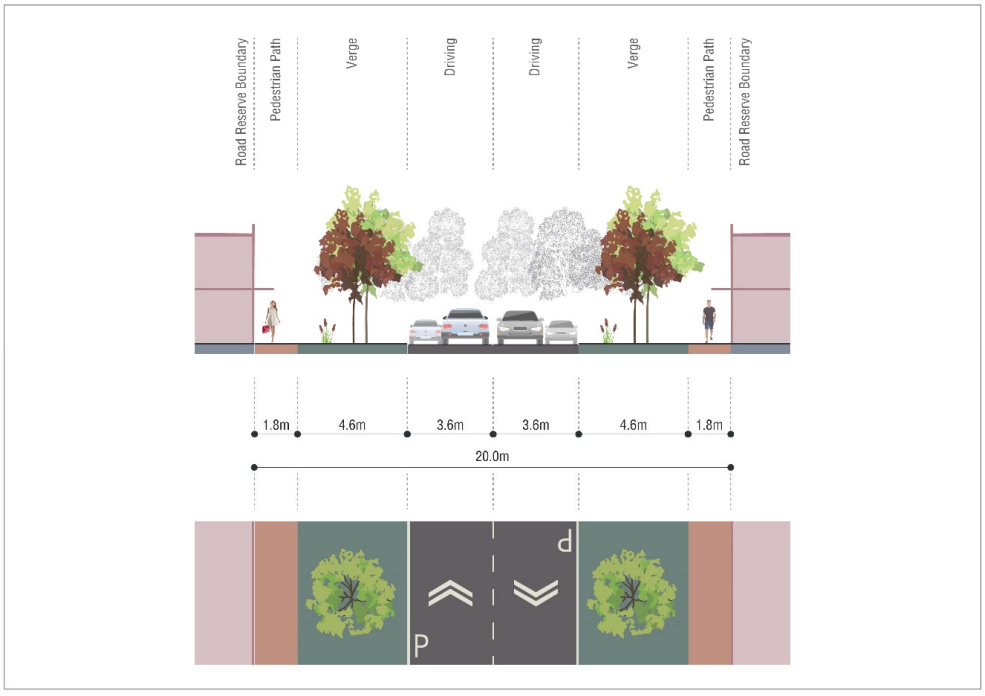
Provide graphics of the proposed internal road cross-section within the Structure Plan Area



Road 5b

Name	Sultana Road West (west of Milner Road)
Projected Traffic Volumes	498 (2031) 1,586 (2050)
Proposed Number of Lanes	One lane per direction
Proposed Road Reservation Width	20.0m
Proposed Road Pavement Width	3.6 per traffic lane
Proposed Median Width	no median
Proposed Pedestrian / Cyclist / Shared Path Width	1.8 pedestrian path on both sides of the road reservation
Proposed Classification	Access Street
Proposed Speed Limit	40kph
Proposed Bus Route Extension / Introduction	NO
If YES Nominate Bus Routes	-
Proposed On-street parking	YES – on street parking

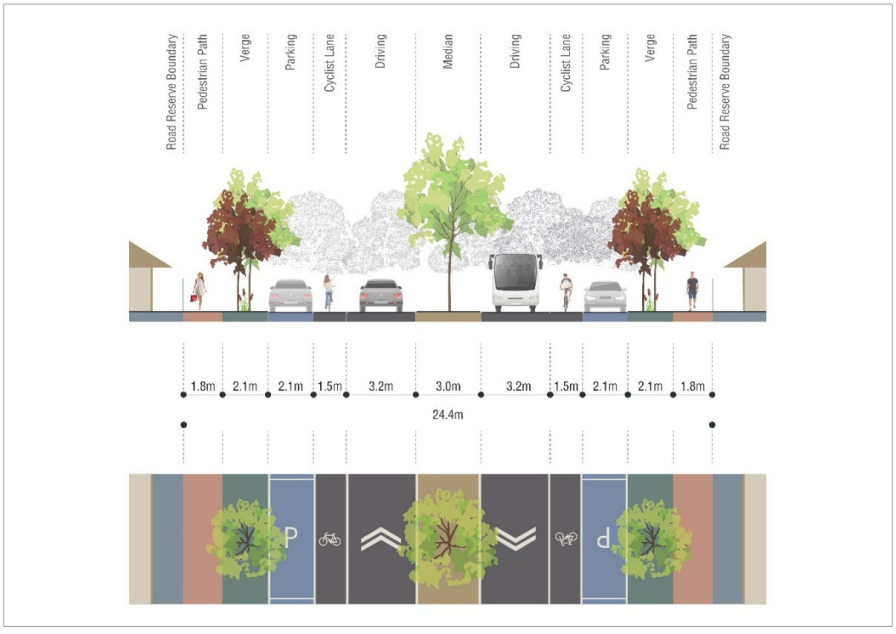
Provide graphics of the proposed internal road cross-section within the Structure Plan Area



Road 06

Name	Raven Street
Projected Traffic Volumes	1,428 (2031) 6,432 (2050)
Proposed Number of Lanes	One lane per direction
Proposed Road Reservation Width	24.4m
Proposed Road Pavement Width	3.2m per traffic lane
Proposed Median Width	3.0m
Proposed Pedestrian / Cyclist / Shared Path Width	1.8 pedestrian path on both sides of the road reservation 1.5m cycling lane on both sides of the road reservation
Equivalent LN Classification	Neighbourhood Connector A
Proposed Speed Limit	50kph
Proposed Bus Route Extension / Introduction	YES
If YES Nominate Bus Routes	It is expected that new Bus Routes will be introduced to/from the High Wycombe Train Station along with an Autonomous Bus Circular Route
Proposed On-street parking	YES (2.1 parallel parking on both sides)

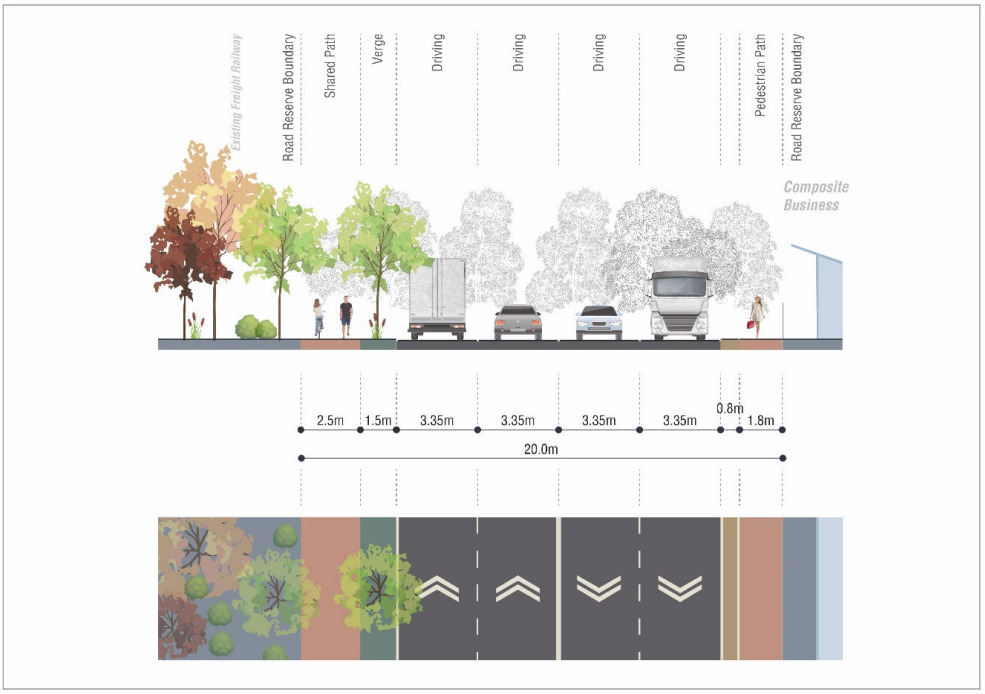
Provide graphics of the proposed internal road cross-section within the Structure Plan Area



Road 7a

Name	Dundas Road (North of Old Dundas Road (Maida Vale Road))
Projected Traffic Volumes	12,471 (2031) 19,964 (2050)
Proposed Number of Lanes	Two lanes per direction
Proposed Road Reservation Width	20.0m
Proposed Road Pavement Width	3.35m per traffic lane
Proposed Median Width	N/A
Proposed Pedestrian / Cyclist / Shared Path Width	1.8 pedestrian path on the eastern side of the road reservation 2.5m shared path on the western side of the road reservation
Proposed Classification	Integrator B
Proposed Speed Limit	60kph
Proposed Bus Route Extension / Introduction	NO
If YES Nominate Bus Routes	-
Proposed On-street parking	NO

Provide graphics of the proposed internal road cross-section within the Structure Plan Area

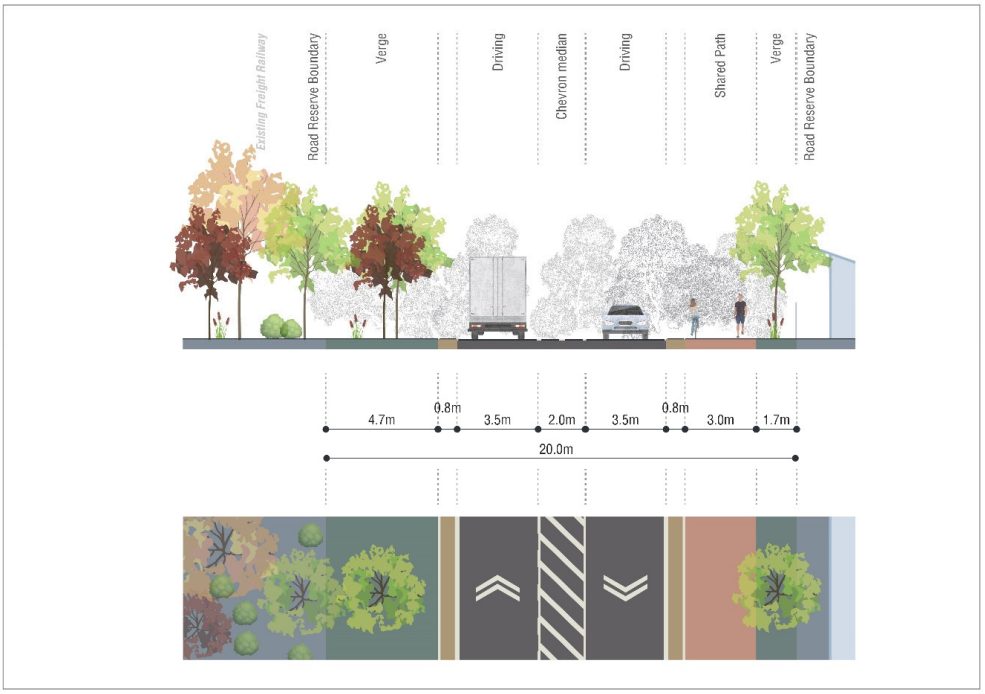


*Note: Cross-section above may be required in 2050; however, this cross-section should only be considered in approach to the final horizon year. The cross-section below (7b), should cater for traffic volumes for the duration of the ASP.

Road 7b

Name	Dundas Road (South of Old Dundas Road)
Projected Traffic Volumes	10,413(2031) 13,680 (2050)
Proposed Number of Lanes	One lane per direction
Proposed Road Reservation Width	20.0m
Proposed Road Pavement Width	3.5m per traffic lane
Proposed Median Width	2.0m painted median
Proposed Pedestrian / Cyclist / Shared Path Width	3.0m shared path on the eastern side of the road reservation
Proposed Classification	Integrator B
Proposed Speed Limit	60kph
Proposed Bus Route Extension / Introduction	NO
If YES Nominate Bus Routes	-
Proposed On-street parking	NO

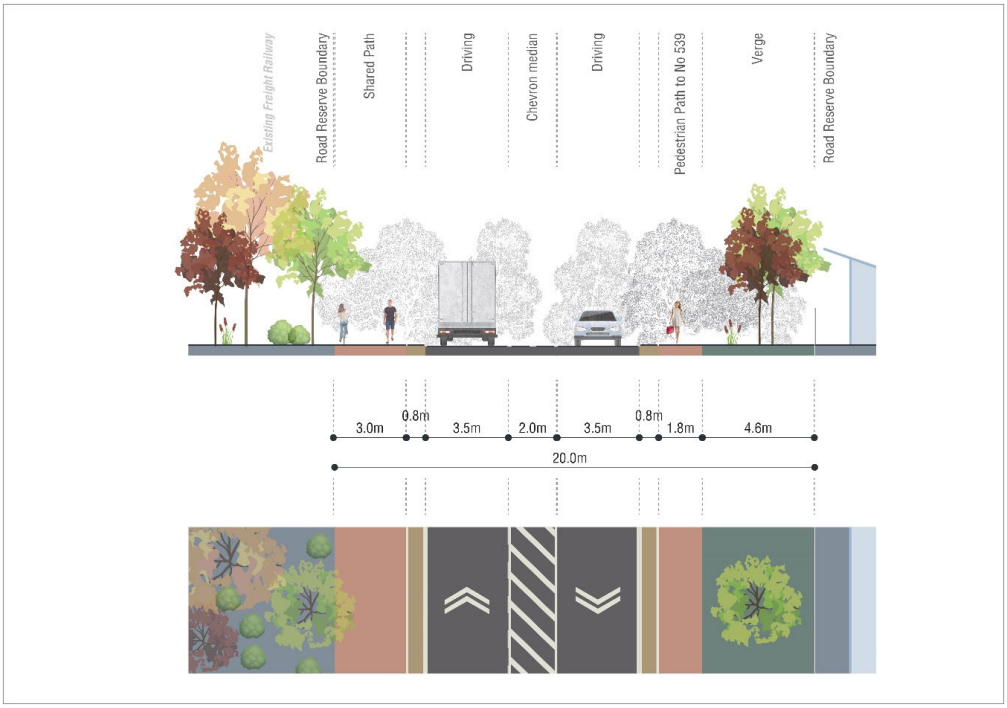
Provide graphics of the proposed internal road cross-section within the Structure Plan Area



Road 7c

Name	Dundas Road (South of Berkshire Road)
Projected Traffic Volumes	11,057(2031) 15,169 (2050)
Proposed Number of Lanes	One lane per direction
Proposed Road Reservation Width	20.0m
Proposed Road Pavement Width	3.5m per traffic lane
Proposed Median Width	2.0m painted median
Proposed Pedestrian / Cyclist / Shared Path Width	1.8m pedestrian path on the eastern side of the road reservation 3.0m shared path on the western side of the road reservation
Proposed Classification	Integrator B
Proposed Speed Limit	60kph
Proposed Bus Route Extension / Introduction	NO
If YES Nominate Bus Routes	-
Proposed On-street parking	NO

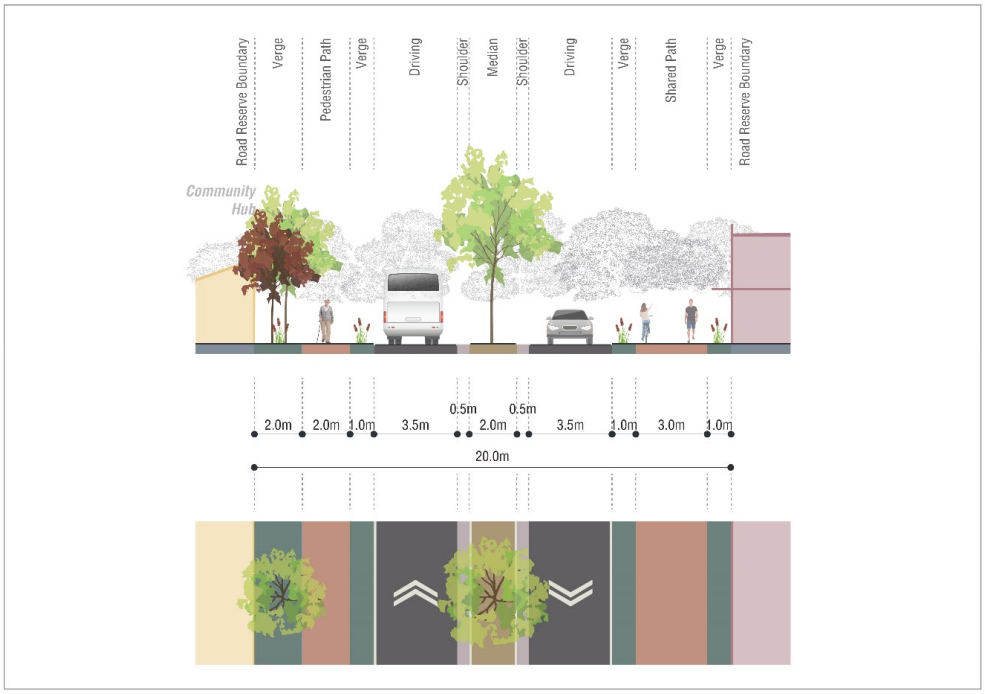
Provide graphics of the proposed internal road cross-section within the Structure Plan Area



Road 7

Name	Enterprise Boulevard (Ibis Place)
Projected Traffic Volumes	1,817 (2031) 3,631 (2050)
Proposed Number of Lanes	One lane per direction
Proposed Road Reservation Width	20.0m
Proposed Road Pavement Width	3.5m per traffic lane + 0.5m shoulder
Proposed Median Width	2.0m
Proposed Pedestrian / Cyclist / Shared Path Width	1.8m pedestrian path on the eastern side of the road reservation 3.0m shared path on the western side of the road reservation
Proposed Classification	Neighbourhood Connector A
Proposed Speed Limit	50kph
Proposed Bus Route Extension / Introduction	YES
If YES Nominate Bus Routes	New bus routes will be introduced to/from the High Wycombe Train Station
Proposed On-street parking	NO

Provide graphics of the proposed internal road cross-section within the Structure Plan Area



2.20. Proposed Intersection Controls

Name the intersection control of the key proposed intersections:

Intersection 1	
Name	Dundas Road / Berkshire Road / Milner Road
Proposed Intersection Control	Roundabout
Intersection 2	
Name	Milner Road / Stewart Road
Proposed Intersection Control	Roundabout
Intersection 3	
Name	Milner Road / Raven Street
Proposed Intersection Control	Roundabout
Intersection 4	
Name	Milner Road / TOD Connector
Proposed Intersection Control	Signalised
Intersection 5	
Name	Milner Road / Sultana Road West
Proposed Intersection Control	T-intersection LILORI (east of Milner Road) Sultana Road West to be converted to cul-de-sac west of Milner Road.
Intersection 6	
Name	Maida Vale Road / Old Dundas Road (North of MVR)
Proposed Intersection Control	Roundabout
Intersection 7	
Name	Dundas Road / Old Dundas Road (North of MVR)
Proposed Intersection Control	Signalised Intersection

All other intersections are to be sign or priority controlled.

2.21. Proposed Internal Transport Networks

Are there any changes/additions to the existing road network?	YES Please refer to Appendices 1 and 2 for the road layout and network configuration. Most notable changes: • TOD Connector construction
Were there any discussions/agreements with MRWA regarding intersections with, or direct access onto roads under their jurisdiction?	N/A
Are there any pedestrian/cycle networks and crossing facilities proposed for the roads within the Structure Plan Area?	YES Shared Paths are proposed for:

	• Milner Road (Sultana Road West-Berkshire Road) • Sultana Road West • Enterprise Boulevard (Ibis Place) Separate Pedestrian Paths are proposed for: • TOD Connector • Milner Road (north of Sultana Road West) • Stewart Road • Raven Street • Sultana Road West • Enterprise Boulevard • Urban Residential Streets Separate Cycle Lanes are proposed for: • Milner Road (north of Sultana Road West) • TOD Connector • Stewart Road • Raven Street There will be a cyclist and pedestrian connection to Littlefield Boulevard over Poison Gully Creek.
Were there any discussions/agreements with the local authority over local road networks and pedestrian and cycle facilities?	YES – all internal road network cross-sections have been developed with input from the City of Kalamunda
Were there any discussions/agreements with PTA / Transperth on new bus services or extensions/alterations to existing bus services to serve the Structure Plan Area?	YES - New routes are expected to be introduced after the completion of High Wycombe Train Station via Maida Vale Road, Berkshire Road and Milner Road to provide surrounding suburbs with a connection to the station. There is an option to consider a local route via Raven Street and TOD Connector.

2.22. Changes to External Transport Networks

Are there any proposed changes of the road network?	YES • In Perth and Peel Transport Plan @3.5million, it is stated that by 2050 Roe Highway will be upgraded to 8-lanes freeway standard. By 2031 Roe Highway is expected to be upgraded to 6 lanes freeway standard. • Realignment of Dundas Road • Grade separation of Kalamunda Road / Roe Highway intersection • Upgrade of the Roe Highway / Maida Vale Road interchange. All of these changes are proposed as a result of a long-term planning outlook. The proposed ASCP does not induce the requirement for these changes, although it will benefit from them,
Are there any proposed changes to the intersection controls?	• Roundabout at the realigned intersection of Dundas Road / Berkshire Road / Milner Road
Are there any proposed changes to the pedestrian/cycle networks and crossing facilities?	Major changes to the pedestrian and cycling network are mostly internal to the ASCP. Dundas Road features a high level shared path, while Maida Vale Road is designated as one of the key links in the City of Kalamunda's cycling strategy.

Are there any proposed changes to the public transport services?	Significant changes are proposed within this ACSP, leading to increases in residential density and improvements in transport infrastructure. Since the entire surrounding area is about to go through significant changes, the public transport will be improved. Notwithstanding the new rail line, additional bus "feeder" routes are anticipated in this area. At this stage of planning, buses will operate along Berkshire Road, Dundas Road, and Maida Vale Road.
Proposed innovative transport initiatives	The option for a driverless shuttle system should be considered. This shuttle could provide a direct connection between the proposed railway station and the residential precinct. This system could be done with larger vehicles or alternatively, with a pool of smaller vehicles stored and operated locally (either as part of the community service or as part of the railway station complex)

2.23. Integration with Surrounding Area

Are there any existing major residential generators of traffic within a minimum of 800 metres from the boundaries of the Structure Plan Area?	YES
If YES, nominate:	High Wycombe to the north;
Are there any existing major non-residential attractors of traffic within a minimum of 800 metres from the boundaries of the Structure Plan Area?	YES
If YES, nominate:	• Existing Industrial Zone south of Berkshire Road • Although not within an 800m radius, the Perth Airport as a regional attractor is located approximately 2km to the west of the structure plan; however, it is one railway stop from the subject site. • Future High Wycombe Train Station
What are the main desire lines between the structure plan land uses and these external attractors/generators?	Via Milner Road south of Sultana Road West, Dundas Road, Abernethy Road, Maida Vale Road and Berkshire Road.
Will the existing transport networks, plus any proposed changes, adequately match these desire lines, particularly for pedestrians, cyclists and public transport users?	YES The entire area will be well connected to the existing and future surrounding major attractors/generators.
Identify any deficiencies or areas for improvement in the surrounding transport networks and/or areas where improvements could be made.	N/A The area is well planned; when all infrastructure is completed, it is expected that the network will function without major issues.

2.24. Analysis of Transport Networks

Determine the year(s) for assessment and the time period(s) for the traffic flow analysis.	2031 (with approximately 15% of Forrestfield North Project Area constructed and operational) 2041 (with approximately 65% of Forrestfield North Project Area constructed and operational) 2050+ (as the full buildout)
Determine structure plan generated traffic.	17,185 VPD The modelling includes generated traffic from all Forrestfield North precincts. Refer to Appendix 4 for more details on traffic modelling.

Structure Plan Impact on Internal and Surrounding External Road Network

Road (Link)	Section	Total Daily Traffic (2031)	External Daily Traffic (2031)	FFN All Precincts Traffic (2031)	Total Daily Traffic (2050)	External Daily Traffic (2050)	FFN All Precincts Traffic (2050
Roe Highway	South of Berkshire Road	120,028	119,171	295	158,183	153,824	1,063
			99.29%	0.25%		97.24%	0.67%
	Between Berkshire Rd and Maida Vale Rd	113,361	113,310	7	140,469	140,182	54
			99.96%	0.01%		99.80%	0.04%
	North of Maida Vale Road	101,905	101,898	7	125,922	125,868	54
			99.99%	0.01%		99.96%	0.04%
	North of Kalamunda Road	108,552	108,026	43	139,007	136256	208
			99.52%	0.04%		98.02%	0.15%
Berkshire Road	West of Roe Hwy	16,404	14,422	1,301	25,843	18,302	3,524
			87.92%	7.93%		70.82%	13.64%
	East of Roe Hwy	17,883	16,721	998	31,805	28,444	2,407
			93.50%	5.58%		89.43%	7.57%
	South of Milner Rd	15,659	13,606	1,308	24,915	17301	3,533
			86.89%	8.35%		69.44%	14.18%
Dundas Road	South of Berkshire Road	11,057	10,443	287	15,169	13,236	757
			94.45%	2.60%		87.26%	4.99%
	South-west from Old Dundas Road alignment	10,413	9,600	813	13,680	12,306	1,374
			92.19%	7.81%		89.96%	10.04%
	North of Old Dundas Road alignment	12,471	10530	1,342	19,964	13407	3,512
			84.44%	10.76%		67.16%	17.59%
Maida Vale Road	East of Ibis Place	3,308	930	1,784	7,620	1,101	3,530
			28.11%	53.93%		14.45%	46.33%

	East of Raven Street	2,856	1,154	1,702	4,422	1,231	3,159
			40.41%	59.59%		27.84%	71.44%
	East of Milner Rd	6,522	4114	1,619	13,072	4178	4,002
			63.08%	24.82%		31.96%	30.62%
Milner Road	South of Maida Vale Rd	3,914	2,864.00	261	9,288	2,747	1,649
			73.17%	6.67%		29.58%	17.75%
	South of TOD Connector	4,045	3,001	162	8,110	2,820	920
			74.19%	4.00%		34.77%	11.34%
	North of Berkshire Rd	6,947	5217	663	12,928	5073	2,426
			75.10%	9.54%		39.24%	18.77%
Enterprise Boulevard	West of Maida Vale Rd	1,817	0	1,817	3,631	0	3,631
			0.00%	100.00%		0.00%	100.00%
	North of TOD Connector	663	0	663	1,924	0	1,924
			0.00%	100.00%		0.00%	100.00%
Raven Street	South of Maida Vale Rd	1,410	456	360	5,602	396	2,132
			32.34%	25.53%		7.07%	38.06%
	South of Milner Rd	1,015	0	85	5,984	0	862
			0.00%	6.03%		0.00%	14.41%
Stewart Road	East of Milner Rd	791	0	63	4,739	0	498
			0.00%	7.96%		0.00%	10.51%
TOD Connector	East of Milner Rd	1,428	319	481	6,432	323	1,961
			22.34%	33.68%		5.02%	30.49%
	South of Ibis Pl	695	0	695	2,322	0	2,322
			0.00%	100.00%		0.00%	100.00%
	South of Brae Rd	733	0	85	4,520	0	811
			0.00%	11.60%		0.00%	17.94%
Sultana Road West	West of Milner Rd	498	0	498	1,586	0	1,586
			0.00%	100.00%		0.00%	100.00%
	East of Milner Rd	786	564	38	1,872	554	243
			71.76%	4.83%		29.59%	12.98%
	West of Brae Street	452	217	53	1,664	180	429
			48.01%	11.73%		10.82%	25.78%
Newburn Road	North of Maida Vale Road	4,131	3534	114	6,568	3593	408
			85.55%	2.76%		54.70%	6.21%
Brae Road	East of TOD Connector	439	0	80	2,599	0	680
			0.00%	18.22%		0.00%	26.16%
Brand Road	North of TOD Connector	247	0	34	2,011	0	291
			0.00%	13.77%		0.00%	14.47%

Identify all schools within the structure plan area and those within 800 metres of the structure plan area.	Only two schools within 800m of the structure plans are the proposed primary schools in Forrestfield North Residential LSP and Edney Primary
Identify the most likely walk and cycle routes to each school from the catchment areas.	The main walk and cycle routes will include the pedestrian and shared paths and cycle lanes on Maida Vale Road to the Maida Vale South School. TOD Connector, Raven Street and Stewart Road will provide connection to Brae Road and Brand Road to the new Forrestfield North Primary School and the District Open Space.

Appendix 1

The Layout of the Proposed Development

LEGEND

Activity Centre Plan Boundary

EXISTING PLANNING FRAMEWORK

Metronet East Redevelopment Area

Project Area: Forrestfield North (RAC-0)

Ecological Conservation (Conservation Category Wetland)

Bush Forever

PROPOSED PLANNING FRAMEWORK

Region Scheme Reserves

Railway

Local Scheme Reserves and Environmental Conservation

Local Open Space

Public Purposes: Community Purpose

Environmental Conservation

Other Categories

District Integrator A (Existing / Proposed)

District Integrator B (Existing / Proposed)

Neighbourhood Connector (Existing / Proposed)

Local Street (Existing / Proposed)

Proposed Roundabout

Proposed Signalised Intersection

A Cul de Sac Head

B No Right Out Intersection

Strategic Site

Hazard seperation required at subsequent planning stages to ensure that no residential land is affected by BAL-40 or BAL-FZ

Indicative Drainage Corridor

Notes

1 Final road widths and indicative cross sections to be determined following a traffic assessment

Precincts

1 Forrestfield Station Precinct

2 Residential Precinct

Sub-Precincts

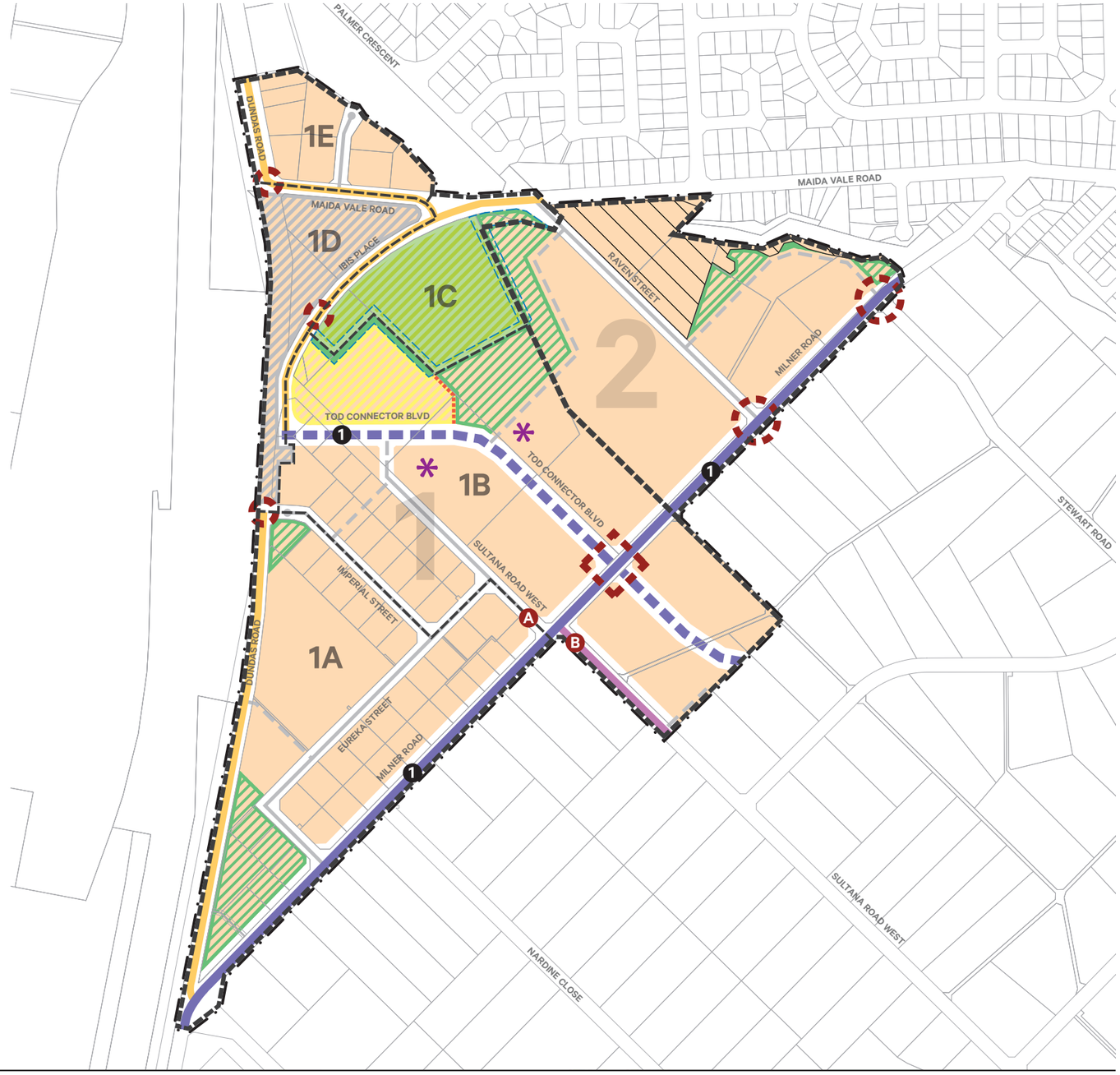
1A Eureka Sub-Precinct

1B Core Sub-Precinct

1C Conservation Sub-Precinct

1D Station Sub-Precinct

1E Maida Vale Sub-Precinct



Activity Centre Plan

Forrestfield North TOD Precinct

DRAFT

Date: 6 Jul 2021 Scale: 1:6000@ A3 1:3000 @ A1 File: **17-527 ST-1** **A** Staff: MC MS GW Checked: MS

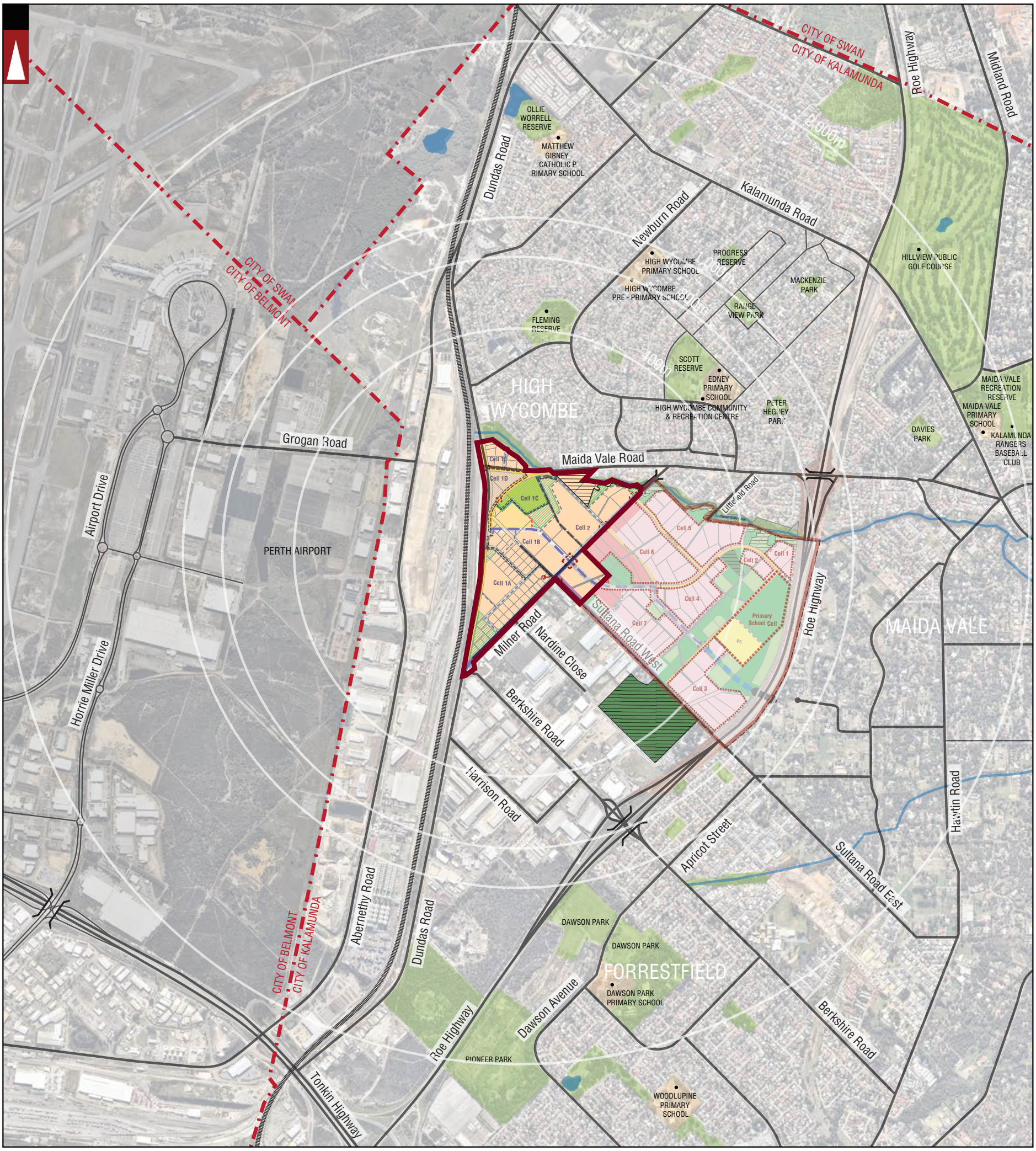
0 60 120m

element.

Level 18, 191 St Georges Terrace, Perth Western Australia 6000.
PO Box 7375 Cloisters Square, Perth Western Australia 6850.
T. +61 8 9289 8300 | **E.** hello@elementwa.com.au **elementwa.com.au**

Appendix 2

Transport Planning and Traffic Plans



No	DATE	AMENDMENT
B	26-07-2021	INFORMATION UPDATED
A	18-10-2018	ISSUED FOR REVIEW

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: LOCALITY PLAN - 2,000M RADIUS

DRAWING NUMBER: KC00604.000_ S01

DRAWN BY: J.S.

Civil & Traffic Engineering Consultants
Suite 7 No 10 Whipple Street Balcatta WA 6021
PH: 08 9441 2700
WEB: www.kctt.com.au
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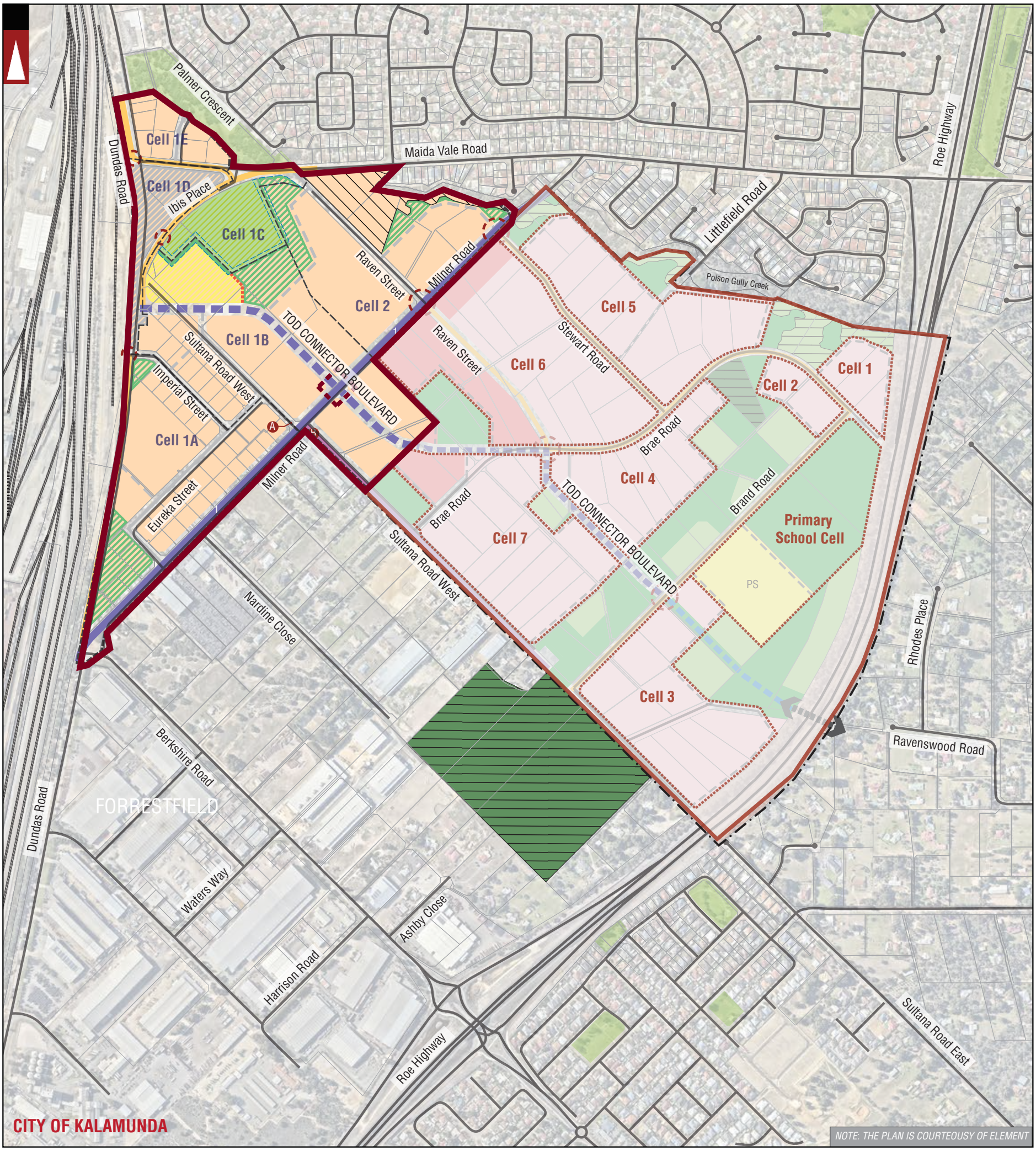

ROAD

Hay Street STREET NAME

RAILWAY

ROAD BRIDGE

TOD PRECINCT BOUNDARY

RESIDENTIAL PRECINCT BOUNDARY

CITY OF KALAMUNDA LOCAL GOVERNMENT NAME

LOCAL AUTHORITY BOUNDARY

LEGEND

No	DATE	AMENDMENT
C	30-08-2021	PROPOSED LAYOUT UPDATED
B	27-07-2021	INFORMATION UPDATED
A	01-11-2018	ISSUED FOR REVIEW

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: LOCALITY PLAN - ZOOM IN

DRAWING NUMBER: KC00604.000_ S01a

DRAWN BY: J.S.

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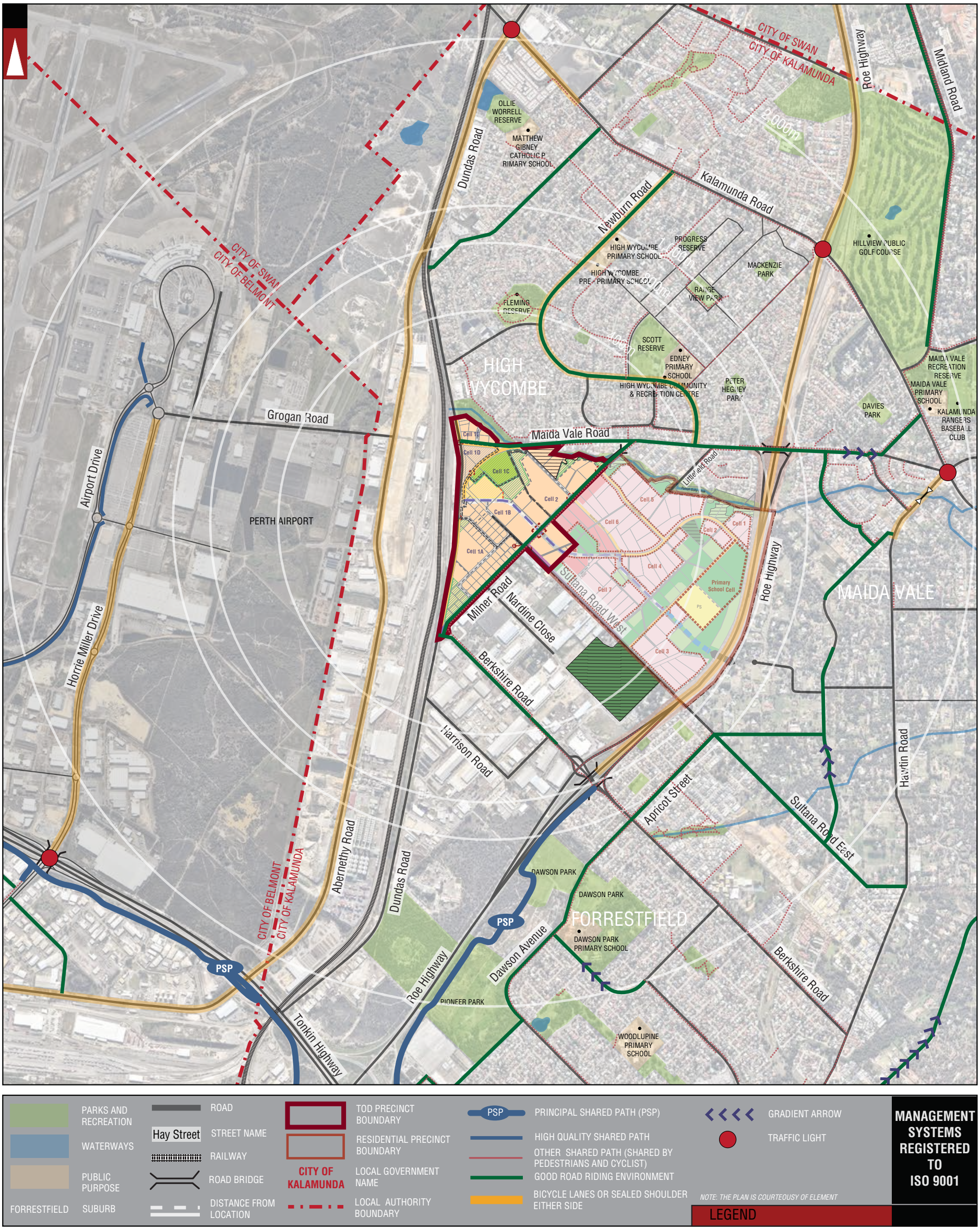

No	DATE	AMENDMENT
B	26-07-2021	INFORMATION UPDATED
A	18-10-2018	ISSUED FOR REVIEW

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: BICYCLE NETWORK PLAN - 2,000M RADIUS

DRAWING NUMBER: KC00604.000_S02

DRAWN BY: J.S.

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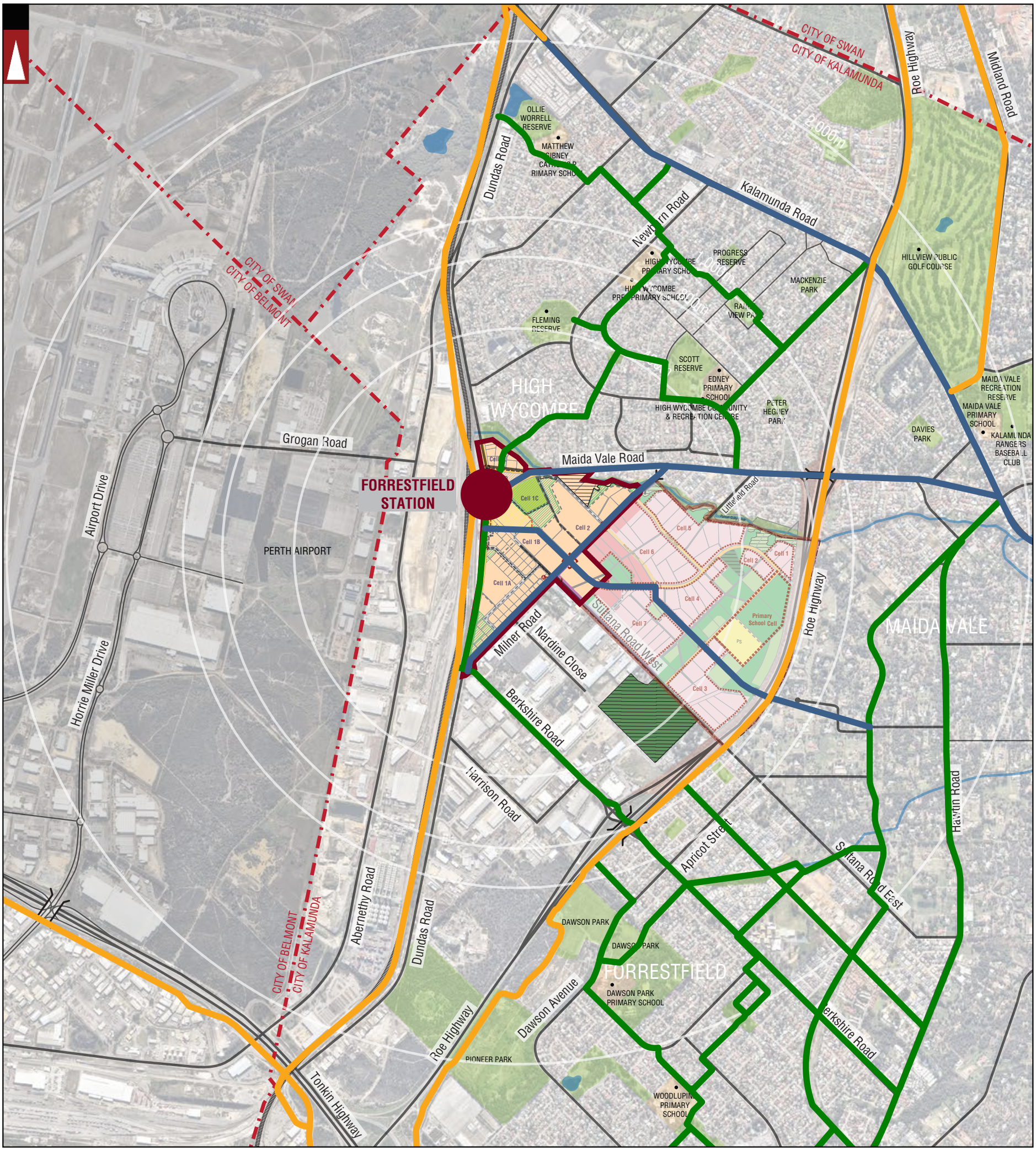

No	DATE	AMENDMENT
B	26-07-2021	INFORMATION UPDATED
A	18-10-2018	ISSUED FOR REVIEW

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: CITY OF KALAMUNDA - ULTIMATE CYCLING NETWORK - 2,000M RADIUS

DRAWING NUMBER: KC00604.000_ S02a

DRAWN BY: J.S.

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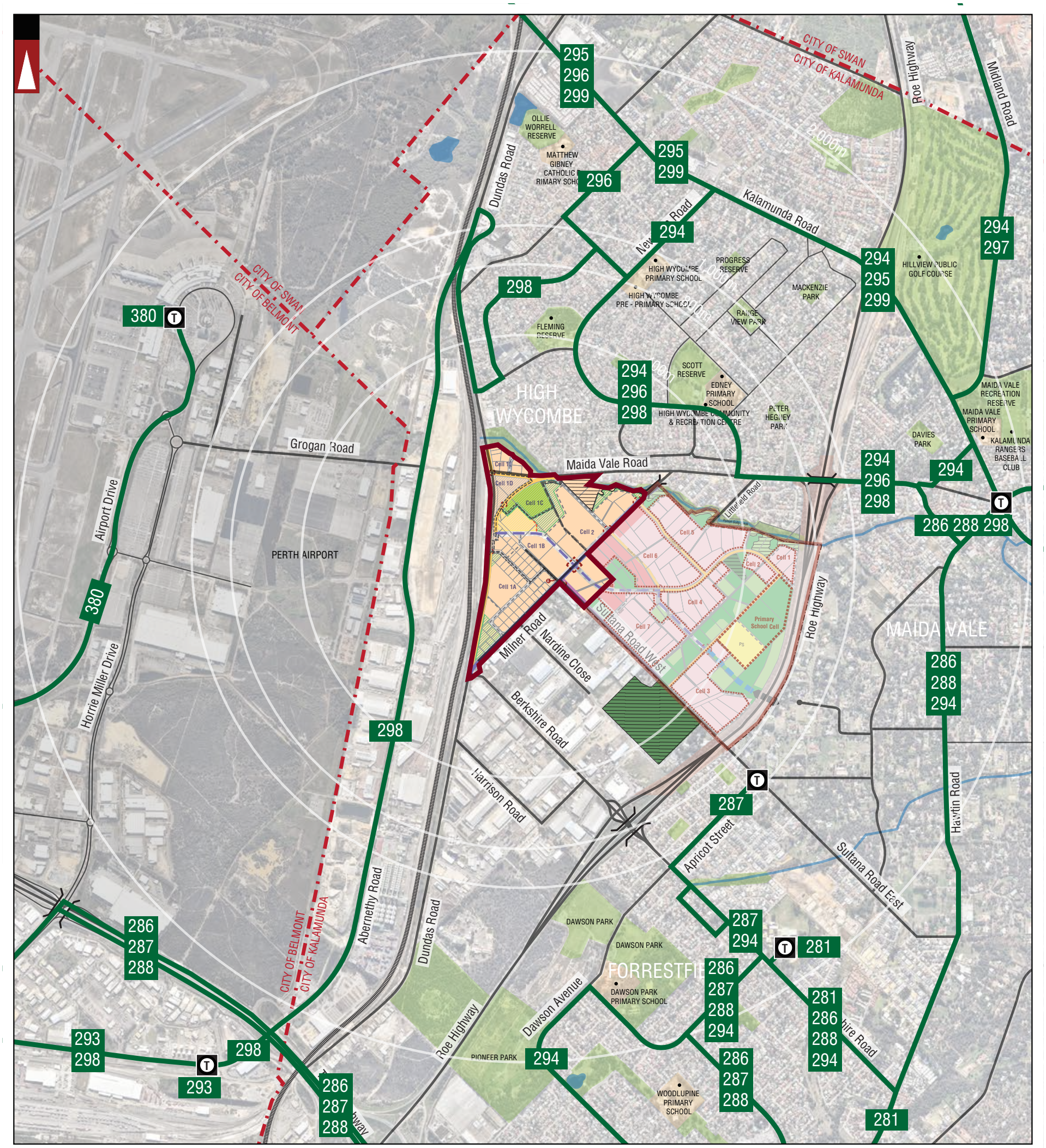

No	DATE	AMENDMENT
B	26-07-2021	INFORMATION UPDATED
A	18-10-2018	ISSUED FOR REVIEW

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: PUBLIC TRANSPORT PLAN - 2,000M RADIUS

DRAWING NUMBER: KC00604.000_ S03

DRAWN BY: J.S.

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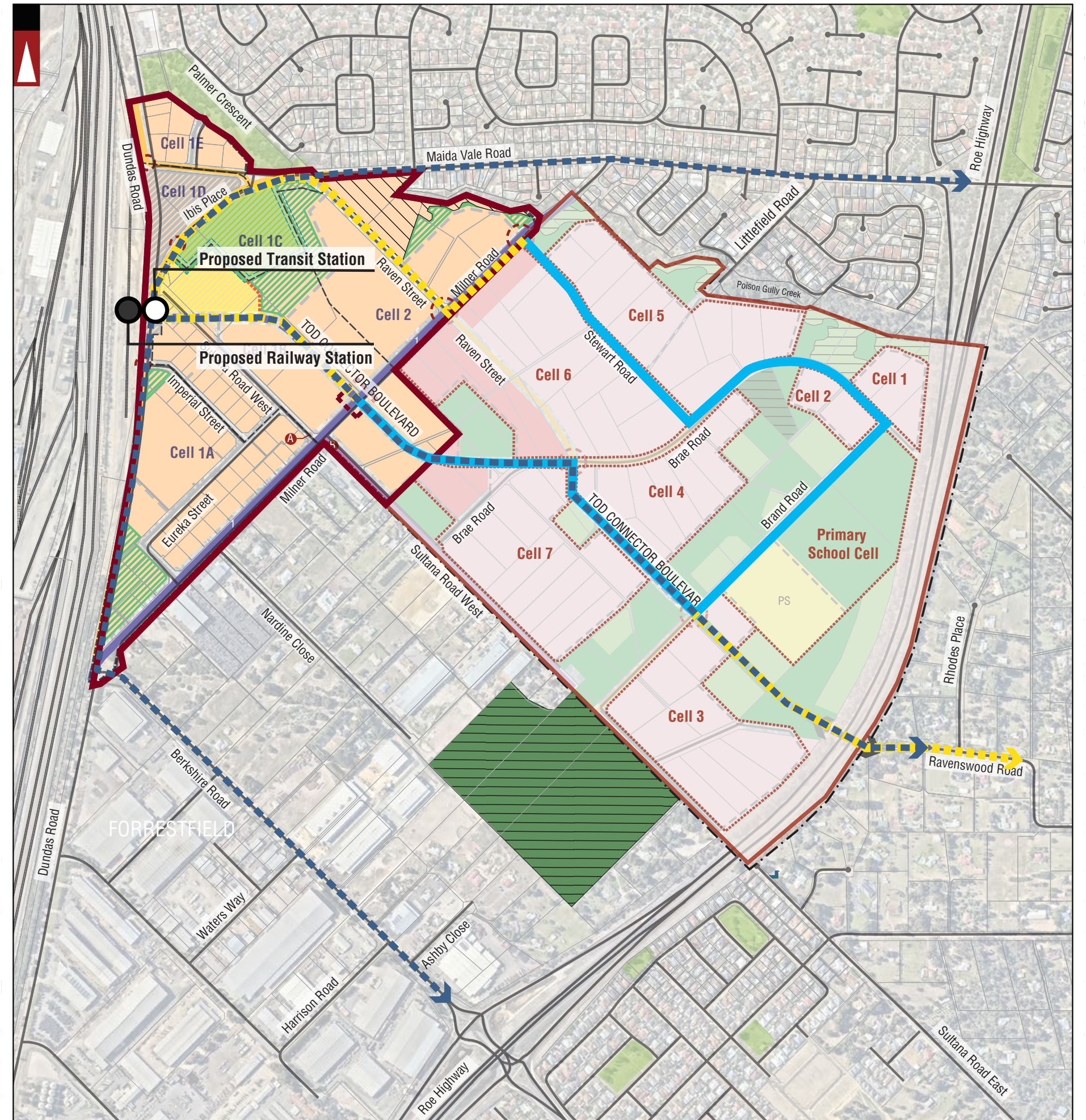

ROAD
Hay Street STREET NAME
RAILWAY
ROAD BRIDGE
TOD PRECINCT BOUNDARY
RESIDENTIAL PRECINCT BOUNDARY
CITY OF KALAMUNDA LOCAL GOVERNMENT NAME
LOCAL AUTHORITY BOUNDARY
POTENTIAL ALIGNMENT OF BUS ROUTES
PROPOSED AUTONOMOUS BUS CIRCULAR ROUTE
POTENTIAL EXTERNAL ALIGNMENT OF AUTONOMOUS BUS CIRCULAR ROUTE

LEGEND

Certified System Quality ISO 9001 SAI GLOBAL

No	DATE	AMENDMENT
C	30-08-2021	PROPOSED LAYOUT UPDATED
B	27-07-2021	INFORMATION UPDATED
A	01-11-2018	ISSUED FOR REVIEW

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: PUBLIC TRANSPORT PLAN - PROPOSED

DRAWING NUMBER: KC00604.000_ S03a

DRAWN BY: J.S.

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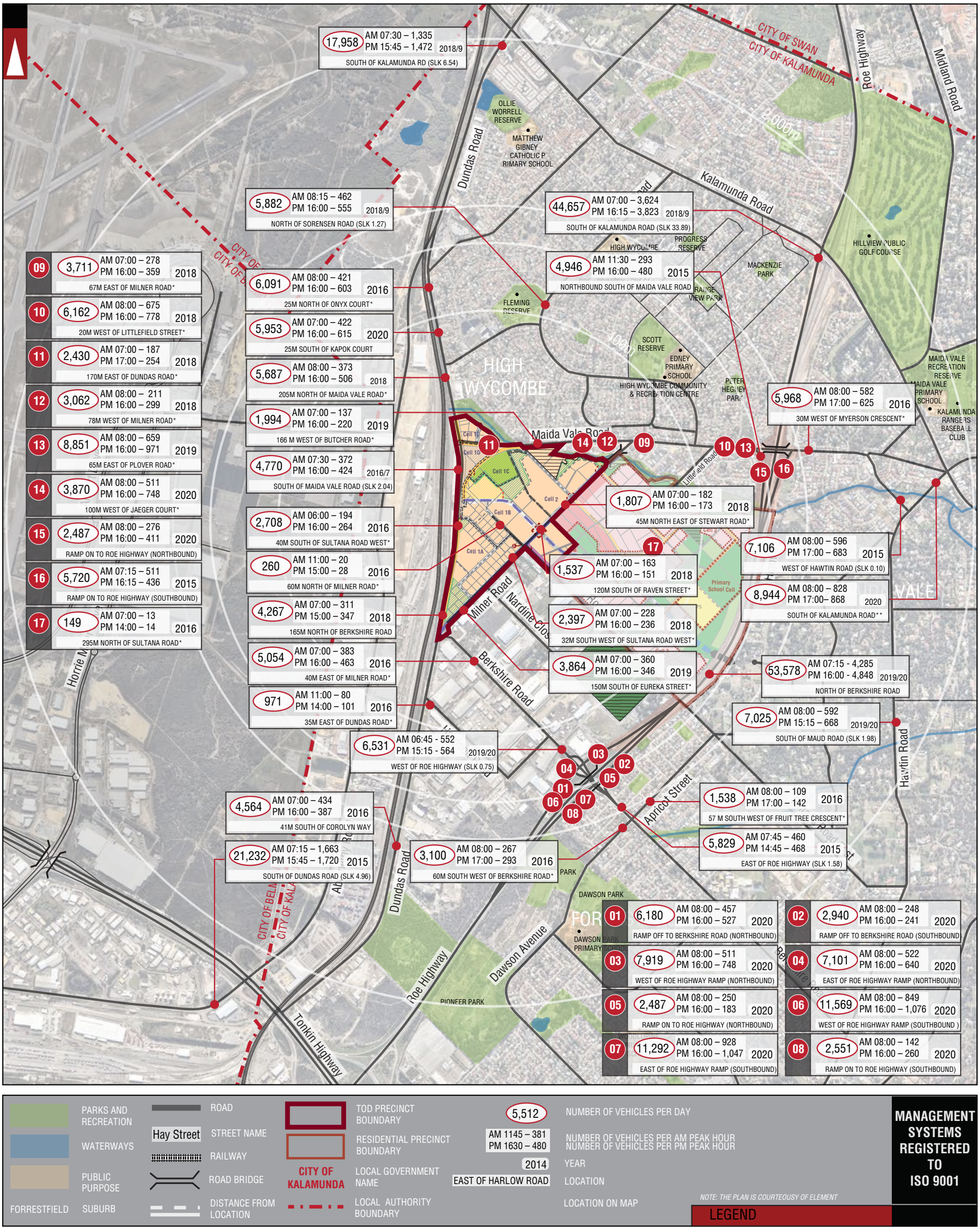

No	DATE	AMENDMENT
B	26-07-2021	INFORMATION UPDATED
A	18-10-2018	ISSUED FOR REVIEW

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: EXISTING TRAFFIC COUNTS - 2,000M RADIUS

DRAWING NUMBER: KC00604.000_ S05

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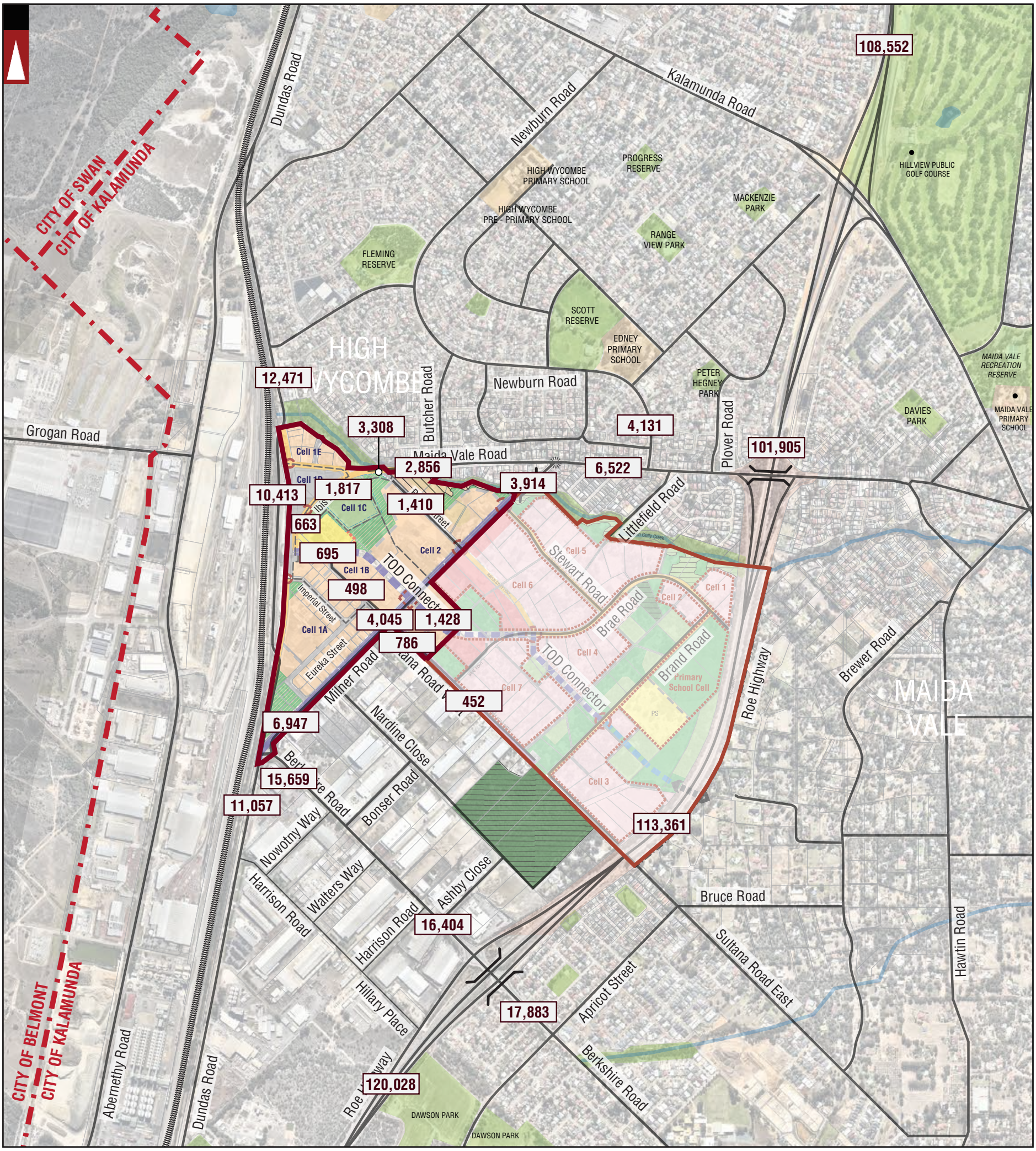

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B	26-07-2021	INFORMATION UPDATED
A	18-10-2018	ISSUED FOR REVIEW
No	DATE	AMENDMENT

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: TOTAL DAILY ANTICIPATED TRAFFIC - 2031

DRAWING NUMBER: KC00604.000_ S06A

DRAWN BY: J.S.

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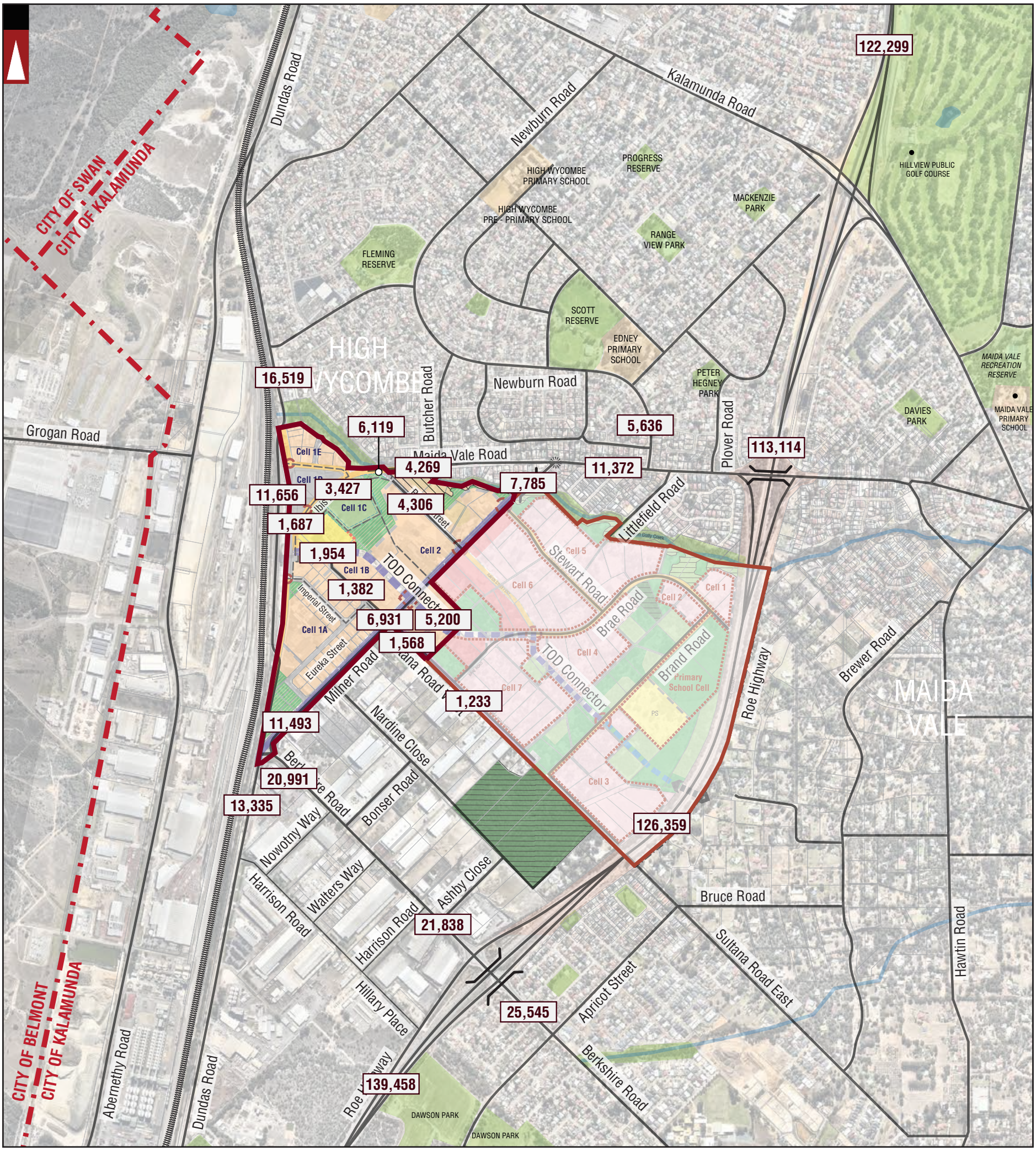

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B	26-07-2021	INFORMATION UPDATED
A	18-10-2018	ISSUED FOR REVIEW
No	DATE	AMENDMENT

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: TOTAL DAILY ANTICIPATED TRAFFIC - 2041

DRAWING NUMBER: KC00604.000_ S06B

DRAWN BY: J.S.

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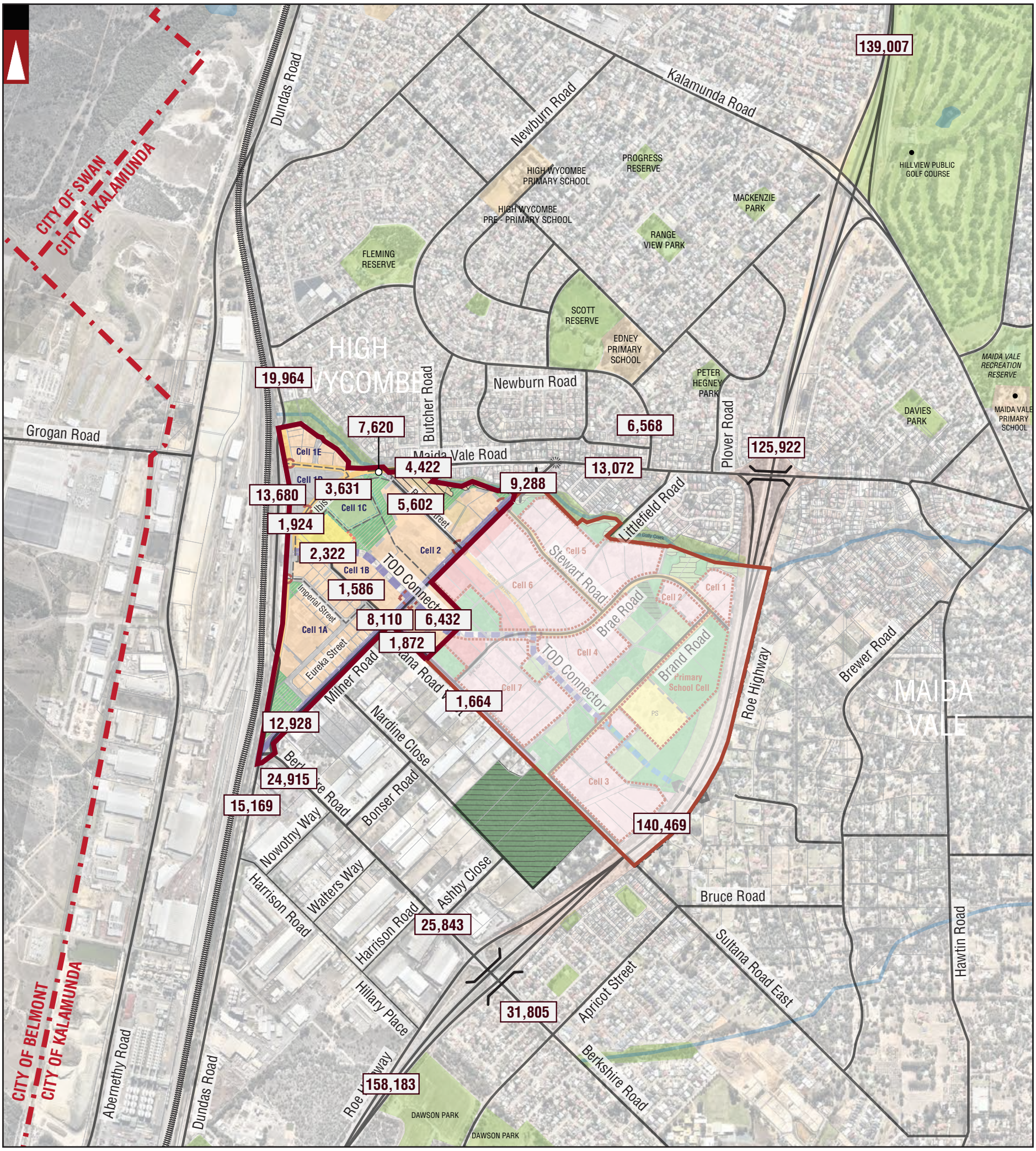

C	31-08-2021	PROPOSED LAYOUT AMENDED
B	26-07-2021	INFORMATION UPDATED
A	18-10-2018	ISSUED FOR REVIEW
No	DATE	AMENDMENT

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: TOTAL DAILY ANTICIPATED TRAFFIC - 2050+

DRAWING NUMBER: KC00604.000_ S06C

DRAWN BY: J.S.

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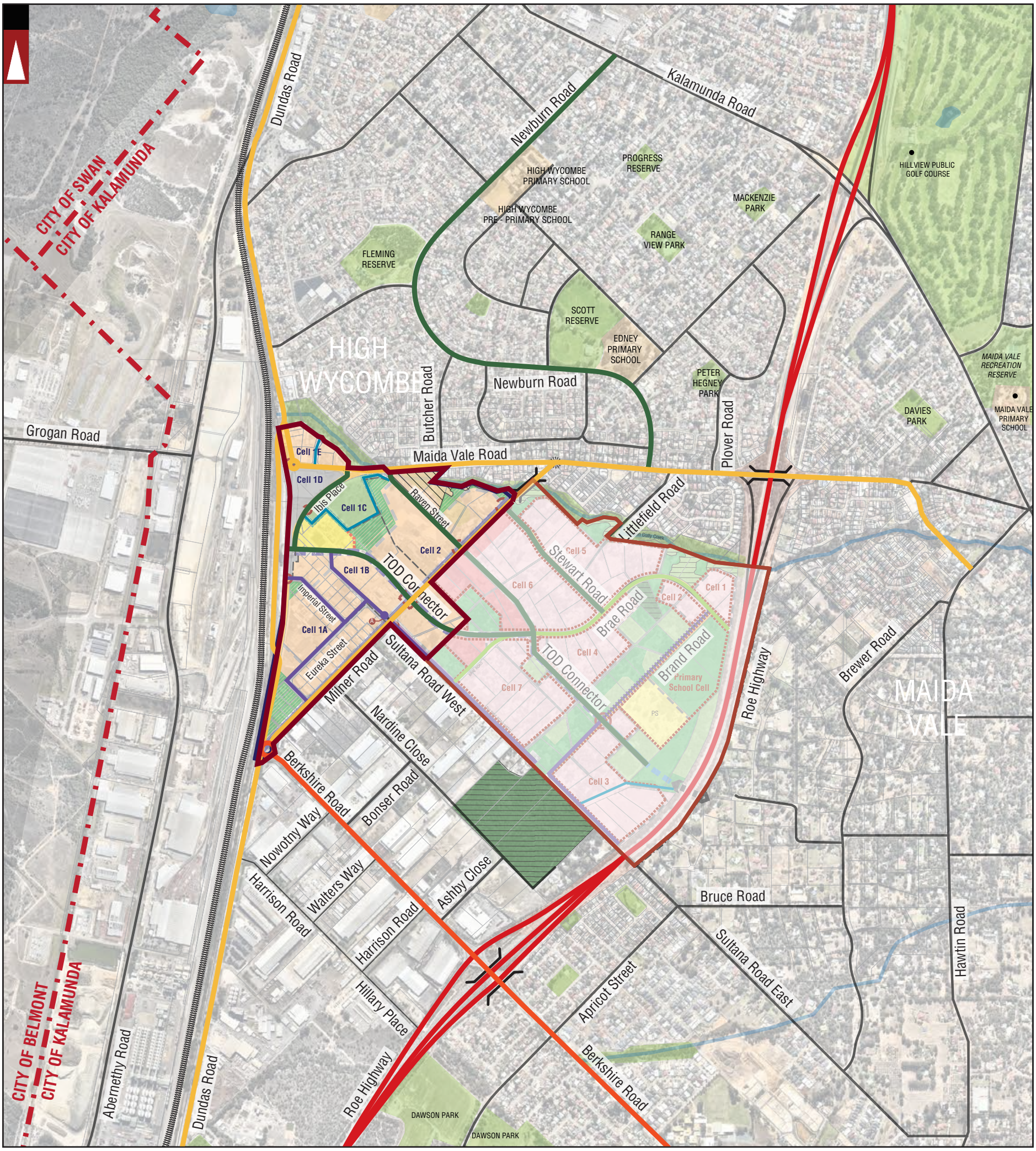

No	DATE	AMENDMENT
C	31-08-2021	PROPOSED LAYOUT AMENDED
B	26-07-2021	INFORMATION UPDATED
A	18-10-2018	ISSUED FOR REVIEW

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: HIERARCHY PLAN

DRAWING NUMBER: KC00604.000_ S10

DRAWN BY: J.S.

Civil & Traffic Engineering Consultants
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PH: 08 9441 2700
WEB: www.kctt.com.au
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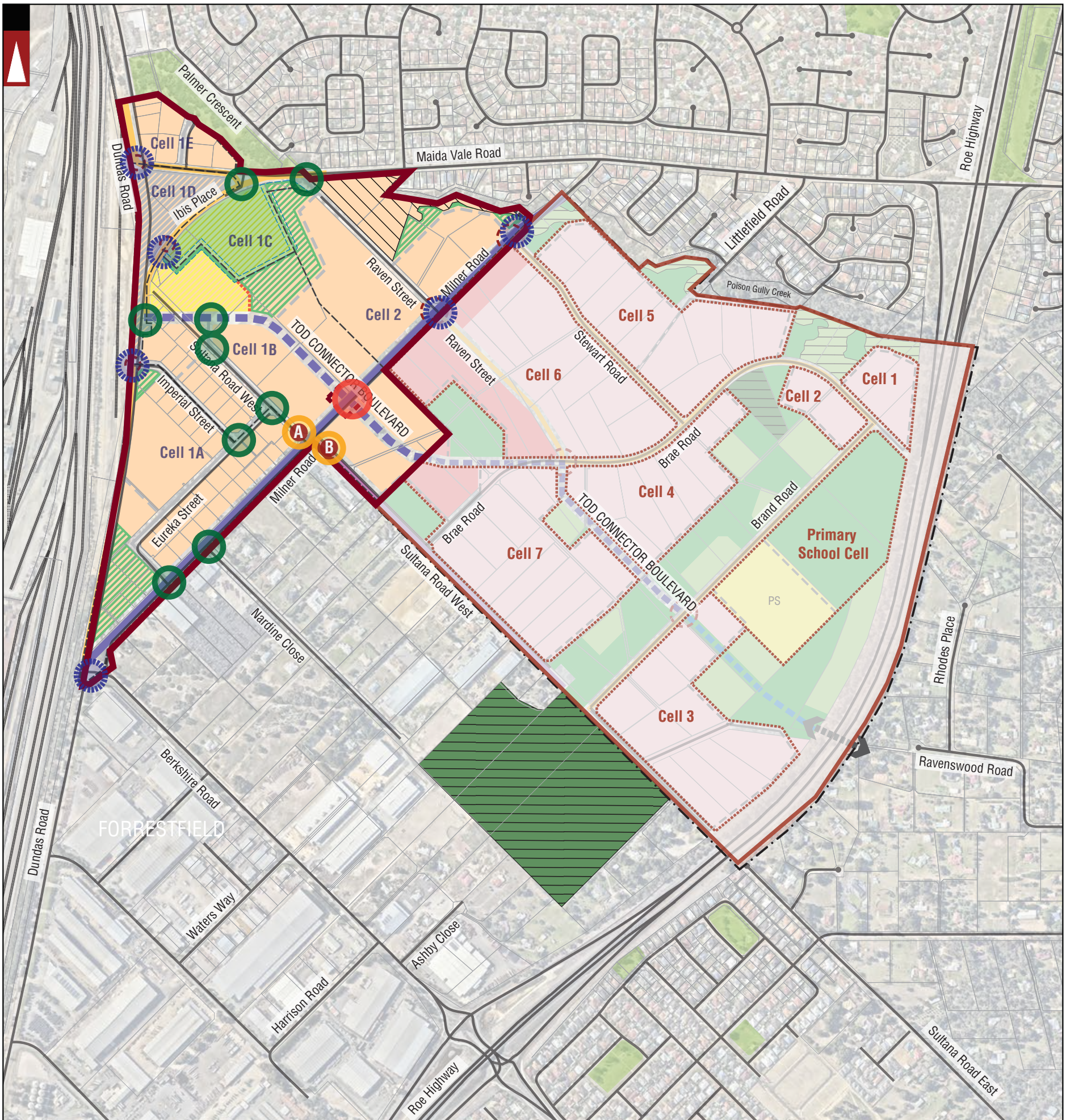

ROAD
Hay Street STREET NAME
RAILWAY
ROAD BRIDGE

TOD PRECINCT BOUNDARY
RESIDENTIAL PRECINCT BOUNDARY
CITY OF KALAMUNDA LOCAL GOVERNMENT NAME
LOCAL AUTHORITY BOUNDARY

PROPOSED ROUNDABOUT
SIGN CONTROLLED / YIELD
PROPOSED SIGNALISED INTERSECTION

A CUL DE SAC HEAD
B NO RIGHT OUT INTERSECTION

NOTE - THE PLAN SHOWS INTERSECTION CONTROL ON MAJOR INTERSECTIONS, ALL OTHER INTERSECTIONS ARE TO BE SIGN CONTROLLED /YIELD.

LEGEND

Certified System Quality ISO 9001 SAI GLOBAL

No	DATE	AMENDMENT
C	30-08-2021	PROPOSED LAYOUT UPDATED
B	27-07-2021	INFORMATION UPDATED
A	01-11-2018	ISSUED FOR REVIEW

PROJECT: FORRESTFIELD NORTH - TOD PRECINCT

TITLE: INTERSECTION CONTROL

DRAWING NUMBER: KC00604.000_ S11

DRAWN BY: J.S.

Civil & Traffic Engineering Consultants
Suite 7 No 10 Whipple Street Balcatta WA 6021
PH: 08 9441 2700
WEB: www.kctt.com.au

kctt

Appendix 3

SIDRA Intersection Analysis

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1. Introduction

This short report provides details on the SIDRA Analysis conducted to support the findings of the report KC00604.000_R01_Rev B. The intersections have been modelled in the AM and PM peak hour for assessment years - 2031 and 2050. It is assumed that by 2031, approximately 15% of Forrestfield North will be developed. In 2050, it is expected that the entire Forrestfield North will be developed.

The dimensions of the existing intersection elements have been scaled from aerial imagery which was obtained through our commercial arrangement with Nearmap and through publicly available Intramaps. These images are suitable for use in concept drafting applications with a level of accuracy to within +/- 10 centimetres. The dimensions of future intersections and roads were sourced from publicly available documents and drawings. Where no data was available, intersection elements were assumed based on the previous experience, regulatory documents and relevant precedents.

2. Traffic Generation and Distribution Analysis

Nominate the analysed intersection	M01. **Interchange of Roe Highway and Maida Vale Road:** M02. **Maida Vale Road / Milner Road – Stop Sign** M03. **Milner Road / Stewart Road – Roundabout** M04. **Milner Road / Raven Street – Roundabout** M05. **Milner Road / TOD Connector - Signals** M06. **Milner Road / Berkshire Road / Dundas Road – Roundabout** M07. **Interchange of Roe Highway and Berkshire Road:**
Nominate the source for SIDRA Intersection turning movements	Input volumes for SIDRA Intersection analysis have been obtained from Paramics modelling results (turn counts). KCTT has utilised the daily volumes of ROM24 2031 and 2041 (MLUFS 1.4 version) to create the demand matrix and distribute the traffic between the 8 major external zones. The traffic demands between any two zones were estimated and calibrated using Paramics modelling software until the traffic flows reflected the ROM24 data as closely as possible. FFN generated traffic was processed separately in the Paramics model, and the turn counts were derived from this model through the analyser component of Paramics and used for the SIDRA models. Refer to Appendix 4 - Traffic Modelling Report for more detail. Input volumes (turn counts) were provided at the end of this document.

Nominate the external zones for traffic distribution

Zone 1. To/from the north via Roe Highway (south of Berkshire Road)
Zone 2. To/from the south via Hawtin Road (south of Berkshire Road)
Zone 3. To/from the east via Kalamunda Road
Zone 4. To/from the north via Roe Highway (north of Kalamunda Road)
Zone 5. To/from the west via Kalamunda Road (west of Abernethy Road)
Zone 6. To/from the south via Abernethy Road (south of Dundas Road)
Zone 7. To/from the south via Dundas Road (south of Berkshire Road
Zone 8. To/from the east via Gooseberry Hill Road (east of Kalamunda Road)

Land Use	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Zone 6	Zone 7	Zone 8	Surrounding Suburbs and Developments	Internal Trips
All Residential	20%	5%	7.5%	15%	10%	8%	6%	2%	1.5%	25%
Primary School	0%	2%	1%	0%	0%	0%	0%	2%	80%	15%
All Non-residential Land uses	5	15%	15%	5%	10%	5%	5%	10%	30%	Varies*

*Note; Non-Residential land use distribution shows only external traffic distribution (calculates to full 100% of external traffic). Internal traffic percentages shown below are calculated from the total traffic generated by individual land uses:

- Retail without Food Component / Showroom Retail: 20%
- Retail with Food Component: 45%
- Medical Centre and Childcare Centre: 25%
- Commercial: 5%
- Community Centre: 20%
- FFN Station: 20%

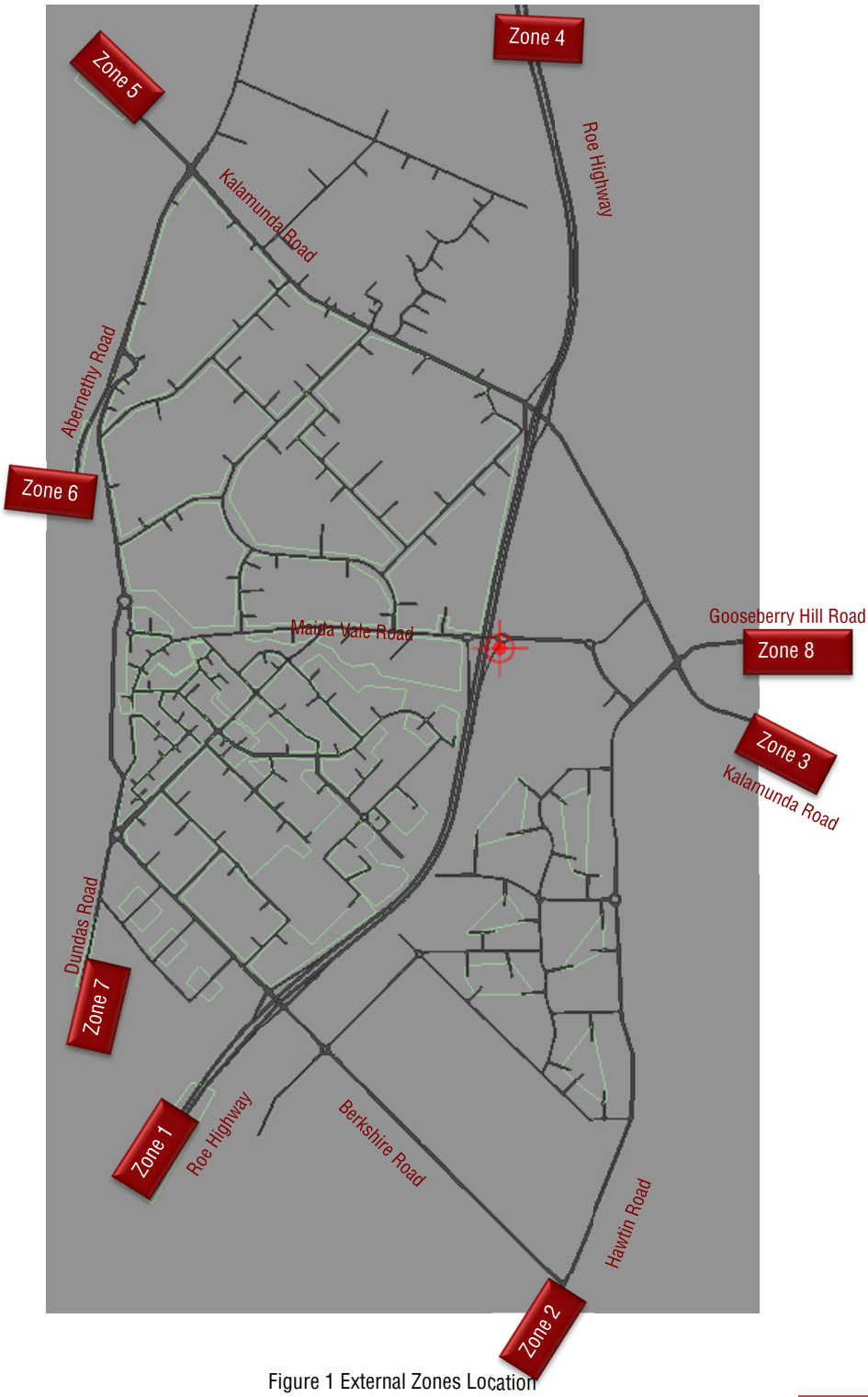


Figure 1 External Zones Location

3. Summary of Results

Nominate the analysed intersections and intersection controls	M01. **Interchange of Roe Highway and Maida Vale Road** M02. **Maida Vale Road / Milner Road – Roundabout** M03. **Milner Road / Stewart Road – Give Way (Potential Roundabout)** M04. **Milner Road / Raven Street – Roundabout** M05. **Milner Road / TOD Connector – Signalised Intersection** M06. **Milner Road / Berkshire Road / Dundas Road – Roundabout** M07. **Interchange of Roe Highway and Berkshire Road**
Describe the models analysed in SIDRA	All intersections were analysed in AM and PM peaks for the years - 2031 and 2050. Input traffic volumes for SIDRA models were obtained through the output of traffic modelling conducted by KCTT (Refer to Appendix 4 for Traffic modelling report). The most notable adjustments to the existing road geometry are: 1) One additional lane per direction has been added for Maida Vale Road (from 2031, near Roe Highway Interchange). 2) Two additional through lanes per direction have been added to Roe Highway, as it is stated in Road Network Plan – Transport @ 3.5 million that Roe Highway will be upgraded to an 8-lane freeway standard by 2050. However, modelling has shown that 4 lanes per direction might be required in 2031 too. 3) Intersections of ramps to/from Roe Highway with Maida Vale Road were modelled as roundabouts. (This is currently being implemented by the City of Kalamunda in 2021). 4) Two lanes per direction for Berkshire Road to cater for the estimated future traffic volumes.
Describe the Level of Service results	M01. The interchange of Roe Highway and Maida Vale Road is generally expected to be satisfying in 2031 and 2050 in both AM and PM peaks with LOS ranging from A – B. In 2031 both roundabouts of Maida Vale Road with Roe Highway ramps were upgraded to 2-lane roundabouts.

M02. The intersection of Maida Vale Road and Milner Road is generally expected to perform at a satisfying level in 2050 in both AM and PM peaks with LOS ranging from A – B. The single-lane roundabout is expected to operate successfully in 2031, and minor addition of a left-turning lane on the Milner Road approach will successfully alleviate potential issues in 2050.

M03. The intersection of Milner Road and Stewart Road is generally expected to perform at a satisfying level in 2050 in both AM, and PM peaks with LOS ranging from A – B. Potential roundabout would also have high LOS A.

M04. The intersection of Milner Road and Raven Street is generally expected to perform at a satisfying level in 2050 in both AM and PM peaks with LOS ranging from A – B.

M05. The intersection of Milner Road and TOD Connector is generally expected to perform at a satisfying level in 2031 and 2050 in both AM and PM peaks with LOS ranging from A – B. The intersection is expected to operate as a signalised intersection in 2031 and 2050.

M06. The intersection of Milner Road, Berkshire Road and Dundas Road is generally expected to perform at a satisfying level in 2031 in both AM and PM peak as a 2-lane roundabout and in 2050 as a 3-lane roundabout.

However, oversizing the intersections too early could have negative effects in the future. KCTT believes that leaving sufficient space and upgrading all intersections gradually, when and if necessary, would be the most effective strategy when planning for a more distant future. A 3-lane roundabout is extremely costly, and it should be used as a last option if traffic volumes reach these values in the distant future.

Additionally, the City of Kalamunda has advised that a 2-lane roundabout is the only acceptable option due to too spatial and other constraints.

M07. The interchange is expected to operate at a satisfying level in 2031. However, if the traffic grows as quickly as anticipated in this report, both signalised intersections on Berkshire Road will show significant queues in both peak times in 2050.
Delays shown in 2050 are generally 1-1.5x of the expected Cycle time of 150s. This is generally common in high traffic volume environments in peak

	times.
Comments and conclusion	KCTT believe that the key will be monitoring the key intersections' performance and acting accordingly. The surrounding area is not developing at speed predicted in the previous decade. Development speed is significantly lower due to the slow-down in the real estate market and general economic conditions in the WA economy. Furthermore, the actual means and efficiency of transportation and associated traffic flow itself are expected to change significantly in the next 2-3 decades (autonomous vehicles, 'smart' infrastructure etc). These are the factors to be considered when committing land for intersection upgrades long before the upgrade is due. Considering these factors will reduce the risk of oversizing the intersection and inefficiently using land and resources. Upgrades shown for 2050 for all models are to be taken with caution, having in mind all the above.

A summary of the results of the SIDRA analysis is shown on the following pages. For purposes of clarity, we will provide intersection summaries below. The full SIDRA output report can be provided on request.

Note - SIDRA graphic is not an accurate representation of the intersection geometry. It is a simplified tool, and its main purpose is to illustrate main intersection elements roughly. The graphic might show median breaks where there are none, in reality, oversized splitter islands and central islands for roundabouts etc. The graphic representation does not influence the calculations nor any other output.*

4. SIDRA Intersection Analysis - Output

USER REPORT FOR NETWORK

All Movement Classes

Project: KC00604.000 M01 Roe Highway - Maida Vale Road **Template: Default Network User Report**

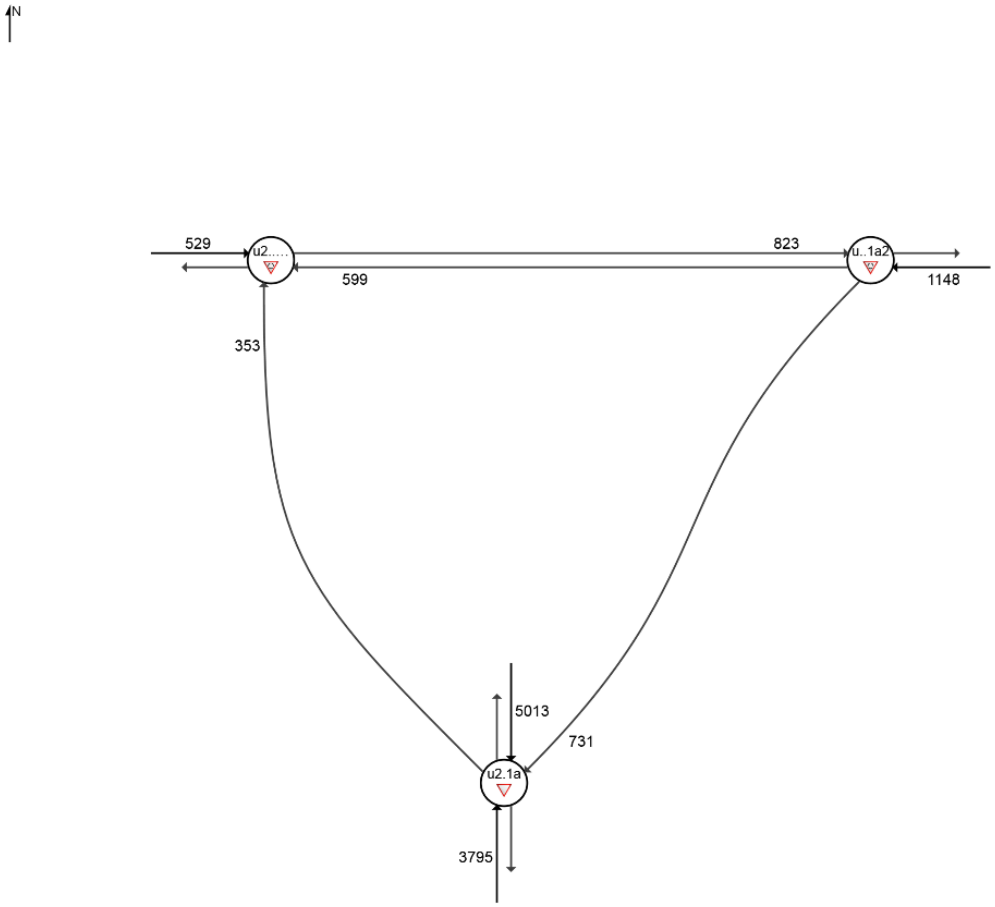
Network: un2.1a [un2.1a Network 2031 AM - Upgrade (Network Folder: 2031)]

un2.1a Network 2031 AM - Upgrade
Network Category: Existing Design

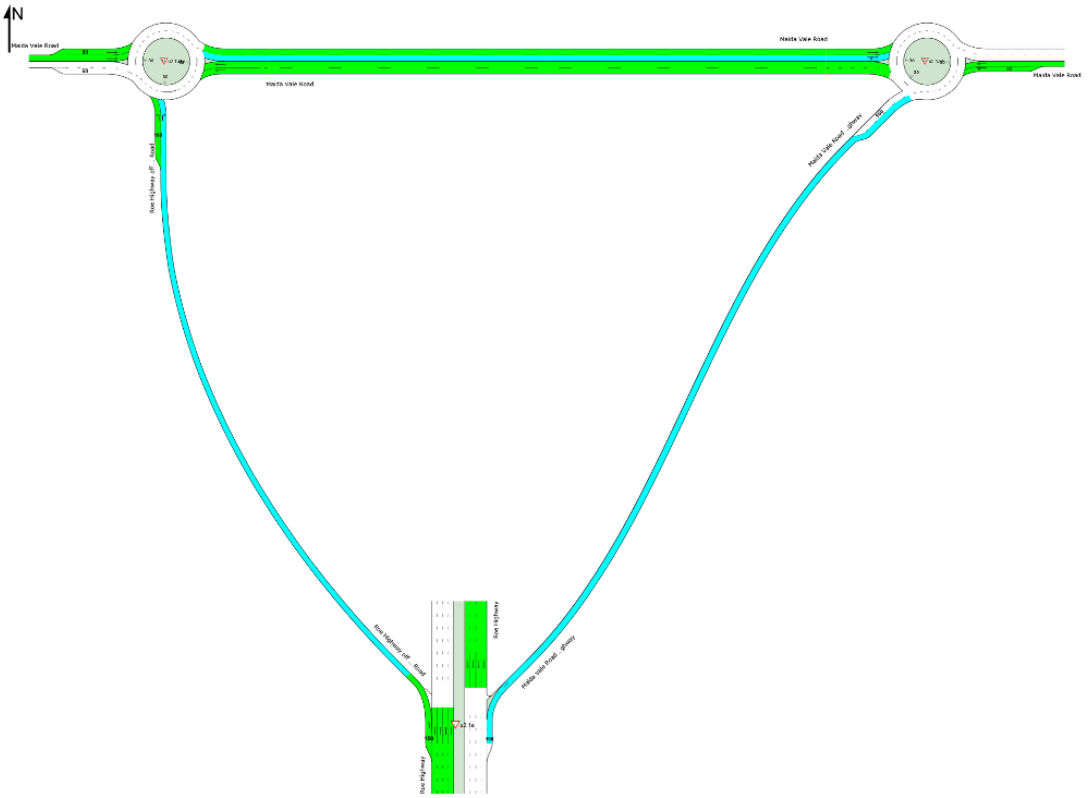
Network Flows - Input

Use the button below to open or close all popup boxes. Click value labels to open selected ones.
Click and drag popup boxes to move to preferred positions.

Open All Popups



Level of Service


N
Hawtin Vale Road
Hawtin Vale Road
Hawtin Vale Road

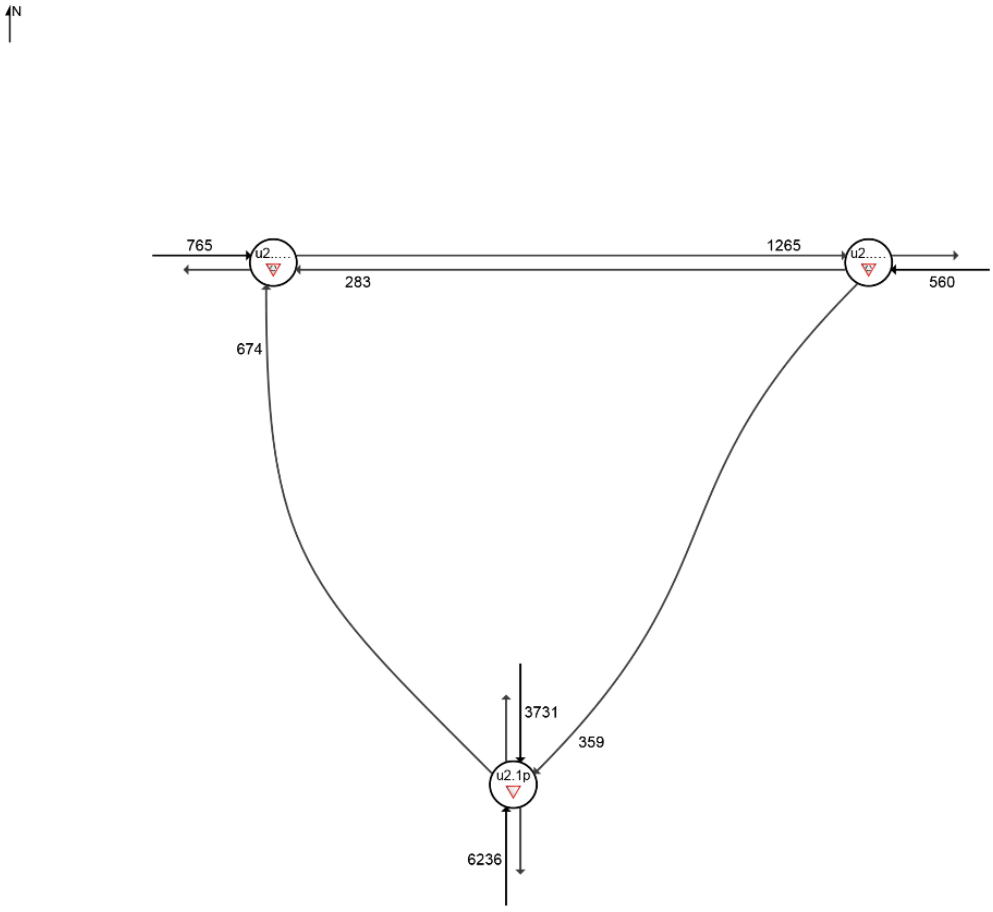
Network: un2.1p [un2.1p Network 2031 PM - Upgrade (Network Folder: 2031)]

un2.1p Network 2031 PM - Upgrade
Network Category: (None)

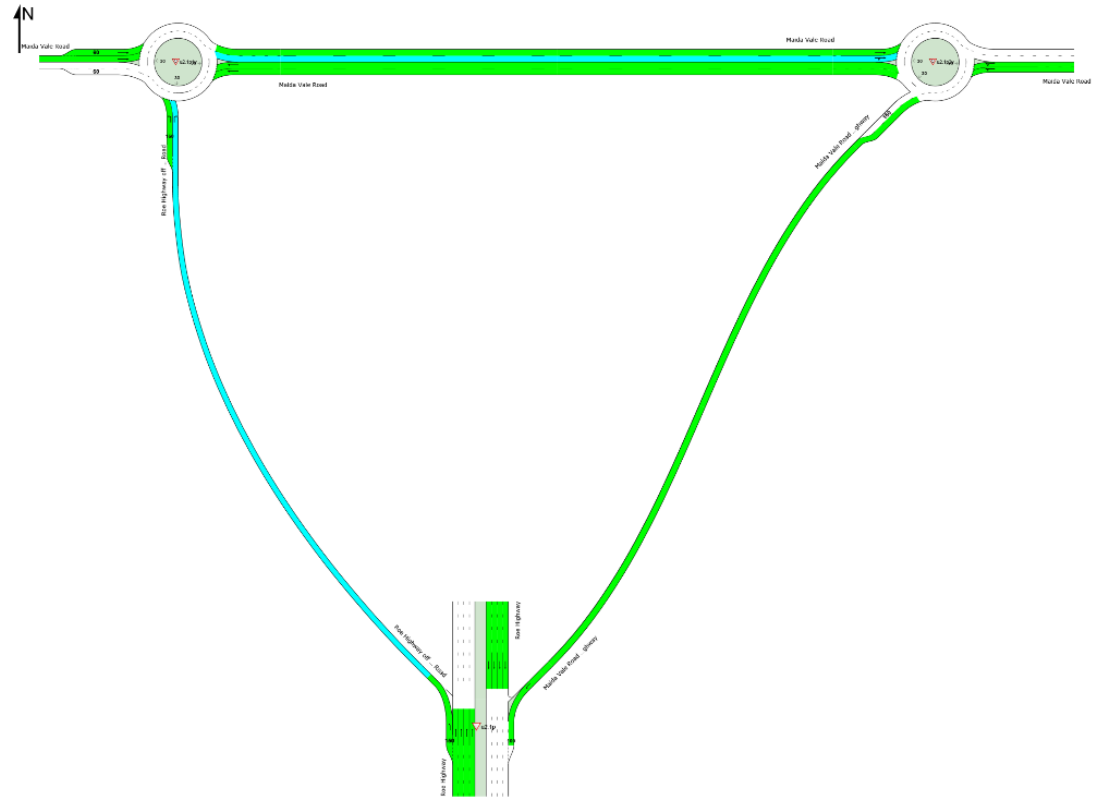
Network Flows - Input

Use the button below to open or close all popup boxes. Click value labels to open selected ones.
Click and drag popup boxes to move to preferred positions.

Open All Popups



Level of Service



Colour code based on Level of Service

LOS A	LOS B	LOS C	LOS D	LOS E	LOS F

Delay Model: SIDRA Standard (Geometric Delay is included).

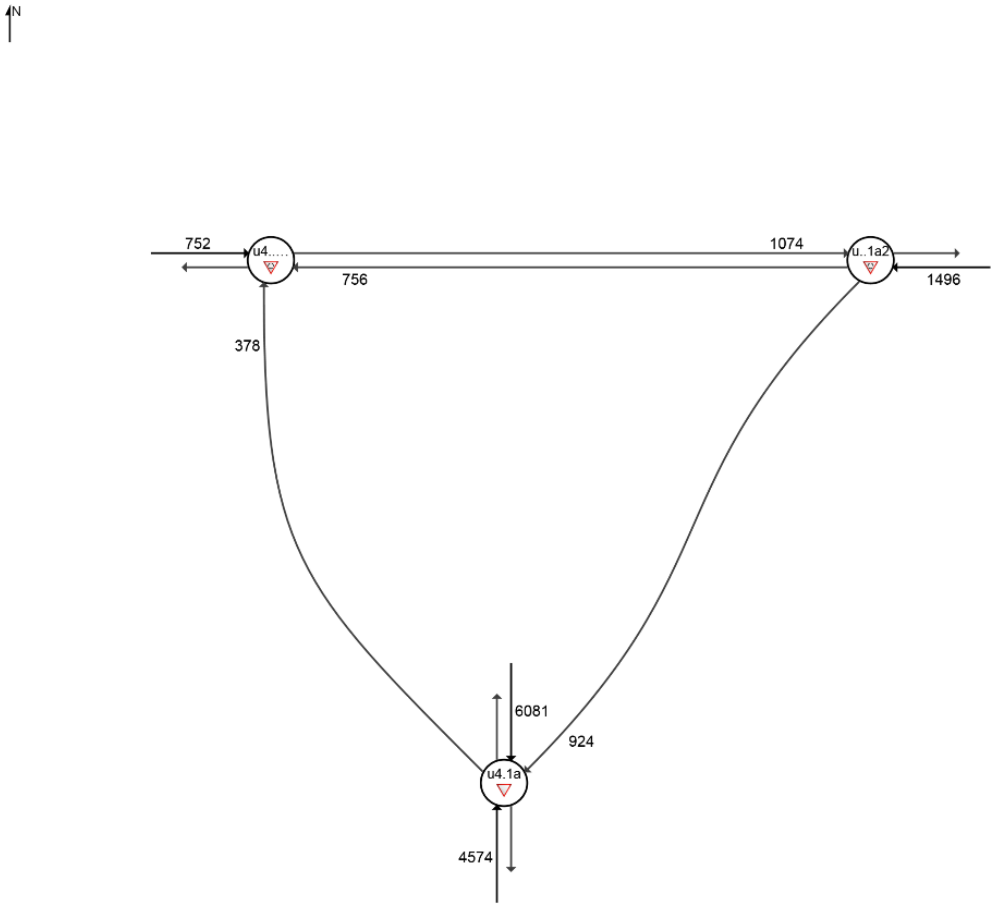
Network: un4.1a [un4.1a Network 2050 AM - Upgrade (Network Folder: 2050)]

un4.1a Network 2050 AM - Upgrade
Network Category: Existing Design

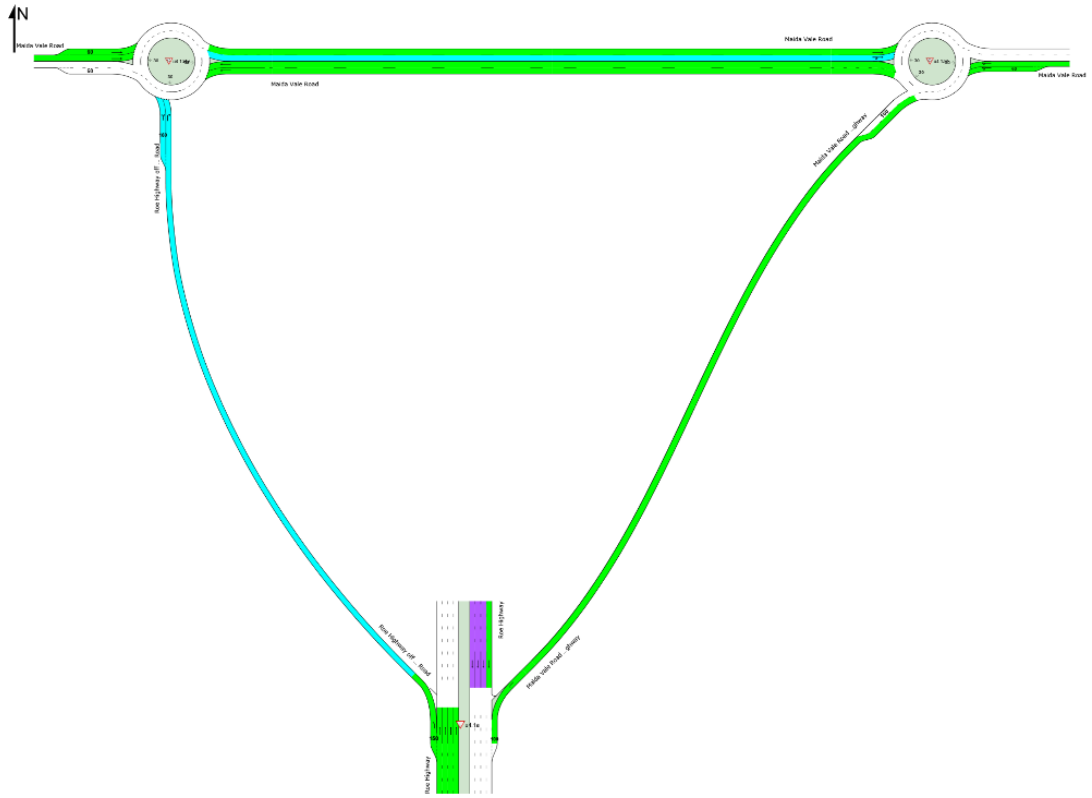
Network Flows - Input

Use the button below to open or close all popup boxes. Click value labels to open selected ones.
Click and drag popup boxes to move to preferred positions.

Open All Popups



Level of Service


N
Mundy Drive
Mundy Drive
Mundy Drive

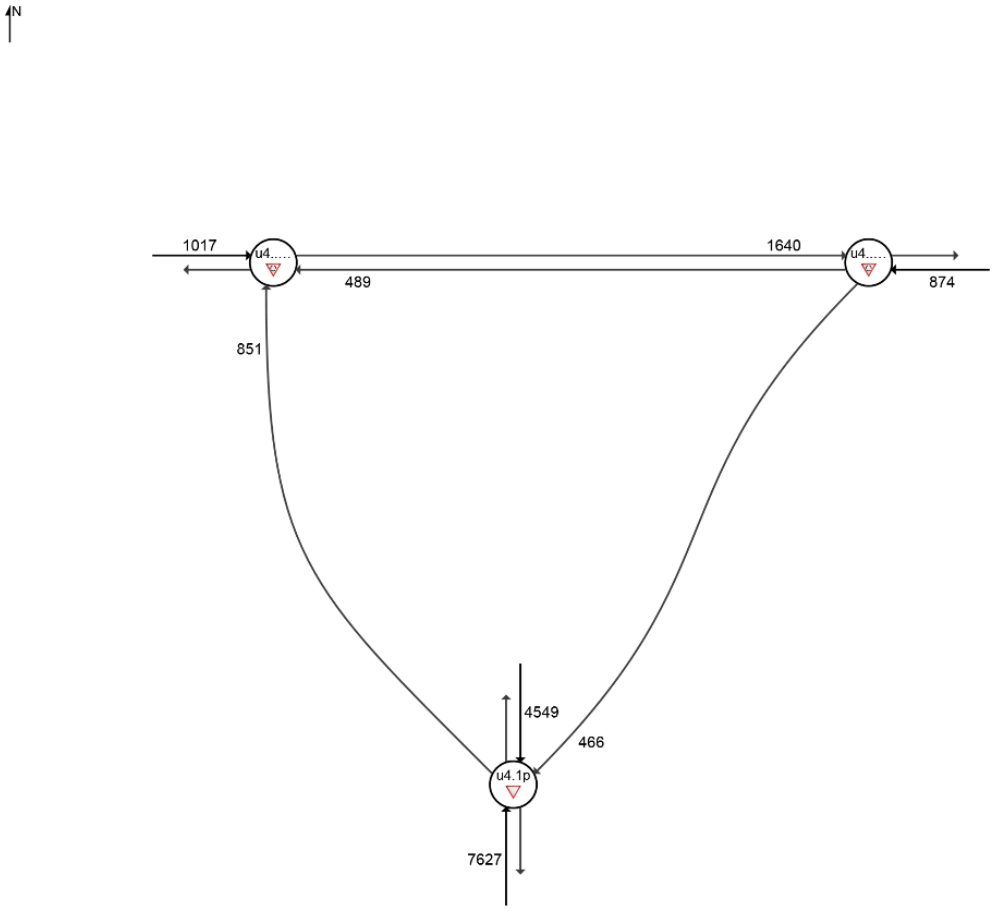
Network: un4.1p [un4.1p Network 2050 PM - Upgrade (Network Folder: 2050)]

un4.1p Network 2050 PM - Upgrade
Network Category: (None)

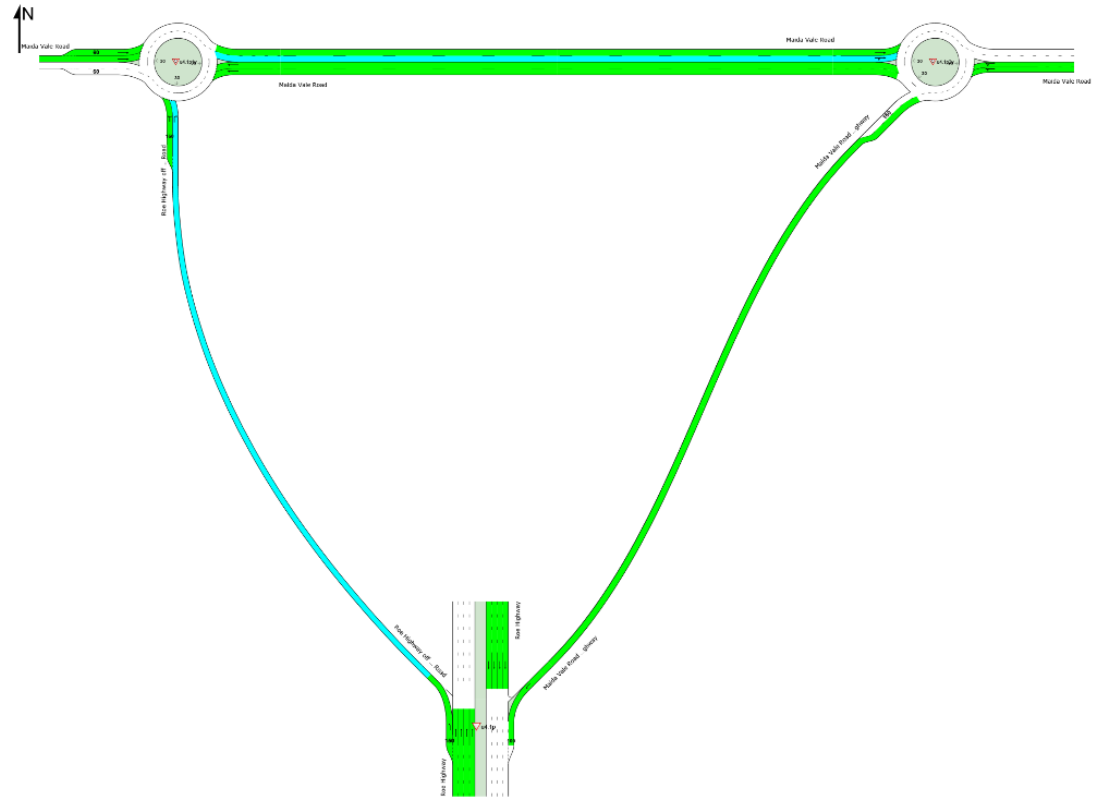
Network Flows - Input

Use the button below to open or close all popup boxes. Click value labels to open selected ones.
Click and drag popup boxes to move to preferred positions.

Open All Popups



Level of Service



Colour code based on Level of Service

LOS A	LOS B	LOS C	LOS D	LOS E	LOS F

Delay Model: SIDRA Standard (Geometric Delay is included).

SIDRA INTERSECTION 9.0 | Copyright © 2000-2020 Akcelik and Associates Pty Ltd | sidrasolutions.com
Organisation: KC TRAFFIC AND TRANSPORT | Licence: NETWORK / 1PC | Created: 28 July, 2021 11:49:03
Project: C:\Users\User\Box\KCTT Projects\KC00000 Current Projects\KC00604.000 Forrestfield North - Structure Plan Preparation\Outgoing \SIDRA\DCP\210708 Rev B\Future Models\KC00604.000 M01 Roe Highway - Maida Vale Road.sip9

USER REPORT FOR NETWORK SITE

All Movement Classes

Project: KC00604.000 M01 Roe Highway - Maida Vale Road **Template: Network Site User Report**

Site: u2.1a [u2.1a Roe Highway Interchange with Maida Vale Road - 2031 AM - Upgrade (Site Folder: 2031 AM)] **Network: 5 [un2.1a Network 2031 AM - Upgrade (Network Folder: 2031)]**

u2.1a Roe Highway Interchange with Maida Vale Road - 2031 AM - Upgrade
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)
S: Roe Highway	3995	3252	0	5
NE: Maida Vale Road on to Roe Highway	769	629	0	0
N: Roe Highway	5277	4248	0	0
Total	10041	8129	0	5

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
South: Roe Highway															

Lane 1	372	11.6	372	11.6	1722	0.216	100	7.9	LOS A	0.0	0.0	Short	150	0.0	NA
Lane 2	906	17.1	906	17.1	1840	0.492	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	906	17.1	906	17.1	1840	0.492	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	906	17.1	906	17.1	1840	0.492	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	906	17.1	906	17.1	1840	0.492	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	3995	16.6	3995	16.6		0.492		0.8	NA	0.0	0.0				
NorthEast: Maida Vale Road on to Roe Highway															
Lane 1	769	17.0	769	17.0	1619	0.475	100	12.8	LOS B	0.0	0.0	Full	400	0.0	0.0
Approach	769	17.0	769	17.0		0.475		12.8	NA	0.0	0.0				
North: Roe Highway															
Lane 1	754	17.7	754	17.7	1842	0.409	50[7]	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	1508	17.7	1508	17.7	1842	0.819	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1508	17.7	1508	17.7	1842	0.819	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1508	17.7	1508	17.7	1842	0.819	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	5277	17.7	5277	17.7		0.819		0.3	NA	0.0	0.0				
Intersection	10041	17.2	10041	17.2		0.819		1.5	NA	0.0	0.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

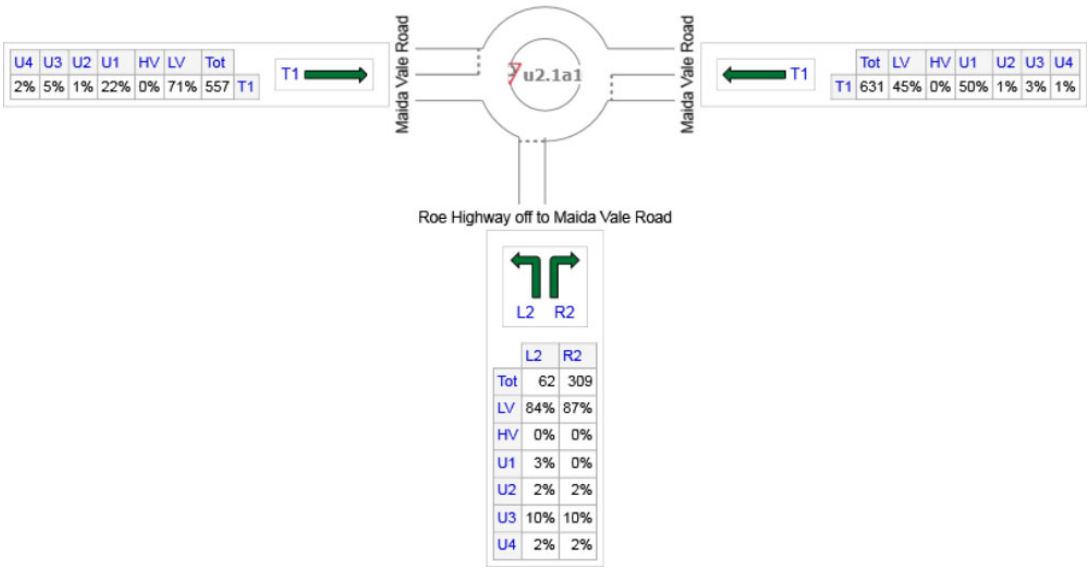
7 Lane under-utilisation specified by the user

Site: u2.1a1v [u2.1a1 Maida Vale Road - Roe Highway off to Maida Vale Road - 2031 AM - Upgrade - Conversion (Site Folder: 2031 AM)]

Network: 5 [un2.1a Network 2031 AM - Upgrade (Network Folder: 2031)]

u2.1a1 Maida Vale Road - Roe Highway off to Maida Vale Road - 2031 AM - Upgrade
Site Category: (None)
Roundabout

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)	A
S: Roe Highway off to Maida Vale Road	372	320	0	2	
E: Maida Vale Road	631	287	0	313	
W: Maida Vale Road	557	394	0	120	
Total	1559	1001	0	435	

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
South: Roe Highway off to Maida Vale Road															
Lane 1	62	11.3	62	11.3	595	0.104	100	8.9	LOS A	0.2	1.3	Short	160	0.0	NA
Lane 2[d]	309	11.7	309	11.7	996	0.311	100	11.9	LOS B	0.6	4.9	Full	400	0.0	0.0
Approach	372	11.6	372	11.6		0.311		11.4	LOS B	0.6	4.9				
East: Maida Vale Road															
Lane 1	182	4.0	182	4.0	1400	0.130	51[6]	4.6	LOS A	0.0	0.0	Full	160	0.0	0.0
Lane 2[d]	449	4.0	449	4.0	1761	0.255	100	3.6	LOS A	0.0	0.0	Full	160	0.0	0.0
Approach	631	4.0	631	4.0		0.255		3.9	LOS A	0.0	0.0				
West: Maida Vale Road															
Lane 1[d]	293	6.3	293	6.3	1153	0.254	100	4.9	LOS A	0.6	4.5	Short	60	0.0	NA
Lane 2	264	6.3	264	6.3	1036	0.254	100	5.1	LOS A	0.6	4.3	Full	120	0.0	0.0

Approach	557	6.3	557	6.3	0.254	5.0	LOS A	0.6	4.5
Intersection	1559	6.6	1559	6.6	0.311	6.1	LOS A	0.6	4.9

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

6 Lane under-utilisation due to downstream effects
d Dominant lane on roundabout approach

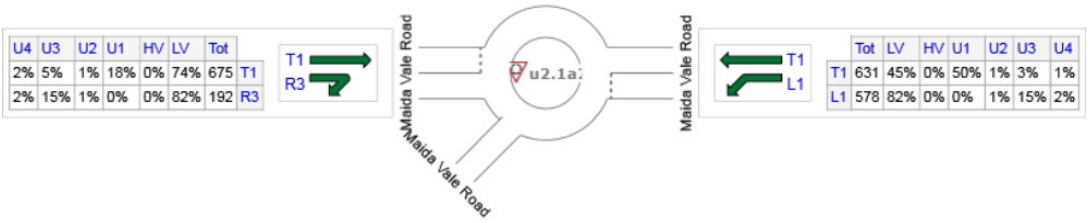
Site: u2.1a2 [u2.1a2 Maida Vale Road - Maida Vale Road on to Roe Highway - 2031 AM - Upgrade - Conversion (Site Folder: 2031 AM)]

Network: 5 [un2.1a Network 2031 AM - Upgrade (Network Folder: 2031)]

u2.1a2 Maida Vale Road - Maida Vale Road on to Roe Highway - 2031 AM - Upgrade - Conversion
Site Category: (None)
Roundabout

OD Arrival Flows

U4	U3	U2	U1	HV	LV	Tot	
2%	5%	1%	18%	0%	74%	675	T1
2%	15%	1%	0%	0%	82%	192	R3



	Tot	LV	HV	U1	U2	U3	U4
T1	631	45%	0%	50%	1%	3%	1%
L1	578	82%	0%	0%	1%	15%	2%

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)	AustroadsClass 2 (
E: Maida Vale Road	1208	759	0	313	13
W: Maida Vale Road	866	657	0	120	12
Total	2075	1417	0	433	25

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Maida Vale Road															
Lane 1	554	17.0	554	17.0	1082	0.512	100	5.1	LOS A	1.5	12.8	Short	60	0.0	NA
Lane 2[d]	655	4.5	655	4.5	1314	0.498	97[6]	5.0	LOS A	1.4	10.8	Full	120	0.0	0.0
Approach	1208	10.2	1208	10.2		0.512		5.0	LOS A	1.5	12.8				
West: Maida Vale Road															
Lane 1[d]	675	6.6	675	6.6	1731	0.390	100	3.6	LOS A	0.0	0.0	Full	160	0.0	0.0
Lane 2	192	17.0	192	17.0	1229	0.156	100	12.1	LOS B	0.0	0.0	Full	160	0.0	0.0
Approach	866	8.9	866	8.9		0.390		5.5	LOS A	0.0	0.0				
Intersection	2075	9.7	2075	9.7		0.512		5.2	LOS A	1.5	12.8				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

6 Lane under-utilisation due to downstream effects
d Dominant lane on roundabout approach

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Project: C:\Users\User\Box\KCTT Projects\KC00000 Current Projects\KC00604.000 Forrestfield North - Structure Plan Preparation\Outgoing

\SIDRA\DCP\210708 Rev B\Future Models\KC00604.000 M01 Roe Highway - Maida Vale Road.sip9

USER REPORT FOR NETWORK SITE

All Movement Classes

Project: KC00604.000 M01 Roe Highway - Maida Vale Road **Template: Network Site User Report**

Site: u2.1p [u2.1p Roe Highway Interchange with Maida Vale Road - 2031 PM - Upgrade (Site Folder: 2031 PM)] **Network: 6 [un2.1p Network 2031 PM - Upgrade (Network Folder: 2031)]**

u2.1p Roe Highway Interchange with Maida Vale Road - 2031 PM - Upgrade
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)
S: Roe Highway	6564	5393	0	5
NE: Maida Vale Road on to Roe Highway	378	309	0	0
N: Roe Highway	3927	3162	0	0
Total	10869	8864	0	5

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
South: Roe Highway															

Lane 1	709	11.7	709	11.7	1721	0.412	100	7.9	LOS A	0.0	0.0	Short	150	0.0	NA
Lane 2	1464	16.4	1464	16.4	1857	0.788	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1464	16.4	1464	16.4	1857	0.788	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1464	16.4	1464	16.4	1857	0.788	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	1464	16.4	1464	16.4	1857	0.788	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	6564	15.9	6564	15.9		0.788		1.1	NA	0.0	0.0				
NorthEast: Maida Vale Road on to Roe Highway															
Lane 1	378	17.0	378	17.0	1619	0.233	100	6.3	LOS A	0.0	0.0	Full	400	0.0	0.0
Approach	378	17.0	378	17.0		0.233		6.3	NA	0.0	0.0				
North: Roe Highway															
Lane 1	561	17.7	561	17.7	1842	0.305	50[7]	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	1122	17.7	1122	17.7	1842	0.609	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1122	17.7	1122	17.7	1842	0.609	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1122	17.7	1122	17.7	1842	0.609	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	3927	17.7	3927	17.7		0.609		0.1	NA	0.0	0.0				
Intersection	10869	16.6	10869	16.6		0.788		0.9	NA	0.0	0.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

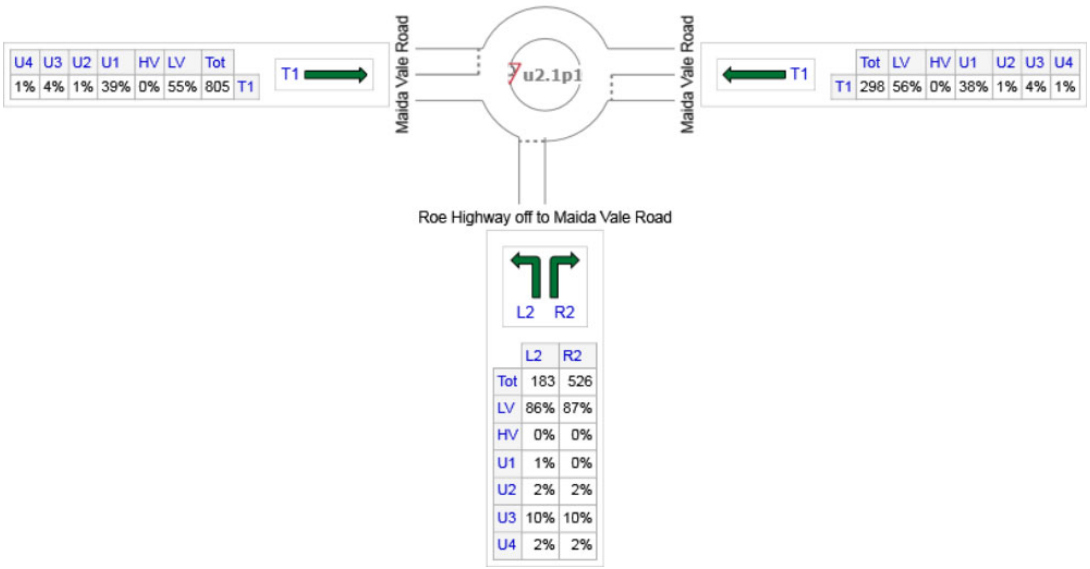
7 Lane under-utilisation specified by the user

Site: u2.1p1v [u2.1p1 Maida Vale Road - Roe Highway off to Maida Vale Road - 2031 PM - Upgrade - Conversion (Site Folder: 2031 PM)]

Network: 6 [un2.1p Network 2031 PM - Upgrade (Network Folder: 2031)]

u2.1p1 Maida Vale Road - Roe Highway off to Maida Vale Road - 2031 PM - Upgrade
Site Category: (None)
Roundabout

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)	A
S: Roe Highway off to Maida Vale Road	709	613	0	2	
E: Maida Vale Road	298	167	0	113	
W: Maida Vale Road	805	442	0	315	
Total	1813	1222	0	429	

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
South: Roe Highway off to Maida Vale Road															
Lane 1	183	11.5	183	11.5	821	0.223	100	6.2	LOS A	0.4	3.3	Short	160	0.0	NA
Lane 2[d]	526	11.7	526	11.7	1180	0.446	100	10.9	LOS B	1.0	8.4	Full	400	0.0	0.0
Approach	709	11.6	709	11.6		0.446		9.7	LOS A	1.0	8.4				
East: Maida Vale Road															
Lane 1	86	5.0	86	5.0	1388	0.062	51[6]	4.0	LOS A	0.0	0.0	Full	160	0.0	0.0
Lane 2[d]	212	5.0	212	5.0	1750	0.121	100	3.6	LOS A	0.0	0.0	Full	160	0.0	0.0
Approach	298	5.0	298	5.0		0.121		3.7	LOS A	0.0	0.0				
West: Maida Vale Road															
Lane 1[d]	432	4.9	432	4.9	991	0.436	100	6.4	LOS A	1.2	9.0	Short	60	0.0	NA
Lane 2	374	4.9	374	4.9	857	0.436	100	7.0	LOS A	1.2	8.7	Full	120	0.0	0.0

Approach	805	4.9	805	4.9	0.436	6.7	LOS A	1.2	9.0
Intersection	1813	7.5	1813	7.5	0.446	7.4	LOS A	1.2	9.0

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

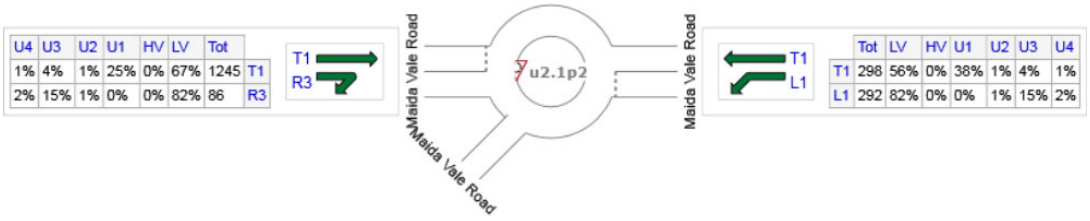
6 Lane under-utilisation due to downstream effects
d Dominant lane on roundabout approach

Site: u2.1p2v [u2.1p2 Maida Vale Road - Maida Vale Road on to Roe Highway - 2031 PM - Upgrade - Conversion (Site Folder: 2031 PM)]

Network: 6 [un2.1p Network 2031 PM - Upgrade (Network Folder: 2031)]

u2.1p2 Maida Vale Road - Maida Vale Road on to Roe Highway - 2031 PM - Upgrade
Site Category: (None)
Roundabout

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)	AustroadsClass 2 (
E: Maida Vale Road	589	406	0	113	7
W: Maida Vale Road	1332	910	0	315	18
Total	1921	1316	0	427	25

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Maida Vale Road															
Lane 1	275	17.0	275	17.0	1214	0.227	100	3.8	LOS A	0.5	4.3	Full	120	0.0	0.0
Lane 2[d]	314	5.6	314	5.6	1422	0.221	97[6]	4.0	LOS A	0.5	3.6	Full	120	0.0	0.0
Approach	589	10.9	589	10.9		0.227		3.9	LOS A	0.5	4.3				
West: Maida Vale Road															
Lane 1[d]	1245	6.0	1245	6.0	1738	0.716	100	3.6	LOS A	0.0	0.0	Full	160	0.0	0.0
Lane 2	86	17.0	86	17.0	1197	0.072	100	11.1	LOS B	0.0	0.0	Full	160	0.0	0.0
Approach	1332	6.7	1332	6.7		0.716		4.1	LOS A	0.0	0.0				
Intersection	1921	8.0	1921	8.0		0.716		4.0	LOS A	0.5	4.3				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

6 Lane under-utilisation due to downstream effects
d Dominant lane on roundabout approach

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USER REPORT FOR NETWORK SITE

All Movement Classes

Project: KC00604.000 M01 Roe Highway - Maida Vale Road **Template: Network Site User Report**

Site: u4.1a [u4.1a Roe Highway Interchange with Maida Vale Road - 2050 AM - Upgrade (Site Folder: 2050 AM)] **Network: 9 [un4.1a Network 2050 AM - Upgrade (Network Folder: 2050)]**

u4.1a Roe Highway Interchange with Maida Vale Road - 2050 AM - Upgrade
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)
S: Roe Highway	4815	3913	0	11
NE: Maida Vale Road on to Roe Highway	973	796	0	0
N: Roe Highway	6401	5152	0	1
Total	12188	9861	0	12

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
South: Roe Highway															

Lane 1	398	11.5	398	11.5	1724	0.231	100	7.9	LOS A	0.0	0.0	Short	150	0.0	NA
Lane 2	1104	17.1	1104	17.1	1840	0.600	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1104	17.1	1104	17.1	1840	0.600	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1104	17.1	1104	17.1	1840	0.600	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	1104	17.1	1104	17.1	1840	0.600	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	4815	16.6	4815	16.6		0.600		0.8	NA	0.0	0.0				
NorthEast: Maida Vale Road on to Roe Highway															
Lane 1	973	17.0	973	17.0	1619	0.601	100	9.3	LOS A	0.0	0.0	Full	400	0.0	0.0
Approach	973	17.0	973	17.0		0.601		9.3	NA	0.0	0.0				
North: Roe Highway															
Lane 1	492	17.7	492	17.7	1842	0.267	25[7]	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	1970	17.7	1970	17.7	1842	1.069	100	32.9	LOS D	0.0	0.0	Full	500	0.0	0.0
Lane 3	1970	17.7	1970	17.7	1842	1.069	100	32.9	LOS D	0.0	0.0	Full	500	0.0	0.0
Lane 4	1970	17.7	1970	17.7	1842	1.069	100	32.9	LOS D	0.0	0.0	Full	500	0.0	0.0
Approach	6401	17.7	6401	17.7		1.069		30.4	NA	0.0	0.0				
Intersection	12188	17.2	12188	17.2		1.069		17.0	NA	0.0	0.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

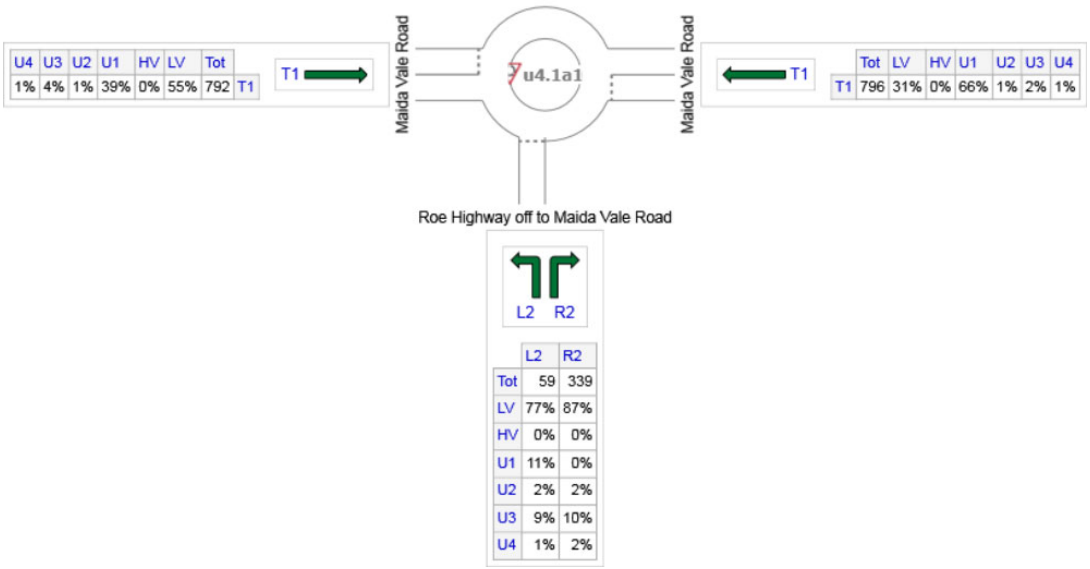
7 Lane under-utilisation specified by the user

Site: u4.1a1v [u4.1a1 Maida Vale Road - Roe Highway off to Maida Vale Road - 2050 AM - Upgrade (Site Folder: 2050 AM)]

Network: 9 [un4.1a Network 2050 AM - Upgrade (Network Folder: 2050)]

u4.1a1 Maida Vale Road - Roe Highway off to Maida Vale Road - 2050 AM - Upgrade
Site Category: (None)
Roundabout

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)	A
S: Roe Highway off to Maida Vale Road	398	339	0	6	
E: Maida Vale Road	796	246	0	523	
W: Maida Vale Road	792	435	0	309	
Total	1985	1020	0	839	

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
South: Roe Highway off to Maida Vale Road															
Lane 1[d]	219	11.4	219	11.4	939	0.234	100	11.2	LOS B	0.4	3.5	Short	160	0.0	NA
Lane 2	179	11.7	179	11.7	764	0.234	100	13.1	LOS B	0.4	3.4	Full	400	0.0	0.0
Approach	398	11.5	398	11.5		0.234		12.1	LOS B	0.4	3.5				
East: Maida Vale Road															
Lane 1	230	2.7	230	2.7	1412	0.163	51[6]	5.0	LOS A	0.0	0.0	Full	160	0.0	0.0
Lane 2[d]	566	2.7	566	2.7	1776	0.319	100	3.5	LOS A	0.0	0.0	Full	160	0.0	0.0
Approach	796	2.7	796	2.7		0.319		4.0	LOS A	0.0	0.0				
West: Maida Vale Road															
Lane 1[d]	418	4.9	418	4.9	1173	0.357	100	4.8	LOS A	0.8	5.8	Short	60	0.0	NA
Lane 2	373	4.9	373	4.9	1046	0.357	100	5.0	LOS A	0.7	5.6	Full	120	0.0	0.0

Approach	792	4.9	792	4.9	0.357	4.9	LOS A	0.8	5.8
Intersection	1985	5.3	1985	5.3	0.357	6.0	LOS A	0.8	5.8

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

6 Lane under-utilisation due to downstream effects
d Dominant lane on roundabout approach

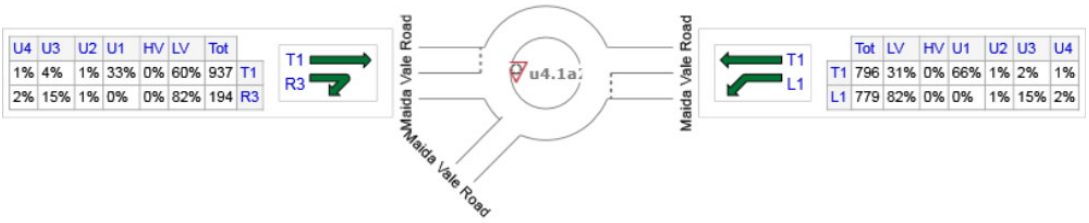
Site: u4.1a2 [u4.1a2 Maida Vale Road - Maida Vale Road on to Roe Highway - 2050 AM - Upgrade - Conversion (Site Folder: 2050 AM)]

Network: 9 [un4.1a Network 2050 AM - Upgrade (Network Folder: 2050)]

u4.1a2 Maida Vale Road - Maida Vale Road on to Roe Highway - 2050 AM - Upgrade - Conversion
Site Category: (None)
Roundabout

OD Arrival Flows

U4	U3	U2	U1	HV	LV	Tot	
1%	4%	1%	33%	0%	60%	937	T1
2%	15%	1%	0%	0%	82%	194	R3



	Tot	LV	HV	U1	U2	U3	U4
T1	796	31%	0%	66%	1%	2%	1%
L1	779	82%	0%	0%	1%	15%	2%

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)	AustroadsClass 2 (
E: Maida Vale Road	1575	883	0	523	14
W: Maida Vale Road	1131	724	0	309	14
Total	2705	1607	0	833	28

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Maida Vale Road															
Lane 1	718	17.0	718	17.0	1090	0.659	100	6.0	LOS A	2.5	21.9	Short	60	0.0	NA
Lane 2[d]	856	3.7	856	3.7	1335	0.642	97[6]	5.4	LOS A	2.3	16.8	Full	120	0.0	0.0
Approach	1575	9.8	1575	9.8		0.659		5.7	LOS A	2.5	21.9				
West: Maida Vale Road															
Lane 1[d]	937	5.4	937	5.4	1745	0.537	100	3.6	LOS A	0.0	0.0	Full	160	0.0	0.0
Lane 2	194	17.0	194	17.0	1197	0.162	100	13.0	LOS B	0.0	0.0	Full	160	0.0	0.0
Approach	1131	7.4	1131	7.4		0.537		5.2	LOS A	0.0	0.0				
Intersection	2705	8.8	2705	8.8		0.659		5.5	LOS A	2.5	21.9				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

6 Lane under-utilisation due to downstream effects
d Dominant lane on roundabout approach

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USER REPORT FOR NETWORK SITE

All Movement Classes

Project: KC00604.000 M01 Roe Highway - Maida Vale Road **Template: Network Site User Report**

Site: u4.1p [u4.1p Roe Highway Interchange with Maida Vale Road - 2050 PM - Upgrade (Site Folder: 2050 PM)] **Network: 10 [un4.1p Network 2050 PM - Upgrade (Network Folder: 2050)]**

u4.1p Roe Highway Interchange with Maida Vale Road - 2050 PM - Upgrade
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)
S: Roe Highway	8028	6561	0	49
NE: Maida Vale Road on to Roe Highway	491	401	0	0
N: Roe Highway	4788	3854	0	1
Total	13307	10816	0	51

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
South: Roe Highway															

Lane 1	896	11.2	896	11.2	1732	0.517	100	7.9	LOS A	0.0	0.0	Short	150	0.0	NA
Lane 2	1783	16.4	1783	16.4	1858	0.960	100	1.6	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1783	16.4	1783	16.4	1858	0.960	100	1.6	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1783	16.4	1783	16.4	1858	0.960	100	1.6	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	1783	16.4	1783	16.4	1858	0.960	100	1.6	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	8028	15.8	8028	15.8		0.960		2.3	NA	0.0	0.0				
NorthEast: Maida Vale Road on to Roe Highway															
Lane 1	491	17.0	491	17.0	1619	0.303	100	5.5	LOS A	0.0	0.0	Full	400	0.0	0.0
Approach	491	17.0	491	17.0		0.303		5.5	NA	0.0	0.0				
North: Roe Highway															
Lane 1	368	17.7	368	17.7	1842	0.200	25[7]	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	1473	17.7	1473	17.7	1842	0.800	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1473	17.7	1473	17.7	1842	0.800	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1473	17.7	1473	17.7	1842	0.800	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	4788	17.7	4788	17.7		0.800		0.3	NA	0.0	0.0				
Intersection	13307	16.5	13307	16.5		0.960		1.7	NA	0.0	0.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

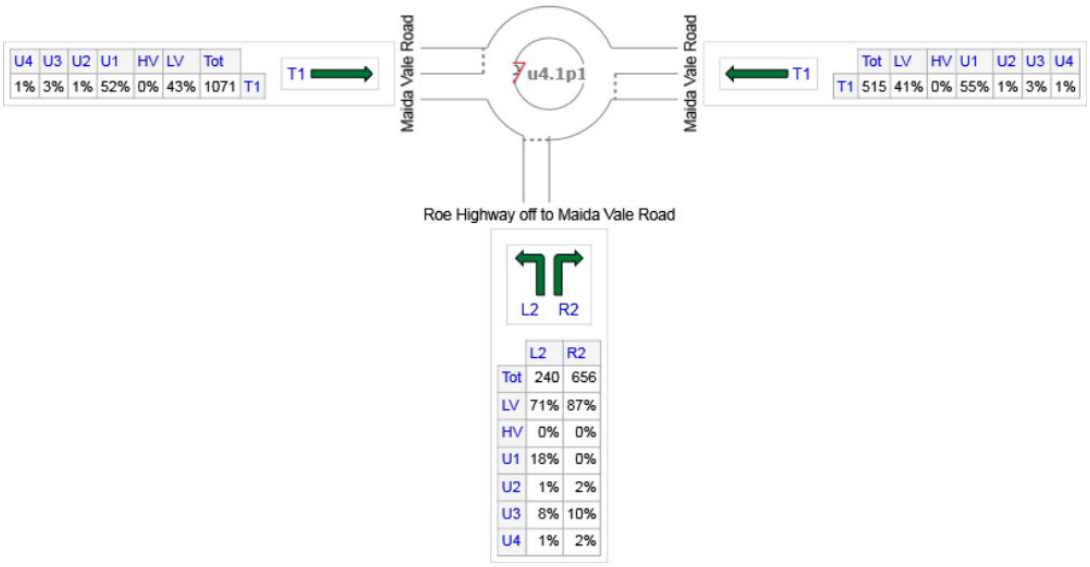
7 Lane under-utilisation specified by the user

Site: u4.1p1v [u4.1p1 Maida Vale Road - Roe Highway off to Maida Vale Road - 2050 PM - Upgrade (Site Folder: 2050 PM)]

Network: 10 [un4.1p Network 2050 PM - Upgrade (Network Folder: 2050)]

u4.1p1 Maida Vale Road - Roe Highway off to Maida Vale Road - 2050 PM - Upgrade
Site Category: (None)
Roundabout

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)	A
S: Roe Highway off to Maida Vale Road	896	739	0	42	
E: Maida Vale Road	515	210	0	282	
W: Maida Vale Road	1071	462	0	558	
Total	2481	1412	0	882	

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
South: Roe Highway off to Maida Vale Road															
Lane 1[d]	484	10.6	484	10.6	1054	0.459	100	9.7	LOS A	1.1	8.6	Short	160	0.0	NA
Lane 2	412	11.7	412	11.7	898	0.459	100	12.8	LOS B	1.1	8.7	Full	400	0.0	0.0
Approach	896	11.1	896	11.1		0.459		11.1	LOS B	1.1	8.7				
East: Maida Vale Road															
Lane 1	148	3.6	148	3.6	1401	0.106	51[6]	4.5	LOS A	0.0	0.0	Full	160	0.0	0.0
Lane 2[d]	366	3.6	366	3.6	1765	0.208	100	3.6	LOS A	0.0	0.0	Full	160	0.0	0.0
Approach	515	3.6	515	3.6		0.208		3.8	LOS A	0.0	0.0				
West: Maida Vale Road															
Lane 1[d]	582	3.8	582	3.8	997	0.583	100	7.1	LOS A	1.7	12.7	Short	60	0.0	NA
Lane 2	489	3.8	489	3.8	838	0.583	100	7.9	LOS A	1.6	12.2	Full	120	0.0	0.0

Approach	1071	3.8	1071	3.8	0.583	7.4	LOS A	1.7	12.7
Intersection	2481	6.4	2481	6.4	0.583	8.0	LOS A	1.7	12.7

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

6 Lane under-utilisation due to downstream effects
d Dominant lane on roundabout approach

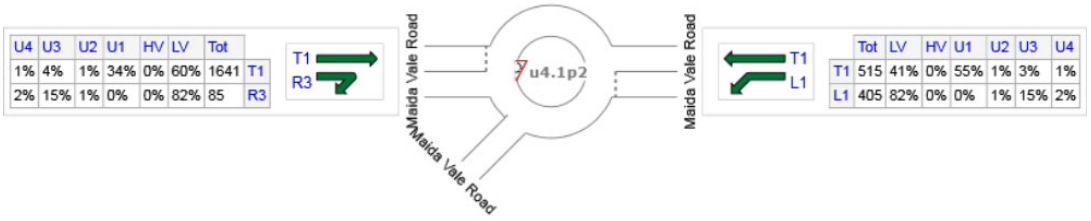
Site: u4.1p2v [u4.1p2 Maida Vale Road - Maida Vale Road on to Roe Highway - 2050 PM - Upgrade (Site Folder: 2050 PM)]

Network: 10 [un4.1p Network 2050 PM - Upgrade (Network Folder: 2050)]

u4.1p2 Maida Vale Road - Maida Vale Road on to Roe Highway - 2050 PM - Upgrade
Site Category: (None)
Roundabout

OD Arrival Flows

U4	U3	U2	U1	HV	LV	Tot	
1%	4%	1%	34%	0%	60%	1641	T1
2%	15%	1%	0%	0%	82%	85	R3



	Tot	LV	HV	U1	U2	U3	U4
T1	515	41%	0%	55%	1%	3%	1%
L1	405	82%	0%	0%	1%	15%	2%

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (U1)	AustroadsClass 2 (
E: Maida Vale Road	920	541	0	282	9
W: Maida Vale Road	1726	1047	0	558	21
Total	2646	1588	0	840	30

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	AVERAGE BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Maida Vale Road															
Lane 1	405	17.0	405	17.0	1197	0.339	97[5]	3.9	LOS A	0.8	7.1	Full	120	0.0	0.0
Lane 2[d]	515	3.6	515	3.6	1470	0.350	100	4.0	LOS A	0.9	6.4	Full	120	0.0	0.0
Approach	920	9.5	920	9.5		0.350		4.0	LOS A	0.9	7.1				
West: Maida Vale Road															
Lane 1[d]	1641	5.3	1641	5.3	1746	0.940	100	3.6	LOS A	0.0	0.0	Full	160	0.0	0.0
Lane 2	85	17.0	85	17.0	1197	0.071	100	11.4	LOS B	0.0	0.0	Full	160	0.0	0.0
Approach	1726	5.9	1726	5.9		0.940		4.0	LOS A	0.0	0.0				
Intersection	2646	7.1	2646	7.1		0.940		4.0	LOS A	0.9	7.1				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

5 Lane under-utilisation found by the program
d Dominant lane on roundabout approach

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Project: C:\Users\User\Box\KCTT Projects\KC00000 Current Projects\KC00604.000 Forrestfield North - Structure Plan Preparation\Outgoing

\SIDRA\DCP\210708 Rev B\Future Models\KC00604.000 M01 Roe Highway - Maida Vale Road.sip9

USER REPORT FOR SITE

All Movement Classes

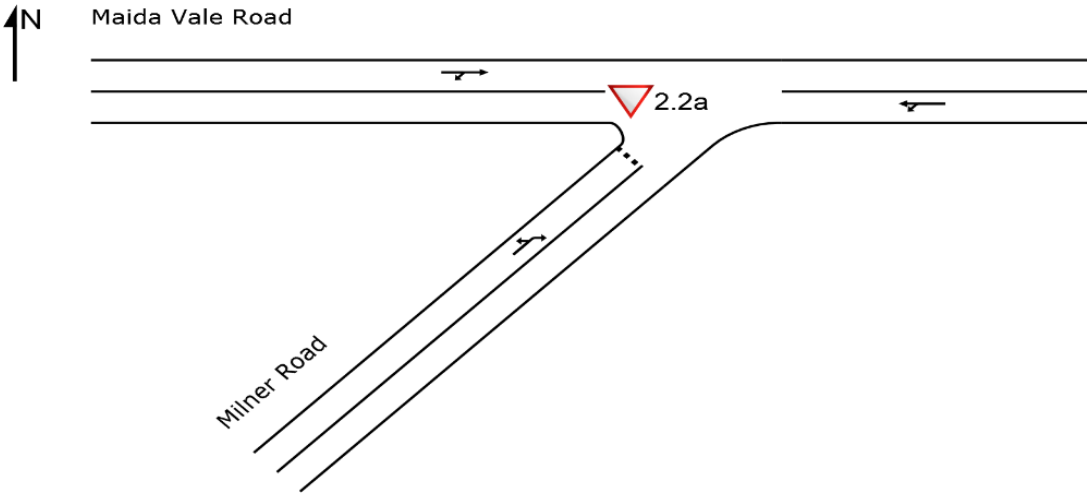
Project: KC00604.000 M02 Maida Vale Road - Milner Road **Template: Site User Report**

Site: 2.2a [2.2a Maida Vale Road - Milner Road - 2031 AM (Site Folder: 2031)]

2.2a Maida Vale Road - Milner Road - 2031 AM
Site Category: Existing Design
Give-Way (Two-Way)

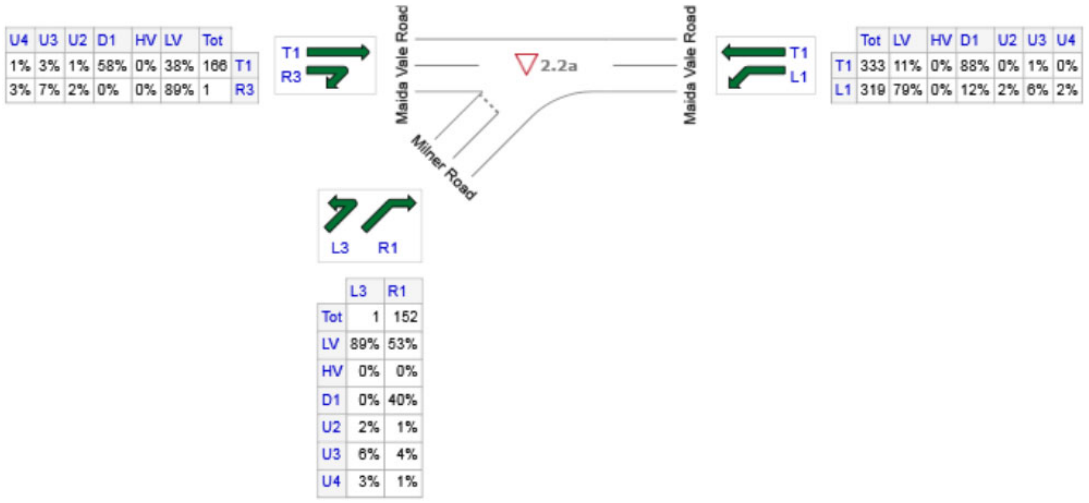
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (
E: Maida Vale Road	652	288	0	330	6
W: Maida Vale Road	167	64	0	96	1
SW: Milner Road	153	82	0	61	2
Total	972	434	0	487	9

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Maida Vale Road													
Lane 1	686	4.4	1880	0.365	100	2.7	LOS A	0.0	0.0	Full	350	0.0	0.0
Approach	686	4.4		0.365		2.7	NA	0.0	0.0				
West: Maida Vale Road													
Lane 1	176	3.4	1908	0.092	100	0.1	LOS A	0.0	0.1	Full	450	0.0	0.0
Approach	176	3.4		0.092		0.1	NA	0.0	0.1				
SouthWest: Milner Road													
Lane 1	161	5.4	763	0.211	100	8.7	LOS A	0.8	5.8	Full	220	0.0	0.0
Approach	161	5.4		0.211		8.7	LOS A	0.8	5.8				
Intersection	1023	4.4		0.365		3.2	NA	0.8	5.8				

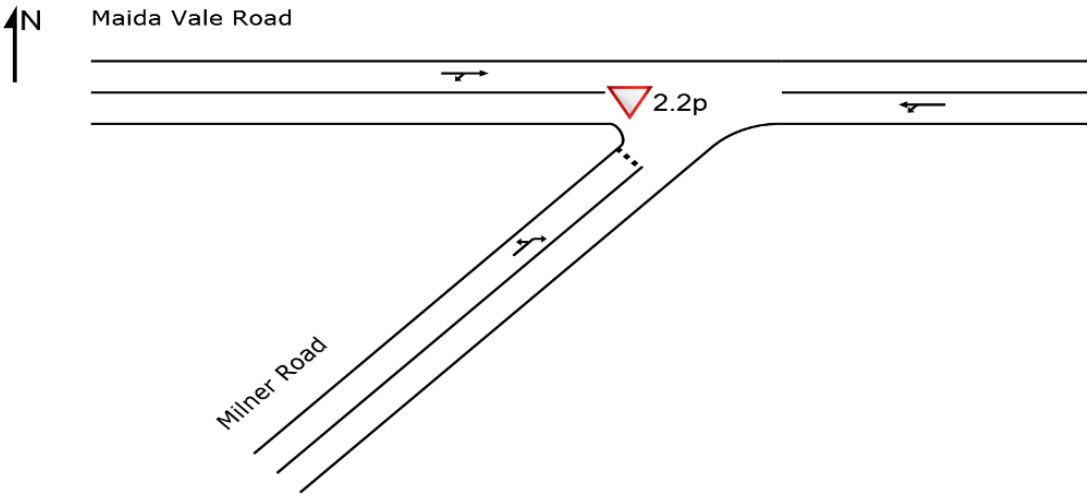
Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Site: 2.2p [2.2p Maida Vale Road - Milner Road - 2031 PM (Site Folder: 2031)]

2.2p Maida Vale Road - Milner Road - 2031 PM
Site Category: Existing Design
Give-Way (Two-Way)

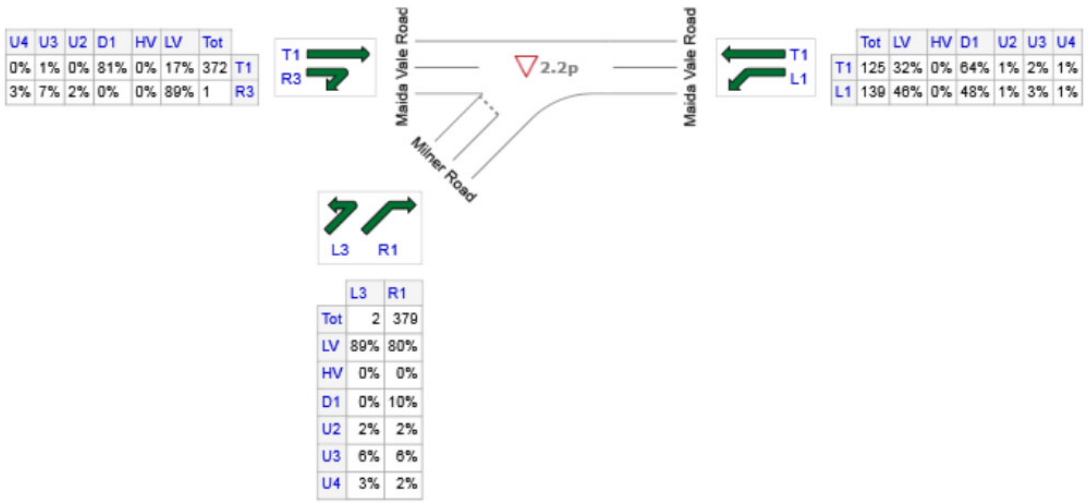
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



U4	U3	U2	D1	HV	LV	Tot	
0%	1%	0%	81%	0%	17%	372	T1
3%	7%	2%	0%	0%	89%	1	R3

	Tot	LV	HV	D1	U2	U3	U4
T1	125	32%	0%	64%	1%	2%	1%
L1	139	46%	0%	48%	1%	3%	1%

	L3	R1
Tot	2	379
LV	89%	80%
HV	0%	0%
D1	0%	10%
U2	2%	2%
U3	6%	6%
U4	3%	2%

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (
E: Maida Vale Road	264	105	0	147	2
W: Maida Vale Road	373	66	0	300	1
SW: Milner Road	381	306	0	38	6
Total	1018	477	0	485	10

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Maida Vale Road													
Lane 1	278	3.8	1895	0.147	100	2.9	LOS A	0.0	0.0	Full	350	0.0	0.0
Approach	278	3.8		0.147		2.9	NA	0.0	0.0				
West: Maida Vale Road													
Lane 1	393	1.6	1974	0.199	100	0.0	LOS A	0.0	0.1	Full	450	0.0	0.0
Approach	393	1.6		0.199		0.0	NA	0.0	0.1				
SouthWest: Milner Road													
Lane 1	401	8.1	844	0.475	100	10.0	LOS B	2.8	21.9	Full	220	0.0	0.0
Approach	401	8.1		0.475		10.0	LOS B	2.8	21.9				
Intersection	1072	4.6		0.475		4.5	NA	2.8	21.9				

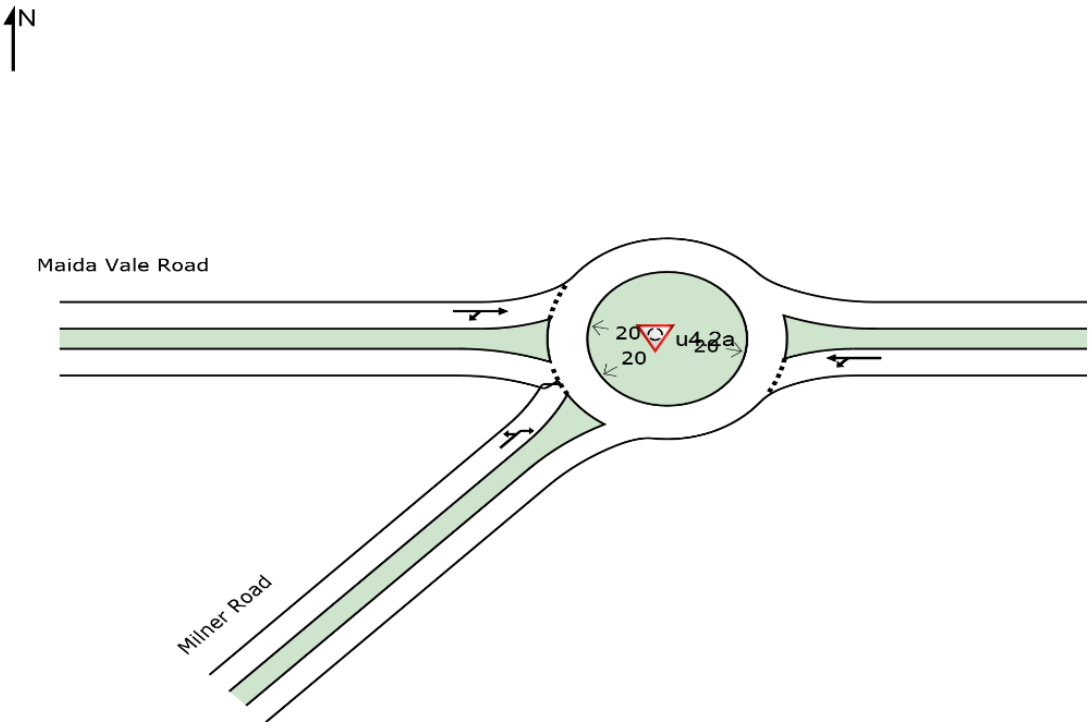
Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Site: u4.2a [u4.2a Maida Vale Road - Milner Road - 2050 AM - Upgrade (Site Folder: 2050)]

u4.2a Maida Vale Road - Milner Road - 2050 AM - Upgrade
Site Category: Existing Design
Roundabout

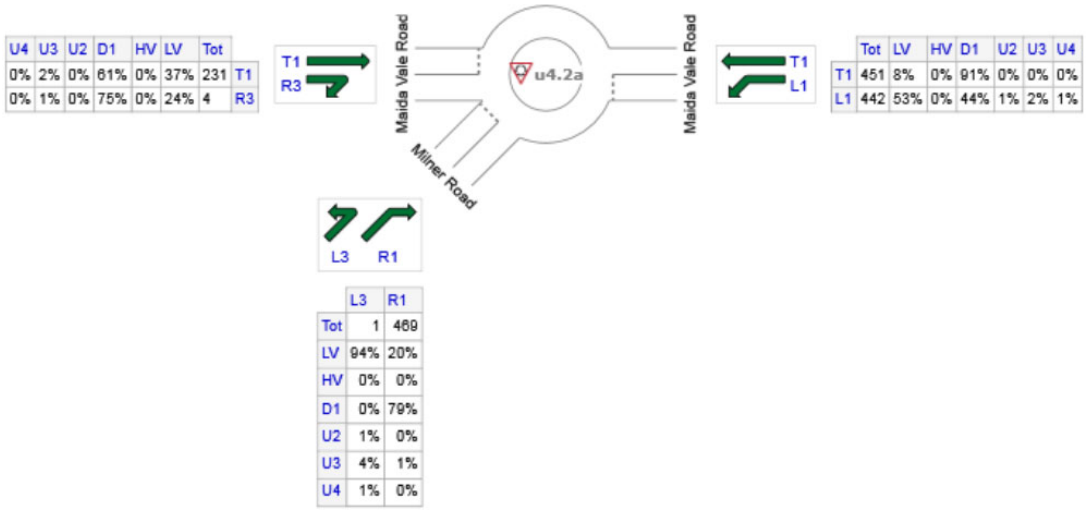
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (
E: Maida Vale Road	893	270	0	606	3
W: Maida Vale Road	235	86	0	143	1
SW: Milner Road	470	94	0	370	1
Total	1598	450	0	1119	5

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Maida Vale Road													
Lane 1[d]	940	1.6	1716	0.548	100	3.9	LOS A	6.1	43.5	Full	350	0.0	0.0
Approach	940	1.6		0.548		3.9	LOS A	6.1	43.5				
West: Maida Vale Road													
Lane 1[d]	247	2.0	861	0.287	100	7.2	LOS A	1.9	14.0	Full	450	0.0	0.0
Approach	247	2.0		0.287		7.2	LOS A	1.9	14.0				
SouthWest: Milner Road													
Lane 1[d]	495	1.1	927	0.533	100	12.9	LOS B	4.5	32.0	Full	220	0.0	0.0
Approach	495	1.1		0.533		12.9	LOS B	4.5	32.0				
Intersection	1682	1.5		0.548		7.1	LOS A	6.1	43.5				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

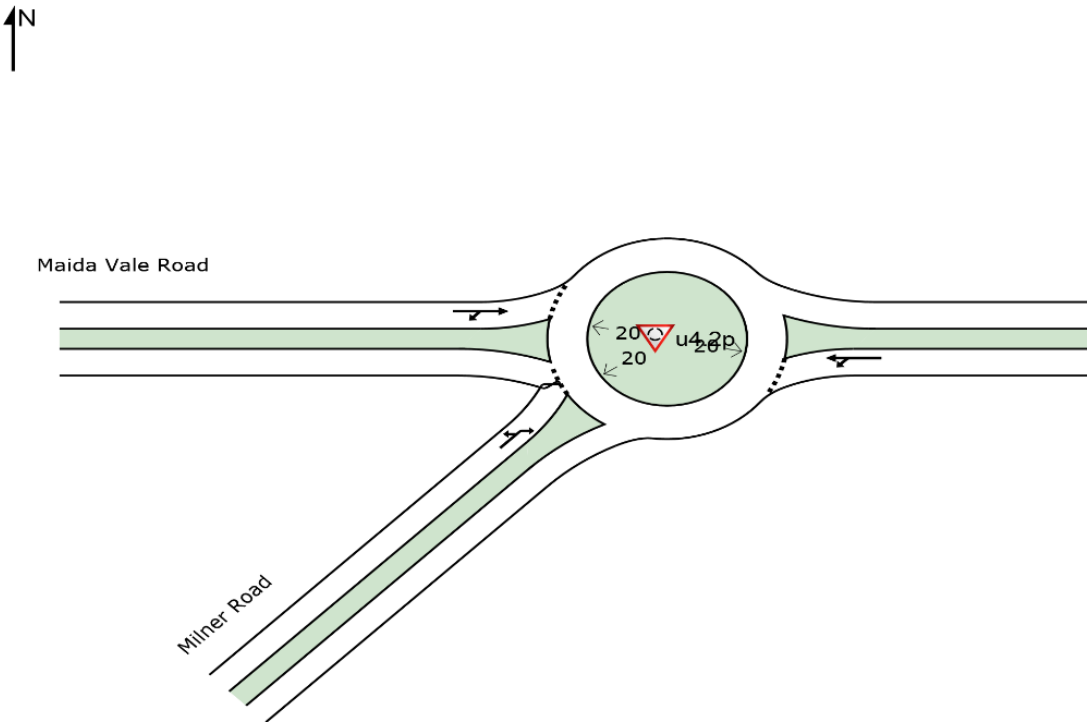
d Dominant lane on roundabout approach

Site: u4.2p [u4.2p Maida Vale Road - Milner Road - 2050 PM - Upgrade (Site Folder: 2050)]

u4.2p Maida Vale Road - Milner Road - 2050 PM - Upgrade
Site Category: Existing Design
Roundabout

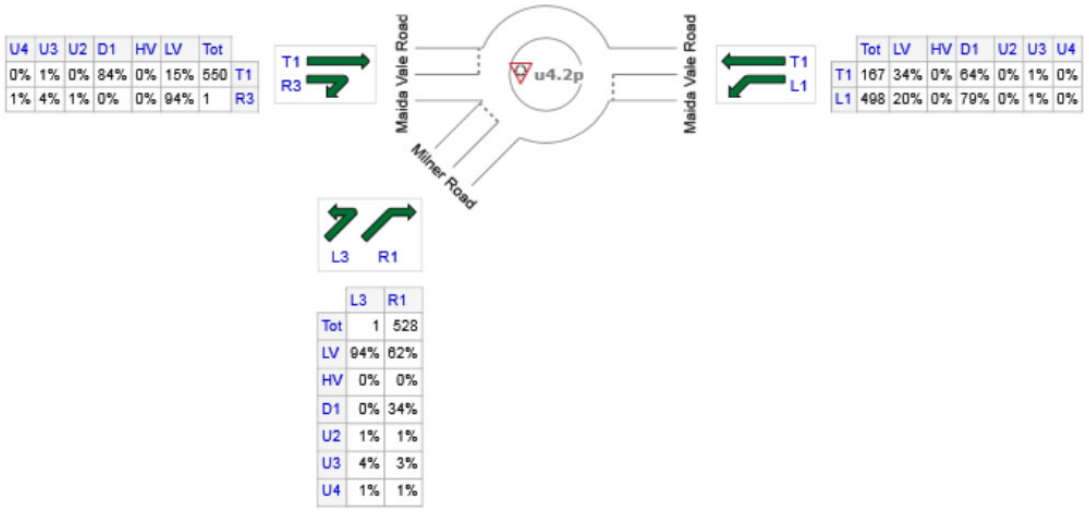
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (
E: Maida Vale Road	665	155	0	500	2
W: Maida Vale Road	551	82	0	464	1
SW: Milner Road	529	329	0	179	4
Total	1745	566	0	1143	6

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Maida Vale Road													
Lane 1[d]	700	1.2	1734	0.404	100	3.8	LOS A	3.8	27.4	Full	350	0.0	0.0
Approach	700	1.2		0.404		3.8	LOS A	3.8	27.4				
West: Maida Vale Road													
Lane 1[d]	580	0.8	848	0.684	100	12.4	LOS B	7.9	56.3	Full	450	0.0	0.0
Approach	580	0.8		0.684		12.4	LOS B	7.9	56.3				
SouthWest: Milner Road													
Lane 1[d]	557	3.3	1224	0.455	100	9.9	LOS A	3.4	25.2	Full	220	0.0	0.0
Approach	557	3.3		0.455		9.9	LOS A	3.4	25.2				
Intersection	1837	1.7		0.684		8.4	LOS A	7.9	56.3				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

d Dominant lane on roundabout approach

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USER REPORT FOR SITE

All Movement Classes

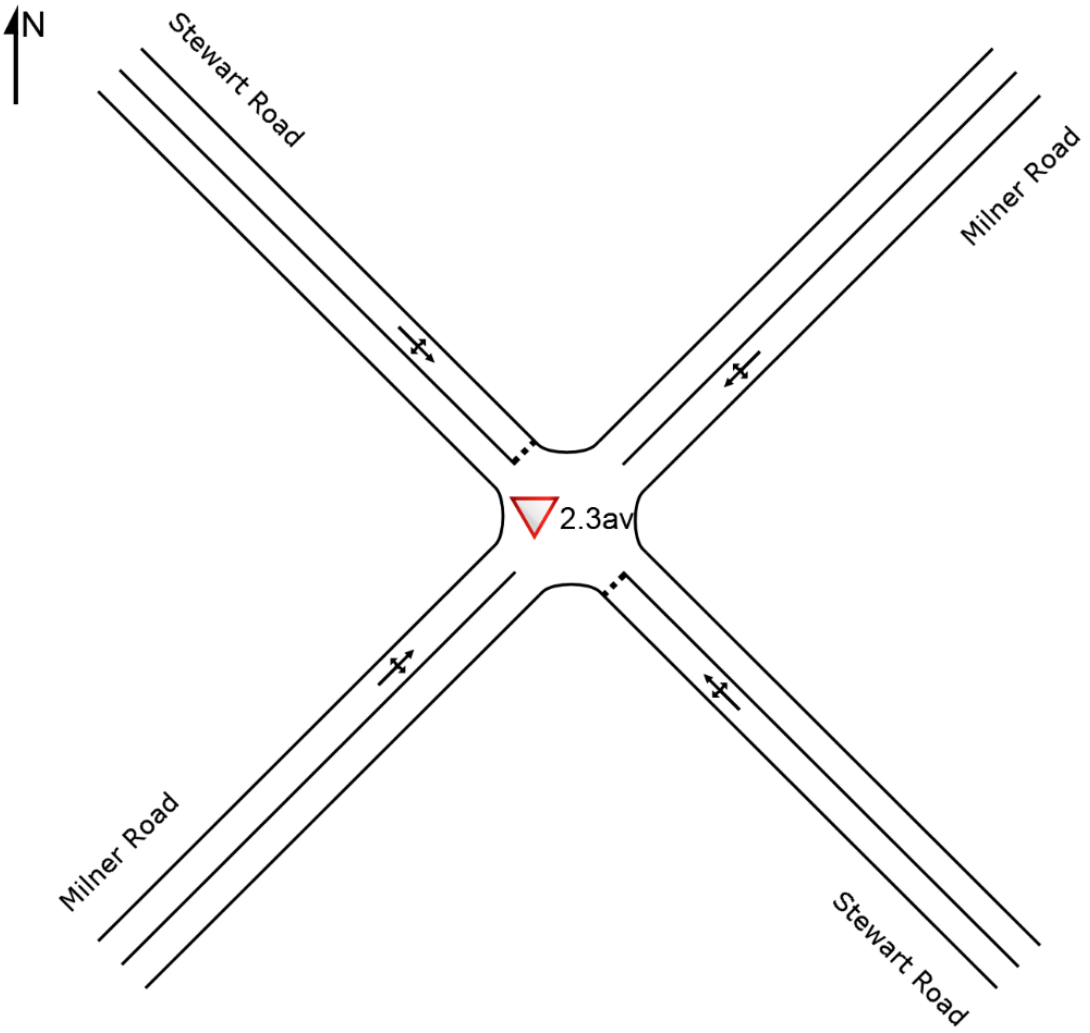
Project: KC00604.000 M03 Milner Road - Stewart Road **Template: Site User Report**

Site: 2.3av [2.3a Milner Road - Stewart Road - 2031 AM (Site Folder: 2031)]

2.3a Milner Road - Stewart Road - 2031 AM
Site Category: (None)
Give-Way (Two-Way)

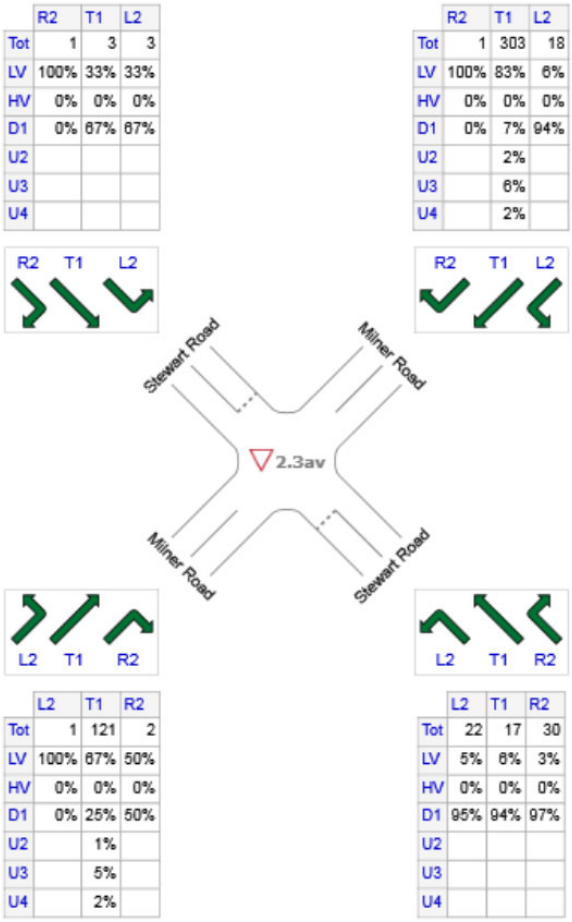
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Stewart Road	69	3	0	66	-
NE: Milner Road	322	254	0	38	5
NW: Stewart Road	7	3	0	4	-
SW: Milner Road	124	83	0	31	2
Total	522	343	0	139	7

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Stewart Road													
Lane 1	73	0.0	812	0.089	100	6.3	LOS A	0.3	2.2	Full	60	0.0	0.0
Approach	73	0.0		0.089		6.3	LOS A	0.3	2.2				
NorthEast: Milner Road													
Lane 1	339	7.9	1750	0.194	100	0.3	LOS A	0.0	0.1	Full	220	0.0	0.0
Approach	339	7.9		0.194		0.3	NA	0.0	0.1				
NorthWest: Stewart Road													
Lane 1	7	0.0	952	0.008	100	5.5	LOS A	0.0	0.2	Full	130	0.0	0.0

Approach	7	0.0		0.008		5.5	LOS A	0.0	0.2				
SouthWest: Milner Road													
Lane 1	131	6.6	1771	0.074	100	0.2	LOS A	0.0	0.2	Full	200	0.0	0.0
Approach	131	6.6		0.074		0.2	NA	0.0	0.2				
Intersection	549	6.4		0.194		1.2	NA	0.3	2.2				

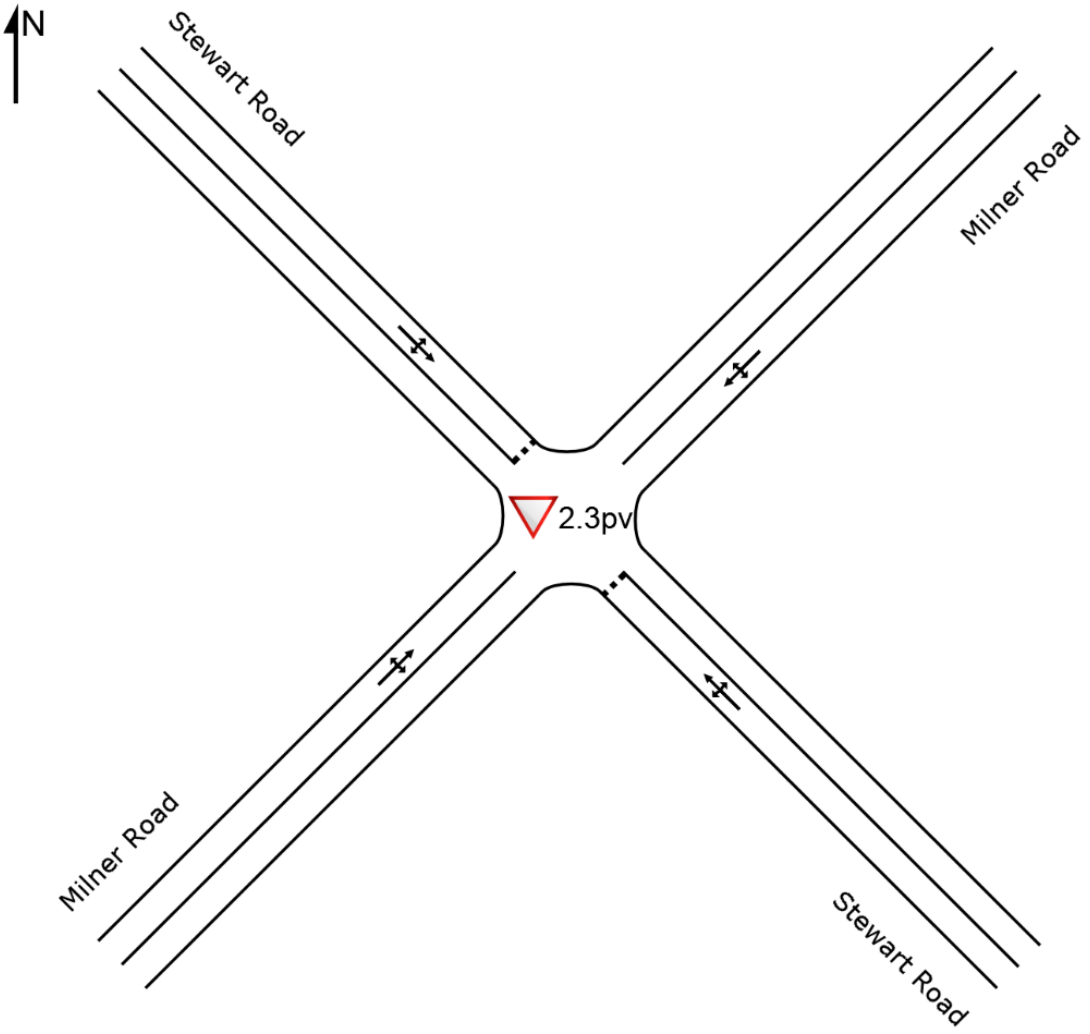
Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Site: 2.3pv [2.3p Milner Road - Stewart Road - 2031 PM (Site Folder: 2031)]

2.3p Milner Road - Stewart Road - 2031 PM
Site Category: (None)
Give-Way (Two-Way)

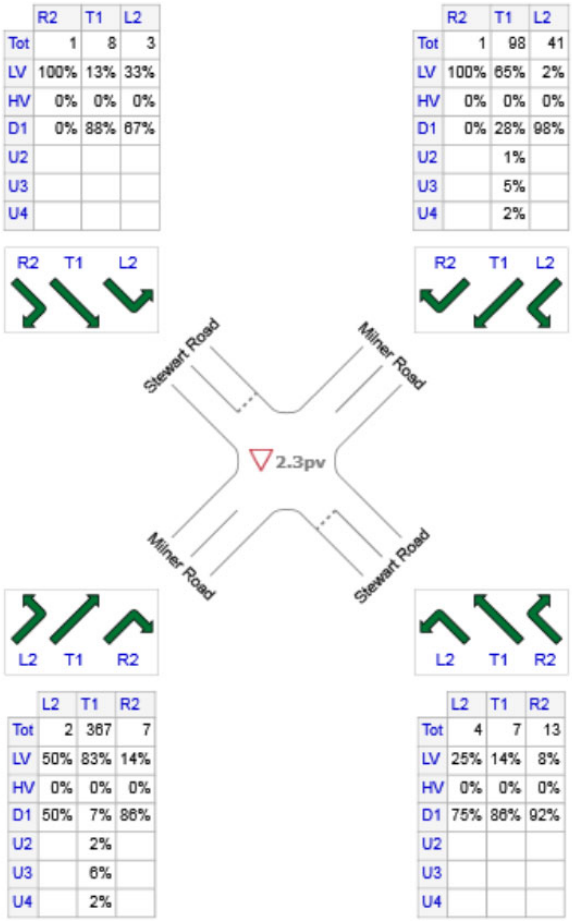
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Stewart Road	24	3	0	21	-
NE: Milner Road	140	65	0	67	1
NW: Stewart Road	12	3	0	9	-
SW: Milner Road	376	308	0	31	6
Total	552	379	0	128	7

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Stewart Road													
Lane 1	25	0.0	711	0.036	100	6.6	LOS A	0.1	0.8	Full	60	0.0	0.0
Approach	25	0.0		0.036		6.6	LOS A	0.1	0.8				
NorthEast: Milner Road													
Lane 1	147	4.6	1800	0.082	100	1.7	LOS A	0.0	0.1	Full	220	0.0	0.0
Approach	147	4.6		0.082		1.7	NA	0.0	0.1				
NorthWest: Stewart Road													
Lane 1	13	0.0	786	0.016	100	6.1	LOS A	0.1	0.4	Full	130	0.0	0.0

Approach	13	0.0		0.016		6.1	LOS A	0.1	0.4				
SouthWest: Milner Road													
Lane 1	396	8.2	1743	0.227	100	0.2	LOS A	0.1	0.5	Full	200	0.0	0.0
Approach	396	8.2		0.227		0.2	NA	0.1	0.5				
Intersection	581	6.8		0.227		1.0	NA	0.1	0.8				

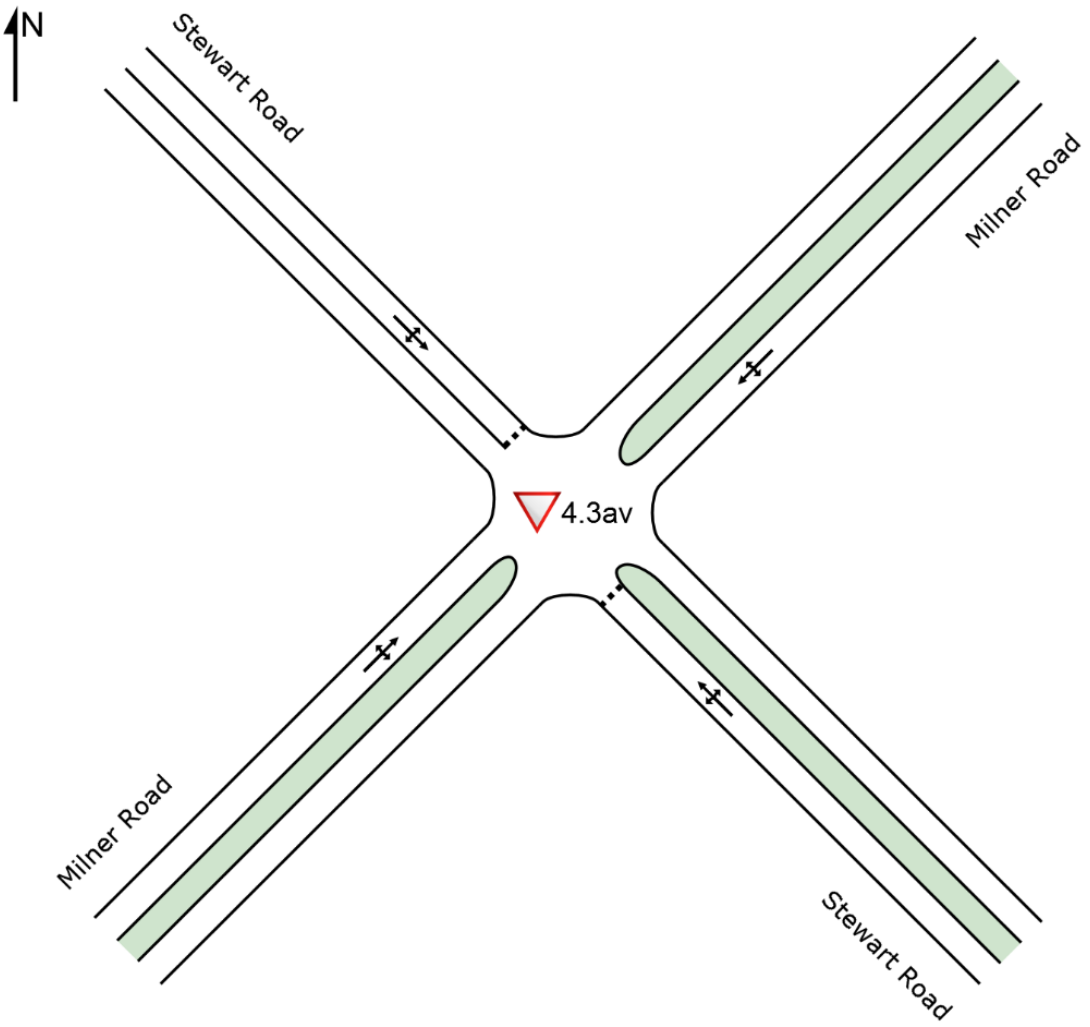
Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Site: 4.3av [4.3a Milner Road - Stewart Road - 2050 AM (Site Folder: 2050)]

4.3a Milner Road - Stewart Road - 2050 AM
Site Category: (None)
Give-Way (Two-Way)

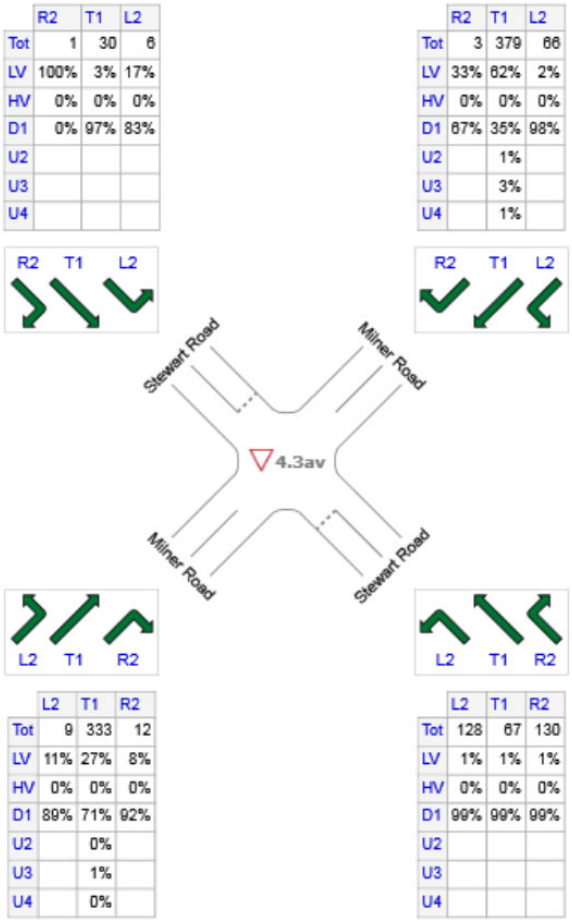
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Stewart Road	325	3	0	322	-
NE: Milner Road	448	235	0	198	2
NW: Stewart Road	37	3	0	34	-
SW: Milner Road	354	93	0	255	1
Total	1164	334	0	809	3

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Stewart Road													
Lane 1	342	0.0	584	0.586	100	12.8	LOS B	3.7	26.0	Full	60	0.0	0.0
Approach	342	0.0		0.586		12.8	LOS B	3.7	26.0				
NorthEast: Milner Road													
Lane 1	472	2.8	1861	0.253	100	0.9	LOS A	0.0	0.3	Full	220	0.0	0.0
Approach	472	2.8		0.253		0.9	NA	0.0	0.3				
NorthWest: Stewart Road													
Lane 1	39	0.0	556	0.070	100	8.3	LOS A	0.2	1.7	Full	130	0.0	0.0

Approach	39	0.0		0.070		8.3	LOS A	0.2	1.7				
SouthWest: Milner Road													
Lane 1	373	1.4	1864	0.200	100	0.6	LOS A	0.2	1.2	Full	200	0.0	0.0
Approach	373	1.4		0.200		0.6	NA	0.2	1.2				
Intersection	1225	1.5		0.586		4.3	NA	3.7	26.0				

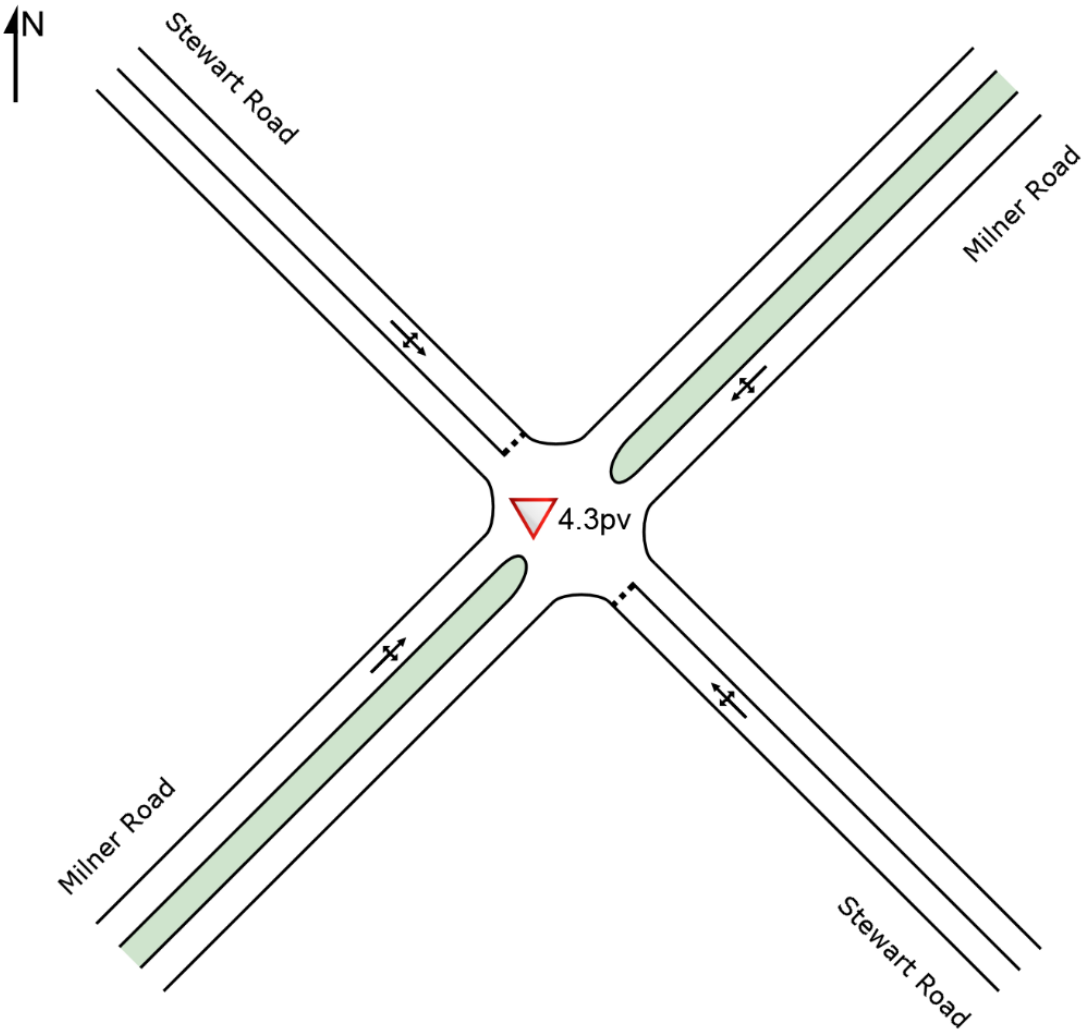
Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Site: 4.3pv [4.3p Milner Road - Stewart Road - 2050 PM (Site Folder: 2050)]

4.3p Milner Road - Stewart Road - 2050 PM
Site Category: (None)
Give-Way (Two-Way)

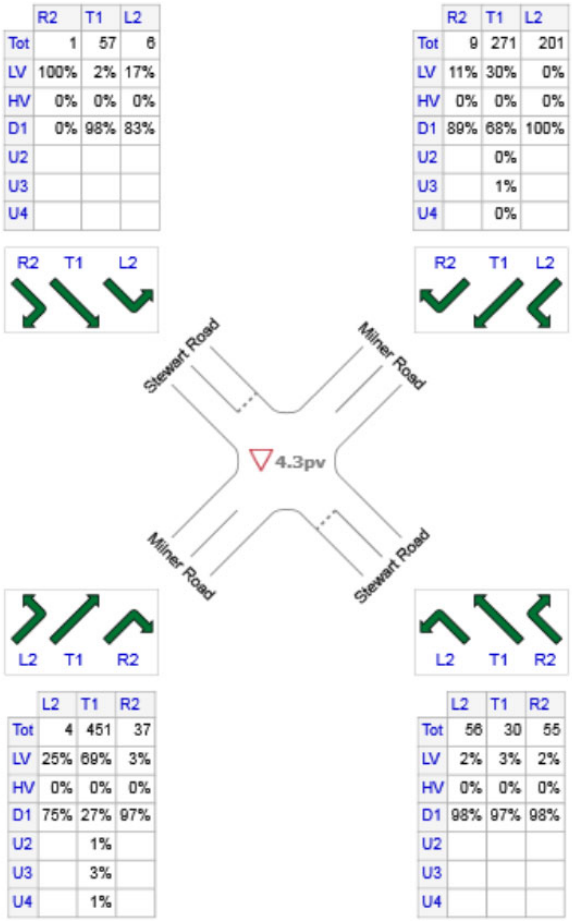
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Stewart Road	141	3	0	138	-
NE: Milner Road	481	83	0	393	1
NW: Stewart Road	64	3	0	61	-
SW: Milner Road	492	313	0	159	3
Total	1178	402	0	751	4

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Stewart Road													
Lane 1	148	0.0	508	0.292	100	10.6	LOS B	1.2	8.1	Full	60	0.0	0.0
Approach	148	0.0		0.292		10.6	LOS B	1.2	8.1				
NorthEast: Milner Road													
Lane 1	506	0.9	1861	0.272	100	2.6	LOS A	0.2	1.2	Full	220	0.0	0.0
Approach	506	0.9		0.272		2.6	NA	0.2	1.2				
NorthWest: Stewart Road													
Lane 1	67	0.0	420	0.160	100	10.8	LOS B	0.5	3.7	Full	130	0.0	0.0

Approach	67	0.0		0.160		10.8	LOS B	0.5	3.7				
SouthWest: Milner Road													
Lane 1	518	3.4	1764	0.294	100	1.0	LOS A	0.5	3.9	Full	200	0.0	0.0
Approach	518	3.4		0.294		1.0	NA	0.5	3.9				
Intersection	1240	1.8		0.294		3.4	NA	1.2	8.1				

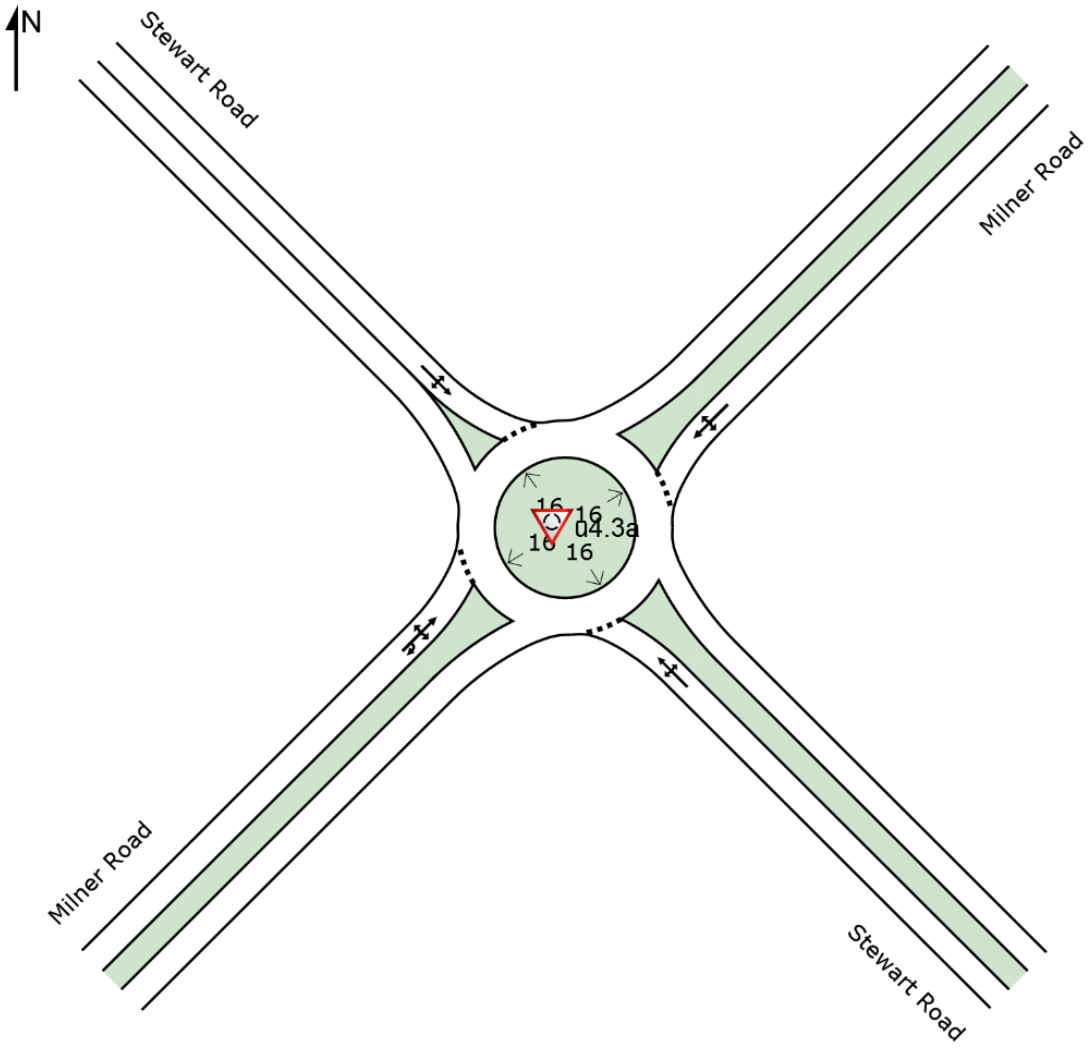
Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Site: u4.3a [u4.3a Milner Road - Stewart Road - 2050 AM - Upgrade (Site Folder: 2050 - Potential upgrade to Roundabout for safety reasons)]

u4.3a Milner Road - Stewart Road - 2050 AM - Upgrade
Site Category: (None)
Roundabout

Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.

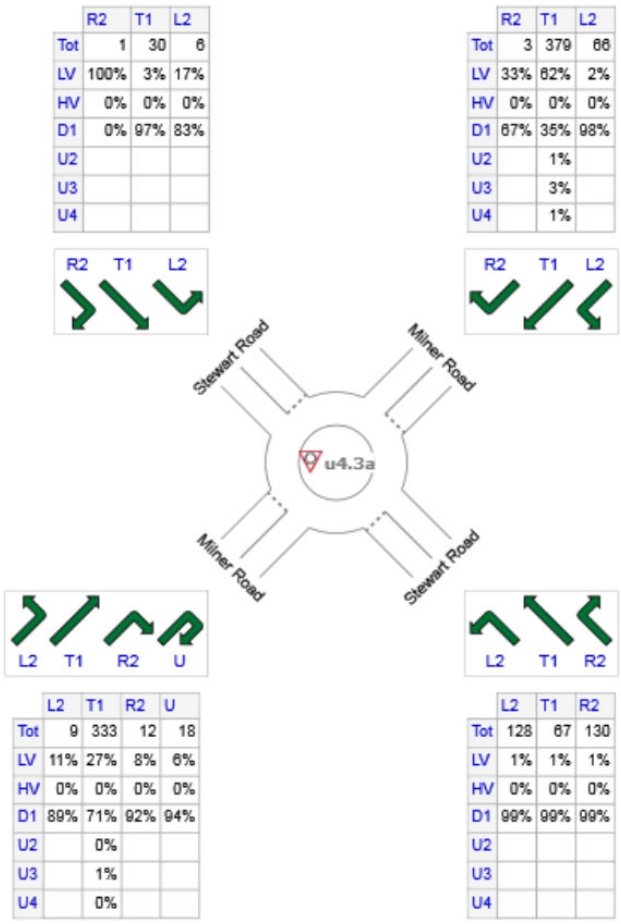


Input Volumes

Volume Display Method: Total and %

	R2	T1	L2
Tot	1	30	6
LV	100%	3%	17%
HV	0%	0%	0%
D1	0%	97%	83%
U2			
U3			
U4			

	R2	T1	L2
Tot	3	379	66
LV	33%	62%	2%
HV	0%	0%	0%
D1	67%	35%	98%
U2		1%	
U3		3%	
U4		1%	



	L2	T1	R2	U
Tot	9	333	12	18
LV	11%	27%	8%	6%
HV	0%	0%	0%	0%
D1	89%	71%	92%	94%
U2		0%		
U3		1%		
U4		0%		

	L2	T1	R2
Tot	128	67	130
LV	1%	1%	1%
HV	0%	0%	0%
D1	99%	99%	99%
U2			
U3			
U4			

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Stewart Road	325	3	0	322	-
NE: Milner Road	448	235	0	198	2
NW: Stewart Road	37	3	0	34	-
SW: Milner Road	372	94	0	272	1
Total	1182	335	0	826	3

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Stewart Road													
Lane 1[d]	342	0.0	901	0.380	100	8.1	LOS A	2.8	19.4	Full	60	0.0	0.0
Approach	342	0.0		0.380		8.1	LOS A	2.8	19.4				
NorthEast: Milner Road													
Lane 1[d]	472	2.8	1400	0.337	100	4.8	LOS A	2.6	19.0	Full	220	0.0	0.0
Approach	472	2.8		0.337		4.8	LOS A	2.6	19.0				

NorthWest: Stewart Road													
Lane 1[d]	39	0.0	815	0.048	100	6.6	LOS A	0.3	2.1	Full	130	0.0	0.0
Approach	39	0.0		0.048		6.6	LOS A	0.3	2.1				
SouthWest: Milner Road													
Lane 1[d]	392	1.3	1112	0.352	100	6.4	LOS A	2.8	19.7	Full	200	0.0	0.0
Approach	392	1.3		0.352		6.4	LOS A	2.8	19.7				
Intersection	1244	1.5		0.380		6.3	LOS A	2.8	19.7				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

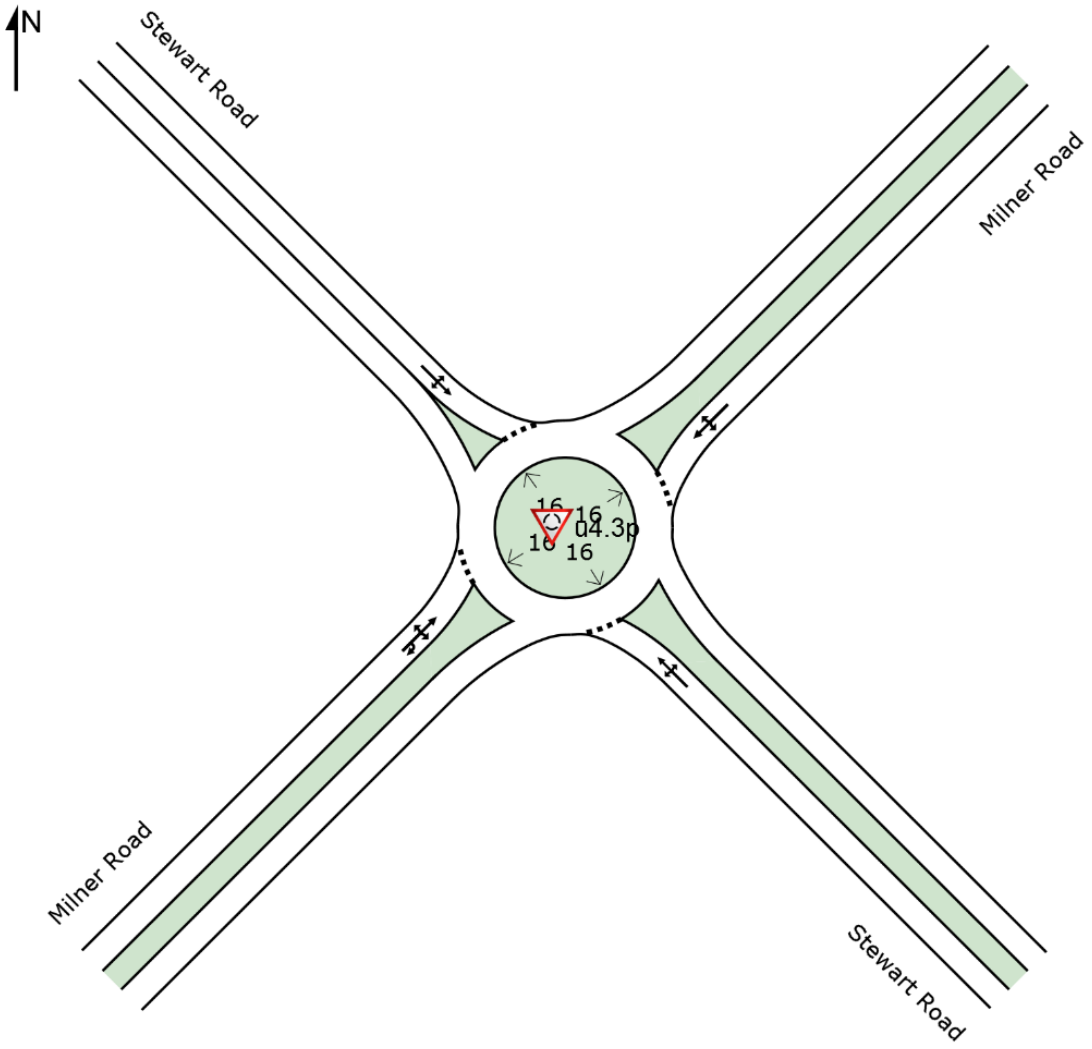
d Dominant lane on roundabout approach

Site: u4.3p [u4.3p Milner Road - Stewart Road - 2050 PM - Upgrade (Site Folder: 2050 - Potential upgrade to Roundabout for safety reasons)]

u4.3p Milner Road - Stewart Road - 2050 PM - Upgrade
Site Category: (None)
Roundabout

Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.

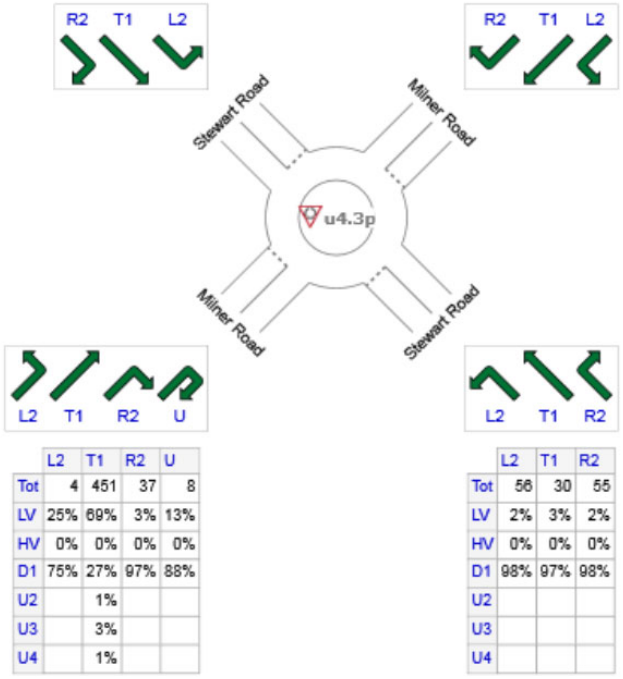


Input Volumes

Volume Display Method: Total and %

	R2	T1	L2
Tot	1	57	6
LV	100%	2%	17%
HV	0%	0%	0%
D1	0%	98%	83%
U2			
U3			
U4			

	R2	T1	L2
Tot	9	271	201
LV	11%	30%	0%
HV	0%	0%	0%
D1	89%	68%	100%
U2		0%	
U3		1%	
U4		0%	



	L2	T1	R2	U
Tot	4	451	37	8
LV	25%	69%	3%	13%
HV	0%	0%	0%	0%
D1	75%	27%	97%	88%
U2		1%		
U3		3%		
U4		1%		

	L2	T1	R2
Tot	56	30	55
LV	2%	3%	2%
HV	0%	0%	0%
D1	98%	97%	98%
U2			
U3			
U4			

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Stewart Road	141	3	0	138	-
NE: Milner Road	481	83	0	393	1
NW: Stewart Road	64	3	0	61	-
SW: Milner Road	500	314	0	166	3
Total	1186	403	0	758	4

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Stewart Road													
Lane 1[d]	148	0.0	999	0.149	100	6.5	LOS A	1.0	6.7	Full	60	0.0	0.0
Approach	148	0.0		0.149		6.5	LOS A	1.0	6.7				
NorthEast: Milner Road													
Lane 1[d]	506	0.9	1318	0.384	100	5.2	LOS A	3.1	22.0	Full	220	0.0	0.0
Approach	506	0.9		0.384		5.2	LOS A	3.1	22.0				

NorthWest: Stewart Road													
Lane 1[d]	67	0.0	769	0.088	100	7.3	LOS A	0.6	3.9	Full	130	0.0	0.0
Approach	67	0.0		0.088		7.3	LOS A	0.6	3.9				
SouthWest: Milner Road													
Lane 1[d]	526	3.3	1316	0.400	100	5.6	LOS A	3.4	24.8	Full	200	0.0	0.0
Approach	526	3.3		0.400		5.6	LOS A	3.4	24.8				
Intersection	1248	1.8		0.400		5.6	LOS A	3.4	24.8				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

d Dominant lane on roundabout approach

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Project: C:\Users\User\Box\KCTT Projects\KC00000 Current Projects\KC00604.000 Forrestfield North - Structure Plan Preparation\Outgoing \SIDRA\DCP\210708 Rev B\Future Models\KC00604.000 M03 Milner Road - Stewart Road.sip9

USER REPORT FOR SITE

All Movement Classes

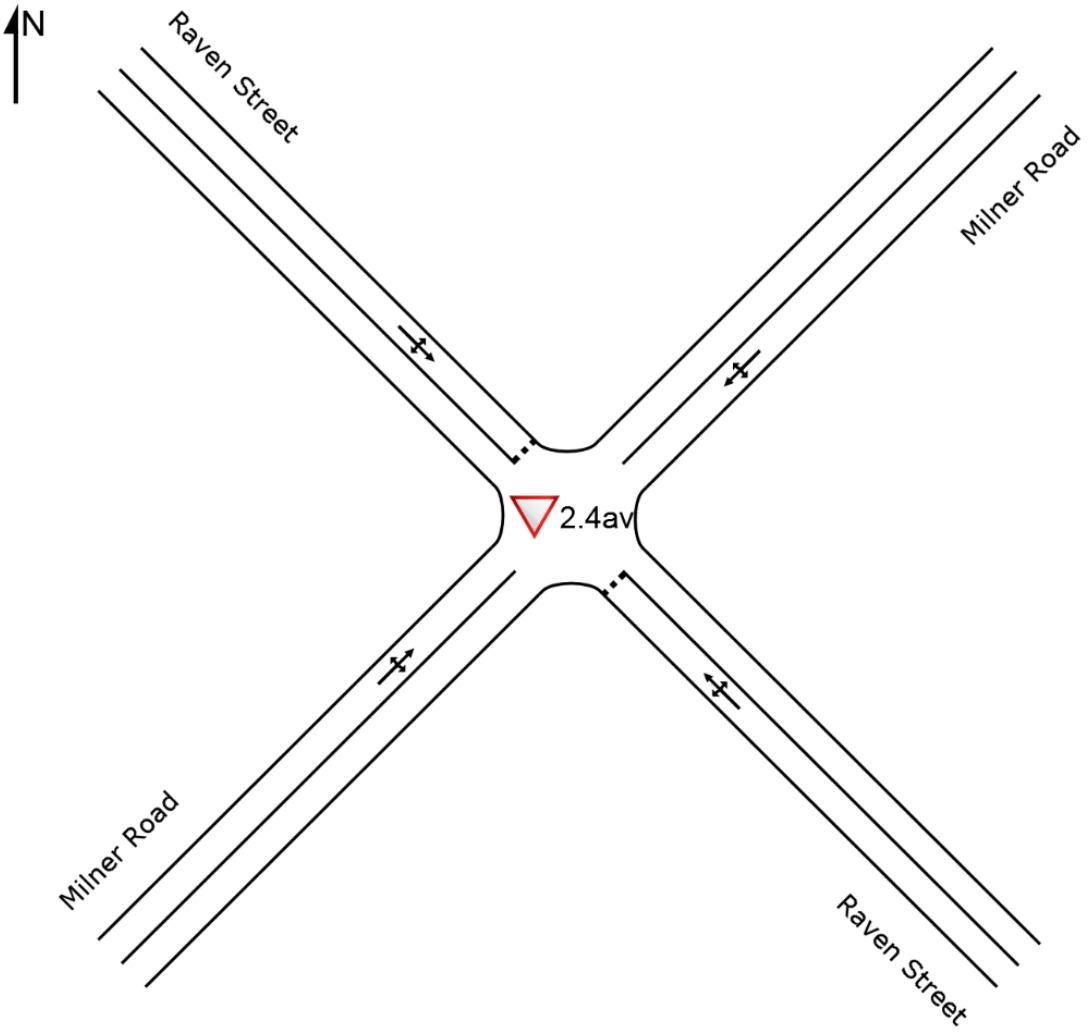
Project: KC00604.000 M04 Milner Road - Raven Street **Template: Site User Report**

Site: 2.4av [2.4a Milner Road - Raven Street - 2031 AM (Site Folder: 2031)]

2.4a Milner Road - Raven Street - 2031 AM
Site Category: (None)
Give-Way (Two-Way)

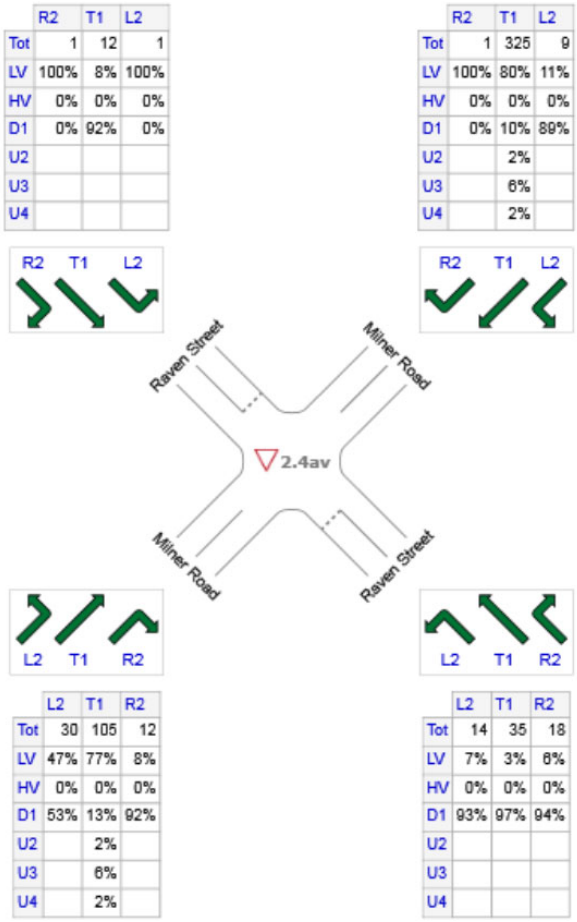
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Raven Street	67	3	0	64	-
NE: Milner Road	335	262	0	42	5
NW: Raven Street	14	3	0	11	-
SW: Milner Road	147	96	0	41	2
Total	563	364	0	158	7

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Raven Street													
Lane 1	71	0.0	784	0.090	100	6.4	LOS A	0.3	2.2	Full	150	0.0	0.0
Approach	71	0.0		0.090		6.4	LOS A	0.3	2.2				
NorthEast: Milner Road													
Lane 1	353	7.8	1754	0.201	100	0.2	LOS A	0.0	0.1	Full	240	0.0	0.0
Approach	353	7.8		0.201		0.2	NA	0.0	0.1				
NorthWest: Raven Street													
Lane 1	15	0.0	792	0.019	100	5.7	LOS A	0.1	0.4	Full	130	0.0	0.0

Approach	15	0.0		0.019		5.7	LOS A	0.1	0.4				
SouthWest: Milner Road													
Lane 1	155	5.6	1726	0.090	100	1.9	LOS A	0.1	1.1	Full	280	0.0	0.0
Approach	155	5.6		0.090		1.9	NA	0.1	1.1				
Intersection	593	6.1		0.201		1.5	NA	0.3	2.2				

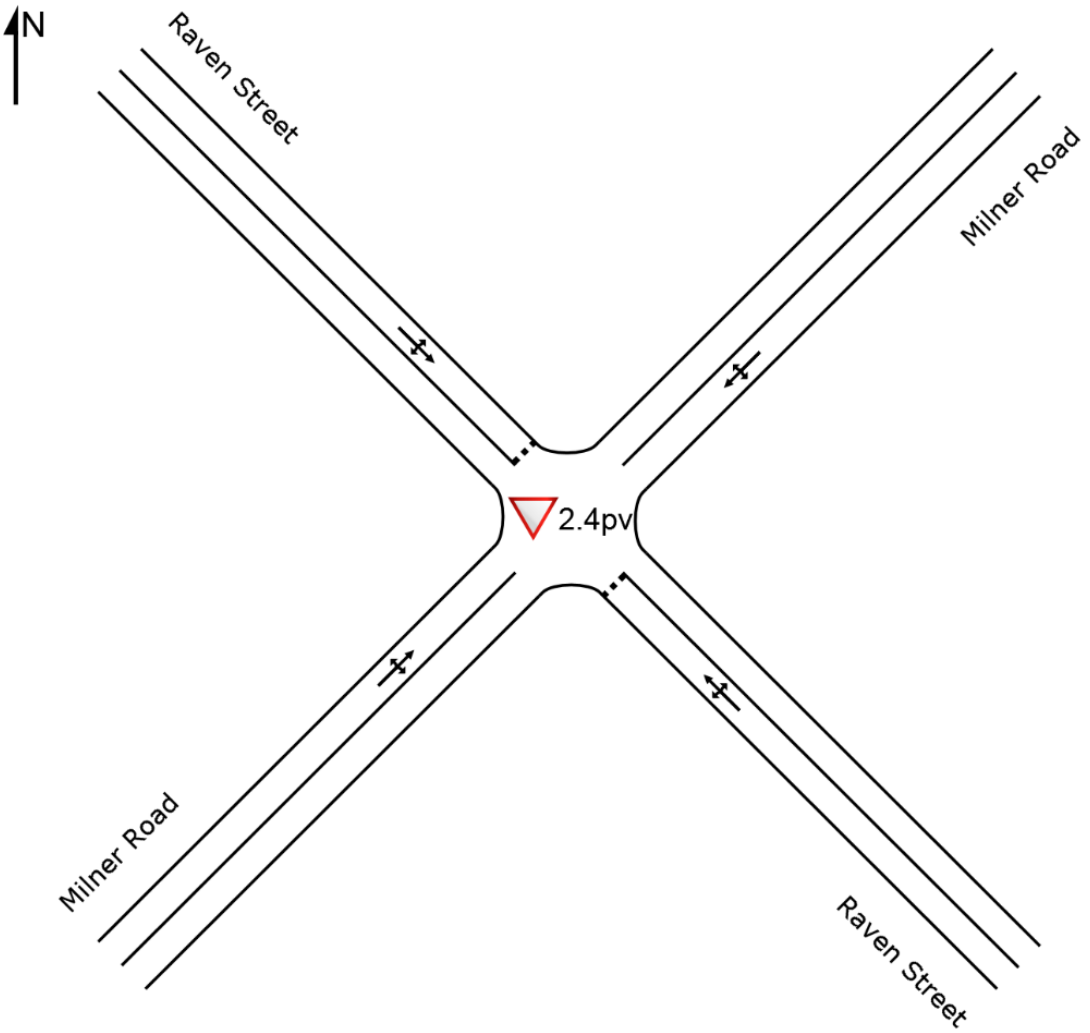
Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Site: 2.4pv [2.4p Milner Road - Raven Street - 2031 PM (Site Folder: 2031)]

2.4p Milner Road - Raven Street - 2031 PM
Site Category: (None)
Give-Way (Two-Way)

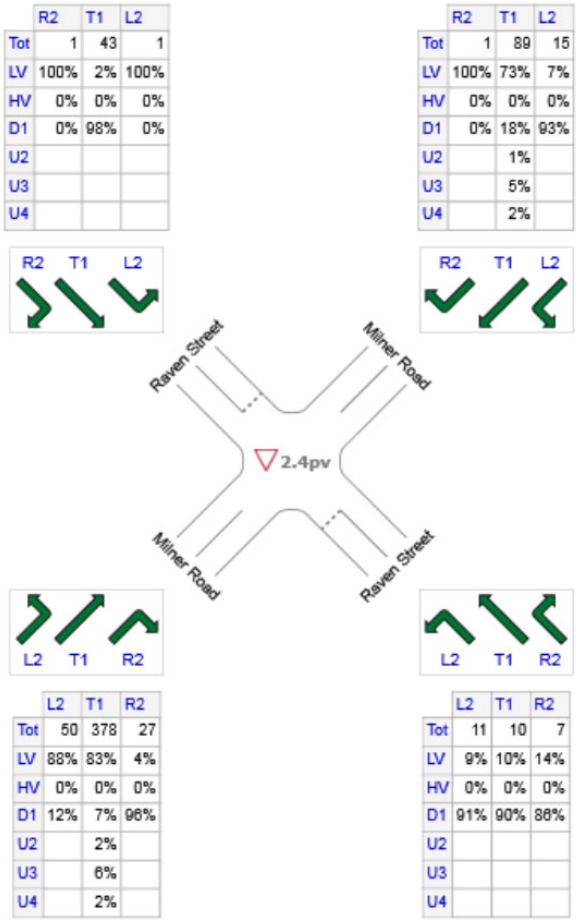
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Raven Street	28	3	0	25	-
NE: Milner Road	105	67	0	30	1
NW: Raven Street	45	3	0	42	-
SW: Milner Road	455	360	0	57	6
Total	633	433	0	154	8

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Raven Street													
Lane 1	29	0.0	818	0.036	100	6.3	LOS A	0.1	0.9	Full	150	0.0	0.0
Approach	29	0.0		0.036		6.3	LOS A	0.1	0.9				
NorthEast: Milner Road													
Lane 1	111	6.3	1770	0.062	100	0.9	LOS A	0.0	0.1	Full	240	0.0	0.0
Approach	111	6.3		0.062		0.9	NA	0.0	0.1				
NorthWest: Raven Street													
Lane 1	47	0.0	708	0.067	100	6.4	LOS A	0.2	1.6	Full	130	0.0	0.0

Approach	47	0.0		0.067		6.4	LOS A	0.2	1.6				
SouthWest: Milner Road													
Lane 1	479	7.0	1753	0.273	100	1.0	LOS A	0.3	2.2	Full	280	0.0	0.0
Approach	479	7.0		0.273		1.0	NA	0.3	2.2				
Intersection	666	6.1		0.273		1.6	NA	0.3	2.2				

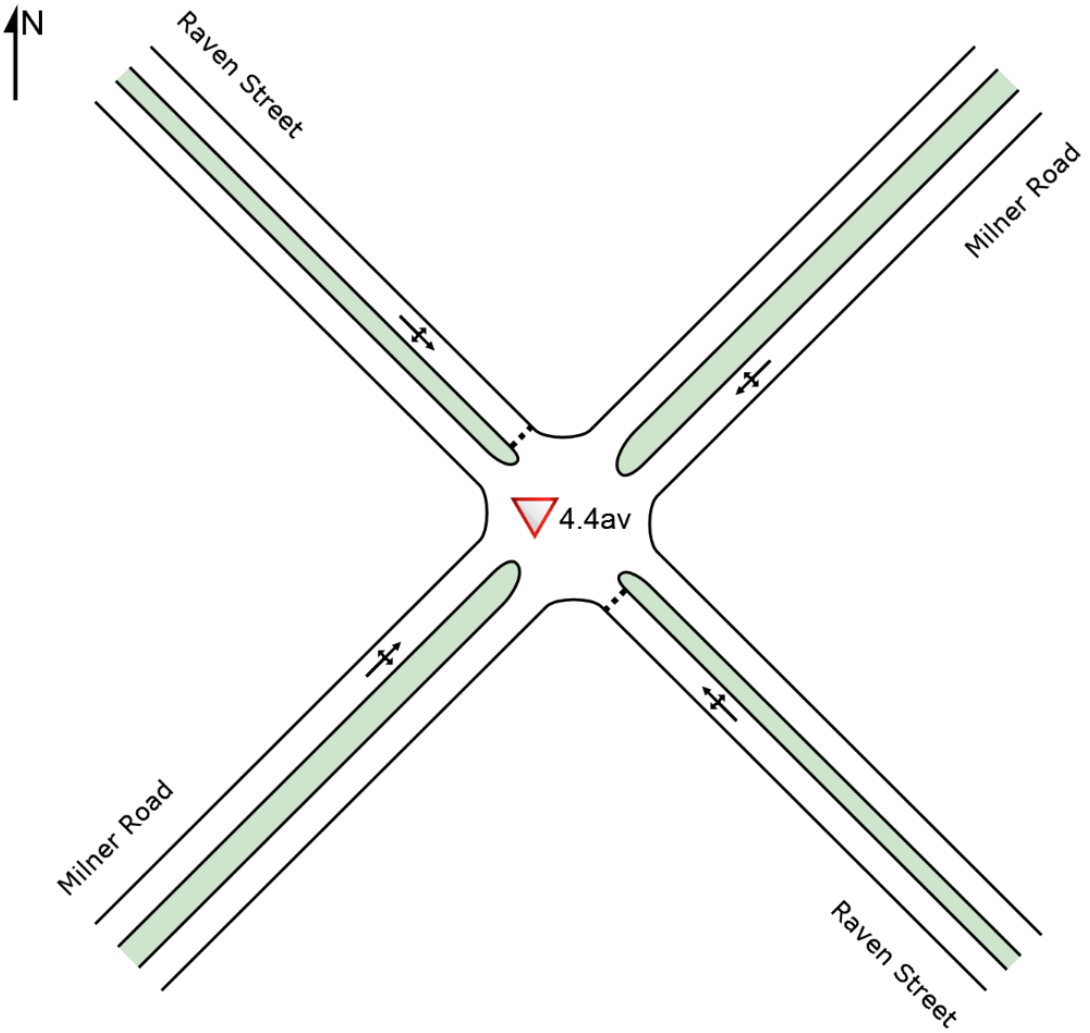
Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Site: 4.4av [4.4a Milner Road - Raven Street - 2050 AM (Site Folder: 2050)]

4.4a Milner Road - Raven Street - 2050 AM
Site Category: (None)
Give-Way (Two-Way)

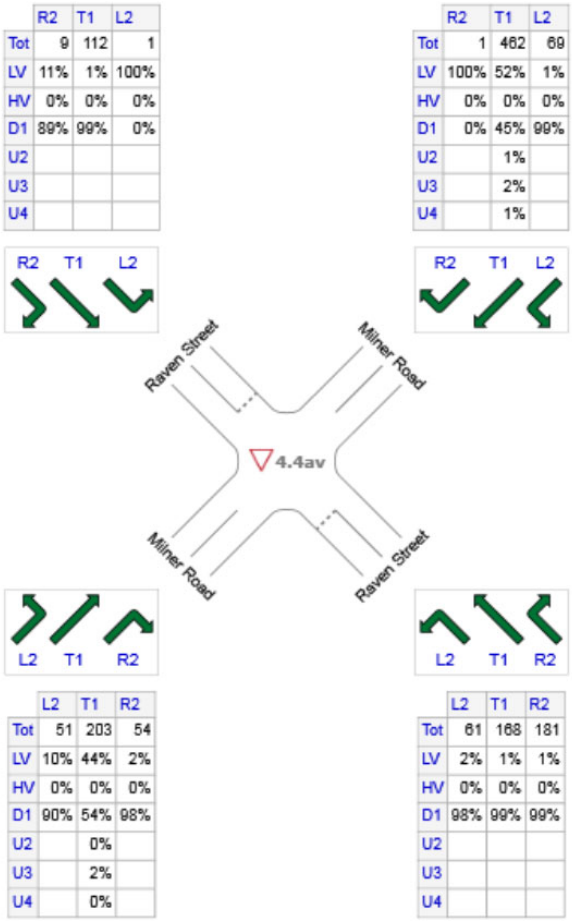
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Raven Street	410	3	0	407	-
NE: Milner Road	532	242	0	275	3
NW: Raven Street	122	3	0	119	-
SW: Milner Road	308	94	0	208	1
Total	1372	342	0	1009	3

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Raven Street													
Lane 1	432	0.0	458	0.942	100	38.2	LOS E	13.9	97.0	Full	150	0.0	0.0
Approach	432	0.0		0.942		38.2	LOS E	13.9	97.0				
NorthEast: Milner Road													
Lane 1	560	2.4	1877	0.298	100	0.7	LOS A	0.0	0.1	Full	240	0.0	0.0
Approach	560	2.4		0.298		0.7	NA	0.0	0.1				
NorthWest: Raven Street													
Lane 1	128	0.0	472	0.272	100	10.6	LOS B	1.0	7.3	Full	130	0.0	0.0

Approach	128	0.0		0.272		10.6	LOS B	1.0	7.3				
SouthWest: Milner Road													
Lane 1	324	1.5	1624	0.200	100	3.4	LOS A	0.8	5.8	Full	280	0.0	0.0
Approach	324	1.5		0.200		3.4	NA	0.8	5.8				
Intersection	1444	1.3		0.942		13.4	NA	13.9	97.0				

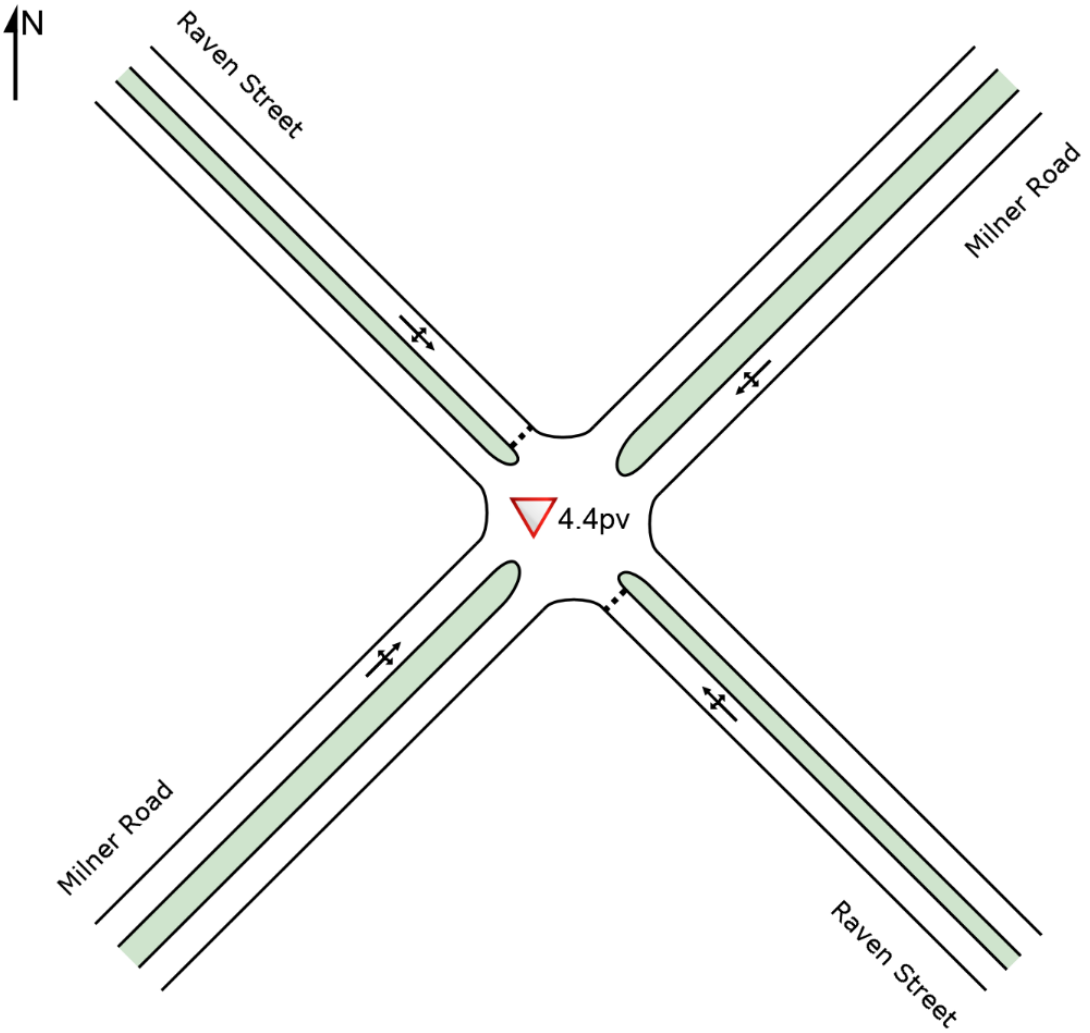
Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Site: 4.4pv [4.4p Milner Road - Raven Street - 2050 PM (Site Folder: 2050)]

4.4p Milner Road - Raven Street - 2050 PM
Site Category: (None)
Give-Way (Two-Way)

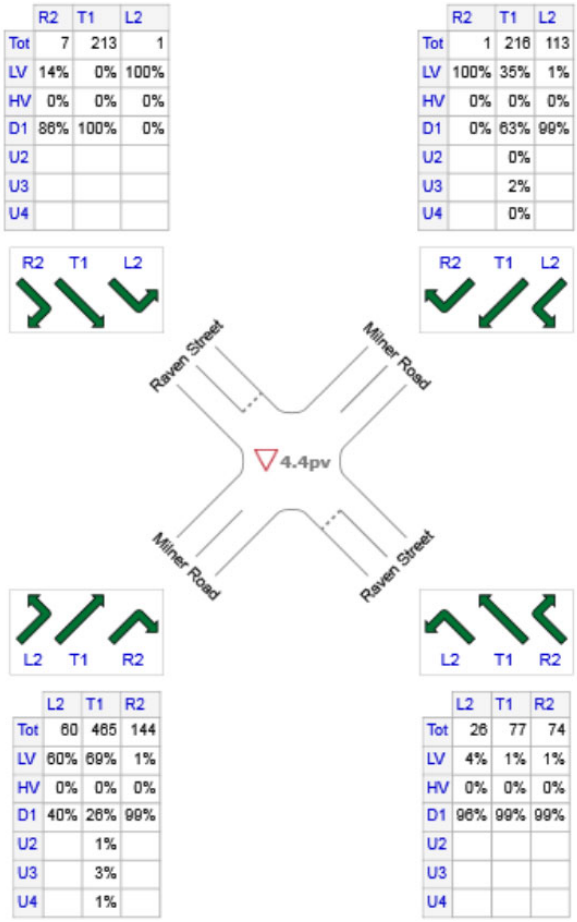
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Raven Street	177	3	0	174	-
NE: Milner Road	330	78	0	247	1
NW: Raven Street	221	3	0	218	-
SW: Milner Road	669	358	0	290	3
Total	1397	443	0	929	4

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Raven Street													
Lane 1	186	0.0	342	0.545	100	18.8	LOS C	2.5	17.8	Full	150	0.0	0.0
Approach	186	0.0		0.545		18.8	LOS C	2.5	17.8				
NorthEast: Milner Road													
Lane 1	347	1.2	1882	0.185	100	1.9	LOS A	0.0	0.1	Full	240	0.0	0.0
Approach	347	1.2		0.185		1.9	NA	0.0	0.1				
NorthWest: Raven Street													
Lane 1	233	0.0	371	0.626	100	18.5	LOS C	3.2	22.7	Full	130	0.0	0.0

Approach	233	0.0		0.626		18.5	LOS C	3.2	22.7				
SouthWest: Milner Road													
Lane 1	704	2.6	1698	0.415	100	2.9	LOS A	2.2	16.0	Full	280	0.0	0.0
Approach	704	2.6		0.415		2.9	NA	2.2	16.0				
Intersection	1471	1.5		0.626		7.2	NA	3.2	22.7				

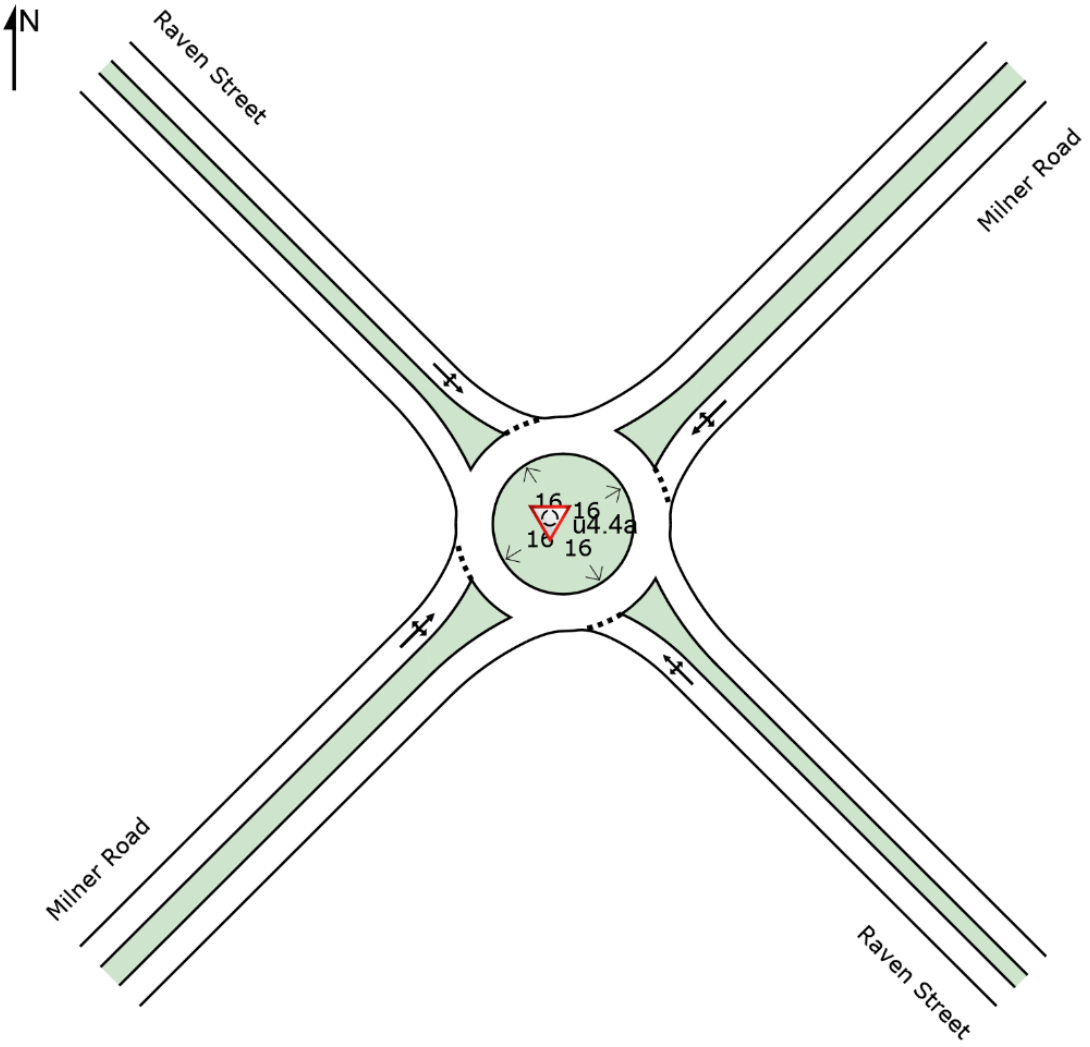
Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Site: u4.4a [u4.4a Milner Road - Raven Street - 2050 AM - Upgrade (Site Folder: 2050)]

u4.4a Milner Road - Raven Street - 2050 AM - Upgrade
Site Category: (None)
Roundabout

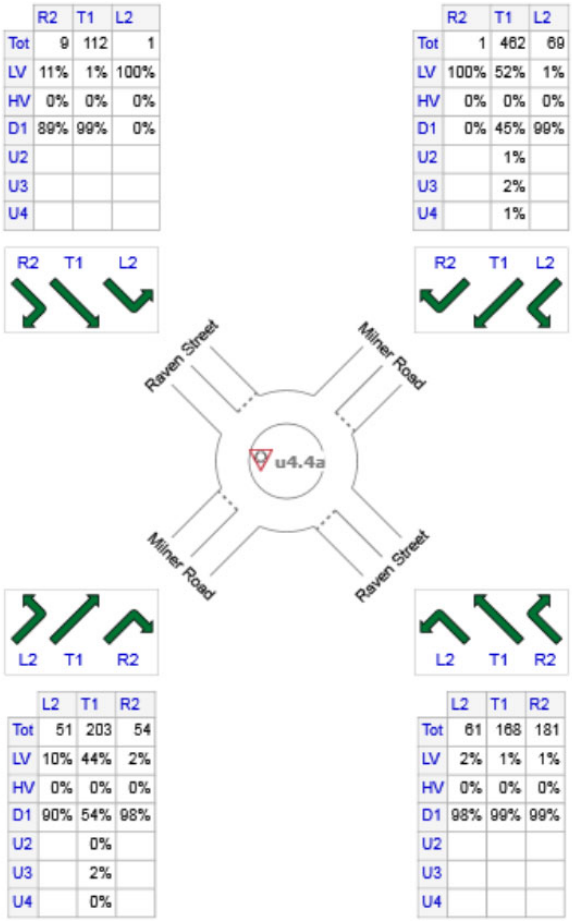
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Raven Street	410	3	0	407	-
NE: Milner Road	532	242	0	275	3
NW: Raven Street	122	3	0	119	-
SW: Milner Road	308	94	0	208	1
Total	1372	342	0	1009	3

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Raven Street													
Lane 1[d]	432	0.0	808	0.534	100	11.0	LOS B	5.0	35.2	Full	150	0.0	0.0
Approach	432	0.0		0.534		11.0	LOS B	5.0	35.2				
NorthEast: Milner Road													
Lane 1[d]	560	2.4	1176	0.476	100	6.0	LOS A	4.1	29.7	Full	240	0.0	0.0
Approach	560	2.4		0.476		6.0	LOS A	4.1	29.7				
NorthWest: Raven Street													
Lane 1[d]	128	0.0	857	0.150	100	6.7	LOS A	1.0	6.8	Full	130	0.0	0.0

Approach	128	0.0		0.150		6.7	LOS A	1.0	6.8				
SouthWest: Milner Road													
Lane 1[d]	324	1.5	907	0.357	100	7.9	LOS A	2.7	19.6	Full	280	0.0	0.0
Approach	324	1.5		0.357		7.9	LOS A	2.7	19.6				
Intersection	1444	1.3		0.534		8.0	LOS A	5.0	35.2				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

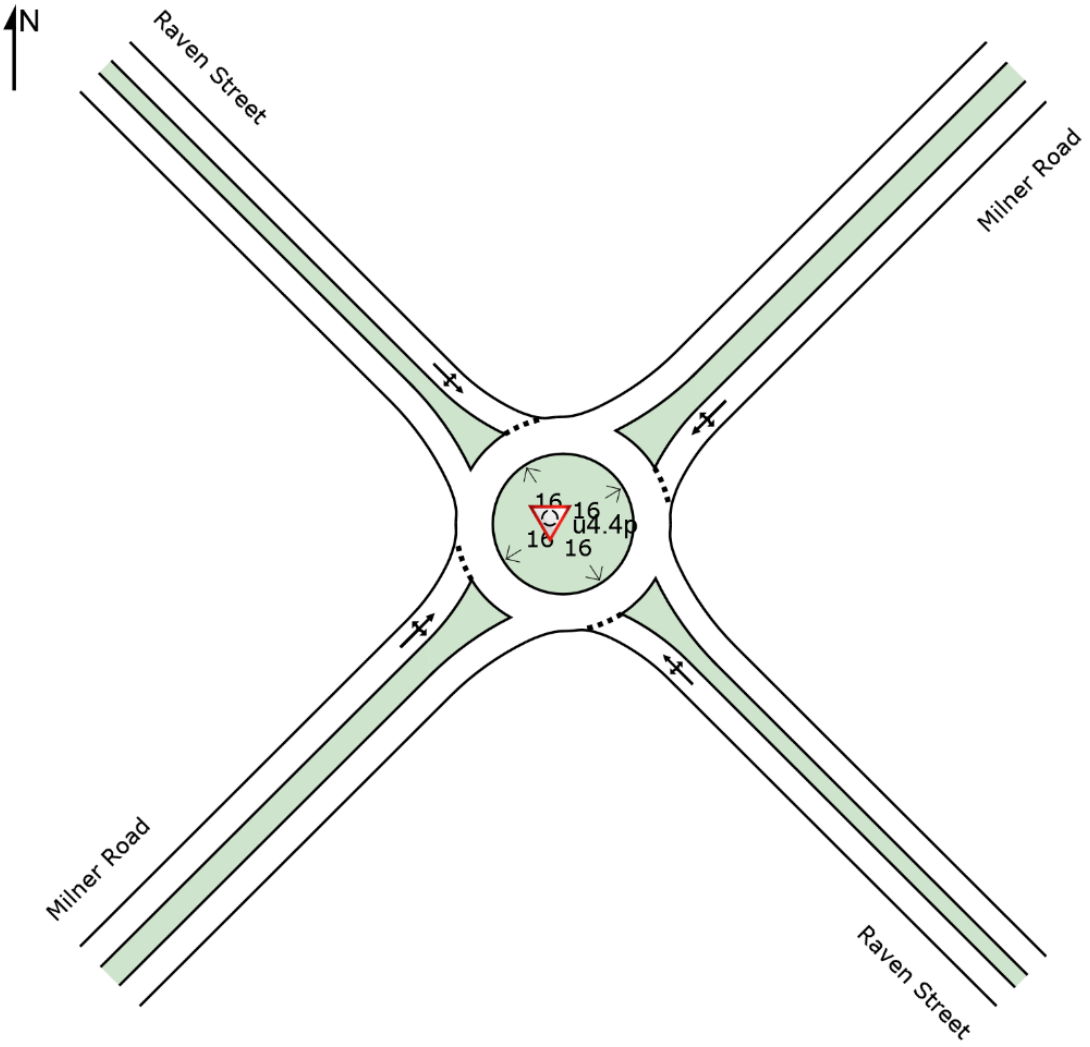
d Dominant lane on roundabout approach

Site: u4.4p [u4.4p Milner Road - Raven Street - 2050 PM - Upgrade (Site Folder: 2050)]

u4.4p Milner Road - Raven Street - 2050 PM - Upgrade
Site Category: (None)
Roundabout

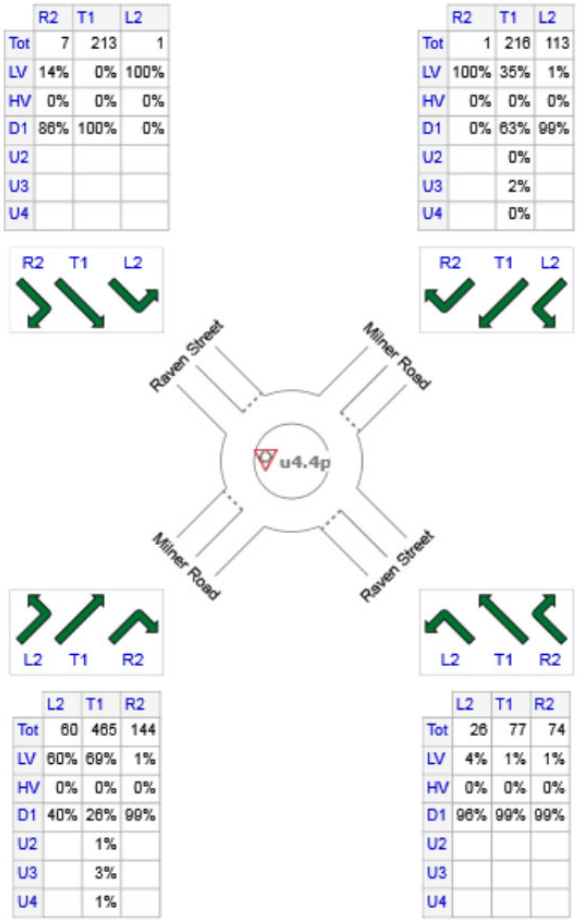
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U2
SE: Raven Street	177	3	0	174	-
NE: Milner Road	330	78	0	247	1
NW: Raven Street	221	3	0	218	-
SW: Milner Road	669	358	0	290	3
Total	1397	443	0	929	4

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Raven Street													
Lane 1[d]	186	0.0	1045	0.178	100	6.5	LOS A	1.2	8.6	Full	150	0.0	0.0
Approach	186	0.0		0.178		6.5	LOS A	1.2	8.6				
NorthEast: Milner Road													
Lane 1[d]	347	1.2	923	0.376	100	7.3	LOS A	2.8	19.7	Full	240	0.0	0.0
Approach	347	1.2		0.376		7.3	LOS A	2.8	19.7				
NorthWest: Raven Street													
Lane 1[d]	233	0.0	632	0.368	100	9.8	LOS A	2.8	19.3	Full	130	0.0	0.0

Approach	233	0.0		0.368		9.8	LOS A	2.8	19.3				
SouthWest: Milner Road													
Lane 1[d]	704	2.6	1224	0.575	100	6.9	LOS A	5.8	42.1	Full	280	0.0	0.0
Approach	704	2.6		0.575		6.9	LOS A	5.8	42.1				
Intersection	1471	1.5		0.575		7.4	LOS A	5.8	42.1				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

d Dominant lane on roundabout approach

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Organisation: KC TRAFFIC AND TRANSPORT | Licence: NETWORK / 1PC | Created: 30 July, 2021 12:31:31
Project: C:\Users\User\Box\KCTT Projects\KC00000 Current Projects\KC00604.000 Forrestfield North - Structure Plan Preparation\Outgoing \SIDRA\DCP\210708 Rev B\Future Models\KC00604.000 M04 Milner Road - Raven Street.sip9

USER REPORT FOR SITE

All Movement Classes

Project: KC00604.000 M05 Milner Road - TOD Connector **Template: Site User Report**

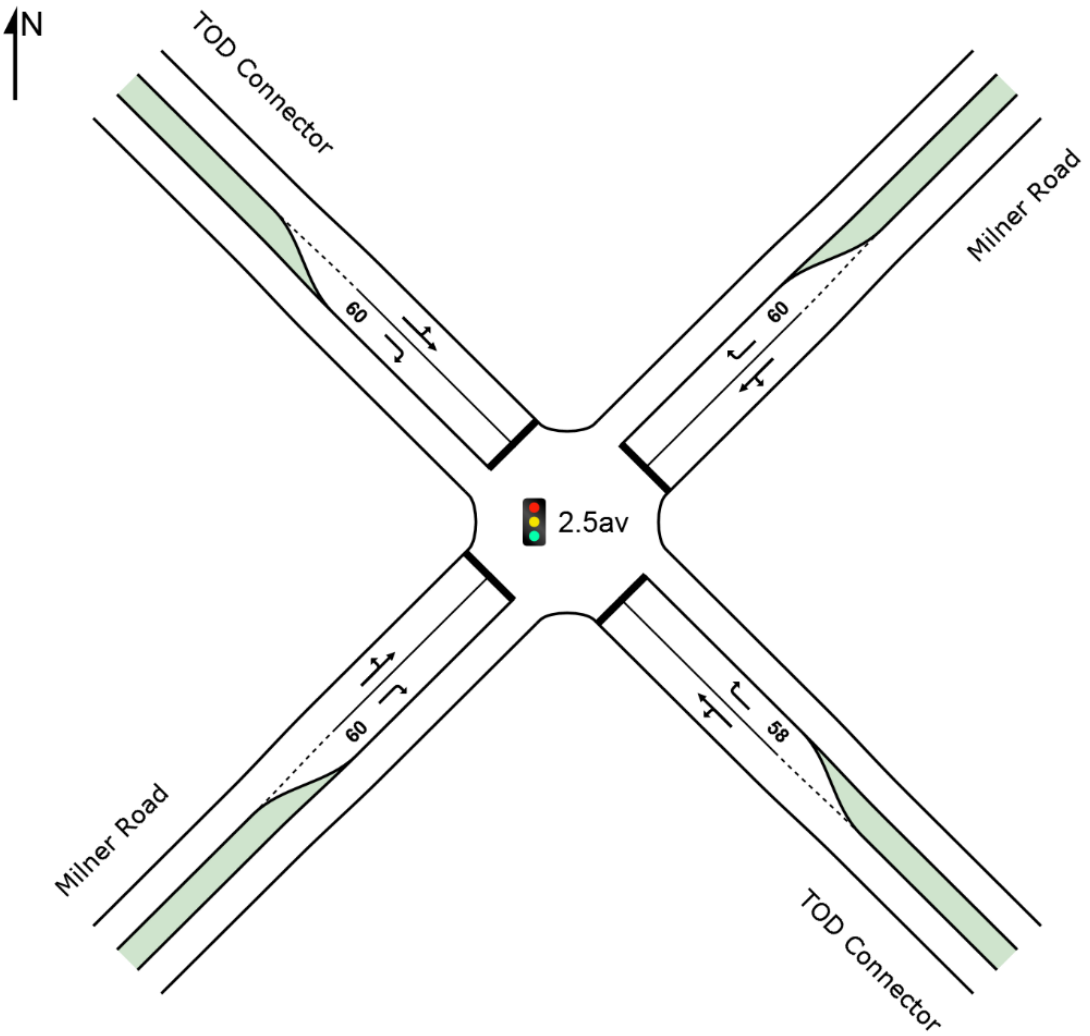
Site: 2.5av [2.5a Milner Road - TOD Connector - 2031 AM - Conversion (Site Folder: 2031)]

2.5a Milner Road - TOD Connector - 2031 AM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 30 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Phase Sequence: Convert Function Default
Reference Phase: Phase B
Input Phase Sequence: A, B
Output Phase Sequence: A, B

Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.

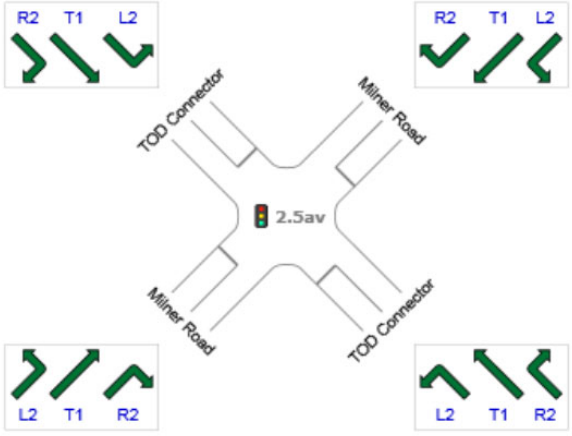


Input Volumes

Volume Display Method: Total and %

	R2	T1	L2
Tot	56	4	5
LV	87%	24%	19%
HV	0%	0%	0%
D1	11%	75%	80%
U2	2%	1%	0%
U3	1%	0%	0%
U4			

	R2	T1	L2
Tot	1	258	9
LV	92%	79%	10%
HV	0%	0%	0%
D1	0%	11%	89%
U2	2%	2%	0%
U3	7%	6%	1%
U4		2%	



	L2	T1	R2
Tot	1	93	5
LV	92%	80%	18%
HV	0%	0%	0%
D1	0%	11%	80%
U2	2%	2%	0%
U3	7%	6%	1%
U4		2%	

	L2	T1	R2
Tot	74	14	53
LV	58%	7%	48%
HV	0%	0%	0%
D1	41%	93%	51%
U2	1%	0%	1%
U3	1%	0%	0%
U4			

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (
SE: TOD Connector	141	69	0	70	1
NE: Milner Road	268	206	0	37	4
NW: TOD Connector	65	50	0	13	1
SW: Milner Road	99	76	0	14	2
Total	573	401	0	134	8

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: TOD Connector													
Lane 1	93	0.5	374	0.248	100	15.7	LOS B	1.2	8.8	Full	300	0.0	0.0
Lane 2	56	0.5	394	0.142	100	16.0	LOS B	0.7	5.1	Short	58	0.0	NA

Approach	148	0.5		0.248		15.8	LOS B	1.2	8.8				
NorthEast: Milner Road													
Lane 1	281	7.7	710	0.396	100	7.5	LOS A	3.1	24.3	Full	250	0.0	0.0
Lane 2	1	6.5	573	0.002	100	12.1	LOS B	0.0	0.1	Short	60	0.0	NA
Approach	282	7.7		0.396		7.5	LOS A	3.1	24.3				
NorthWest: TOD Connector													
Lane 1	9	0.2	381	0.025	100	13.5	LOS B	0.1	0.8	Full	155	0.0	0.0
Lane 2	59	0.9	334	0.176	100	17.1	LOS B	0.8	5.7	Short	60	0.0	NA
Approach	68	0.8		0.176		16.6	LOS B	0.8	5.7				
SouthWest: Milner Road													
Lane 1	99	8.0	708	0.140	100	6.5	LOS A	1.0	7.6	Full	120	0.0	0.0
Lane 2	5	1.3	452	0.012	100	13.8	LOS B	0.1	0.4	Short	60	0.0	NA
Approach	104	7.7		0.140		6.9	LOS A	1.0	7.6				
Intersection	603	5.2		0.396		10.5	LOS B	3.1	24.3				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

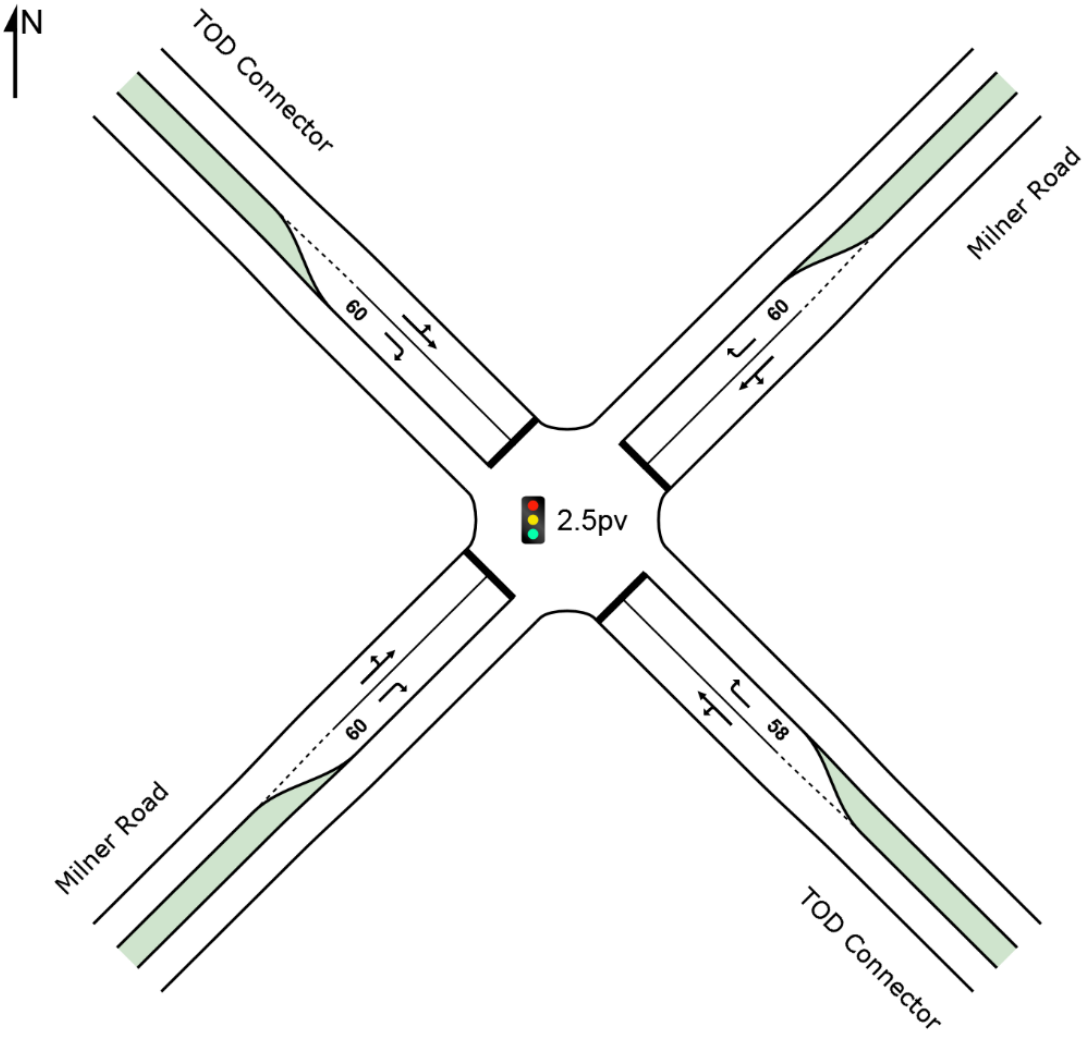
Site: 2.5pv [2.5p Milner Road - TOD Connector - 2031 PM - Conversion (Site Folder: 2031)]

2.5p Milner Road - TOD Connector - 2031 PM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 30 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Phase Sequence: Convert Function Default
Reference Phase: Phase B
Input Phase Sequence: A, B
Output Phase Sequence: A, B

Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.

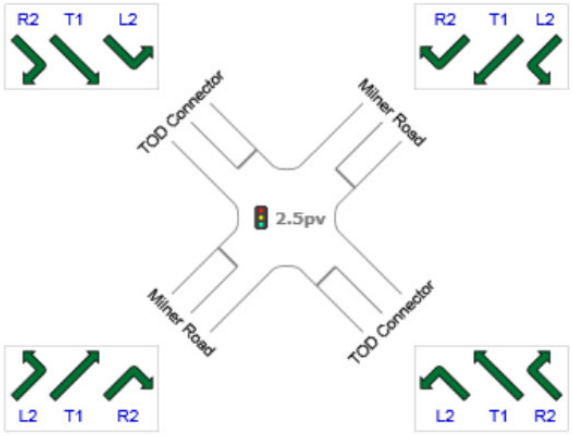


Input Volumes

Volume Display Method: Total and %

	R2	T1	L2
Tot	23	3	6
LV	76%	32%	16%
HV	0%	0%	0%
D1	22%	67%	83%
U2	2%	1%	0%
U3	1%	0%	0%
U4			

	R2	T1	L2
Tot	1	80	17
LV	92%	83%	5%
HV	0%	0%	0%
D1	0%	8%	94%
U2	2%	2%	0%
U3	7%	6%	0%
U4		2%	



	L2	T1	R2
Tot	3	360	40
LV	92%	81%	83%
HV	0%	0%	0%
D1	0%	9%	10%
U2	2%	2%	2%
U3	7%	6%	6%
U4		2%	

	L2	T1	R2
Tot	17	9	109
LV	6%	67%	81%
HV	0%	0%	0%
D1	94%	33%	17%
U2	0%	0%	2%
U3	0%	0%	1%
U4			

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (
SE: TOD Connector	135	95	0	37	2
NE: Milner Road	98	68	0	22	1
NW: TOD Connector	32	19	0	12	0
SW: Milner Road	403	327	0	38	7
Total	668	509	0	109	10

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: TOD Connector													
Lane 1	27	0.0	381	0.072	100	14.2	LOS B	0.3	2.4	Full	300	0.0	0.0
Lane 2	115	0.8	392	0.293	100	16.5	LOS B	1.5	11.0	Short	58	0.0	NA
Approach	142	0.7		0.293		16.0	LOS B	1.5	11.0				
NorthEast: Milner Road													
Lane 1	102	6.9	712	0.143	100	7.4	LOS A	1.0	7.7	Full	250	0.0	0.0
Lane 2	1	6.5	369	0.003	100	14.6	LOS B	0.0	0.1	Short	60	0.0	NA

Approach	103	6.9		0.143		7.5	LOS A	1.0	7.7				
NorthWest: TOD Connector													
Lane 1	9	0.2	379	0.025	100	14.0	LOS B	0.1	0.8	Full	155	0.0	0.0
Lane 2	24	0.8	380	0.064	100	15.7	LOS B	0.3	2.2	Short	60	0.0	NA
Approach	34	0.6		0.064		15.2	LOS B	0.3	2.2				
SouthWest: Milner Road													
Lane 1	382	8.1	707	0.541	100	7.9	LOS A	4.5	35.9	Full	120	0.0	0.0
Lane 2	42	5.9	573	0.073	100	12.6	LOS B	0.4	3.2	Short	60	0.0	NA
Approach	424	7.9		0.541		8.4	LOS A	4.5	35.9				
Intersection	703	6.0		0.541		10.1	LOS B	4.5	35.9				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

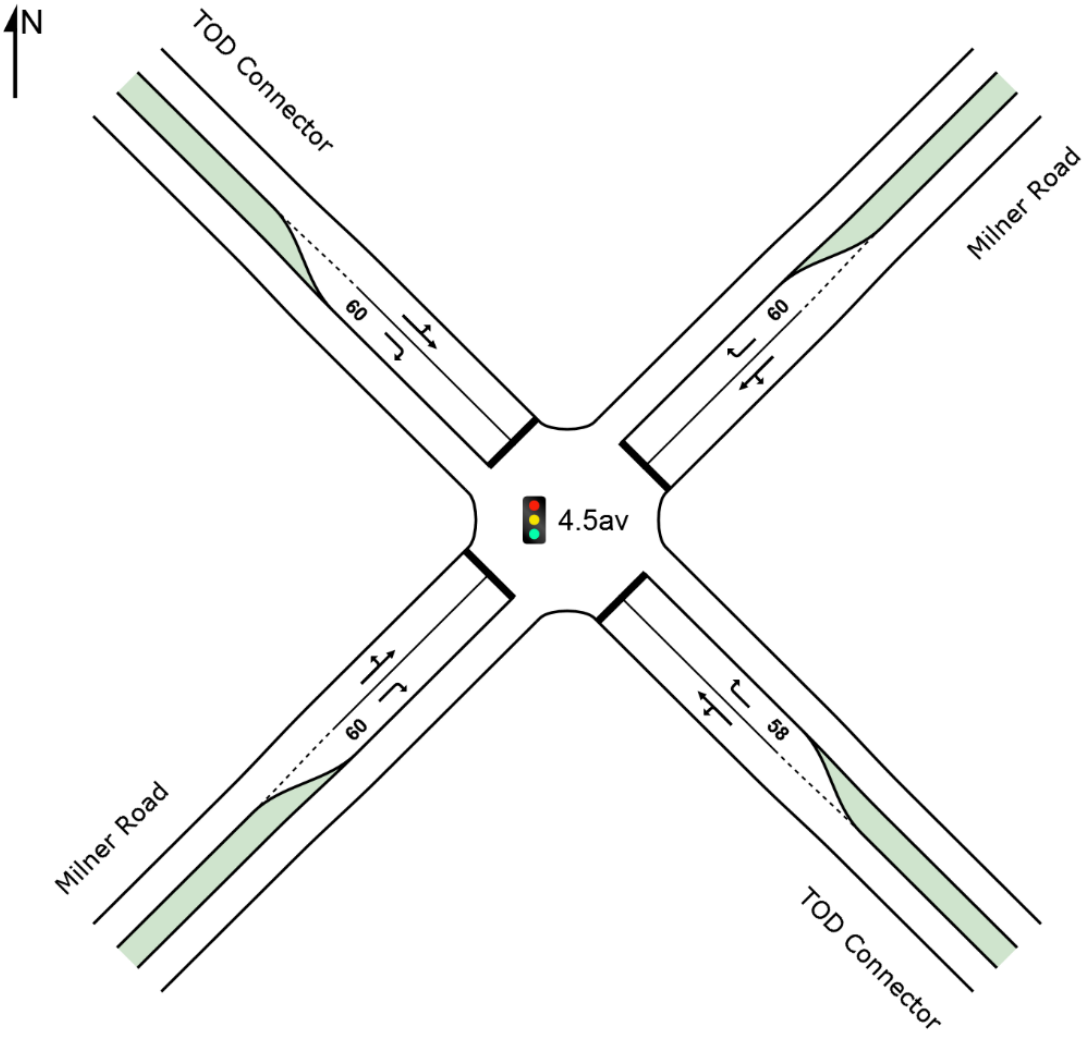
Site: 4.5av [4.5a Milner Road - TOD Connector - 2050 AM - Conversion (Site Folder: 2050)]

4.5a Milner Road - TOD Connector - 2050 AM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 30 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Phase Sequence: Convert Function Default
Reference Phase: Phase B
Input Phase Sequence: A, B
Output Phase Sequence: A, B

Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.

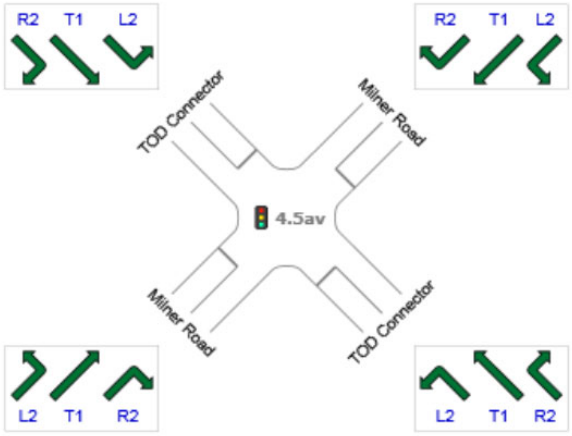


Input Volumes

Volume Display Method: Total and %

	R2	T1	L2
Tot	62	26	21
LV	47%	4%	5%
HV	0%	0%	0%
D1	52%	96%	95%
U2	1%	0%	0%
U3	0%	0%	0%
U4			

	R2	T1	L2
Tot	9	438	123
LV	10%	60%	1%
HV	0%	0%	0%
D1	89%	33%	99%
U2	0%	1%	0%
U3	1%	4%	0%
U4		2%	



	L2	T1	R2
Tot	1	154	72
LV	92%	51%	8%
HV	0%	0%	0%
D1	0%	43%	92%
U2	2%	1%	0%
U3	7%	4%	1%
U4		1%	

	L2	T1	R2
Tot	271	61	136
LV	15%	2%	10%
HV	0%	0%	0%
D1	85%	98%	90%
U2	0%	0%	0%
U3	0%	0%	0%
U4			

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (
SE: TOD Connector	468	55	0	411	1
NE: Milner Road	570	263	0	275	5
NW: TOD Connector	109	31	0	77	1
SW: Milner Road	227	85	0	132	2
Total	1374	434	0	895	9

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: TOD Connector													
Lane 1	349	0.1	440	0.794	100	19.4	LOS B	6.0	41.9	Full	300	0.0	0.0
Lane 2	143	0.1	413	0.346	100	15.8	LOS B	1.9	13.3	Short	58	0.0	NA
Approach	493	0.1		0.794		18.3	LOS B	6.0	41.9				
NorthEast: Milner Road													
Lane 1	591	4.7	671	0.881	100	19.9	LOS B	11.9	89.7	Full	250	0.0	0.0
Lane 2	9	0.7	503	0.019	100	13.0	LOS B	0.1	0.7	Short	60	0.0	NA

Approach	600	4.6		0.881		19.8	LOS B	11.9	89.7				
NorthWest: TOD Connector													
Lane 1	49	0.0	449	0.110	100	12.4	LOS B	0.6	4.2	Full	155	0.0	0.0
Lane 2	65	0.5	223	0.292	100	20.5	LOS C	1.0	7.2	Short	60	0.0	NA
Approach	115	0.3		0.292		17.0	LOS B	1.0	7.2				
SouthWest: Milner Road													
Lane 1	163	5.2	673	0.242	100	7.5	LOS A	1.7	13.2	Full	120	0.0	0.0
Lane 2	76	0.5	239	0.317	100	21.4	LOS C	1.2	8.3	Short	60	0.0	NA
Approach	239	3.7		0.317		11.9	LOS B	1.7	13.2				
Intersection	1446	2.6		0.881		17.8	LOS B	11.9	89.7				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

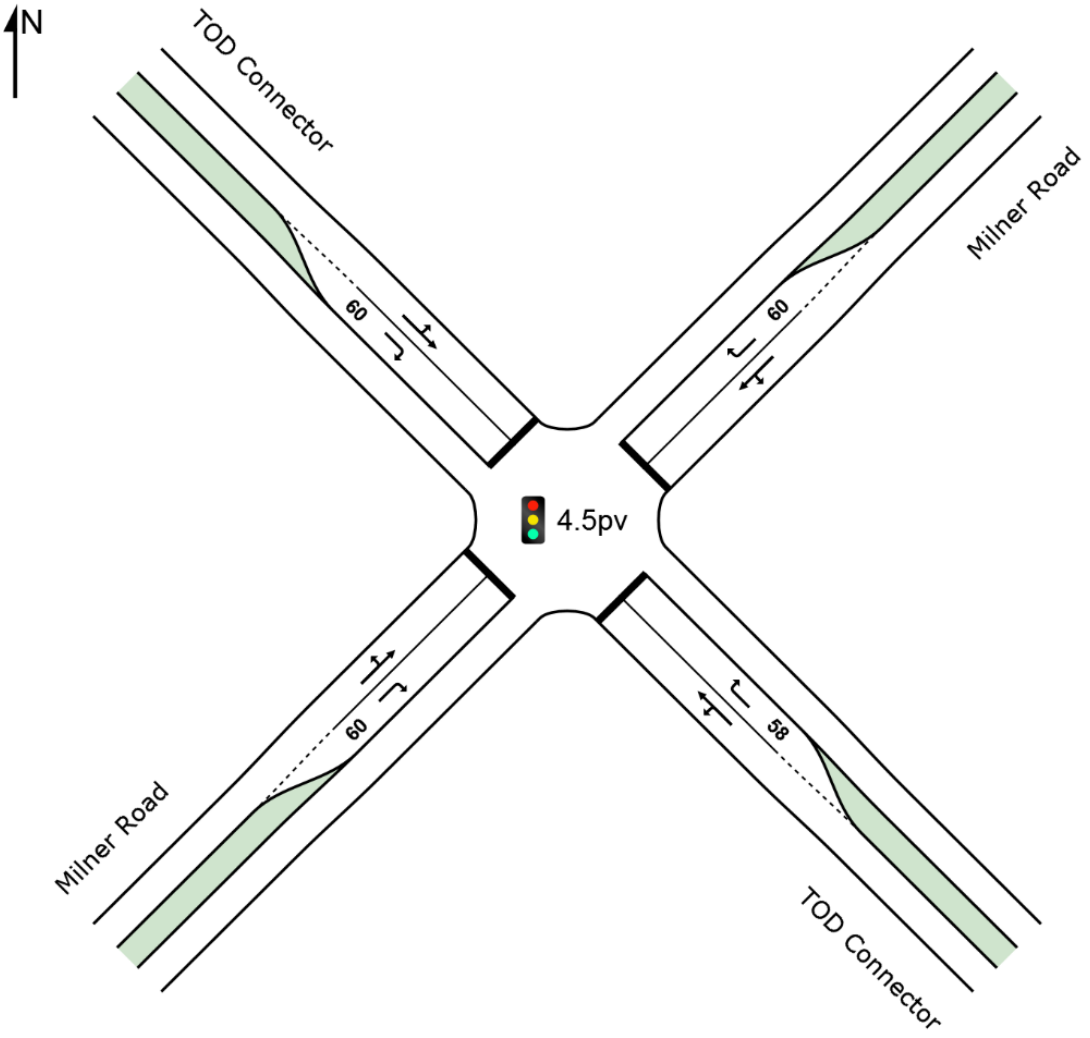
Site: 4.5pv [4.5p Milner Road - TOD Connector - 2050 PM - Conversion (Site Folder: 2050)]

4.5p Milner Road - TOD Connector - 2050 PM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 30 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Phase Sequence: Convert Function Default
Reference Phase: Phase B
Input Phase Sequence: A, B
Output Phase Sequence: A, B

Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.

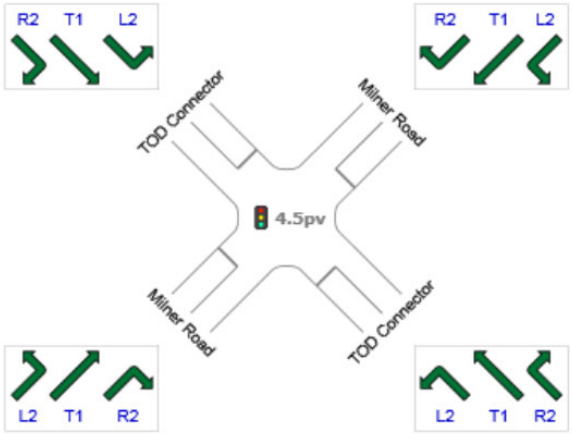


Input Volumes

Volume Display Method: Total and %

	R2	T1	L2
Tot	36	78	56
LV	35%	1%	2%
HV	0%	0%	0%
D1	64%	99%	98%
U2	1%	0%	0%
U3	0%	0%	0%
U4	0%	0%	

	R2	T1	L2
Tot	5	148	98
LV	18%	50%	1%
HV	0%	0%	0%
D1	80%	44%	99%
U2	0%	1%	0%
U3	1%	4%	0%
U4		1%	



	L2	T1	R2
Tot	4	464	125
LV	92%	60%	28%
HV	0%	0%	0%
D1	0%	33%	70%
U2	2%	1%	1%
U3	7%	4%	2%
U4		2%	

	L2	T1	R2
Tot	104	31	166
LV	6%	19%	49%
HV	0%	0%	0%
D1	94%	81%	50%
U2	0%	0%	1%
U3	0%	0%	1%
U4			

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (
SE: TOD Connector	301	92	0	206	2
NE: Milner Road	251	76	0	166	2
NW: TOD Connector	170	15	0	155	0
SW: Milner Road	593	317	0	239	6
Total	1315	499	0	766	10

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: TOD Connector													
Lane 1	142	0.1	379	0.375	100	15.7	LOS B	2.0	13.8	Full	300	0.0	0.0
Lane 2	175	0.5	307	0.569	100	18.6	LOS B	2.7	18.8	Short	58	0.0	NA
Approach	317	0.3		0.569		17.3	LOS B	2.7	18.8				
NorthEast: Milner Road													
Lane 1	259	3.1	741	0.349	100	9.3	LOS A	2.8	20.4	Full	250	0.0	0.0
Lane 2	5	1.3	321	0.016	100	16.5	LOS B	0.1	0.5	Short	60	0.0	NA

Approach	264	3.0		0.349		9.5	LOS A	2.8	20.4				
NorthWest: TOD Connector													
Lane 1	141	0.0	386	0.366	100	14.0	LOS B	1.9	13.6	Full	155	0.0	0.0
Lane 2	38	0.4	305	0.124	100	17.0	LOS B	0.5	3.6	Short	60	0.0	NA
Approach	179	0.1		0.366		14.7	LOS B	1.9	13.6				
SouthWest: Milner Road													
Lane 1	493	6.1	726	0.679	100	9.5	LOS A	6.7	51.5	Full	120	0.0	0.0
Lane 2	132	2.0	473	0.278	100	14.1	LOS B	1.5	11.0	Short	60	0.0	NA
Approach	624	5.2		0.679		10.4	LOS B	6.7	51.5				
Intersection	1384	3.0		0.679		12.4	LOS B	6.7	51.5				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

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Organisation: KC TRAFFIC AND TRANSPORT | Licence: NETWORK / 1PC | Created: 30 July, 2021 12:34:55
Project: C:\Users\User\Box\KCTT Projects\KC00000 Current Projects\KC00604.000 Forrestfield North - Structure Plan Preparation\Outgoing \SIDRA\DCP\210708 Rev B\Future Models\KC00604.000 M05 Milner Road - TOD Connector.sip9

USER REPORT FOR SITE

All Movement Classes

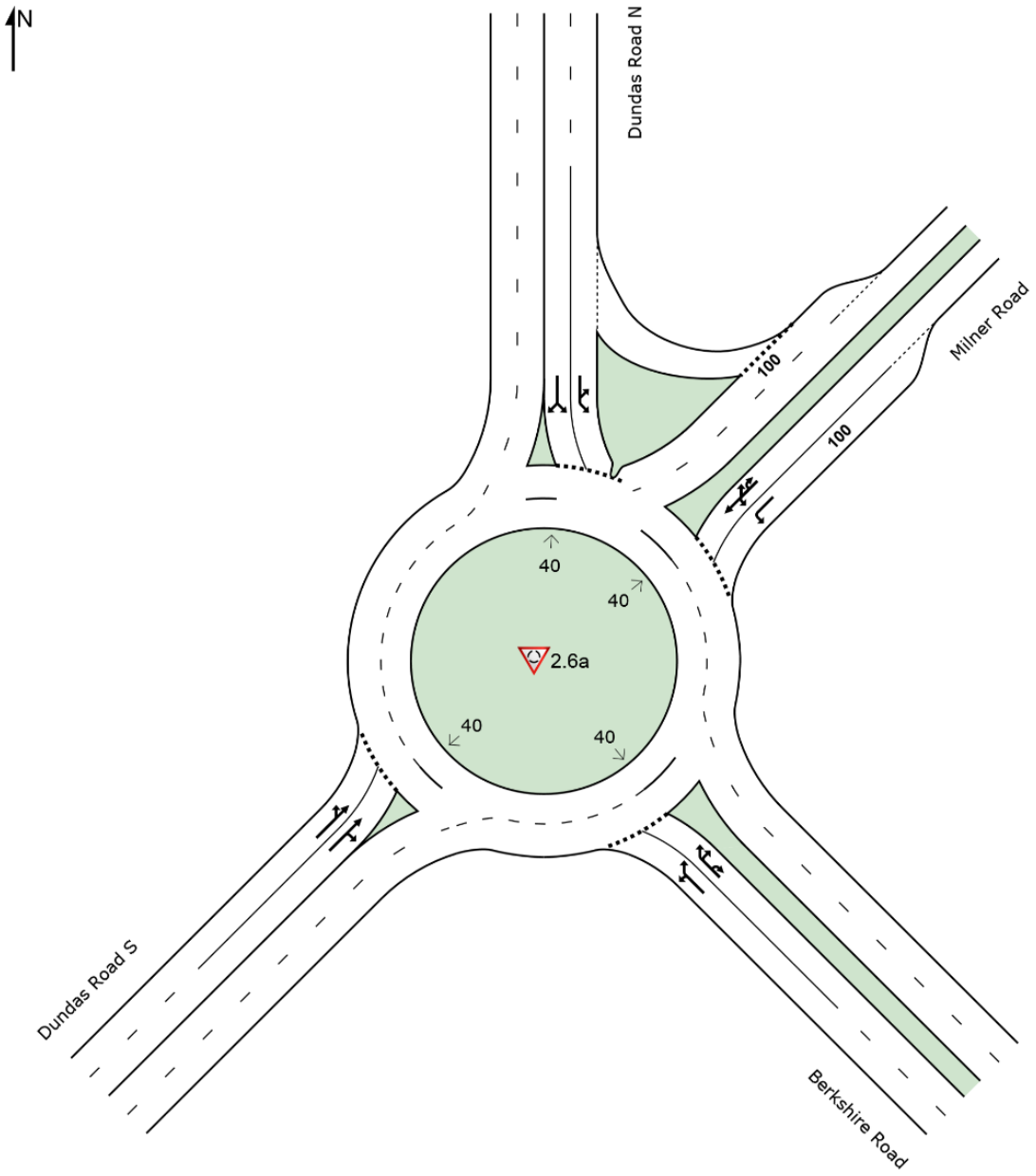
Project: KC00604.000 M06 Milner Road - Berkshire Road - Dundas Road **Template: Site User Report**

Site: 2.6a [2.6a Milner Road - Berkshire Road - Dundas Road - 2031 AM (Site Folder: 2031)]

2.6a Milner Road - Berkshire Road - Dundas Road - 2031 AM
Site Category: (None)
Roundabout

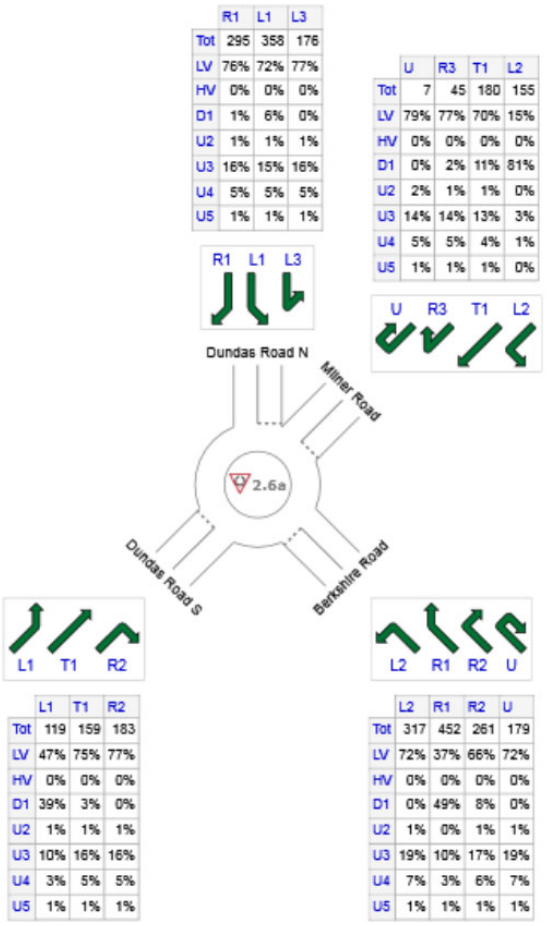
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U
SE: Berkshire Road	1209	696	0	243	9
NE: Milner Road	387	189	0	146	4
N: Dundas Road N	829	618	0	27	8
SW: Dundas Road S	461	316	0	50	4
Total	2886	1820	0	466	24

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road													
Lane 1[d]	712	20.0	903	0.789	100	16.3	LOS B	11.1	101.9	Full	500	0.0	0.0
Lane 2	560	23.6	710	0.789	100	22.9	LOS C	10.0	95.6	Full	500	0.0	0.0
Approach	1273	21.6		0.789		19.2	LOS B	11.1	101.9				
NorthEast: Milner Road													
Lane 1	163	3.7	566	0.288	80[5]	8.4	LOS A	1.6	11.6	Short	100	0.0	NA
Lane 2[d]	244	18.3	678	0.360	100	9.7	LOS A	2.2	20.0	Full	500	0.0	0.0
Approach	407	12.5		0.360		9.2	LOS A	2.2	20.0				
North: Dundas Road N													

Lane 1	414	21.2	665	0.623	100	11.2	LOS B	5.1	47.1	Full	500	0.0	0.0
Lane 2[d]	458	21.3	736	0.623	100	14.6	LOS B	5.6	52.2	Full	500	0.0	0.0
Approach	873	21.3		0.623		13.0	LOS B	5.6	52.2				
SouthWest: Dundas Road S													
Lane 1	182	16.0	515	0.353	78[6]	7.8	LOS A	1.9	16.4	Full	500	0.0	0.0
Lane 2[d]	303	21.8	673	0.451	100	11.4	LOS B	2.9	26.9	Full	500	0.0	0.0
Approach	485	19.6		0.451		10.1	LOS B	2.9	26.9				
Intersection	3038	20.0		0.789		14.6	LOS B	11.1	101.9				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

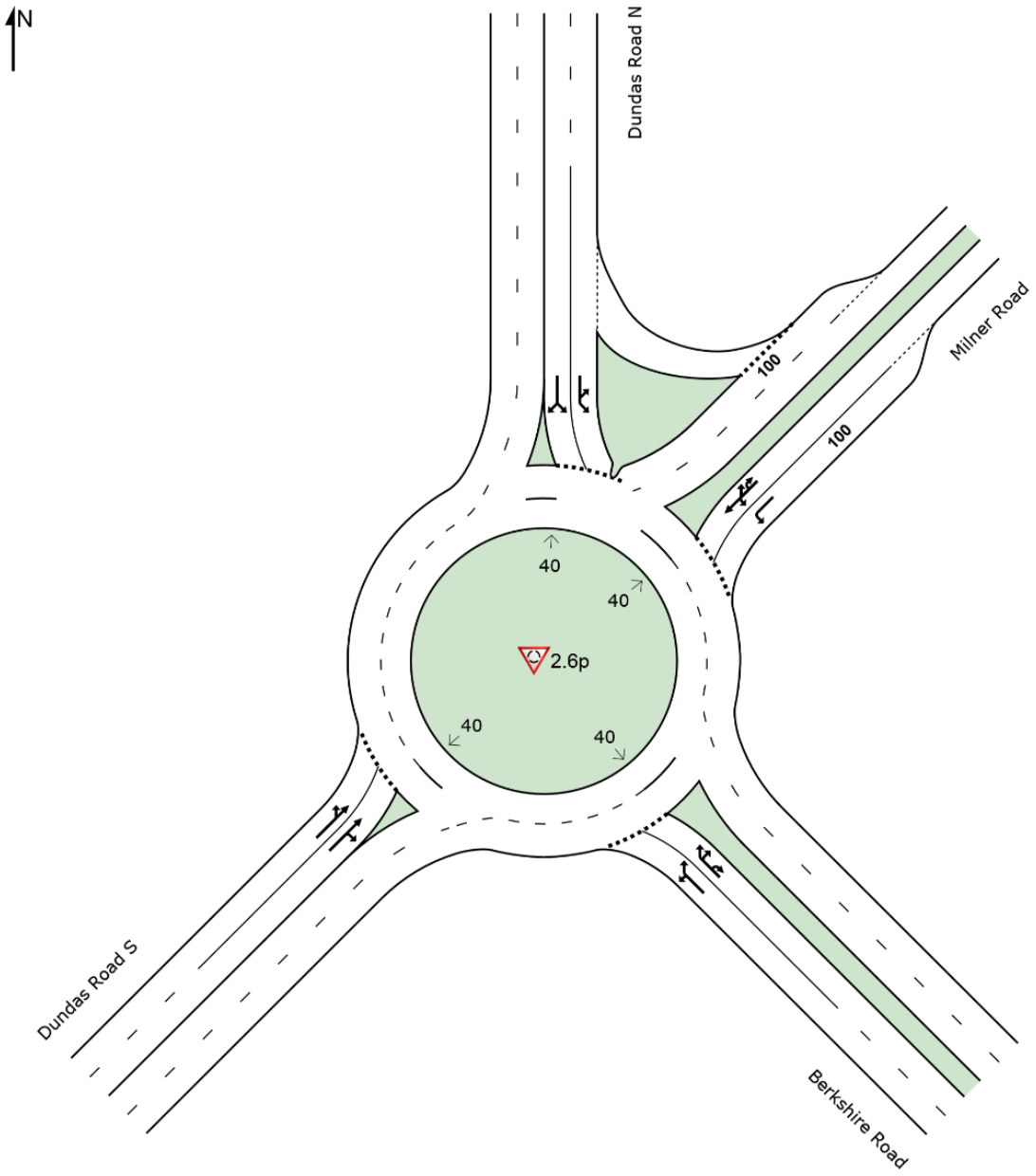
5 Lane under-utilisation found by the program
6 Lane under-utilisation due to downstream effects
d Dominant lane on roundabout approach

Site: 2.6p [2.6p Milner Road - Berkshire Road - Dundas Road - 2031 PM (Site Folder: 2031)]

2.6p Milner Road - Berkshire Road - Dundas Road - 2031 PM
Site Category: (None)
Roundabout

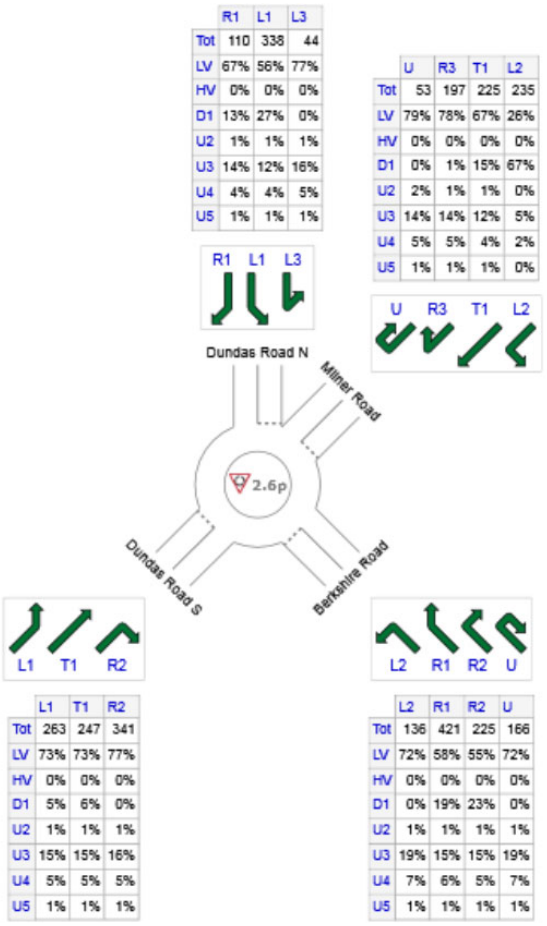
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U
SE: Berkshire Road	948	588	0	132	7
NE: Milner Road	710	406	0	193	8
N: Dundas Road N	492	299	0	104	4
SW: Dundas Road S	851	634	0	27	8
Total	3001	1927	0	456	27

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road													
Lane 1[d]	554	23.2	785	0.705	100	16.0	LOS B	8.1	77.5	Full	500	0.0	0.0
Lane 2	444	23.3	629	0.705	100	21.3	LOS C	7.3	69.9	Full	500	0.0	0.0
Approach	998	23.2		0.705		18.3	LOS B	8.1	77.5				
NorthEast: Milner Road													
Lane 1	247	6.6	597	0.414	61[5]	9.6	LOS A	2.5	19.1	Short	100	0.0	NA
Lane 2[d]	500	18.5	737	0.678	100	16.7	LOS B	6.2	56.6	Full	500	0.0	0.0
Approach	747	14.6		0.678		14.4	LOS B	6.2	56.6				
North: Dundas Road N													

Lane 1	233	17.3	467	0.498	100	14.5	LOS B	3.6	31.8	Full	500	0.0	0.0
Lane 2[d]	285	17.4	573	0.498	100	15.6	LOS B	4.2	36.9	Full	500	0.0	0.0
Approach	518	17.3		0.498		15.1	LOS B	4.2	36.9				
SouthWest: Dundas Road S													
Lane 1	321	20.9	462	0.696	78[6]	14.1	LOS B	5.0	46.0	Full	500	0.0	0.0
Lane 2[d]	574	21.5	646	0.889	100	25.0	LOS C	11.3	105.7	Full	500	0.0	0.0
Approach	896	21.3		0.889		21.1	LOS C	11.3	105.7				
Intersection	3159	19.7		0.889		17.7	LOS B	11.3	105.7				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

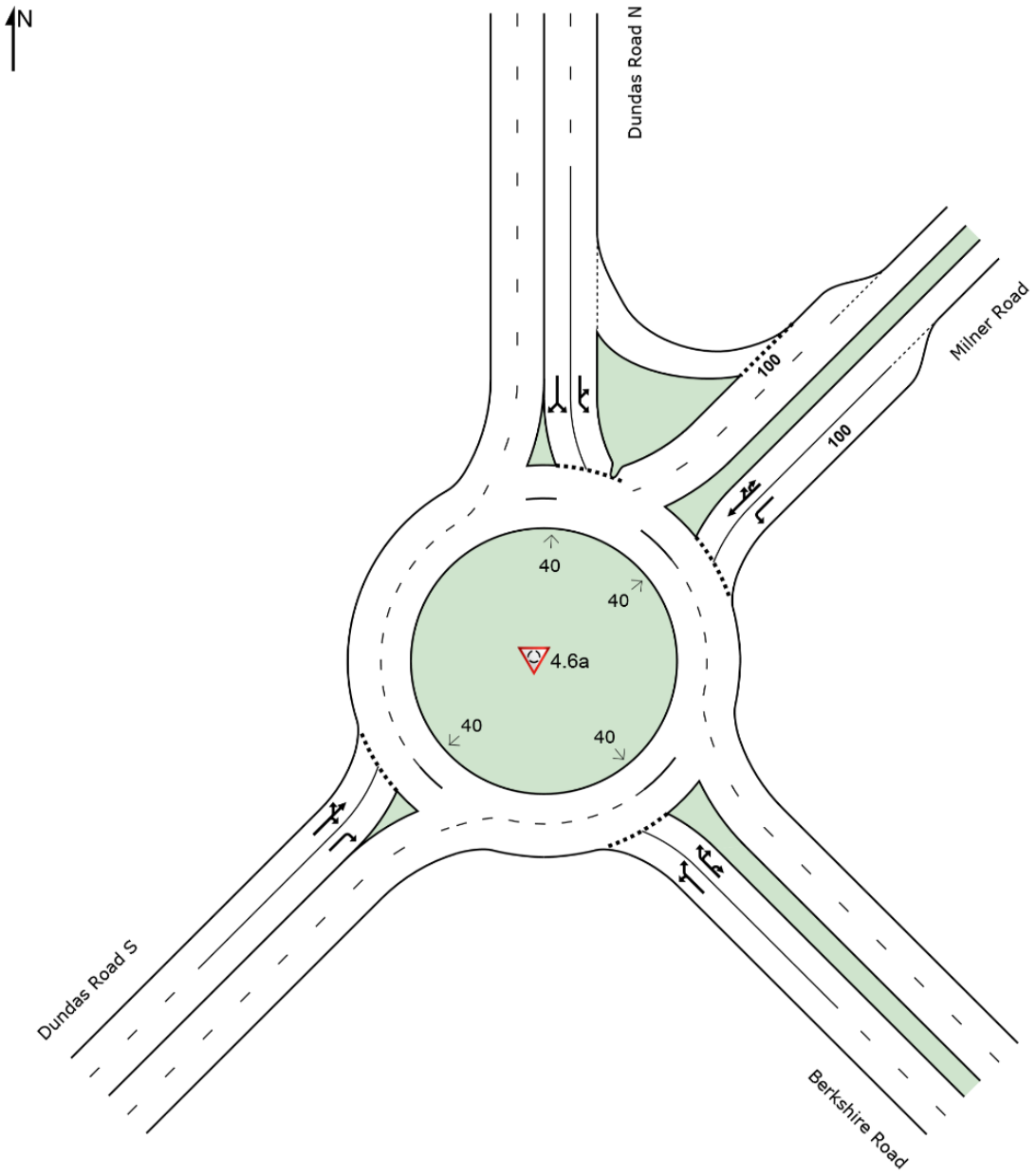
5 Lane under-utilisation found by the program
6 Lane under-utilisation due to downstream effects
d Dominant lane on roundabout approach

Site: 4.6a [4.6a Milner Road - Berkshire Road - Dundas Road - 2050 AM (Site Folder: 2050)]

4.6a Milner Road - Berkshire Road - Dundas Road - 2050 AM
Site Category: (None)
Roundabout

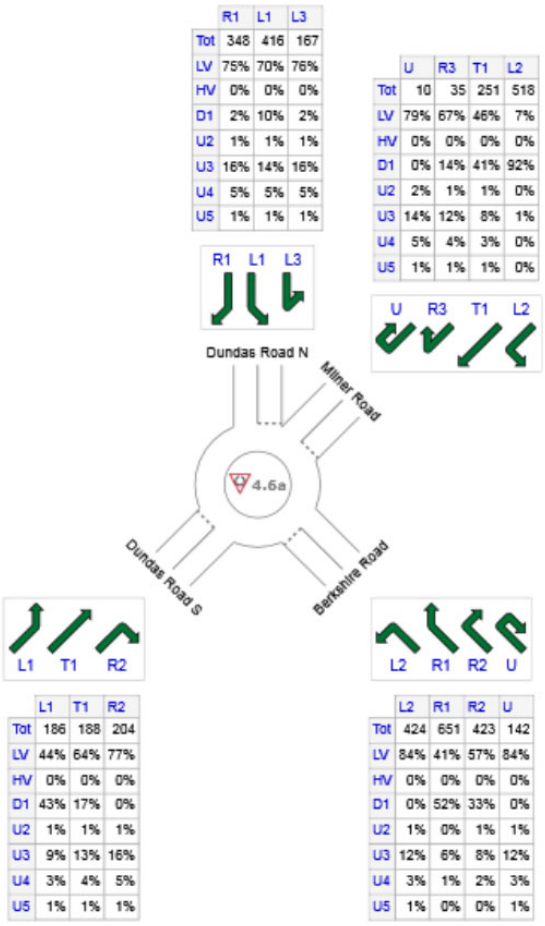
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U
SE: Berkshire Road	1640	981	0	474	10
NE: Milner Road	814	181	0	583	3
N: Dundas Road N	931	678	0	50	9
SW: Dundas Road S	578	359	0	112	5
Total	3963	2199	0	1219	27

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road													
Lane 1[d]	958	10.9	870	1.101	100	121.7	LOS F	81.5	662.0	Full	500	0.0	14.0
Lane 2	768	10.4	698	1.101	100	128.8	LOS F	67.2	542.9	Full	500	0.0	7.5
Approach	1726	10.7		1.101		124.9	LOS F	81.5	662.0				
NorthEast: Milner Road													
Lane 1[d]	545	1.7	731	0.746	100	14.7	LOS B	8.1	58.1	Short	100	0.0	NA
Lane 2	312	12.6	474	0.658	100	16.5	LOS B	5.0	42.0	Full	500	0.0	0.0
Approach	857	5.7		0.746		15.4	LOS B	8.1	58.1				
North: Dundas Road N													

Lane 1	447	20.6	612	0.730	100	13.3	LOS B	6.3	58.3	Full	500	0.0	0.0
Lane 2[d]	533	21.0	731	0.730	100	16.0	LOS B	7.0	65.4	Full	500	0.0	0.0
Approach	980	20.8		0.730		14.8	LOS B	7.0	65.4				
SouthWest: Dundas Road S													
Lane 1[d]	394	15.4	618	0.637	100	10.7	LOS B	4.9	42.8	Full	500	0.0	0.0
Lane 2	215	22.0	413	0.520	82[5]	17.5	LOS B	3.1	28.7	Full	500	0.0	0.0
Approach	608	17.7		0.637		13.1	LOS B	4.9	42.8				
Intersection	4172	13.1		1.101		60.2	LOS E	81.5	662.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

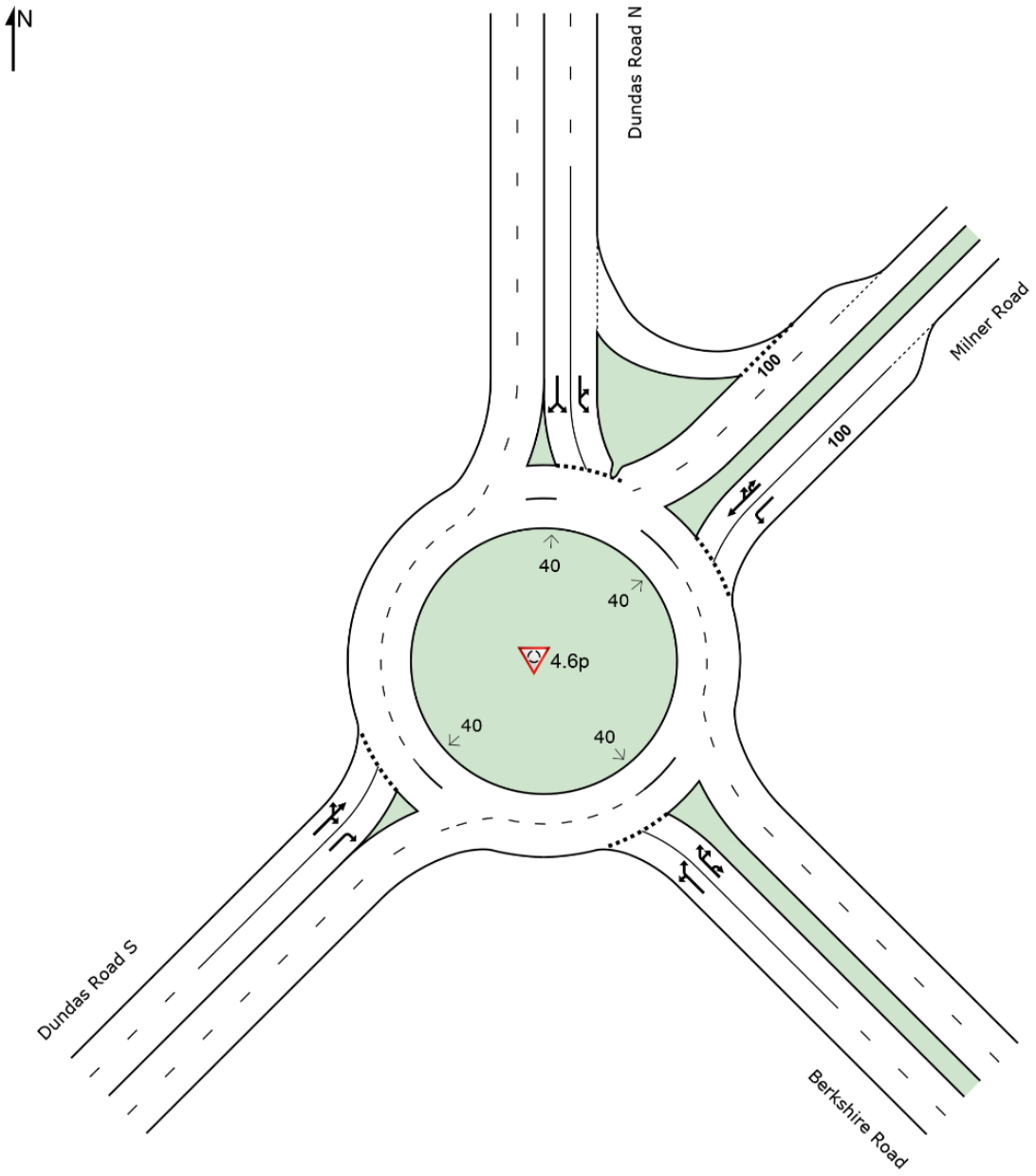
5 Lane under-utilisation found by the program
d Dominant lane on roundabout approach

Site: 4.6p [4.6p Milner Road - Berkshire Road - Dundas Road - 2050 PM (Site Folder: 2050)]

4.6p Milner Road - Berkshire Road - Dundas Road - 2050 PM
Site Category: (None)
Roundabout

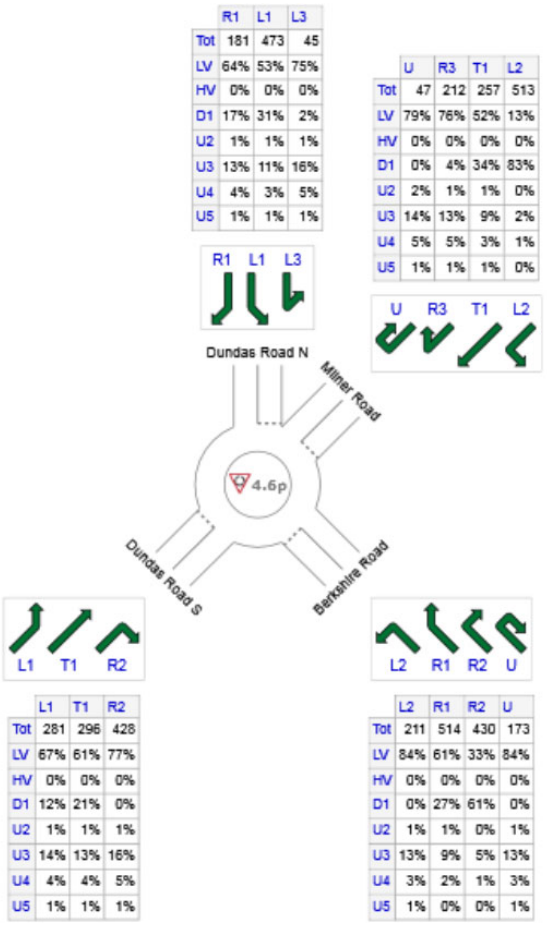
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U
SE: Berkshire Road	1328	775	0	400	9
NE: Milner Road	1029	399	0	521	8
N: Dundas Road N	699	400	0	180	5
SW: Dundas Road S	1005	700	0	96	9
Total	4061	2273	0	1197	31

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road													
Lane 1[d]	763	12.5	747	1.021	98[5]	72.7	LOS F	43.3	359.0	Full	500	0.0	0.0
Lane 2	635	8.8	611	1.039	100	90.4	LOS F	41.7	329.2	Full	500	0.0	0.0
Approach	1398	10.8		1.039		80.7	LOS F	43.3	359.0				
NorthEast: Milner Road													
Lane 1	540	3.4	516	1.047	100	91.5	LOS F	34.6	255.5	Short	100	0.0	NA
Lane 2[d]	543	16.3	642	0.846	100	28.0	LOS C	11.1	97.8	Full	500	0.0	0.0
Approach	1083	9.9		1.047		59.6	LOS E	34.6	255.5				
North: Dundas Road N													

Lane 1	315	16.1	453	0.696	100	18.3	LOS B	5.5	47.8	Full	500	0.0	0.0
Lane 2[d]	421	16.5	604	0.696	100	18.6	LOS B	6.5	57.4	Full	500	0.0	0.0
Approach	736	16.3		0.696		18.5	LOS B	6.5	57.4				
SouthWest: Dundas Road S													
Lane 1[d]	635	18.5	551	1.153	100	162.4	LOS F	66.1	594.9	Full	500	0.0	10.4
Lane 2	423	22.0	367	1.153	100	174.4	LOS F	45.7	429.2	Full	500	0.0	0.7
Approach	1058	19.9		1.153		167.2	LOS F	66.1	594.9				
Intersection	4275	13.8		1.153		86.1	LOS F	66.1	594.9				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

5 Lane under-utilisation found by the program
d Dominant lane on roundabout approach

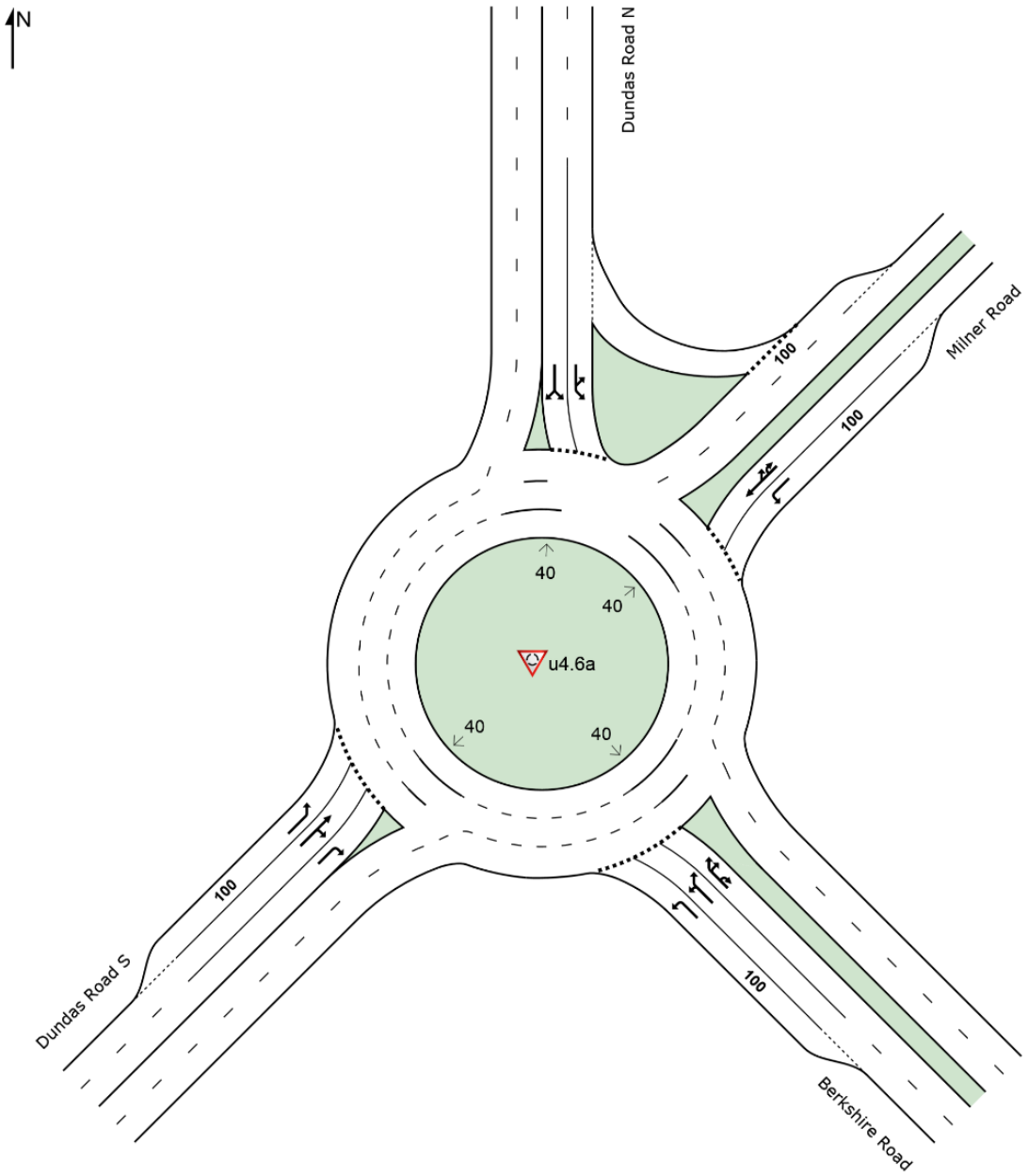
Site: u4.6a [u4.6a Milner Road - Berkshire Road - Dundas Road - 2050 AM - Upgrade (Site Folder: 2050)]

u4.6a Milner Road - Berkshire Road - Dundas Road - 2050 AM - Upgrade

Site Category: (None)
Roundabout

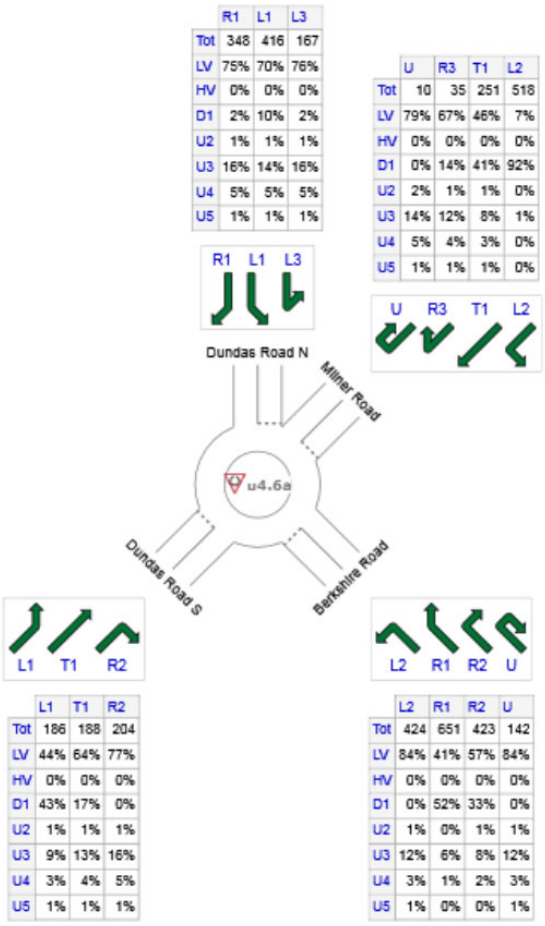
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U
SE: Berkshire Road	1640	981	0	474	10
NE: Milner Road	814	181	0	583	3
N: Dundas Road N	931	678	0	50	9
SW: Dundas Road S	578	359	0	112	5
Total	3963	2199	0	1219	27

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road													
Lane 1	446	15.0	737	0.606	79[5]	10.4	LOS B	5.0	43.0	Short	100	0.0	NA
Lane 2[d]	685	7.3	985	0.696	91[5]	15.4	LOS B	7.9	61.2	Full	500	0.0	0.0
Lane 3	595	11.3	775	0.768	100	20.8	LOS C	9.0	73.5	Full	500	0.0	0.0
Approach	1726	10.7		0.768		16.0	LOS B	9.0	73.5				
NorthEast: Milner Road													
Lane 1[d]	545	1.7	733	0.744	100	10.0	LOS B	5.7	40.7	Short	100	0.0	NA
Lane 2	312	12.6	511	0.609	100	11.2	LOS B	3.3	28.1	Full	500	0.0	0.0
Approach	857	5.7		0.744		10.5	LOS B	5.7	40.7				

North: Dundas Road N													
Lane 1	478	20.5	612	0.781	100	12.8	LOS B	6.5	59.5	Full	500	0.0	0.0
Lane 2[d]	502	21.1	642	0.781	100	16.7	LOS B	6.8	62.8	Full	500	0.0	0.0
Approach	980	20.8		0.781		14.8	LOS B	6.8	62.8				
SouthWest: Dundas Road S													
Lane 1	196	12.5	535	0.366	100	6.6	LOS A	1.8	14.9	Short	100	0.0	NA
Lane 2[d]	243	19.0	709	0.343	100	8.3	LOS A	1.8	16.6	Full	500	0.0	0.0
Lane 3	169	22.0	494	0.343	100	13.8	LOS B	1.6	15.0	Full	500	0.0	0.0
Approach	608	17.7		0.366		9.3	LOS A	1.8	16.6				
Intersection	4172	13.1		0.781		13.6	LOS B	9.0	73.5				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

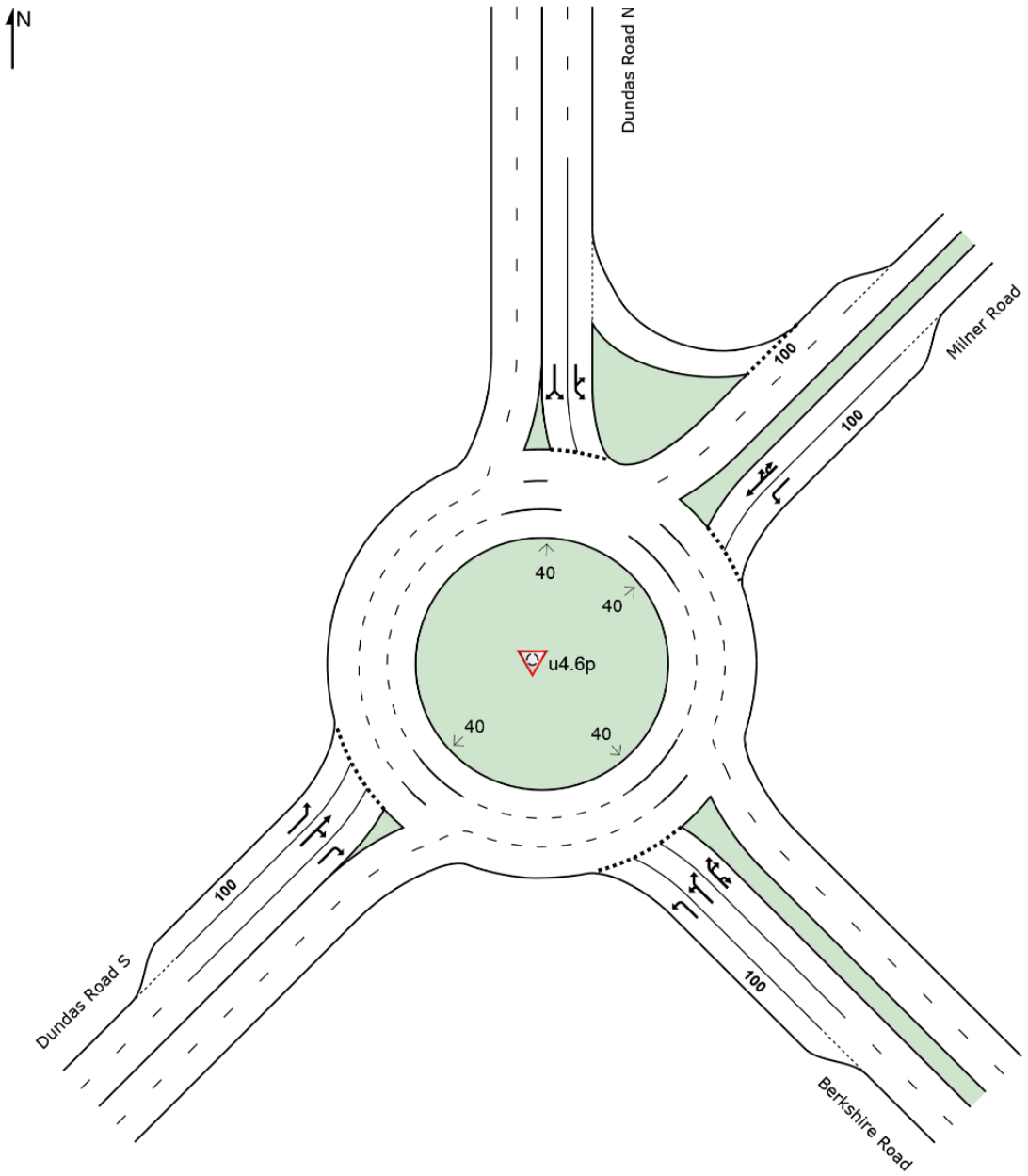
5 Lane under-utilisation found by the program
d Dominant lane on roundabout approach

Site: u4.6p [u4.6p Milner Road - Berkshire Road - Dundas Road - 2050 PM - Upgrade (Site Folder: 2050)]

u4.6p Milner Road - Berkshire Road - Dundas Road - 2050 PM - Upgrade
Site Category: (None)
Roundabout

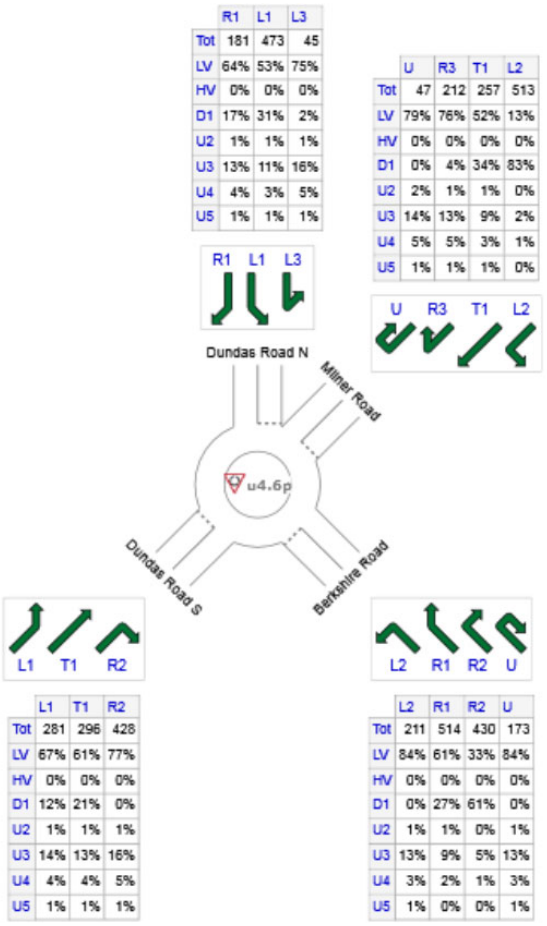
Site Layout

Layout pictures are schematic functional drawings reflecting input data. They are not design drawings.



Input Volumes

Volume Display Method: Total and %



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development LV (D1)	AustroadsClass 2 (U
SE: Berkshire Road	1328	775	0	400	9
NE: Milner Road	1029	399	0	521	8
N: Dundas Road N	699	400	0	180	5
SW: Dundas Road S	1005	700	0	96	9
Total	4061	2273	0	1197	31

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road													
Lane 1	222	15.5	606	0.366	46[5]	8.3	LOS A	2.2	18.8	Short	100	0.0	NA
Lane 2	541	11.3	686	0.789	100	21.2	LOS C	9.7	79.2	Full	500	0.0	0.0
Lane 3[d]	635	8.8	870	0.729	92[5]	19.5	LOS B	9.0	71.5	Full	500	0.0	0.0
Approach	1398	10.8		0.789		18.4	LOS B	9.7	79.2				
NorthEast: Milner Road													
Lane 1	540	3.4	606	0.891	100	18.7	LOS B	8.8	65.2	Short	100	0.0	NA
Lane 2[d]	543	16.3	653	0.832	100	18.1	LOS B	7.0	62.1	Full	500	0.0	0.0
Approach	1083	9.9		0.891		18.4	LOS B	8.8	65.2				

North: Dundas Road N													
Lane 1	337	16.0	454	0.743	100	17.3	LOS B	5.4	46.9	Full	500	0.0	0.0
Lane 2[d]	399	16.6	537	0.743	100	18.9	LOS B	6.0	52.6	Full	500	0.0	0.0
Approach	736	16.3		0.743		18.2	LOS B	6.0	52.6				
SouthWest: Dundas Road S													
Lane 1	296	19.3	493	0.599	100	8.7	LOS A	3.2	29.4	Short	100	0.0	NA
Lane 2[d]	455	18.9	710	0.640	100	11.0	LOS B	4.2	37.7	Full	500	0.0	0.0
Lane 3	308	22.0	481	0.640	100	16.3	LOS B	3.5	33.2	Full	500	0.0	0.0
Approach	1058	19.9		0.640		11.9	LOS B	4.2	37.7				
Intersection	4275	13.8		0.891		16.8	LOS B	9.7	79.2				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).
Roundabout LOS Method: SIDRA Roundabout LOS.
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Roundabout Capacity Model: SIDRA Standard.
Delay Model: SIDRA Standard (Geometric Delay is included).
Queue Model: SIDRA Standard.
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

5 Lane under-utilisation found by the program
d Dominant lane on roundabout approach

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Organisation: KC TRAFFIC AND TRANSPORT | Licence: NETWORK / 1PC | Created: 30 July, 2021 11:44:30
Project: C:\Users\User\Box\KCTT Projects\KC00000 Current Projects\KC00604.000 Forrestfield North - Structure Plan Preparation\Outgoing \SIDRA\DCP\210708 Rev B\Future Models\KC00604.000 M06 Milner Road - Berkshire Road - Dundas Road.sip9

USER REPORT FOR NETWORK

All Movement Classes

Project: KC00604.000 M07 Roe Highway - Berkshire Road **Template: Default Network User Report**

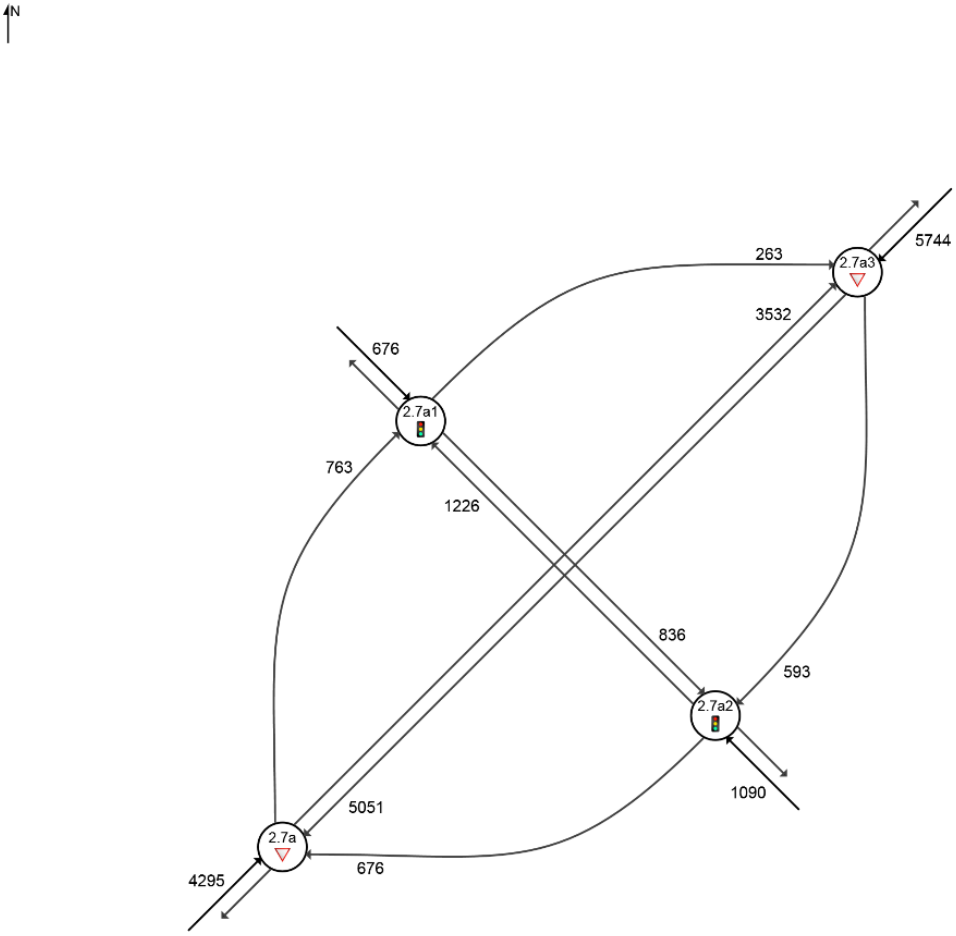
Network: n2.2a [n2.2a Roe Highway - Berkshire Road - 2031 AM (Network Folder: 2031)]

n2.2a Roe Highway - Berkshire Road - 2031 AM
Network Category: (None)

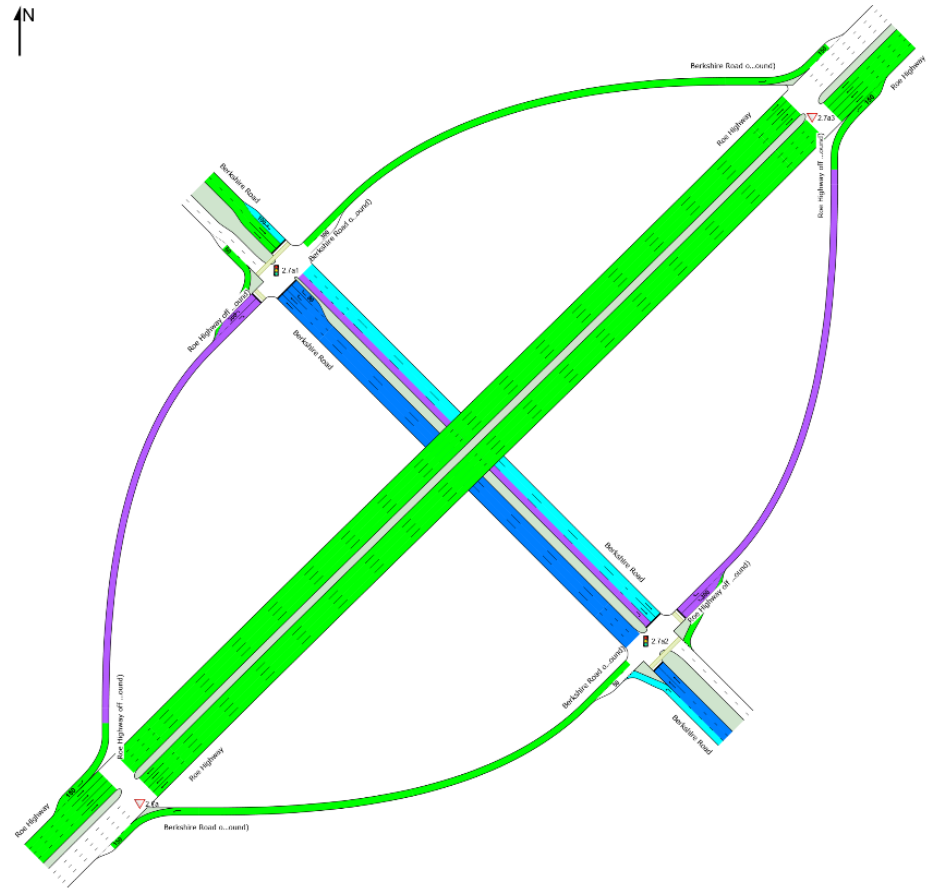
Network Flows - Input

Use the button below to open or close all popup boxes. Click value labels to open selected ones.
Click and drag popup boxes to move to preferred positions.

Open All Popups



Level of Service


N
Berkshire Road o...ound)
Roe Highway
Berkshire Road
Roe Highway off ...ound)
Berkshire Road o...ound)
Roe Highway
Berkshire Road
Roe Highway off ...ound)
Roe Highway
Roe Highway
Berkshire Road o...ound)
Berkshire Road
Colour code based on Level of Service
LOS A
LOS B
LOS C
LOS D
LOS E
LOS F
Delay Model: SIDRA Standard (Geometric Delay is included).

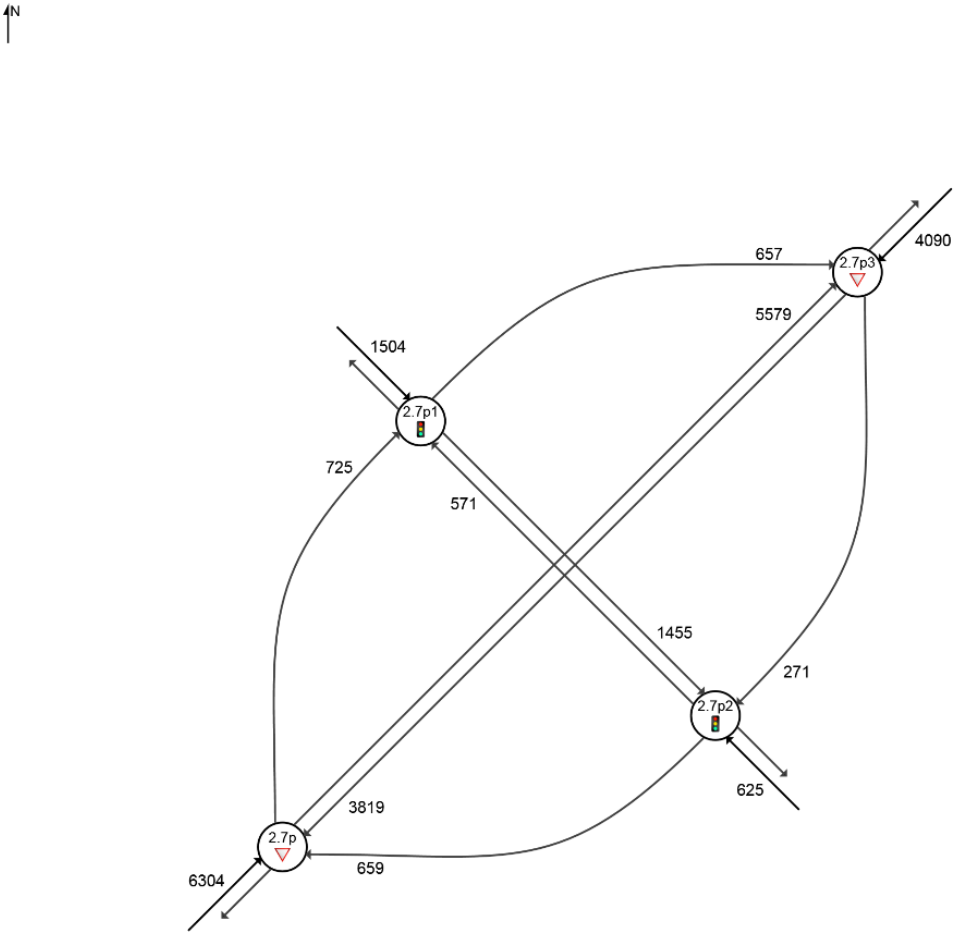
Network: n2.2p [n2.2p Roe Highway - Berkshire Road - 2031 PM (Network Folder: 2031)]

n2.2p Roe Highway - Berkshire Road - 2031 PM
Network Category: (None)

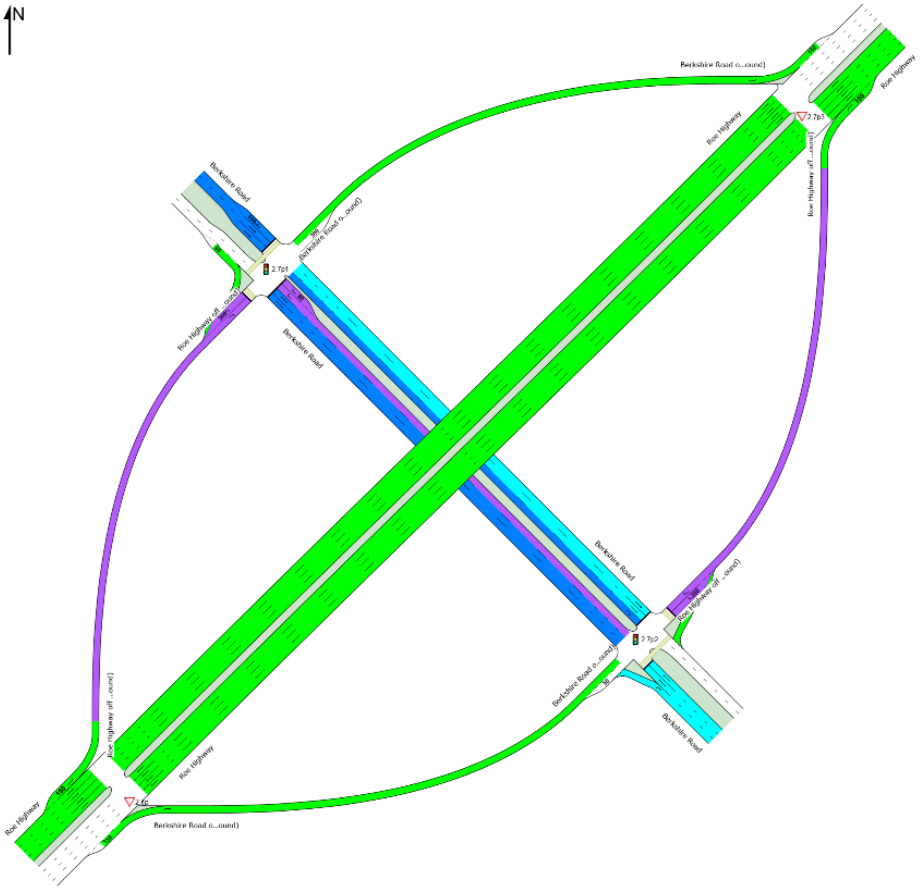
Network Flows - Input

Use the button below to open or close all popup boxes. Click value labels to open selected ones.
Click and drag popup boxes to move to preferred positions.

Open All Popups



Level of Service



Colour code based on Level of Service

LOS A | LOS B | LOS C | LOS D | LOS E | LOS F

Delay Model: SIDRA Standard (Geometric Delay is included).

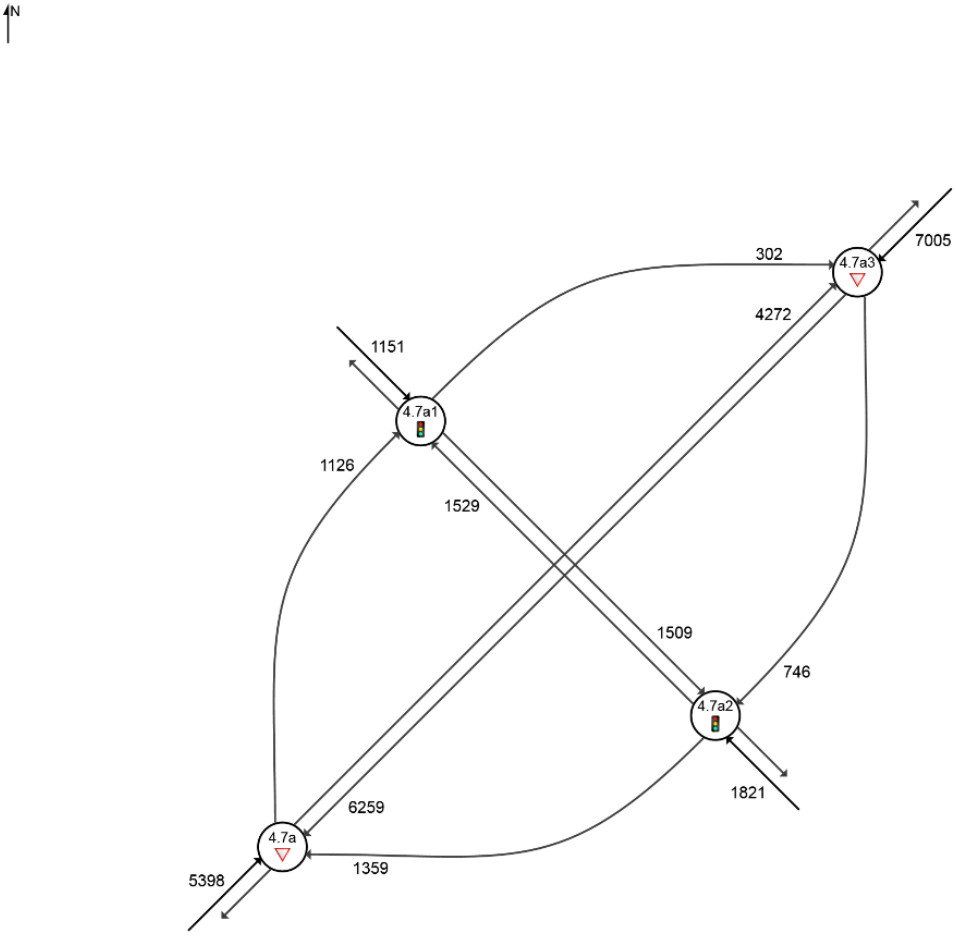
Network: n4.2a [n4.2a Roe Highway - Berkshire Road - 2050 AM (Network Folder: 2050)]

n4.2a Roe Highway - Berkshire Road - 2050 AM
Network Category: (None)

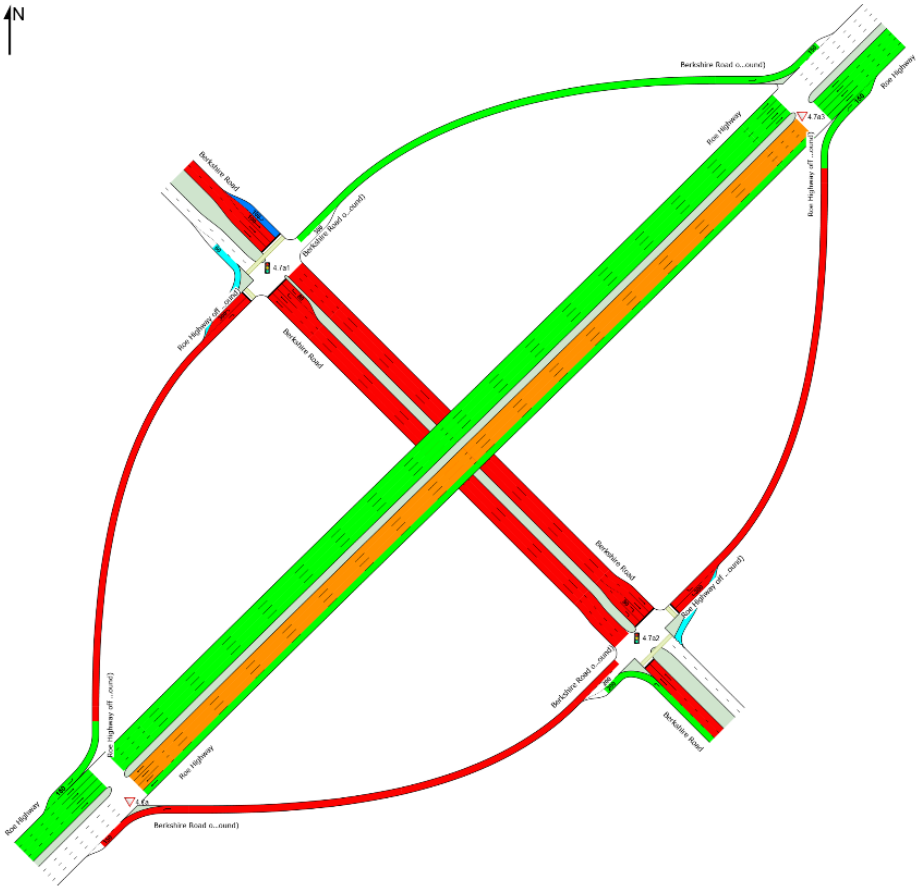
Network Flows - Input

Use the button below to open or close all popup boxes. Click value labels to open selected ones.
Click and drag popup boxes to move to preferred positions.

Open All Popups



Level of Service



Colour code based on Level of Service

LOS A	LOS B	LOS C	LOS D	LOS E	LOS F

Delay Model: SIDRA Standard (Geometric Delay is included).

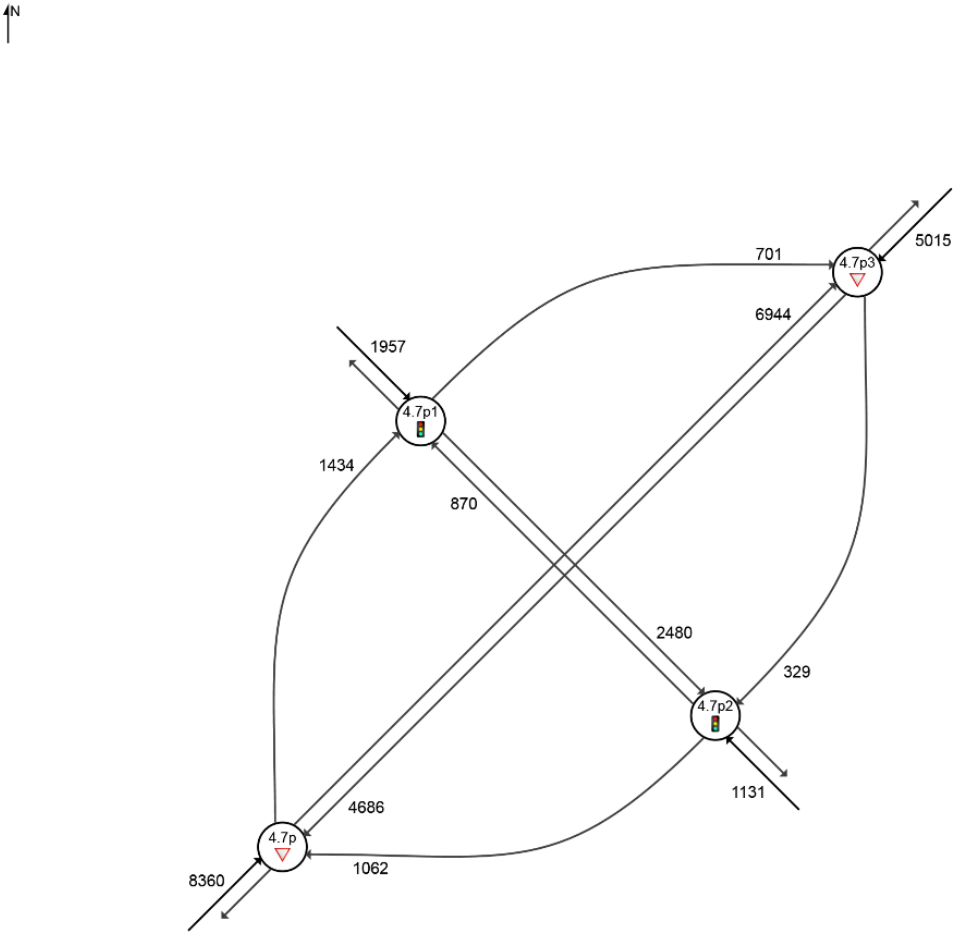
Network: n4.2p [n4.2p Roe Highway - Berkshire Road - 2050 PM (Network Folder: 2050)]

n4.2p Roe Highway - Berkshire Road - 2050 PM
Network Category: (None)

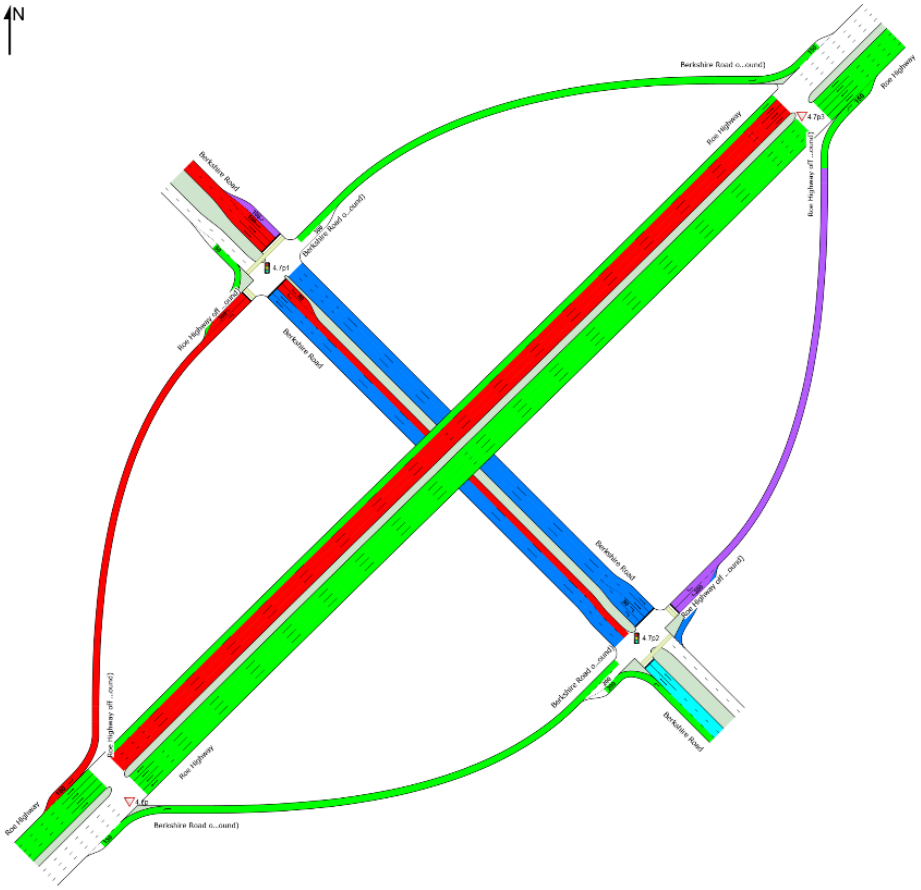
Network Flows - Input

Use the button below to open or close all popup boxes. Click value labels to open selected ones.
Click and drag popup boxes to move to preferred positions.

Open All Popups



Level of Service



Colour code based on Level of Service

LOS A	LOS B	LOS C	LOS D	LOS E	LOS F

Delay Model: SIDRA Standard (Geometric Delay is included).

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Project: C:\Users\User\Box\KCTT Projects\KC00000 Current Projects\KC00604.000 Forrestfield North - Structure Plan Preparation\Outgoing \SIDRA\DCP\210708 Rev B\Future Models\KC00604.000 M07 Roe Highway - Berkshire Road.sip9

USER REPORT FOR NETWORK SITE

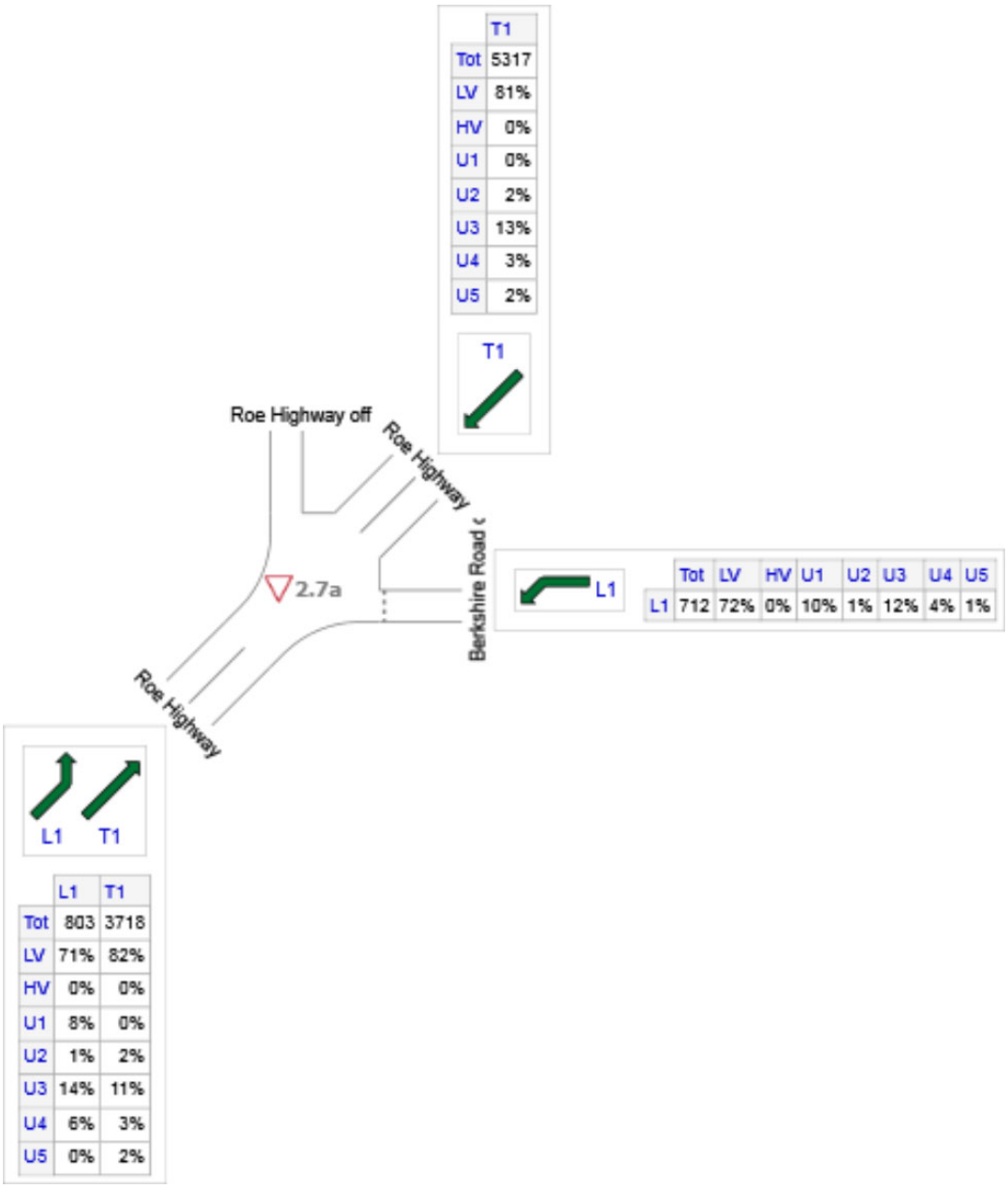
All Movement Classes

Project: KC00604.000 M07 Roe Highway - Berkshire Road **Template: Network Site User Report**

Site: 2.7a [2.7a Roe Highway Interchange with Berkshire Road - 2031 AM (South) (Site Folder: 2031 AM)] **Network: 1 [n2.2a Roe Highway - Berkshire Road - 2031 AM (Network Folder: 2031)]**

2.7a Roe Highway Interchange with Berkshire Road - 2031 AM (South)
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development
E: Berkshire Road on to Roe Highway (Southbound)	712	514	0	74
NE: Roe Highway	5317	4280	0	0
SW: Roe Highway	4521	3609	0	64
Total	10549	8403	0	138

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Berkshire Road on to Roe Highway (Southbound)															
Lane 1	712	16.5	712	16.5	1577	0.451	100	6.5	LOS A	0.0	0.0	Full	500	0.0	0.0

Approach	712	16.5	712	16.5		0.451		6.5	NA	0.0	0.0			
NorthEast: Roe Highway														
Lane 1	409	17.7	409	17.7	1842	0.222	25[7]	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	1636	17.7	1636	17.7	1842	0.888	100	0.6	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1636	17.7	1636	17.7	1842	0.888	100	0.6	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1636	17.7	1636	17.7	1842	0.888	100	0.6	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	5317	17.7	5317	17.7		0.888		0.5	NA	0.0	0.0			
SouthWest: Roe Highway														
Lane 1	803	19.9	803	19.9	1540	0.521	100	8.2	LOS A	0.0	0.0	Short	150	0.0	NA
Lane 2	929	16.4	929	16.4	1857	0.500	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	929	16.4	929	16.4	1857	0.500	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	929	16.4	929	16.4	1857	0.500	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	929	16.4	929	16.4	1857	0.500	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	4521	17.0	4521	17.0		0.521		1.5	NA	0.0	0.0			
Intersection	10549	17.3	10549	17.3		0.888		1.4	NA	0.0	0.0			

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

7 Lane under-utilisation specified by the user

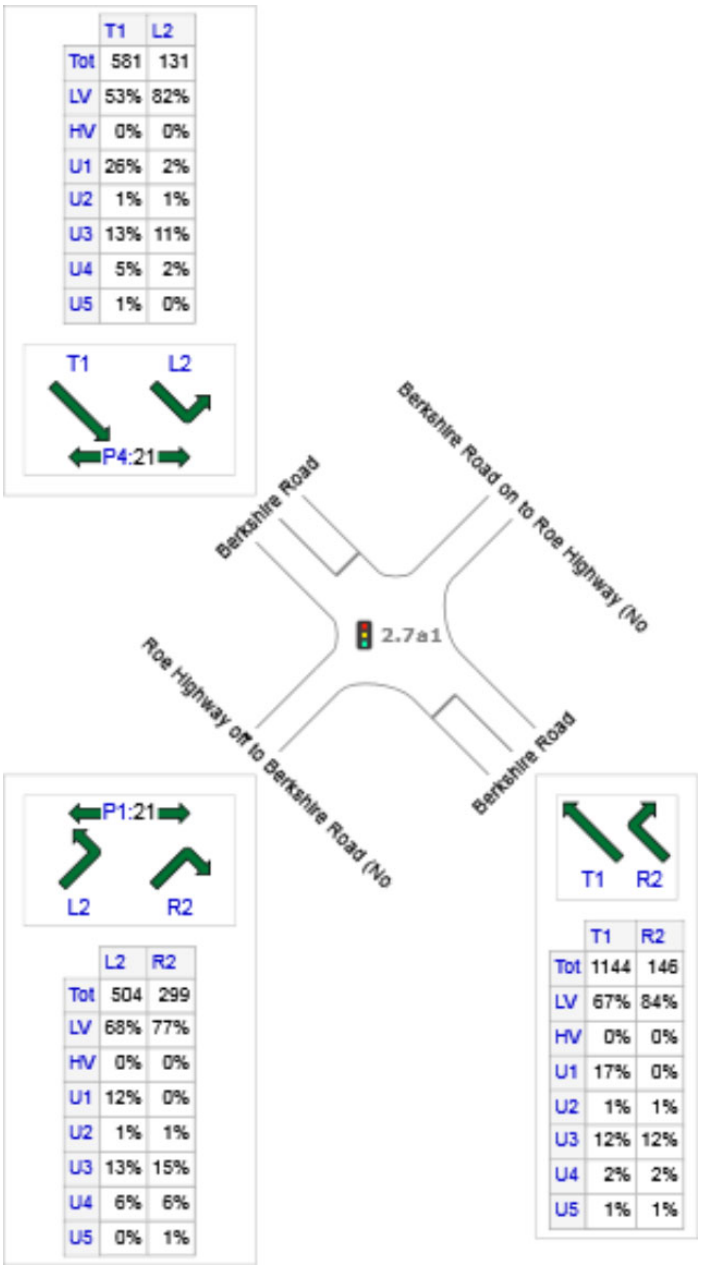
Site: 2.7a1 [2.7a1 Berkshire Road - Roe Highway off to Berkshire Road - 2031 AM (Site Folder: 2031 AM)]

Network: 1 [n2.2a Roe Highway - Berkshire Road - 2031 AM (Network Folder: 2031)]

2.7a1 Berkshire Road - Roe Highway off to Berkshire Road - 2031 AM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 50 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Downstream lane blockage effects included in determining phase times
Phase Sequence: Leading Right Turn
Reference Phase: Phase B
Input Phase Sequence: A, B, C, D
Output Phase Sequence: A, B, C, D

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developme
SE: Berkshire Road	1291	885	0	1
NW: Berkshire Road	712	418	0	1
SW: Roe Highway off to Berkshire Road (Northbound)	803	574	0	0
Total	2805	1876	0	4

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road															
Lane 1	572	15.8	572	15.8	654	0.874	100	25.7	LOS C	17.5	151.4	Full	150	0.0	5.8

Lane 2	572	15.8	572	15.8	654	0.874	100	25.7	LOS C	17.5	151.4	Full	150	0.0	5.8
Lane 3	73	14.6	73	14.6	188	0.390	100	29.7	LOS C	1.8	15.4	Full	150	0.0	0.0
Lane 4	73	14.6	73	14.6	188	0.390	100	30.1	LOS C	1.8	15.4	Short	80	0.0	NA
Approach	1291	15.6	1291	15.6		0.874		26.2	LOS C	17.5	151.4				
NorthWest: Berkshire Road															
Lane 1	238	16.6	238	16.6	438	0.544	100	12.2	LOS B	2.8	24.3	Short	100	0.0	NA
Lane 2	237	19.4	237	19.4	435	0.544	100	9.1	LOS A	2.7	25.2	Full	500	0.0	0.0
Lane 3	237	19.4	237	19.4	435	0.544	100	9.1	LOS A	2.7	25.2	Full	500	0.0	0.0
Approach	712	18.5	712	18.5		0.544		10.2	LOS B	2.8	25.2				
SouthWest: Roe Highway off to Berkshire Road (Northbound)															
Lane 1	504	18.9	504	18.9	1471	0.343	100	8.4	LOS A	0.0	0.0	Short	120	0.0	NA
Lane 2	149	21.6	149	21.6	171	0.873	100	38.8	LOS D	4.6	43.2	Short	300	0.0	NA
Lane 3	149	21.6	149	21.6	171	0.873	100	38.8	LOS D	4.6	43.2	Full	500	0.0	0.0
Approach	803	19.9	803	19.9		0.873		19.7	LOS B	4.6	43.2				
Intersection	2805	17.6	2805	17.6		0.874		20.3	LOS C	17.5	151.4				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

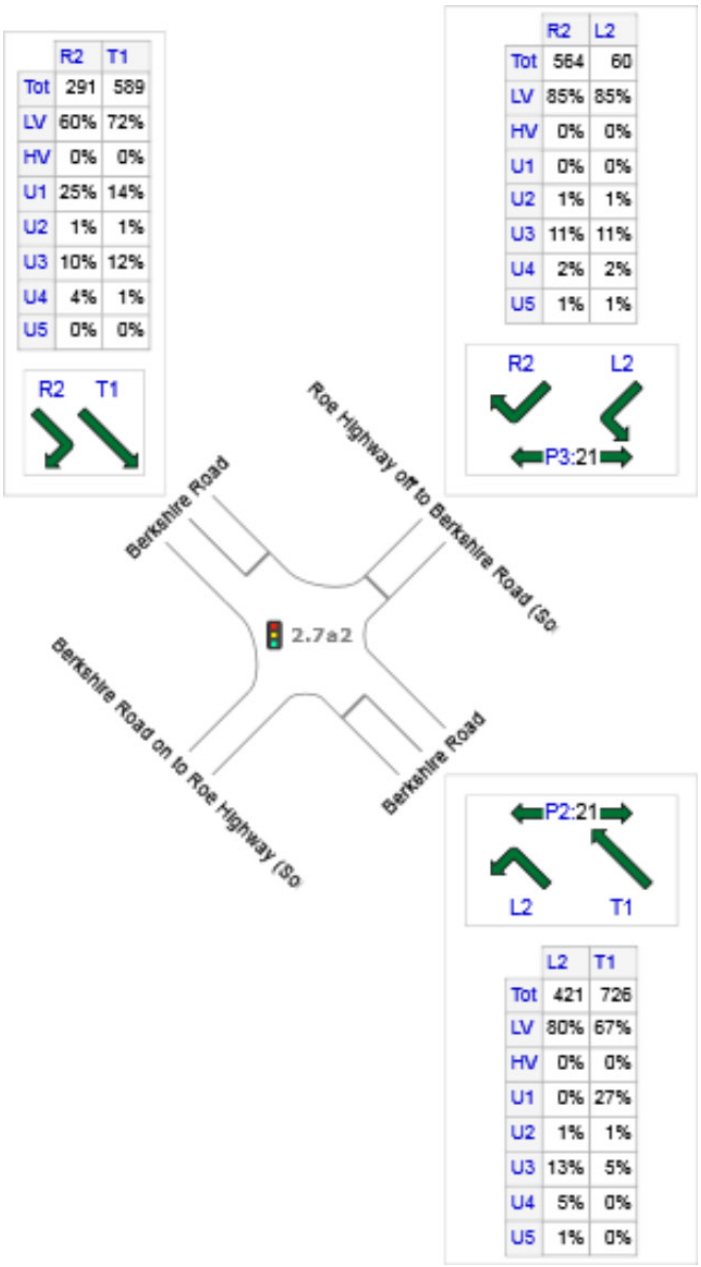
Site: 2.7a2 [2.7a2 Berkshire Road - Berkshire Road on to Roe Highway - 2031 AM (Site Folder: 2031 AM)]

Network: 1 [n2.2a Roe Highway - Berkshire Road - 2031 AM (Network Folder: 2031)]

2.7a2 Berkshire Road - Berkshire Road on to Roe Highway - 2031 AM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 70 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Downstream lane blockage effects included in determining phase times
Phase Sequence: Split Phasing
Reference Phase: Phase B
Input Phase Sequence: A, B, C, D
Output Phase Sequence: A, B, C, D

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developme
SE: Berkshire Road	1147	828	0	1
NE: Roe Highway off to Berkshire Road (Southbound)	624	530	0	
NW: Berkshire Road	880	601	0	1
Total	2652	1959	0	3

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road															
Lane 1	421	18.4	421	18.4	1014	0.415	100	10.9	LOS B	4.9	44.6	Full	300	0.0	0.0

Lane 2	363	5.1	363	5.1	400	0.907	100	29.0	LOS C	9.8	73.3	Full	300	-5.8[N3]	0.0
Lane 3	363	5.1	363	5.1	400	0.907	100	29.0	LOS C	9.8	73.3	Full	300	-5.8[N3]	0.0
Approach	1147	10.0	1147	10.0		0.907		22.3	LOS C	9.8	73.3				
NorthEast: Roe Highway off to Berkshire Road (Southbound)															
Lane 1	60	14.0	60	14.0	952	0.063	100	7.7	LOS A	0.5	3.9	Short	120	0.0	NA
Lane 2	274	14.0	274	14.0	318	0.861	100	44.5	LOS D	11.1	94.1	Short	300	-5.8[N3]	NA
Lane 3	291	14.0	291	14.0	337	0.861	100	44.1	LOS D	11.7	99.2	Full	500	0.0	0.0
Approach	624	14.0	624	14.0		0.861		40.8	LOS D	11.7	99.2				
NorthWest: Berkshire Road															
Lane 1	295	13.1	295	13.1	713	0.414	100	15.8	LOS B	7.2	59.1	Full	150	0.0	0.0
Lane 2	295	13.1	295	13.1	713	0.414	100	15.8	LOS B	7.2	59.1	Full	150	0.0	0.0
Lane 3	291	13.7	291	13.7	335	0.867	100	44.7	LOS D	11.8	100.7	Full	150	0.0	0.0
Approach	880	13.3	880	13.3		0.867		25.4	LOS C	11.8	100.7				
Intersection	2652	12.0	2652	12.0		0.907		27.7	LOS C	11.8	100.7				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

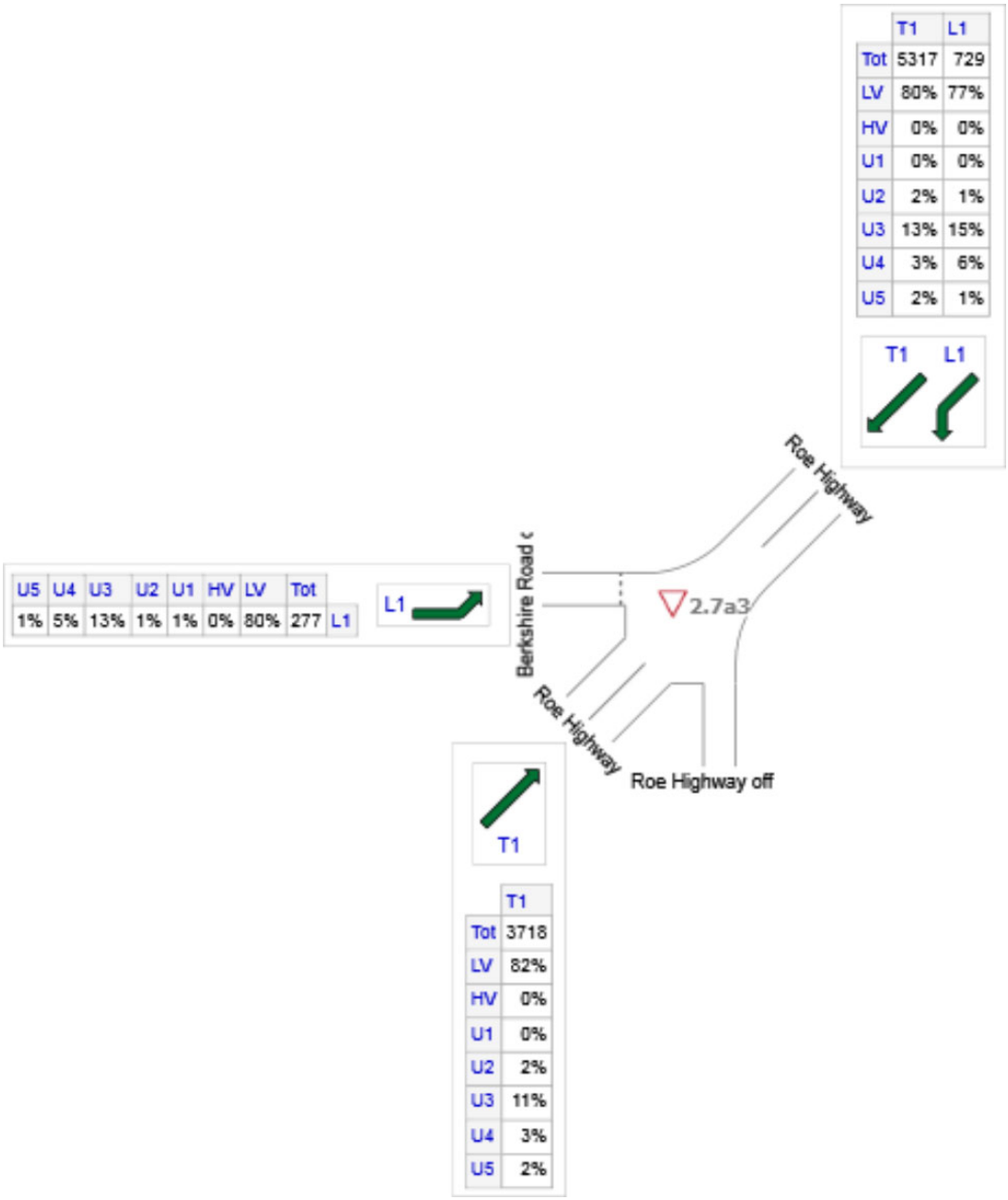
N3 Capacity Adjustment due to downstream lane blockage determined by the program.

Site: 2.7a3 [2.7a3 Roe Highway Interchange with Berkshire Road - 2031 AM (Site Folder: 2031 AM)]

Network: 1 [n2.2a Roe Highway - Berkshire Road - 2031 AM (Network Folder: 2031)]

2.7a3 Roe Highway Interchange with Berkshire Road - 2031 AM
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developmen

NE: Roe Highway	6046	4845	0	0
W: Berkshire Road on to Roe Highway (Northbound)	277	220	0	3
SW: Roe Highway	3718	3036	0	2
Total	10041	8101	0	5

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
NorthEast: Roe Highway															
Lane 1	729	21.6	729	21.6	1512	0.482	100	8.3	LOS A	0.0	0.0	Short	150	0.0	NA
Lane 2	1329	17.7	1329	17.7	1842	0.722	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1329	17.7	1329	17.7	1842	0.722	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1329	17.7	1329	17.7	1842	0.722	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	1329	17.7	1329	17.7	1842	0.722	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	6046	18.2	6046	18.2		0.722		1.2	NA	0.0	0.0				
West: Berkshire Road on to Roe Highway (Northbound)															
Lane 1	277	18.2	277	18.2	1547	0.179	100	5.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	277	18.2	277	18.2		0.179		5.0	NA	0.0	0.0				
SouthWest: Roe Highway															
Lane 1	286	16.4	286	16.4	1857	0.154	25[7]	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	1144	16.4	1144	16.4	1857	0.616	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1144	16.4	1144	16.4	1857	0.616	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1144	16.4	1144	16.4	1857	0.616	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	3718	16.4	3718	16.4		0.616		0.1	NA	0.0	0.0				
Intersection	10041	17.5	10041	17.5		0.722		0.9	NA	0.0	0.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

7 Lane under-utilisation specified by the user

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Project: C:\Users\User\Box\KCTT Projects\KC00000 Current Projects\KC00604.000 Forrestfield North - Structure Plan Preparation\Outgoing \SIDRA\DCP\210708 Rev B\Future Models\KC00604.000 M07 Roe Highway - Berkshire Road.sip9

USER REPORT FOR NETWORK SITE

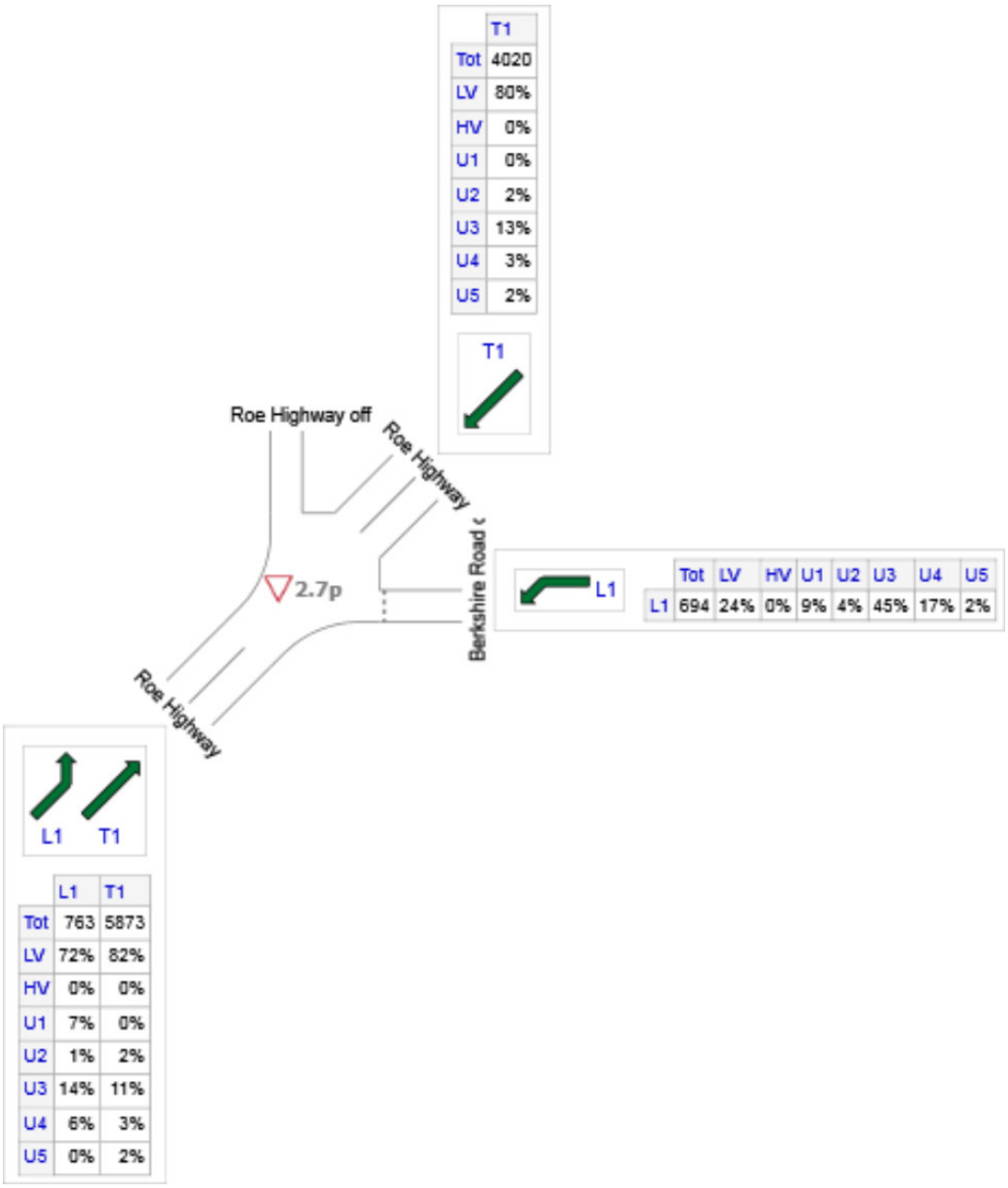
All Movement Classes

Project: KC00604.000 M07 Roe Highway - Berkshire Road **Template: Network Site User Report**

Site: 2.7p [2.7p Roe Highway Interchange with Berkshire Road - 2031 PM (South) (Site Folder: 2031 PM)] **Network: 2 [n2.2p Roe Highway - Berkshire Road - 2031 PM (Network Folder: 2031)]**

2.7p Roe Highway Interchange with Berkshire Road - 2031 PM (South)
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development
E: Berkshire Road on to Roe Highway (Southbound)	694	166	0	60
NE: Roe Highway	4020	3236	0	0
SW: Roe Highway	6636	5343	0	59
Total	11349	8745	0	119

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Berkshire Road on to Roe Highway (Southbound)															
Lane 1	694	63.6	694	63.6	1019	0.681	100	6.5	LOS A	0.0	0.0	Full	500	0.0	0.0

Approach	694	63.6	694	63.6		0.681		6.5	NA	0.0	0.0				
NorthEast: Roe Highway															
Lane 1	309	17.7	309	17.7	1842	0.168	25[7]	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	1237	17.7	1237	17.7	1842	0.672	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1237	17.7	1237	17.7	1842	0.672	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1237	17.7	1237	17.7	1842	0.672	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	4020	17.7	4020	17.7		0.672		0.1	NA	0.0	0.0				
SouthWest: Roe Highway															
Lane 1	763	20.0	763	20.0	1539	0.496	100	8.2	LOS A	0.0	0.0	Short	150	0.0	NA
Lane 2	1468	16.4	1468	16.4	1857	0.790	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1468	16.4	1468	16.4	1857	0.790	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1468	16.4	1468	16.4	1857	0.790	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	1468	16.4	1468	16.4	1857	0.790	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	6636	16.8	6636	16.8		0.790		1.2	NA	0.0	0.0				
Intersection	11349	20.0	11349	20.0		0.790		1.1	NA	0.0	0.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

7 Lane under-utilisation specified by the user

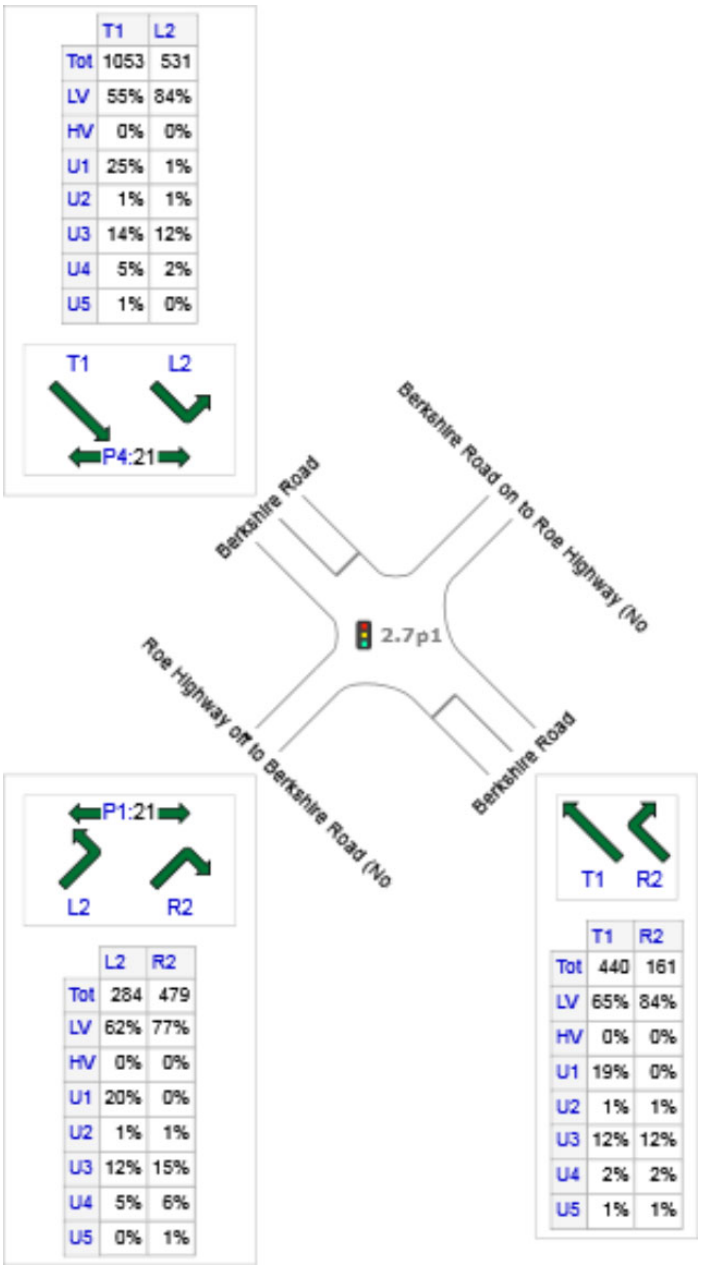
Site: 2.7p1 [2.7p1 Berkshire Road - Roe Highway off to Berkshire Road - 2031 PM (Site Folder: 2031 PM)]

Network: 2 [n2.2p Roe Highway - Berkshire Road - 2031 PM (Network Folder: 2031)]

2.7p1 Berkshire Road - Roe Highway off to Berkshire Road - 2031 PM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 70 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Downstream lane blockage effects included in determining phase times
Phase Sequence: Leading Right Turn
Reference Phase: Phase B
Input Phase Sequence: A, B, C, D
Output Phase Sequence: A, B, C, D

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developme
SE: Berkshire Road	601	422	0	
NW: Berkshire Road	1583	1022	0	2
SW: Roe Highway off to Berkshire Road (Northbound)	763	547	0	
Total	2947	1991	0	4

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road															
Lane 1	220	15.5	220	15.5	539	0.408	100	20.1	LOS C	6.0	51.4	Full	150	0.0	0.0

Lane 2	220	15.5	220	15.5	539	0.408	100	20.1	LOS C	6.0	51.4	Full	150	0.0	0.0
Lane 3	81	14.6	81	14.6	134	0.601	100	42.7	LOS D	2.9	25.1	Full	150	0.0	0.0
Lane 4	81	14.6	81	14.6	134	0.601	100	44.3	LOS D	2.9	25.1	Short	80	0.0	NA
Approach	601	15.2	601	15.2		0.601		26.3	LOS C	6.0	51.4				
NorthWest: Berkshire Road															
Lane 1	533	14.5	533	14.5	604	0.882	100	29.7	LOS C	15.0	128.0	Short	100	0.0	NA
Lane 2	525	19.9	525	19.9	596	0.882	100	24.1	LOS C	14.9	137.1	Full	500	0.0	0.0
Lane 3	525	19.9	525	19.9	596	0.882	100	24.1	LOS C	14.9	137.1	Full	500	0.0	0.0
Approach	1583	18.1	1583	18.1		0.882		26.0	LOS C	15.0	137.1				
SouthWest: Roe Highway off to Berkshire Road (Northbound)															
Lane 1	284	17.3	284	17.3	1499	0.190	100	6.4	LOS A	0.0	0.0	Short	120	0.0	NA
Lane 2	239	21.6	239	21.6	265	0.903	100	51.2	LOS D	10.5	98.5	Short	300	0.0	NA
Lane 3	239	21.6	239	21.6	265	0.903	100	51.2	LOS D	10.5	98.5	Full	500	0.0	0.0
Approach	763	20.0	763	20.0		0.903		34.5	LOS C	10.5	98.5				
Intersection	2947	18.0	2947	18.0		0.903		28.3	LOS C	15.0	137.1				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

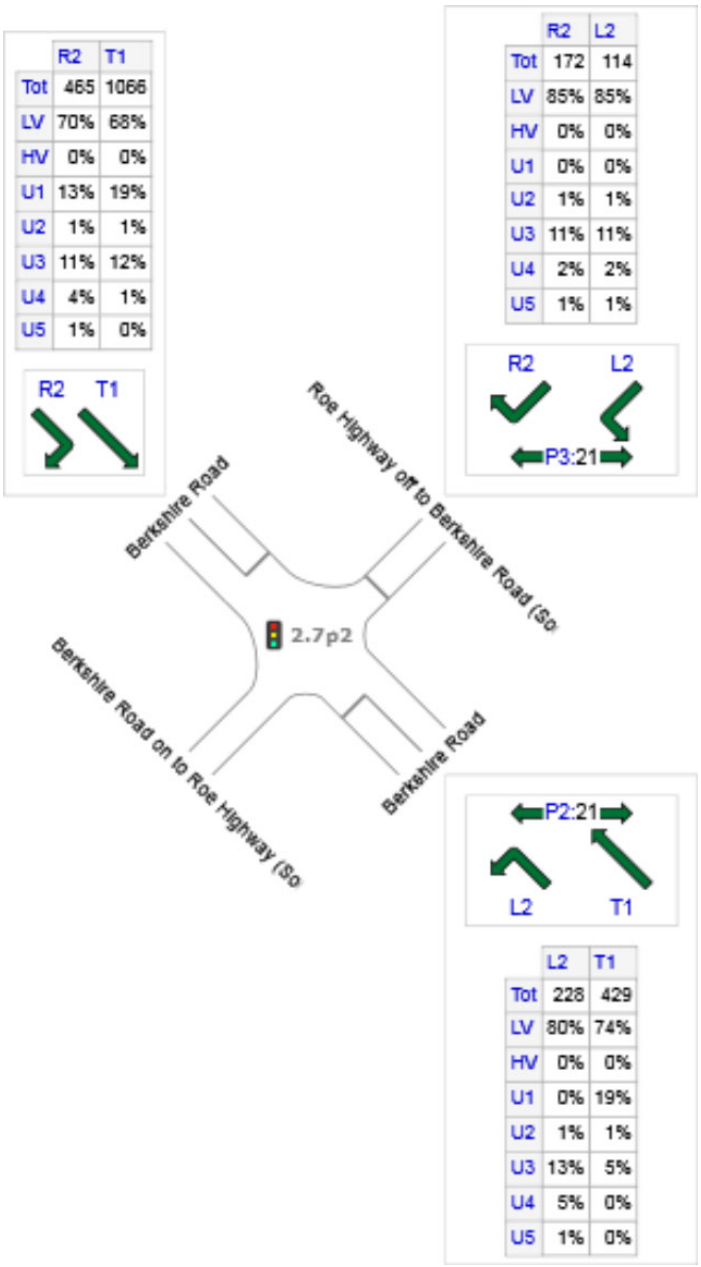
Site: 2.7p2 [2.7p2 Berkshire Road - Berkshire Road on to Roe Highway - 2031 PM (Site Folder: 2031 PM)]

Network: 2 [n2.2p Roe Highway - Berkshire Road - 2031 PM (Network Folder: 2031)]

2.7p2 Berkshire Road - Berkshire Road on to Roe Highway - 2031 PM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 70 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Downstream lane blockage effects included in determining phase times
Phase Sequence: Split Phasing
Reference Phase: Phase B
Input Phase Sequence: A, B, C, D
Output Phase Sequence: A, B, C, D

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developme
SE: Berkshire Road	658	503	0	8
NE: Roe Highway off to Berkshire Road (Southbound)	285	242	0	
NW: Berkshire Road	1532	1053	0	2
Total	2475	1798	0	3

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road															
Lane 1	228	18.4	228	18.4	817	0.280	100	13.3	LOS B	3.6	32.1	Full	300	0.0	0.0

Lane 2	215	5.6	215	5.6	317	0.678	100	19.3	LOS B	5.4	40.6	Full	300	0.0	0.0
Lane 3	215	5.6	215	5.6	317	0.678	100	19.3	LOS B	5.4	40.6	Full	300	0.0	0.0
Approach	658	10.0	658	10.0		0.678		17.2	LOS B	5.4	40.6				
NorthEast: Roe Highway off to Berkshire Road (Southbound)															
Lane 1	114	14.0	114	14.0	646	0.176	100	9.7	LOS A	1.4	11.7	Short	120	0.0	NA
Lane 2	86	14.0	86	14.0	135	0.636	100	43.1	LOS D	3.2	26.8	Short	300	0.0	NA
Lane 3	86	14.0	86	14.0	135	0.636	100	43.1	LOS D	3.2	26.8	Full	500	0.0	0.0
Approach	285	14.0	285	14.0		0.636		29.8	LOS C	3.2	26.8				
NorthWest: Berkshire Road															
Lane 1	533	12.4	533	12.4	990	0.538	100	10.0	LOS B	11.3	92.1	Full	150	0.0	0.0
Lane 2	533	12.4	533	12.4	990	0.538	100	10.0	LOS B	11.3	92.1	Full	150	0.0	0.0
Lane 3	465	16.0	465	16.0	608	0.765	100	28.7	LOS C	15.2	133.4	Full	150	0.0	0.0
Approach	1532	13.5	1532	13.5		0.765		15.7	LOS B	15.2	133.4				
Intersection	2475	12.6	2475	12.6		0.765		17.7	LOS B	15.2	133.4				

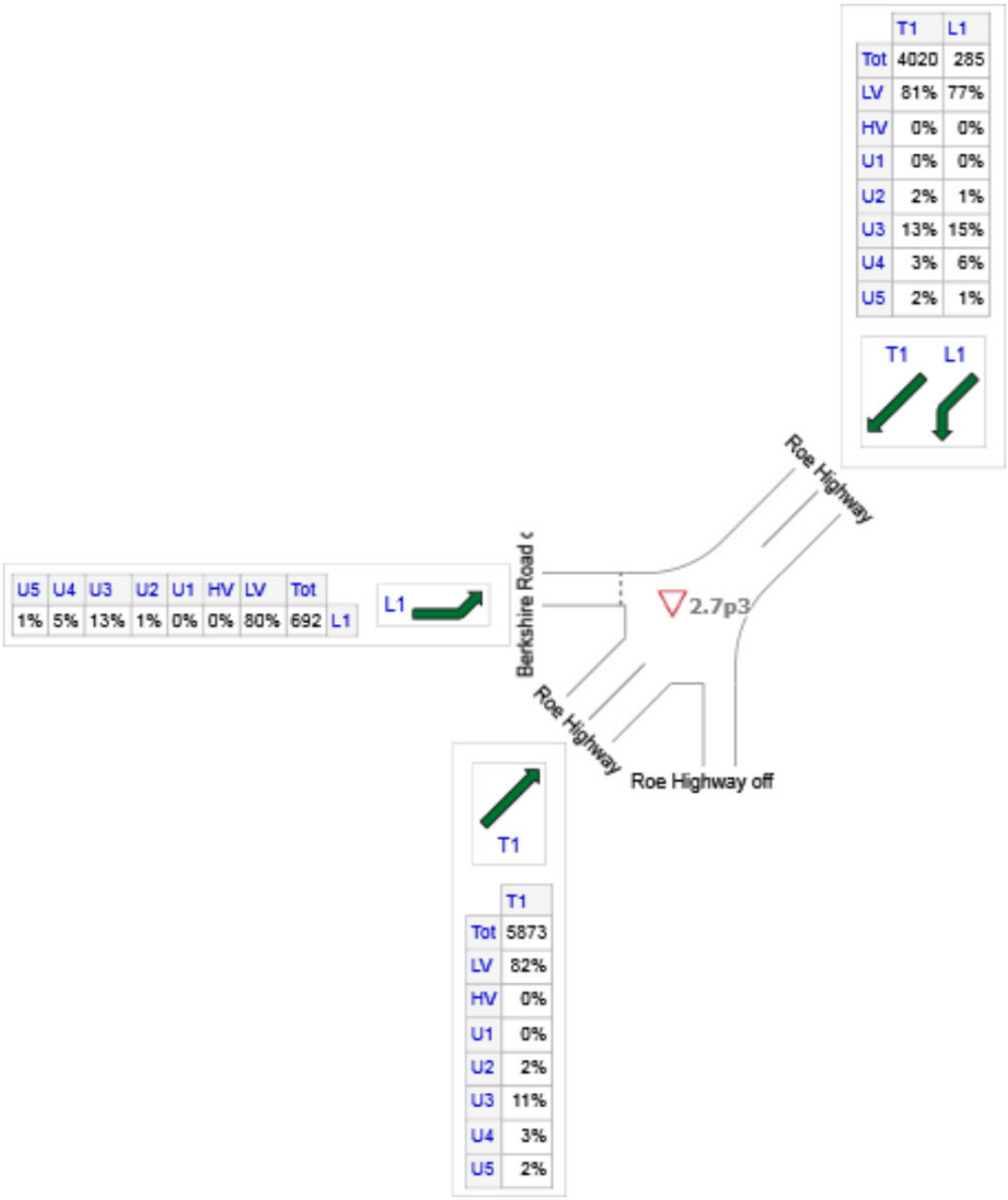
Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Site: 2.7p3 [2.7p3 Roe Highway Interchange with Berkshire Road - 2031 PM (Site Folder: 2031 PM)]

Network: 2 [n2.2p Roe Highway - Berkshire Road - 2031 PM (Network Folder: 2031)]

2.7p3 Roe Highway Interchange with Berkshire Road - 2031 PM
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developmen

NE: Roe Highway	4305	3457	0	0
W: Berkshire Road on to Roe Highway (Northbound)	692	554	0	3
SW: Roe Highway	5873	4796	0	2
Total	10869	8807	0	5

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
NorthEast: Roe Highway															
Lane 1	285	21.6	285	21.6	1512	0.189	100	8.2	LOS A	0.0	0.0	Short	150	0.0	NA
Lane 2	1005	17.7	1005	17.7	1842	0.546	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1005	17.7	1005	17.7	1842	0.546	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1005	17.7	1005	17.7	1842	0.546	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	1005	17.7	1005	17.7	1842	0.546	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	4305	18.0	4305	18.0		0.546		0.6	NA	0.0	0.0				
West: Berkshire Road on to Roe Highway (Northbound)															
Lane 1	692	18.3	692	18.3	1545	0.448	100	6.8	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	692	18.3	692	18.3		0.448		6.8	NA	0.0	0.0				
SouthWest: Roe Highway															
Lane 1	452	16.4	452	16.4	1857	0.243	25[7]	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	1807	16.4	1807	16.4	1857	0.973	100	2.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1807	16.4	1807	16.4	1857	0.973	100	2.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1807	16.4	1807	16.4	1857	0.973	100	2.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	5873	16.4	5873	16.4		0.973		2.1	NA	0.0	0.0				
Intersection	10869	17.1	10869	17.1		0.973		1.8	NA	0.0	0.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

7 Lane under-utilisation specified by the user

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Project: C:\Users\User\Box\KCTT Projects\KC00000 Current Projects\KC00604.000 Forrestfield North - Structure Plan Preparation\Outgoing \SIDRA\DCP\210708 Rev B\Future Models\KC00604.000 M07 Roe Highway - Berkshire Road.sip9

USER REPORT FOR NETWORK SITE

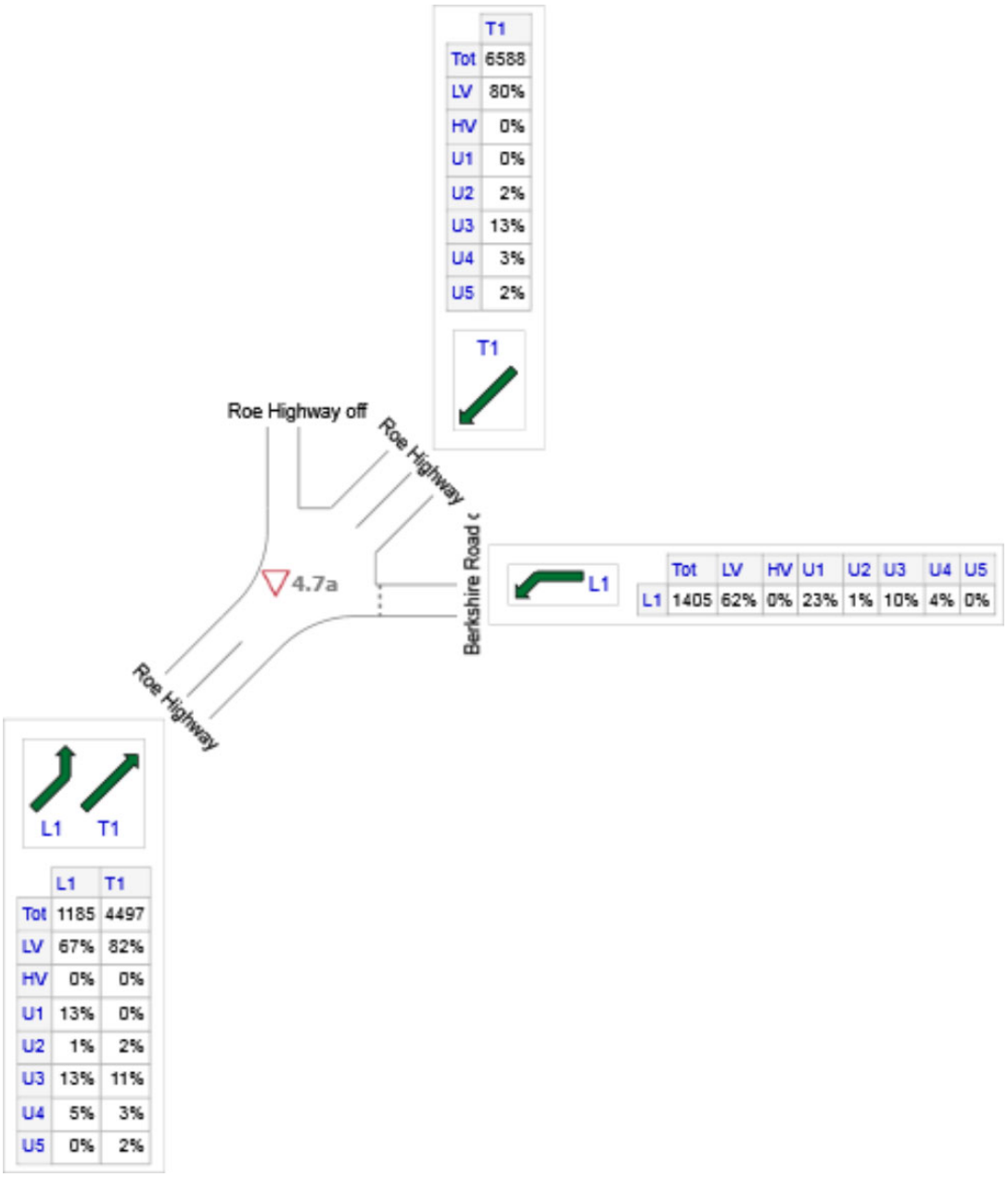
All Movement Classes

Project: KC00604.000 M07 Roe Highway - Berkshire Road **Template: Network Site User Report**

Site: 4.7a [4.7a Roe Highway Interchange with Berkshire Road - 2050 AM (South) (Site Folder: 2050 AM)] **Network: 5 [n4.2a Roe Highway - Berkshire Road - 2050 AM (Network Folder: 2050)]**

4.7a Roe Highway Interchange with Berkshire Road - 2050 AM (South)
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



Some reduced upstream exit flow rates exist due to capacity constraint applied to oversaturated approach lanes. See Arrival Flows in Lane Summary reports.

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development
E: Berkshire Road on to Roe Highway (Southbound)	1405	871	0	322
NE: Roe Highway	6588	5304	0	0
SW: Roe Highway	5682	4466	0	162
Total	13675	10641	0	484

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Berkshire Road on to Roe Highway (Southbound)															

Lane 1	1431	14.1	1405	14.2	1621	0.867	100	162.4	LOS F	0.0	0.0	Full	500	0.0	0.0
Approach	1431	14.1	1405 N1	14.2		0.867		162.4	NA	0.0	0.0				
NorthEast: Roe Highway															
Lane 1	507	17.7	507	17.7	1842	0.275	25 7	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	2027	17.7	2027	17.7	1842	1.101	100	47.0	LOS E	0.0	0.0	Full	500	0.0	0.0
Lane 3	2027	17.7	2027	17.7	1842	1.101	100	47.0	LOS E	0.0	0.0	Full	500	0.0	0.0
Lane 4	2027	17.7	2027	17.7	1842	1.101	100	47.0	LOS E	0.0	0.0	Full	500	0.0	0.0
Approach	6588	17.7	6588	17.7		1.101		43.4	NA	0.0	0.0				
SouthWest: Roe Highway															
Lane 1	1185	18.8	1185	18.8	1561	0.759	100	8.3	LOS A	0.0	0.0	Short	150	0.0	NA
Lane 2	1124	16.4	1124	16.4	1858	0.605	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1124	16.4	1124	16.4	1858	0.605	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1124	16.4	1124	16.4	1858	0.605	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	1124	16.4	1124	16.4	1858	0.605	100	0.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	5682	16.9	5682	16.9		0.759		1.8	NA	0.0	0.0				
Intersection	13701	17.0	13675 N1	17.0		1.101		38.3	NA	0.0	0.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

7 Lane under-utilisation specified by the user
N1 Arrival Flow value is reduced due to capacity constraint at oversaturated upstream lanes.

Site: 4.7a1 [4.7a1 Berkshire Road - Roe Highway off to Berkshire Road - 2050 AM (Site Folder: 2050 AM)]

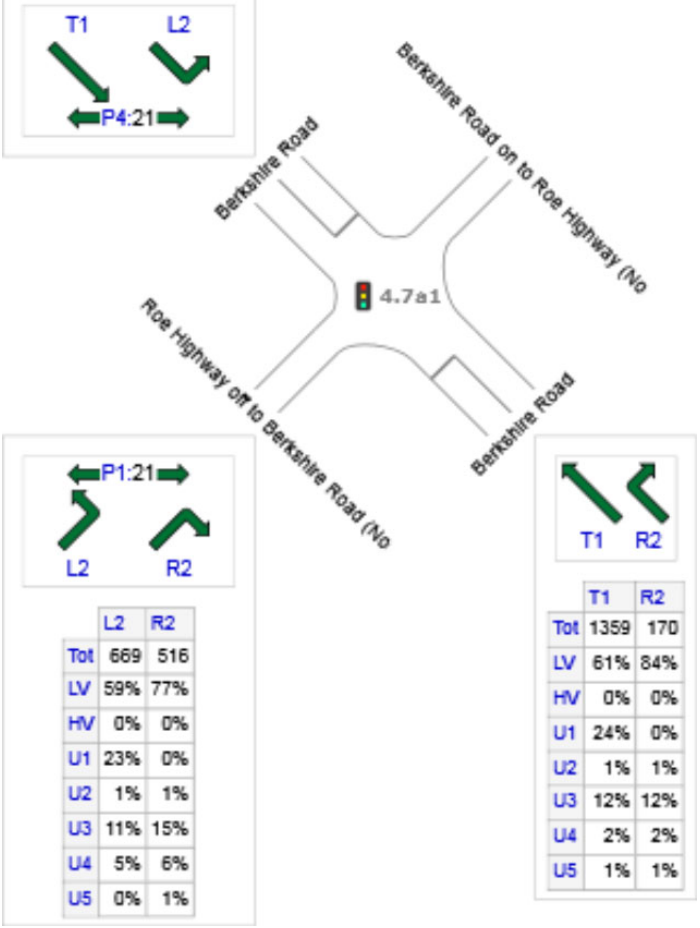
Network: 5 [n4.2a Roe Highway - Berkshire Road - 2050 AM (Network Folder: 2050)]

4.7a1 Berkshire Road - Roe Highway off to Berkshire Road - 2050 AM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 150 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Downstream lane blockage effects included in determining phase times
Phase Sequence: Leading Right Turn
Reference Phase: Phase B
Input Phase Sequence: A, B, C, D
Output Phase Sequence: A, B, C, D

OD Arrival Flows

	T1	L2
Tot	1073	139
LV	36%	82%
HV	0%	0%
U1	50%	3%
U2	0%	1%
U3	9%	11%
U4	3%	2%
U5	1%	0%



	L2	R2
Tot	669	516
LV	59%	77%
HV	0%	0%
U1	23%	0%
U2	1%	1%
U3	11%	15%
U4	5%	6%
U5	0%	1%

	T1	R2
Tot	1359	170
LV	61%	84%
HV	0%	0%
U1	24%	0%
U2	1%	1%
U3	12%	12%
U4	2%	2%
U5	1%	1%

Some reduced upstream exit flow rates exist due to capacity constraint applied to oversaturated approach lanes. See Arrival Flows in Lane Summary reports.

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developme
SE: Berkshire Road	1530	969	0	3
NW: Berkshire Road	1212	503	0	5
SW: Roe Highway off to Berkshire Road (Northbound)	1185	797	0	1
Total	3927	2268	0	1(

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road															

Lane 1	715	14.4	680	14.8	683	0.995	100	97.5	LOS F	28.6[N4]	244.8[N4]	Full	150	0.0	50.0
Lane 2	715	14.4	680	14.8	683	0.995	100	97.5	LOS F	28.6[N4]	244.8[N4]	Full	150	0.0	50.0
Lane 3	89	14.6	85	14.9	94	0.911	100	98.5	LOS F	7.2	61.9	Full	150	0.0	0.0
Lane 4	89	14.6	85	14.9	94	0.911	100	98.9	LOS F	7.2	61.9	Short	80	0.0	NA
Approach	1609	14.5	1530[N 1]	14.8		0.995		97.6	LOS F	28.6	244.8				
NorthWest: Berkshire Road															
Lane 1	139	14.2	139	14.2	671	0.207	100	24.7	LOS C	5.0	42.4	Short	100	0.0	NA
Lane 2	358	13.2	358	13.2	355	1.006	100	93.3	LOS F	42.8[N5]	362.5[N5]	Short	100	-50.0[N3]	NA
Lane 3	358	13.2	358	13.2	355	1.006	100	93.3	LOS F	30.1	254.8	Full	500	-50.0[N3]	0.0
Lane 4	358	13.2	358	13.2	355	1.006	100	93.3	LOS F	30.1	254.8	Full	500	-50.0[N3]	0.0
Approach	1212	13.3	1212	13.3		1.006		85.4	LOS F	42.8	362.5				
SouthWest: Roe Highway off to Berkshire Road (Northbound)															
Lane 1	669	16.6	669	16.6	1511	0.443	100	10.3	LOS B	0.0	0.0	Short	120	0.0	NA
Lane 2	258	21.6	258	21.6	252	1.023	100	145.0	LOS F	31.0	290.4	Short	300	-50.0[N3]	NA
Lane 3	258	21.6	258	21.6	252	1.023	100	145.0	LOS F	31.0	290.4	Full	500	-50.0[N3]	0.0
Approach	1185	18.8	1185	18.8		1.023		68.9	LOS E	31.0	290.4				
Intersection	4006	15.4	3927[N 1]	15.7		1.023		85.2	LOS F	42.8	362.5				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

N1 Arrival Flow value is reduced due to capacity constraint at oversaturated upstream lanes.
N3 Capacity Adjustment due to downstream lane blockage determined by the program.
N4 Average back of queue has been restricted to the available queue storage space.
N5 Continuous Lane results determined by Back of Queue values of downstream lanes (proportional to lane movement flows).

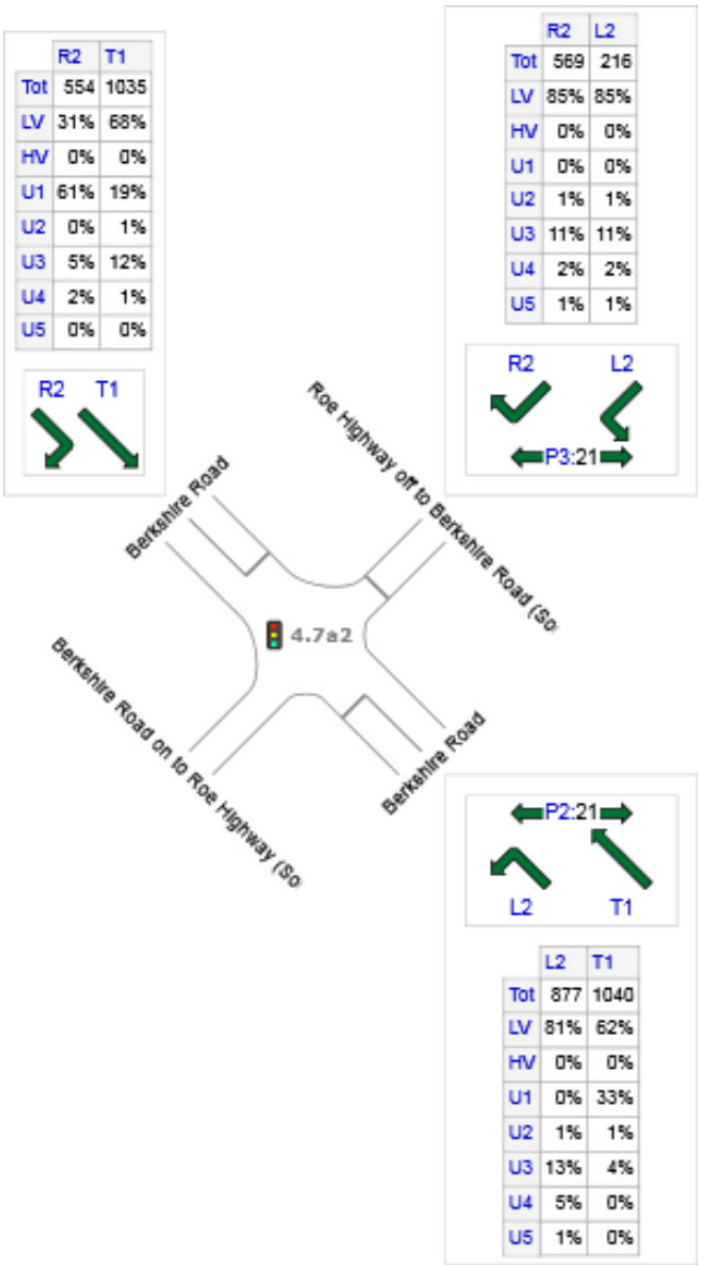
Site: 4.7a2 [4.7a2 Berkshire Road - Berkshire Road on to Roe Highway - 2050 AM (Site Folder: 2050 AM)]

Network: 5 [n4.2a Roe Highway - Berkshire Road - 2050 AM (Network Folder: 2050)]

4.7a2 Berkshire Road - Berkshire Road on to Roe Highway - 2050 AM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 150 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Downstream lane blockage effects included in determining phase times
Phase Sequence: Split Phasing
Reference Phase: Phase B
Input Phase Sequence: A, B, C, D
Output Phase Sequence: A, B, C, D

OD Arrival Flows



Some reduced upstream exit flow rates exist due to capacity constraint applied to oversaturated approach lanes. See Arrival Flows in Lane Summary reports.

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developme
SE: Berkshire Road	1917	1347	0	3
NE: Roe Highway off to Berkshire Road (Southbound)	785	666	0	
NW: Berkshire Road	1588	873	0	5
Total	4291	2886	0	8

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road															

Lane 1	877	18.4	877	18.4	1479	0.593	100	7.3	LOS A	0.0	0.0	Full	300	0.0	0.0
Lane 2	520	4.6	520	4.6	448	1.160	100	210.1	LOS F	68.3	508.8	Full	300	-50.0 N3	53.6
Lane 3	520	4.6	520	4.6	448	1.160	100	210.1	LOS F	68.3	508.8	Full	300	-50.0 N3	53.6
Approach	1917	10.9	1917	10.9		1.160		117.4	LOS F	68.3	508.8				
NorthEast: Roe Highway off to Berkshire Road (Southbound)															
Lane 1	216	14.0	216	14.0	1041	0.207	100	16.2	LOS B	5.9	49.9	Short	120	0.0	NA
Lane 2	190	14.0	190	14.0	168	1.130	100	220.1	LOS F	27.2	229.8	Short	300	-50.0 N3	NA
Lane 3	380	14.0	380	14.0	336	1.130	100	208.6	LOS F	52.1	440.3	Full	500	0.0	0.0
Approach	785	14.0	785	14.0		1.130		158.5	LOS F	52.1	440.3				
NorthWest: Berkshire Road															
Lane 1	576	12.3	576	12.3	509	1.132	100	199.9	LOS F	30.1 N4	244.8 N4	Full	150	0.0	50.0
Lane 2	459	12.3	459	12.3	405 1	1.132	100	203.5	LOS F	30.1 N4	244.8 N4	Full	150	0.0	50.0 8
Lane 3	276	7.2	276	7.2	248 1	1.115	100	199.7	LOS F	31.4 N4	244.8 N4	Short	80	0.0	NA
Lane 4	278	7.2	278	7.2	249	1.115	100	197.3	LOS F	31.4 N4	244.8 N4	Full	150	0.0	50.0
Approach	1588	10.5	1588	10.5		1.132		200.5	LOS F	31.4	244.8				
Intersection	4291	11.3	4291	11.3		1.160		155.7	LOS F	68.3	508.8				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

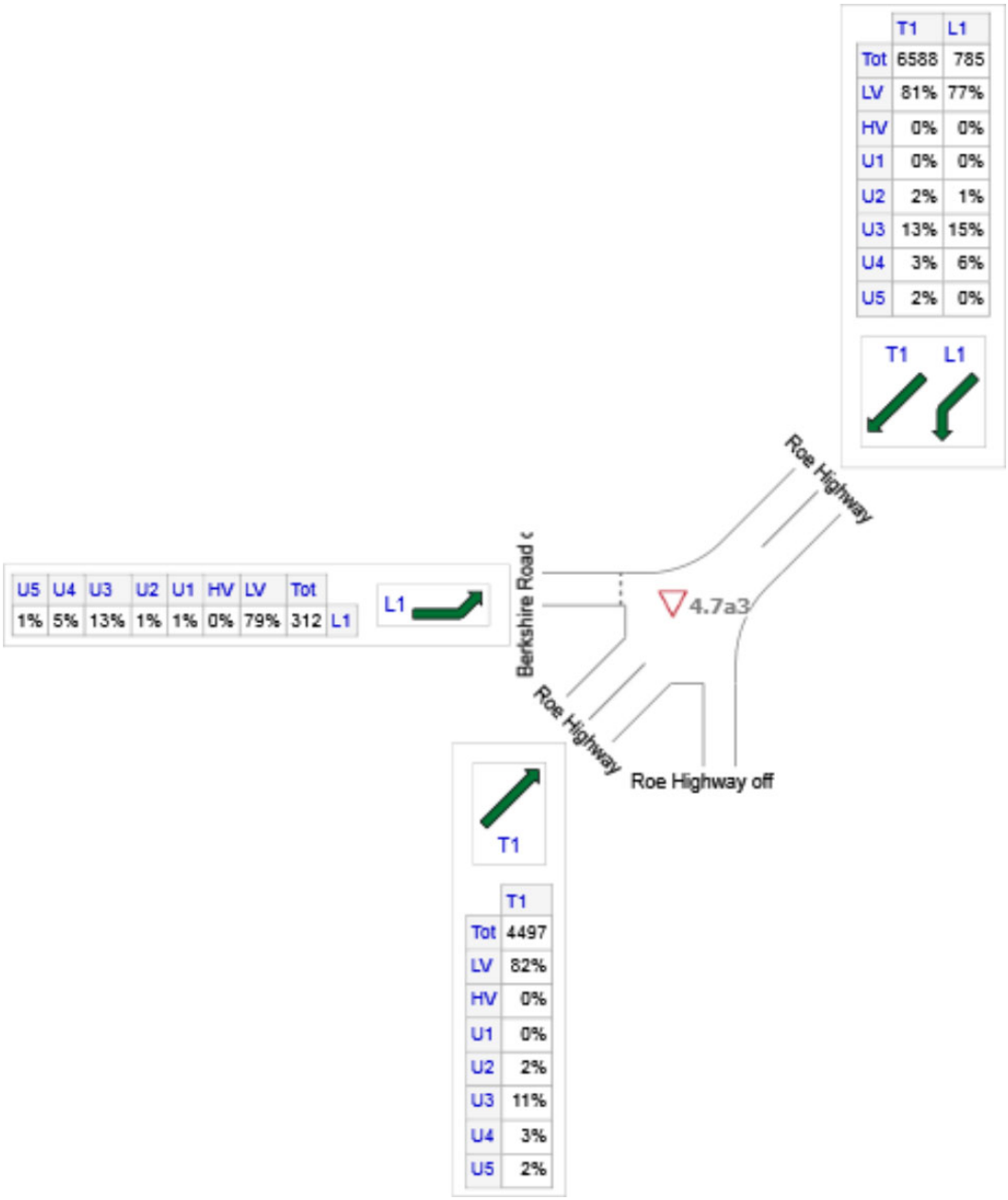
1 Reduced capacity due to a short lane effect. Short lane queues may extend into the full-length lanes. Some upstream delays at entry to short lanes are not included.
8 Probability of Blockage has been set on the basis of a queue that overflows from a short lane.
N3 Capacity Adjustment due to downstream lane blockage determined by the program.
N4 Average back of queue has been restricted to the available queue storage space.

Site: 4.7a3 [4.7a3 Roe Highway Interchange with Berkshire Road - 2050 AM (Site Folder: 2050 AM)]

Network: 5 [n4.2a Roe Highway - Berkshire Road - 2050 AM (Network Folder: 2050)]

4.7a3 Roe Highway Interchange with Berkshire Road - 2050 AM
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



Some reduced upstream exit flow rates exist due to capacity constraint applied to oversaturated approach lanes. See Arrival Flows in Lane Summary reports.

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developmen
NE: Roe Highway	7374	5911	0	1
W: Berkshire Road on to Roe Highway (Northbound)	312	247	0	4
SW: Roe Highway	4497	3669	0	6
Total	12182	9826	0	12

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
NorthEast: Roe Highway															
Lane 1	785	21.6	785	21.6	1513	0.519	100	8.3	LOS A	0.0	0.0	Short	150	0.0	NA
Lane 2	1647	17.7	1647	17.7	1842	0.894	100	0.6	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1647	17.7	1647	17.7	1842	0.894	100	0.6	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1647	17.7	1647	17.7	1842	0.894	100	0.6	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	1647	17.7	1647	17.7	1842	0.894	100	0.6	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	7374	18.1	7374	18.1		0.894		1.4	NA	0.0	0.0				
West: Berkshire Road on to Roe Highway (Northbound)															
Lane 1	318	18.2	312	18.3	1544	0.202	100	5.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	318	18.2	312 N1	18.3		0.202		5.2	NA	0.0	0.0				
SouthWest: Roe Highway															
Lane 1	346	16.4	346	16.4	1858	0.186	25 7	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	1384	16.4	1384	16.4	1858	0.745	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1384	16.4	1384	16.4	1858	0.745	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1384	16.4	1384	16.4	1858	0.745	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	4497	16.4	4497	16.4		0.745		0.2	NA	0.0	0.0				
Intersection	12188	17.5	12182 N1	17.5		0.894		1.1	NA	0.0	0.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

7 Lane under-utilisation specified by the user
N1 Arrival Flow value is reduced due to capacity constraint at oversaturated upstream lanes.

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Project: C:\Users\User\Box\KCTT Projects\KC00000 Current Projects\KC00604.000 Forrestfield North - Structure Plan Preparation\Outgoing \SIDRA\DCP\210708 Rev B\Future Models\KC00604.000 M07 Roe Highway - Berkshire Road.sip9

USER REPORT FOR NETWORK SITE

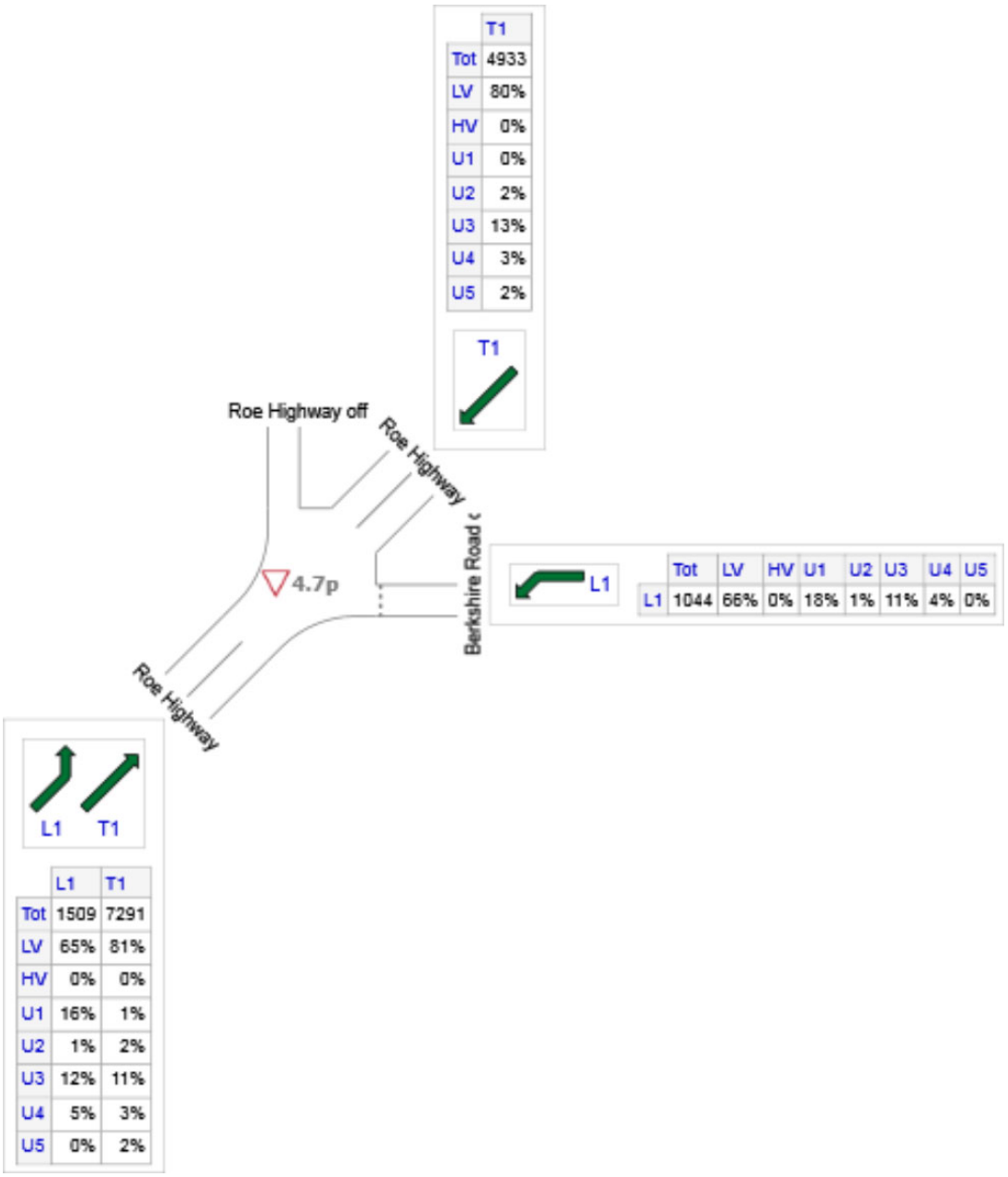
All Movement Classes

Project: KC00604.000 M07 Roe Highway - Berkshire Road **Template: Network Site User Report**

Site: 4.7p [4.7p Roe Highway Interchange with Berkshire Road - 2050 PM (South) (Site Folder: 2050 PM)] **Network: 6 [n4.2p Roe Highway - Berkshire Road - 2050 PM (Network Folder: 2050)]**

4.7p Roe Highway Interchange with Berkshire Road - 2050 PM (South)
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



Some reduced upstream exit flow rates exist due to capacity constraint applied to oversaturated approach lanes. See Arrival Flows in Lane Summary reports.

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Development
E: Berkshire Road on to Roe Highway (Southbound)	1044	691	0	186
NE: Roe Highway	4933	3971	0	0
SW: Roe Highway	8800	6902	0	285
Total	14777	11564	0	471

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
East: Berkshire Road on to Roe Highway (Southbound)															

Lane 1	1118	15.1	1044	15.1	1603	0.651	100	8.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	1118	15.1	1044 N1	15.1		0.651		8.3	NA	0.0	0.0				
NorthEast: Roe Highway															
Lane 1	379	17.7	379	17.7	1842	0.206	25 7	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	1518	17.7	1518	17.7	1842	0.824	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1518	17.7	1518	17.7	1842	0.824	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1518	17.7	1518	17.7	1842	0.824	100	0.3	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	4933	17.7	4933	17.7		0.824		0.3	NA	0.0	0.0				
SouthWest: Roe Highway															
Lane 1	1509	18.1	1509	18.1	1206	1.252	100	116.3	LOS F	0.0	0.0	Short	150	-23.3 N3	NA
Lane 2	1823	16.3	1823	16.3	1859	0.980	100	2.8	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1823	16.3	1823	16.3	1859	0.980	100	2.8	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1823	16.3	1823	16.3	1859	0.980	100	2.8	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	1823	16.3	1823	16.3	1859	0.980	100	2.8	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	8800	16.6	8800	16.6		1.252		22.3	NA	0.0	0.0				
Intersection	14851	16.9	14777 N1	16.9		1.252		14.0	NA	0.0	0.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

7 Lane under-utilisation specified by the user
N1 Arrival Flow value is reduced due to capacity constraint at oversaturated upstream lanes.
N3 Capacity Adjustment due to downstream lane blockage determined by the program.

Site: 4.7p1 [4.7p1 Berkshire Road - Roe Highway off to Berkshire Road - 2050 PM (Site Folder: 2050 PM)]

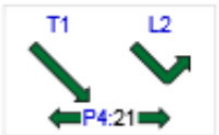
Network: 6 [n4.2p Roe Highway - Berkshire Road - 2050 PM (Network Folder: 2050)]

4.7p1 Berkshire Road - Roe Highway off to Berkshire Road - 2050 PM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 150 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Downstream lane blockage effects included in determining phase times
Phase Sequence: Leading Right Turn
Reference Phase: Phase B
Input Phase Sequence: A, B, C, D
Output Phase Sequence: A, B, C, D

OD Arrival Flows

	T1	L2
Tot	1573	487
LV	45%	83%
HV	0%	0%
U1	38%	2%
U2	1%	1%
U3	11%	12%
U4	4%	2%
U5	1%	0%



	L2	R2
Tot	393	864
LV	37%	77%
HV	0%	0%
U1	52%	0%
U2	0%	1%
U3	7%	15%
U4	3%	6%
U5	0%	1%

	T1	R2
Tot	665	251
LV	59%	84%
HV	0%	0%
U1	27%	0%
U2	1%	1%
U3	11%	12%
U4	2%	2%
U5	1%	1%

Some reduced upstream exit flow rates exist due to capacity constraint applied to oversaturated approach lanes. See Arrival Flows in Lane Summary reports.

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developme
SE: Berkshire Road	916	602	0	1
NW: Berkshire Road	2060	1113	0	6
SW: Roe Highway off to Berkshire Road (Northbound)	1257	816	0	2
Total	4232	2531	0	9

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road															

Lane 1	333	13.9	333	13.9	712	0.467	100	32.7	LOS C	17.2	145.6	Full	150	0.0	2.3
Lane 2	333	13.9	333	13.9	712	0.467	100	32.7	LOS C	17.2	145.6	Full	150	0.0	2.3
Lane 3	125	14.6	125	14.6	115	1.092	100	182.2	LOS F	15.4	130.7	Full	150	0.0	0.0
Lane 4	125	14.6	125	14.6	115	1.092	100	183.8	LOS F	15.4	130.7	Short	80	0.0	NA
Approach	916	14.1	916	14.1		1.092		73.8	LOS E	17.2	145.6				
NorthWest: Berkshire Road															
Lane 1	487	14.4	487	14.4	669	0.728	100	36.2	LOS D	20.0	169.7	Short	100	0.0	NA
Lane 2	372	16.4	372	16.4	343	1.087	100	150.2	LOS F	39.0	344.1	Short	100	-50.0 N3	NA
Lane 3	455	16.4	455	16.4	419	1.087	100	145.5	LOS F	46.6	411.0	Full	500	-38.9 N3	0.0
Lane 4	745	16.4	745	16.4	685	1.087	100	134.1	LOS F	71.9	633.9	Full	500	0.0	26.6
Approach	2060	15.9	2060	15.9		1.087		116.4	LOS F	71.9	633.9				
SouthWest: Roe Highway off to Berkshire Road (Northbound)															
Lane 1	472	10.5	393	10.5	1628	0.241	100	6.7	LOS A	0.0	0.0	Short	120	0.0	NA
Lane 2	394	21.6	328	21.6	297	1.105	100	196.5	LOS F	45.6	426.4	Short	300	-38.9 N3	NA
Lane 3	644	21.6	536	21.6	485	1.105	100	186.2	LOS F	71.8	671.7	Full	500	0.0	31.9
Approach	1509	18.1	1257 N1	18.1		1.105		132.8	LOS F	71.8	671.7				
Intersection	4485	16.3	4232 N1	17.3		1.105		112.1	LOS F	71.9	671.7				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

N1 Arrival Flow value is reduced due to capacity constraint at oversaturated upstream lanes.
N3 Capacity Adjustment due to downstream lane blockage determined by the program.

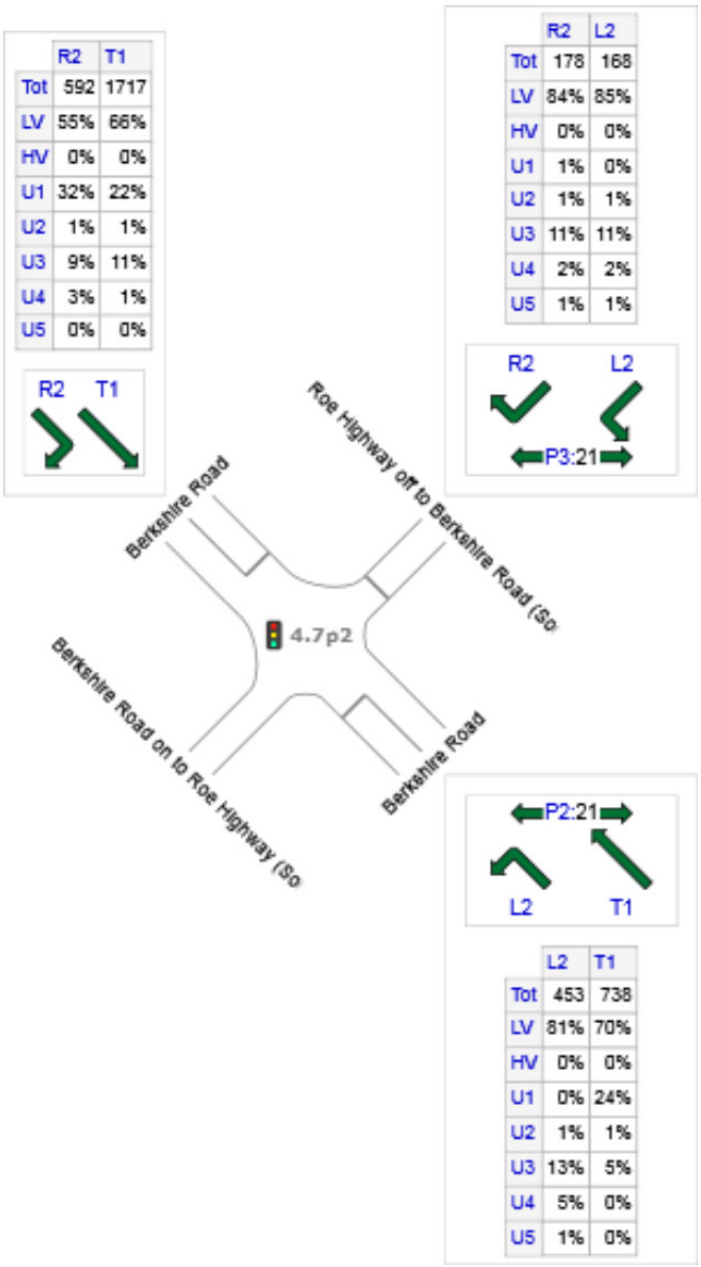
Site: 4.7p2 [4.7p2 Berkshire Road - Berkshire Road on to Roe Highway - 2050 PM (Site Folder: 2050 PM)]

Network: 6 [n4.2p Roe Highway - Berkshire Road - 2050 PM (Network Folder: 2050)]

4.7p2 Berkshire Road - Berkshire Road on to Roe Highway - 2050 PM
Site Category: (None)
Signals - EQUISAT (Fixed-Time/SCATS) Isolated Cycle Time = 80 seconds (Site Practical Cycle Time)

Timings based on settings in the Site Phasing & Timing dialog
Phase Times determined by the program
Downstream lane blockage effects included in determining phase times
Phase Sequence: Split Phasing
Reference Phase: Phase B
Input Phase Sequence: A, B, C, D
Output Phase Sequence: A, B, C, D

OD Arrival Flows



Some reduced upstream exit flow rates exist due to capacity constraint applied to oversaturated approach lanes. See Arrival Flows in Lane Summary reports.

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developme
SE: Berkshire Road	1191	880	0	1
NE: Roe Highway off to Berkshire Road (Southbound)	346	293	0	
NW: Berkshire Road	2309	1452	0	5
Total	3846	2625	0	7

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
SouthEast: Berkshire Road															

Lane 1	453	18.4	453	18.4	1479	0.306	100	6.7	LOS A	0.0	0.0	Full	300	0.0	0.0
Lane 2	369	5.2	369	5.2	566	0.652	100	15.0	LOS B	8.6	64.8	Full	300	-2.3 N3	0.0
Lane 3	369	5.2	369	5.2	566	0.652	100	15.0	LOS B	8.6	64.8	Full	300	-2.3 N3	0.0
Approach	1191	10.2	1191	10.2		0.652		11.8	LOS B	8.6	64.8				
NorthEast: Roe Highway off to Berkshire Road (Southbound)															
Lane 1	168	14.0	168	14.0	469	0.359	100	23.9	LOS C	5.2	44.1	Short	120	0.0	NA
Lane 2	88	13.9	88	13.9	115	0.762	100	51.2	LOS D	3.9	32.6	Short	300	-2.3 N3	NA
Lane 3	90	13.9	90	13.9	118	0.762	100	51.0	LOS D	3.9	33.3	Full	500	0.0	0.0
Approach	346	14.0	346	14.0		0.762		37.9	LOS D	5.2	44.1				
NorthWest: Berkshire Road															
Lane 1	1066	12.1	941	11.9	1089	0.864	100	21.1	LOS C	30.3 N4	244.8 N4	Full	150	0.0	50.0
Lane 2	880	12.1	777	11.9	899 1	0.864	100	21.3	LOS C	26.9	217.3	Full	150	0.0	38.9
Lane 3	333	12.9	296	12.5	496	0.597	100	32.7	LOS C	10.0	83.7	Short	80	0.0	NA
Lane 4	333	12.9	296	12.5	496	0.597	100	31.3	LOS C	10.0	83.7	Full	150	0.0	0.0
Approach	2611	12.3	2309 N 1	12.0		0.864		24.0	LOS C	30.3	244.8				
Intersection	4147	11.8	3846 N 1	12.8		0.864		21.5	LOS C	30.3	244.8				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Intersection and Approach LOS values are based on average delay for all lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

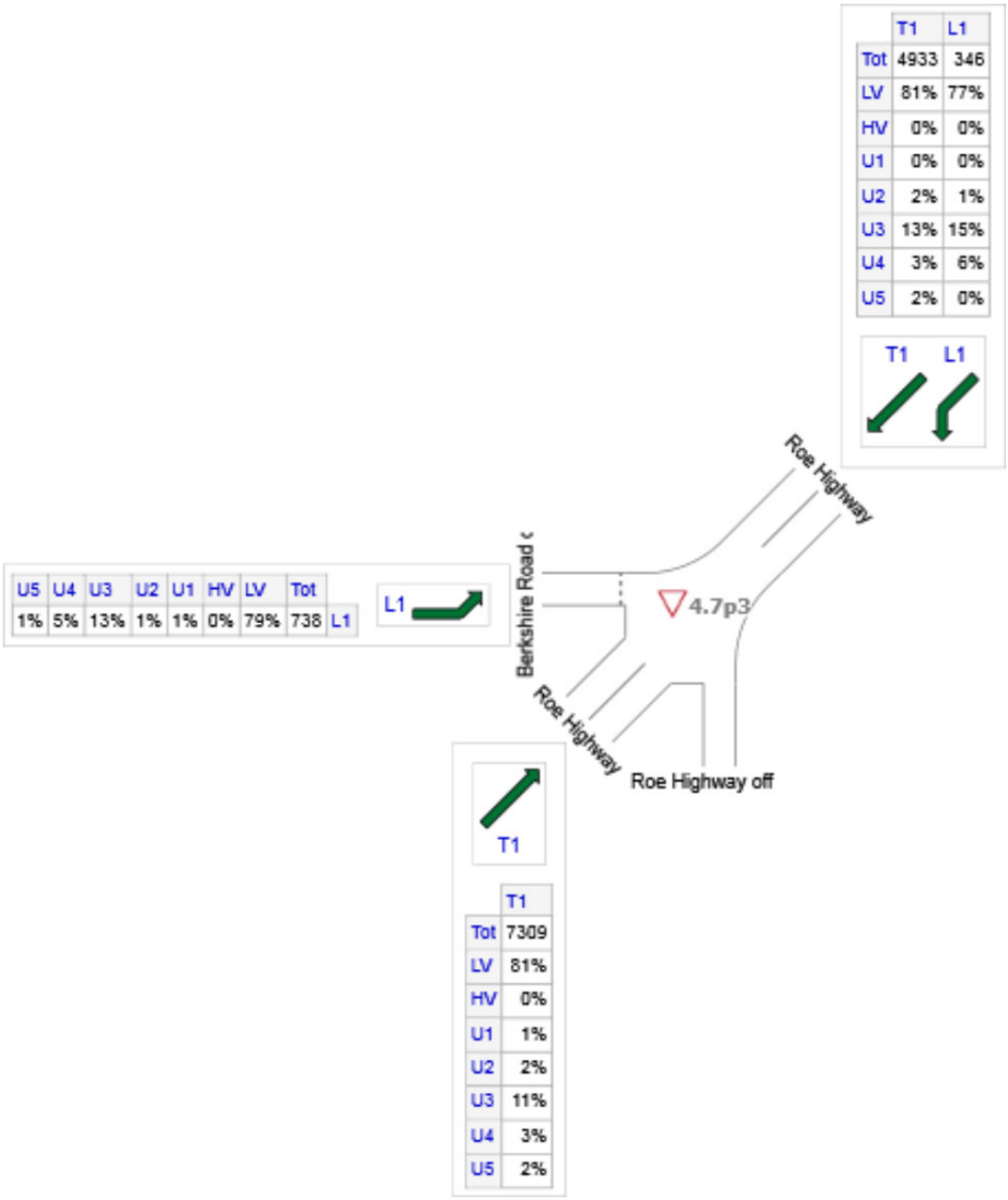
1 Reduced capacity due to a short lane effect. Short lane queues may extend into the full-length lanes. Some upstream delays at entry to short lanes are not included.
N1 Arrival Flow value is reduced due to capacity constraint at oversaturated upstream lanes.
N3 Capacity Adjustment due to downstream lane blockage determined by the program.
N4 Average back of queue has been restricted to the available queue storage space.

Site: 4.7p3 [4.7p3 Roe Highway Interchange with Berkshire Road - 2050 PM (Site Folder: 2050 PM)]

Network: 6 [n4.2p Roe Highway - Berkshire Road - 2050 PM (Network Folder: 2050)]

4.7p3 Roe Highway Interchange with Berkshire Road - 2050 PM
Site Category: (None)
Give-Way (Two-Way)

OD Arrival Flows



Some reduced upstream exit flow rates exist due to capacity constraint applied to oversaturated approach lanes. See Arrival Flows in Lane Summary reports.

	All MCs	Light Vehicles (LV)	Heavy Vehicles (HV)	Developmen
NE: Roe Highway	5279	4238	0	1
W: Berkshire Road on to Roe Highway (Northbound)	738	584	0	7
SW: Roe Highway	7309	5935	0	4
Total	13326	10757	0	5

Lane Use and Performance

	DEMAND FLOWS [Total veh/h	HV] %	ARRIVAL FLOWS [Total veh/h	HV] %	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Aver. Delay sec	Level of Service	95% BACK OF QUEUE [Veh	Dist] m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
NorthEast: Roe Highway															
Lane 1	346	21.6	346	21.6	1512	0.229	100	8.2	LOS A	0.0	0.0	Short	150	0.0	NA
Lane 2	1233	17.7	1233	17.7	1842	0.670	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	1233	17.7	1233	17.7	1842	0.670	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 4	1233	17.7	1233	17.7	1842	0.670	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 5	1233	17.7	1233	17.7	1842	0.670	100	0.2	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	5279	18.0	5279	18.0		0.670		0.7	NA	0.0	0.0				
West: Berkshire Road on to Roe Highway (Northbound)															
Lane 1	738	18.7	738	18.7	1538	0.480	100	8.1	LOS A	0.0	0.0	Full	500	0.0	0.0
Approach	738	18.7	738	18.7		0.480		8.1	NA	0.0	0.0				
SouthWest: Roe Highway															
Lane 1	562	16.3	562	16.3	1858	0.303	25[7]	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	2249	16.3	2249	16.3	1858	1.210	100	96.9	LOS F	0.0	0.0	Full	500	0.0	0.0
Lane 3	2249	16.3	2249	16.3	1858	1.210	100	96.9	LOS F	0.0	0.0	Full	500	0.0	0.0
Lane 4	2249	16.3	2249	16.3	1858	1.210	100	96.9	LOS F	0.0	0.0	Full	500	0.0	0.0
Approach	7309	16.3	7309	16.3		1.210		89.4	NA	0.0	0.0				
Intersection	13326	17.1	13326	17.1		1.210		49.8	NA	0.0	0.0				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Network Data dialog (Network tab).
Lane LOS values are based on average delay per lane.
Minor Road Approach LOS values are based on average delay for all lanes.
NA: Intersection LOS and Major Road Approach LOS values are Not Applicable for two-way sign control since the average delay is not a good LOS measure due to zero delays associated with major road lanes.
Delay Model: SIDRA Standard (Geometric Delay is included).
Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).
HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

7 Lane under-utilisation specified by the user

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Appendix 4

Traffic Modelling Report

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1. Introduction

This short report provides details on traffic modelling methodology. The existing and proposed roads for Forrestfield North and the surrounding network were modelled in Paramics. Distribution of traffic throughout the network was based on expected routes for accessing and egressing the structure plan area from the TIA report, which this Appendix accompanies. Routes were based on the City of Kalamunda's profile.id (for employment) and research on shopping, education and recreation opportunities in the surrounding area. The existing routes were slightly modified to account for the known proposed developments in the area.

While this report focused on the TOD Precinct, the traffic impact from the future Residential Precint was accounted for.

2. Trip Generation

Nominate the assessment year(s)	2031 and 2050 The assumption for 2031 is that 15% of the entire Forrestfield North area would be completed. In 2050 the entire area is expected to reach its full traffic generation.
Base Traffic Volumes forecasting for the assessment year(s)	Traffic from external to external zones was modelled to reflect, as close as possible, obtained data from the latest Main Roads WA ROM MLUFS 1.4 version Model for 2031 and 2041.
Development Generated Trips	Development Generated Trips were assessed utilising the following relevant documents: • WAPC Transport Assessment Guidelines for Developments – Volume 5 – Technical Guidance for residential and retail AM and PM peaks rates • NSW RTA Guide to Traffic Generating Developments for daily residential and retail trip rates • Transportation Engineers (ITE) Common Trip Generation Rates (9th edition) Along with Forrestfield North LSP traffic, trip generation from the proposed development Maida Vale South LSP was included in the model.

2.1 Base Traffic Volumes

Nominate the source(s) for obtaining the traffic data	Main Roads website; Traffic Data from the City of Kalamunda Main Roads WA ROM Model – 2031 and 2041

2.2 Traffic Volumes

Road Name	Location of Traffic Count	Vehicles Per Day (VPD)	Vehicles per Peak Hour (VPH)				Heavy Vehicle %	Year	
			AM Peak VPH	AM Peak Time	PM Peak VPH	PM Peak Time	*If HV count is Not Available, are HV likely to be in higher volumes than generally expected?*	Date of Traffic Count	*If older than 3 years, multiply with a growth rate (3% growth rate per annum)*
Berkshire Road	West of Roe Highway (SLK 0.75)	6,531	552	06:45	564	15:15	26.8%	2019/ 2020	5,035
	East of Roe Highway (SLK 1.58)	5,829	460	07:45	468	14:45	11.2%	Dec 2015	6,757
	40m East of Milner Road*	5,054	383	07:00	463	16:00	15.9%	Aug 2016	5,688
	West of Roe Highway Ramp (Northbound)**	7,919	511	08:00	748	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
	East of Roe Highway Ramp (Northbound)**	7,101	522	08:00	640	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
	Ramp on to Roe Highway (Northbound) North of Berkshire Road**	2,487	250	08:00	183	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
	West of Roe Highway Ramp (Southbound)**	11,569	849	08:00	1,076	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
	East of Roe Highway Ramp (Southbound)**	11,292	928	08:00	1,047	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
	Ramp on to Roe Highway (Southbound) South of Berkshire Road**	2,551	142	08:00	260	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
Roe Highway	South of Kalamunda Road (SLK 33.94)	44,657	3,624	07:00	3,823	16:15	12.7%	2018/ 2019	-
	North of Berkshire Road	53,578	4,285	07:15	4,848	16:00	17.1%	2019/ 2020	-
	Ramp off to	4,946	293	11:30	480	16:00	11.7%	Dec	5,733

	Maida Vale Road (Northbound) South of Maida Vale Road (SLK 0.26)							2015	
	Ramp off to Berkshire Road (Northbound) South of Berkshire Road**	6,180	457	08:00	527	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
	Ramp off to Berkshire Road (Southbound) North of Berkshire Road**	2,940	248	08:00	241	16:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
Newburn Road	South of Kalamunda Road (SLK 0.08)	5,882	462	08:15	555	16:00	4.8%	2018/ 2019	-
Hawtin Road	South of Maud Road (SLK 1.98)	7,025	592	08:00	668	15:15	8.3%	2019/ 2020	-
	South of Kalamunda Road**	8,944	828	08:00	868	17:00	N/A – HV not likely to be in higher volumes than generally expected	Feb 2020	-
Maida Vale Road	166 m West of Butcher Road*	1,994	137	07:00	220	16:00	9.1%	Jun 2019	-
	67m East of Milner Road*	3,711	278	07:00	359	16:00	8.0%	Mar 2018	-
	20m West of Littlefield Street*	6,162	675	08:00	778	16:00	7.3%	Nov 2018	-
	170m East of Dundas Road*	2,430	187	07:00	254	17:00	7.0%	Mar 2018	-
	78m West of Milner Road*	3,062	211	08:00	299	16:00	7.3%	Apr 2018	-
	65m East of Plover Road*	8,851	659	08:00	971	16:00	7.6%	Sep 2019	-
	100m West of Jaeger Court*	3,870	276	08:00	411	16:00	8.3%	Nov 2020	-
	West of Hawtin Road (SLK 0.10)	7,106	596	08:00	683	17:00	9.2%	Dec 2015	8,237
	Ramp on to Roe Highway (Southbound) South of Maida Vale Road (SLK 0.23)	5,720	511	07:15	436	16:15	17.0%	Dec 2015	6,631
Abernethy Road	South of Dundas Road (SLK 4.96)	21,232	1,663	07:15	1,720	15:45	23.6%	Dec 2015	24,613
	South of Kalamunda Rd (SLK 6.54)	17,958	1,335	07:30	1,472	15:45	21.7%	2018/ 2019	-
Apricot Street	57 M South West of Fruit Tree Crescent*	1,538	109	08:00	142	17:00	5.6%	Apr 2016	1,731
Dundas Road	205m North of Maida Vale Road*	5,687	373	08:00	506	16:00	12.3%	Mar 2018	-

	165m North of Berkshire Road*	4,267	311	07:00	347	15:00	19.4%	Mar 2018	-
	25m South of Kapok Court*	5,953	422	07:00	615	16:00	11.5%	Nov 2020	-
Sultana Road West	60m North of Milner Road *	260	20	11:00	28	15:00	18.9%	Oct 2016	292
Harrison Road	35m East of Dundas Road *	971	80	11:00	101	14:00	24.1%	Aug 2016	1,092
Milner Road	32m South West of Sultana Road West *	2,397	228	07:00	236	16:00	14.1%	Mar 2018	-
	120m South of Raven Street *	1,537	163	07:00	151	16:00	9.2%	Mar 2018	-
	45m North East of Stewart Road *	1807	182	07:00	173	16:00	9.4%	Mar 2018	-
	150m South of Eureka Street *	3,864	360	07:00	346	16:00	19.3%	Sep 2019	3,003
Brae Road	295m North of Sultana Road *	149	13	07:00	14	14:00	17.1%	Oct 2016	167

Note - These traffic counts have been received from the City of Kalamunda*
*Note**- These traffic volumes have been derived from SCATS data obtained through Main Roads. Although SCATS should not be used as a sole source of data, it is a good tool to verify fluctuations inflow.*

2.3 All Precincts Generated Trips – TOD, Activity Centre and Residential Precincts

2.3.1 Residential Precinct

Land Use Type	Rate above	Yield	Daily Traffic Generation VPD	Peak Hour Traffic Generation	
				AM VPH	PM VPH
Residential (R30 and R40 Houses)	8.0 VPD per unit Peak - 10% of VPD	938	7,504	750	750
Residential (R60 Houses and Apartments)	6.5 VPD per unit Peak - 10% of VPD	1,056 units	6,864	686	686
Residential (R80 Houses and Apartments)	5.5 VPD per unit Peak - 10% of VPD	423 units (150 apartments)	2,327	233	233
	Total Residential:	***2,417 units***	***16,691***	***1,669***	***1,669***
Primary School	2 VPD per student	540 students	216	108	108
	Peak – 1 VPH per student *(based the local use - 80% reciprocity applied**)*		1,080	540	540
District Open Space (Sporting Precinct)	71.33 VPD per court	2 courts (as per Sporting Precinct Preliminary Concept Plan)	71	2	21
	AM Peak – 1.4 VPH per court PM Peak – 20.67 VPH per court *(based on the proximity of the primary school and the local use - 50% reciprocity applied)*		143	3	41
	Total traffic with reciprocity applied:		**16,978**	**1,779**	**1,798**
	Total traffic without reciprocity:		**17,914**	**2,212**	**2,250**

Note * - Strategy City of Kalamunda Forrestfield North Structure Plan Community Infrastructure Strategy *outlines concept plans for the Education and Sporting Precincts and the Community Centre and Town Park Precinct and the yields have been sourced from this document.*

Note ** - *It is anticipated that only 20% of school trips will be generated from outside of the LSP area.*

The estimated yield of 2,417 dwellings is expected within the LSP area. The LSP report indicates an estimated population of 5,998 in the FFN Residential Precinct based on the information received from the City of Kalamunda. According to the City of Kalamunda's household summary on profile.id.com.au, 9% of the City's residents are of primary school age.

80% of the assumed 540 children is 432, which is approximately 7.2% of the expected number of future FFN residents, less than the City of Kalamunda average. This more than accounts for the likely reduction in school-age children within higher-density housing forms. Even if the ratio is changed in favour of trips outside of the LSP area, this will not significantly impact traffic distribution and the surrounding road network. The school is expected to generate only up to 1,080 VPD, which is less than 6.5% of the total Residential Precinct traffic generation.

Based on the above, KCTT believe that the distribution of 80% local / 20% outside LSP area) should be applied.

It should also be noted that while the cumulative peak volumes represent the most conservative scenario, in reality, peak times of various uses may coincide only partially.

Does the site have existing trip generation/attraction?	YES
No of Daily Trips	50 units * 9 VPD = 450 VPD
No of AM Peak Hour Trips	50 units * 0.8 VPH = 40 VPH
No of PM Peak Hour Trips	50 units * 0.8 VPH = 40 VPH

What is the total impact of the new proposed development?	High impact
Total additional daily traffic to the surrounding road network	16,978 – 450 = **16,528 VPD**
Total additional AM peak traffic to the surrounding road network	1,779 – 40 = **1,739 VPH**
Total additional PM peak traffic to the surrounding road network	1,798 – 40 = **1,758 VPH**

2.3.2 TOD Precinct

Development Type	Rate Above	Yield	Total VPD	PM Peak VPH	AM Peak VPH
Residential *25% expected to be attracted by the TOD precinct cells	5.5 vehicular trips per dwelling and 5 vehicular trips per dwelling Peak – 0.55 and 0.5 vehicular trips per 100m^2 per hour	207 - 5.5 vpd dwellings; 301 - 5 vpd dwellings	2,644* (1,983)**	265* (199)**	265* (199)**
Commercial *5% expected to be generated by the future residents of FFN area	10 vehicular trips per 100m^2 Peak – 2 vehicular trips per 100m^2 per hour	1,200m^2	120* (114)**	24* (23)**	24* (23)**
Retail with Food Component *45% expected to be generated by the future residents of FFN area	121 vehicular trips per 100m^2 AM Peak – 2.5 vehicular trips per 100m^2 per hour PM Peak – 10 vehicular trips per 100m^2 per hour	Assumed 70% of total retail = 3,612m^2	4,371* (2,404)**	90* (50)**	361* (199)**

Retail without Food Component (including Showroom Retail) *20% expected to be generated by the future residents of FFN area	17 vehicular trips per 100m^2 AM Peak – 2.7 vehicular trips per 100m^2 per hour PM Peak – 2.7 vehicular trips per 100m^2 per hour	Assumed 30% of total retail = 1,548m^2 + 2,500 Showroom = 4,048m^2	688* (551)**	110* (96)**	110* (96)**
Medical Centre *25% expected to be generated by the future residents of FFN area	72 vehicular trips per GP; 30 vehicular trips per day per specialist 15% VPD = VPH	4 General Practitioners; 8 Specialists	528* (396)**	79* (59)**	79* (59)**
Childcare Centre *25% expected to be generated by the future residents of FFN area	4 VPD per child and 2 VPD per staff member AM Peak - 0.8 VPH per child PM Peak - 0.7 VPH per child	125 children; 25 staff members	550* (413)**	100* (75)**	88* (66)**
Community Facilities (Potential Aquatic Centre) *20% expected to be generated by future residents of the FFN area	1.5 vehicular trips per day per person; 0.15 vehicular trips per person per hour	Assumed 1,256 persons per day as per information received from the planner	1,884* (1,507)**	188* (150)**	188* (150)**
Forrestfield Station Parking *20% expected to be generated	Peak Traffic obtained from 'Forrestfield Station Multi-Storey Car Park Traffic Impact	1,200 spaces	2,400* (1,920)**	840* (672)**	840* (672)**

by future residents of the FFN area	Assessment'; Daily Traffic assumed 2 VPD per parking space				
Kiss'n'Ride Parking *20% expected to be generated by future residents of the FFN area	Peak Traffic obtained from 'Forrestfield Station - Multi-Storey Car Park Traffic Impact Assessment'	N/A	4,000* (3,200)**	1,000* (800)**	1,000* (800)**
Total - The Proposed Development			**17,185* (12,488)****	**2,696* (2,124**)**	**2,955* (2,264)****

3. Trip Distribution

3.1 Zones

How many zones were created for the model?

47 Zones were created in Paramics.

External zones were created to match the expected desire lines in accordance with the existing statistical data (sourced from Community ID profile for the City of Kalamunda).

Nominate external zones

33 zones external to the Forrestfield North area:

- **Zone 1.** To/from the north via Roe Highway (south of Berkshire Road)
- **Zone 2.** To/from the south via Hawtin Road (south of Berkshire Road)
- **Zone 3.** To/from the east via Kalamunda Road
- **Zone 4.** To/from the north via Roe Highway (north of Kalamunda Road)
- **Zone 5.** To/from the west via Kalamunda Road (west of Abernethy Road)
- **Zone 6.** To/from the south via Abernethy Road (south of Dundas Road)
- **Zone 7.** To/from the south via Dundas Road (south of Berkshire Road)
- **Zone 8.** To/from the east via Gooseberry Hill Road (east of Kalamunda Road)

Forrestfield North Industrial Precinct Zones:

Zones 23 – 26 Forrestfield North Industrial Precinct Zones

- **Zone 27.** Maida Vale Cell 1
- **Zone 28.** Maida Vale Cell 2
- **Zone 29.** Maida Vale Cell 3
- **Zone 30.** Maida Vale Cell 4
- **Zone 31.** Maida Vale Cell 5
- **Zone 32.** Maida Vale Cell 6
- **Zone 33.** Maida Vale Neighbourhood Centre
- **Zone 34.** Maida Vale Primary School

Zones 35 – 44 External Residential zones located north to the Forrestfield North DSP (High Wycombe)

Zones 45-47 Industrial developments south of Berkshire Road

Nominate internal zones and traffic generation

14 internal zones:

Zone 9.	Residential Zone	Cell 1 - Residential Precinct
Zone 10.	Residential Zone	Cell 2- Residential Precinct
Zone 11.	Residential Zone	Cell 3 - Residential Precinct
Zone 12.	Residential Zone	Cell 4 - Residential Precinct
Zone 13.	Residential Zone	Cell 5 - Residential Precinct
Zone 14.	Residential Zone	Cell 6 - Residential Precinct
Zone 15.	Residential Zone	Cell 7 - Residential Precinct
Zone 16.	Residential Zone	Cell 2 - TOD Precinct
Zone 17.	Residential Zone	Cell 1a - TOD Precinct
Zone 18.	Mixed-use zone: Community	Cell 1b - TOD Precinct

	facilities; Medical Centre; Childcare Centre; Residential; Retail	
Zone 19.	Ecological Precinct	Cell 1c - TOD Precinct
Zone 20.	Forrestfield North Station	Cell 1d - TOD Precinct
Zone 21.	Showroom Retail	Cell 1e - TOD Precinct
Zone 22.	Primary School and DOS	Residential Precinct



Figure 1 External Zones Location

3.2 Traffic Flow Distribution onto External Road Networks

ZONE	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Zone 6	Zone 7	Zone 8	FFN Industrial	Internal Traffic
Forrestfield North Residential Zones	20%	5%	7.5%	15%	10%	8%	6%	2%	1.5%	25%

**Note: It was assumed that internal trips will make approximately 25% of the total generated trips because of the large attractors such as Forrestfield North Station, large Activity Centre and other education, shopping and employment opportunities within the Forrestfield North area.*

3.3 Traffic Flow Distribution onto Internal Road Network

ZONE	Zone 17 - Retail	Zone 18 - Mixed Use Cell	Zone 19 - Conservation Sub-Precinct	Zone 20 - Forrestfield Station	Zone 21 - Showroom Retail	Primary School
Residential Zones	0.5% (20%)	13.5% (35%)	0% (0%)	6% (20%)	0.5% (15%)	4.5% (80%)

**Note: It was assumed that internal trips will make 25% of the total generated trips because of the large attractors such as Forrestfield North Station, large Activity Centre and other education, shopping and employment opportunities within the Forrestfield North area.*
***Note: Percentage in the brackets represent the share of FFN residential zones in the total traffic generated by the above land uses.*

3.4 External to External Traffic Distribution

Traffic from external to external zones was modelled to reflect, as closely as possible, the obtained data from the latest Main Roads WA ROM MLUFS 1.4 Model for 2031 and 2041. The difference between results may occur due to the network level of detail. Furthermore, KCTT has modified the projected volumes on Dundas Road and Berkshire Road based on the following items:

- Zone 585 comprises Forrestfield North Project Area, Forrestfield North Industrial Area, a portion of the residential area in the southern portion of High Wycombe and a portion of the Forrestfield Industrial Area. The image, for ease of reference, is shown below.



Given the location of the links in question (Berkshire Road, west of Roe Highway; Dundas Road, south of Berkshire Road and Dundas Road, north of Berkshire Road), and their relative position to the Zone 585; in the traffic discharged from the Zone 585 to either of these links is improbable. Therefore, to estimate the impact of Zone 585 on this section of the network, discharged traffic can be added.

Based on the ROM model plots, Zone 585 discharges cumulatively 46,000 VPD on these three links via one of the connectors. This volume does not include traffic discharged on Maida Vale Road as the key remaining distributor.

Proposal for further modelling:

Passing traffic is to be established by removing the immediate impact of zone 585 on the links mentioned above from the ROM plots. The network model is finer grain than the ROM model; therefore, Zone 585 was split between several zones. The Industrial, High Wycombe residential, and Forrestfield North Project Area are modelled as separate zones.

The most recent traffic count data indicate annual growth of 2.22% on Dundas Road. This growth is in line with our initial assumptions. The most recent traffic count data on Berkshire Road indicate a significant growth (approximately 11% per annum); however, early traffic counts were taken at the time when the Berkshire Road / Roe Highway interchange was in construction. For this reason, recorded traffic growth may appear to be skewed.

We ran a quick analysis on Dundas Road south of Berkshire Road as follows:

- The ROM model shows total anticipated traffic as 23,100 VPD;
- Approximately 17,400 VPD originates from Zone 585:
- Therefore, passing traffic can be estimated as 5,700 VPD
- The most recent traffic recorded on this section of Dundas Road by MRWA is 3,974 VPD. Applying a 2.22% growth rate over 20 years, we arrive at 5,863 VPD. This volume corresponds to a 5,700 VPD estimate, notwithstanding that some traffic from the Forrestfield Industrial Area may be included in this set of counts.

In summary, we propose that passing traffic targets should be derived by removing Zone 585 impacts as recorded in the ROM model.

-

KCTT have utilised the ROM24 2031 daily volumes to create the demand matrix and distribute the traffic between the 8 external zones. The traffic demands between any two zones were estimated and calibrated using Paramics modelling software until the traffic flows reflected the ROM24 data as closely as possible, except for the volumes specified above.

The annual growth rate was derived from calibration plot sheets and Model 2031 plot sheets through a comparison of the projected traffic volumes on the same locations. Base traffic volumes for 2050 were calculated by applying the derived annual growth rate to the traffic volumes for 2031 and 2041.

4. Structure Plan Impact on Internal and Surrounding External Road Network

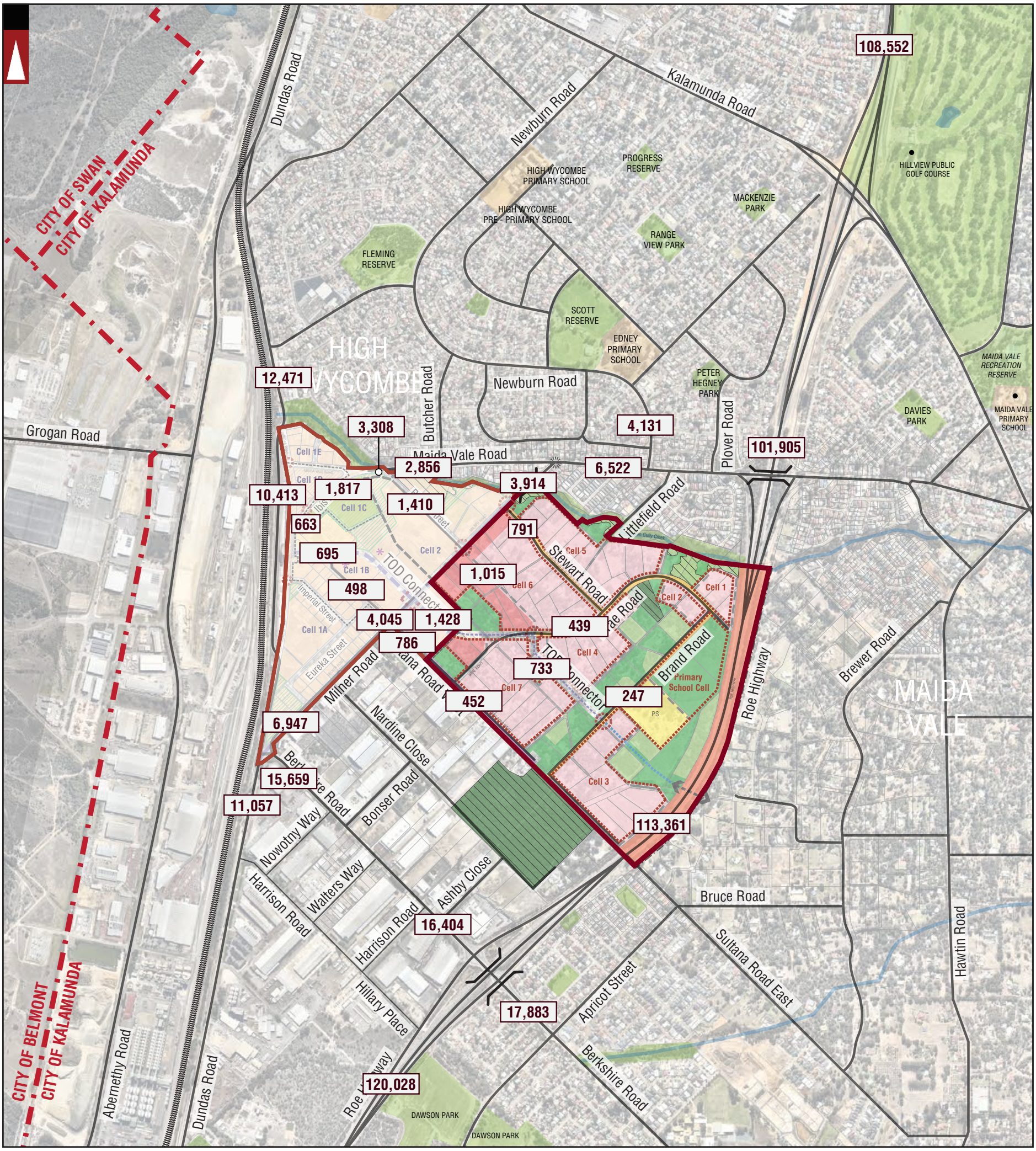
The table below shows anticipated traffic on all major roads within the precinct. Traffic generated by the Residential Precinct was expressed as volume and percentage of total traffic volumes.

Road (Link)	Section	Total Daily Traffic (2031)	External Daily Traffic (2031)	TOD Traffic (2031)	Total Daily Traffic (2050)	External Daily Traffic (2050)	TOD Traffic (2050)
Roe Highway	South of Berkshire Road	120,028	119,171	295	158,183	153,824	1,063
			99.29%	0.25%		97.24%	0.67%
	Between Berkshire Rd and Maida Vale Rd	113,361	113,310	7	140,469	140,182	54
			99.96%	0.01%		99.80%	0.04%
	North of Maida Vale Road	101,905	101,898	7	125,922	125,868	54
			99.99%	0.01%		99.96%	0.04%
	North of Kalamunda Road	108,552	108,026	43	139,007	136256	208
			99.52%	0.04%		98.02%	0.15%
Berkshire Road	West of Roe Hwy	16,404	14,422	1,301	25,843	18,302	3,524
			87.92%	7.93%		70.82%	13.64%
	East of Roe Hwy	17,883	16,721	998	31,805	28,444	2,407
			93.50%	5.58%		89.43%	7.57%
	South of Milner Rd	15,659	13,606	1,308	24,915	17301	3,533
			86.89%	8.35%		69.44%	14.18%
Dundas Road	South of Berkshire Road	11,057	10,443	287	15,169	13,236	757
			94.45%	2.60%		87.26%	4.99%
	South-west from Old Dundas Road alignment	10,413	9,600	813	13,680	12,306	1,374
			92.19%	7.81%		89.96%	10.04%
	North of Old Dundas Road alignment	12,471	10530	1,342	19,964	13407	3,512
			84.44%	10.76%		67.16%	17.59%
Maida Vale Road	East of Ibis Place	3,308	930	1,784	7,620	1,101	3,530
			28.11%	53.93%		14.45%	46.33%
	East of Raven Street	2,856	1,154	1,702	4,422	1,231	3,159
			40.41%	59.59%		27.84%	71.44%
	East of Milner Rd	6,522	4114	1,619	13,072	4178	4,002
			63.08%	24.82%		31.96%	30.62%
Milner Road	South of Maida Vale Rd	3,914	2,864.00	261	9,288	2,747	1,649
			73.17%	6.67%		29.58%	17.75%

	South of TOD Connector	4,045	3,001	162	8,110	2,820	920
			74.19%	4.00%		34.77%	11.34%
	North of Berkshire Rd	6,947	5217	663	12,928	5073	2,426
			75.10%	9.54%		39.24%	18.77%
Enterprise Boulevard	West of Maida Vale Rd	1,817	0	1,817	3,631	0	3,631
			0.00%	100.00%		0.00%	100.00%
	North of TOD Connector	663	0	663	1,924	0	1,924
			0.00%	100.00%		0.00%	100.00%
Raven Street	South of Maida Vale Rd	1,410	456	360	5,602	396	2,132
			32.34%	25.53%		7.07%	38.06%
	South of Milner Rd	1,015	0	85	5,984	0	862
			0.00%	6.03%		0.00%	14.41%
Stewart Road	East of Milner Rd	791	0	63	4,739	0	498
			0.00%	7.96%		0.00%	10.51%
TOD Connector	East of Milner Rd	1,428	319	481	6,432	323	1,961
			22.34%	33.68%		5.02%	30.49%
	South of Ibis Pl	695	0	695	2,322	0	2,322
			0.00%	100.00%		0.00%	100.00%
	South of Brae Rd	733	0	85	4,520	0	811
			0.00%	11.60%		0.00%	17.94%
Sultana Road West	West of Milner Rd	498	0	498	1,586	0	1,586
			0.00%	100.00%		0.00%	100.00%
	East of Milner Rd	786	564	38	1,872	554	243
			71.76%	4.83%		29.59%	12.98%
	West of Brae Street	452	217	53	1,664	180	429
			48.01%	11.73%		10.82%	25.78%
Newburn Road	North of Maida Vale Road	4,131	3534	114	6,568	3593	408
			85.55%	2.76%		54.70%	6.21%
Brae Road	East of TOD Connector	439	0	80	2,599	0	680
			0.00%	18.22%		0.00%	26.16%
Brand Road	North of TOD Connector	247	0	34	2,011	0	291
			0.00%	13.77%		0.00%	14.47%

5. Paramics Output

The following pages show graphs with extracted traffic volumes from Paramics models for 2031 and 2050. Graphs are also divided into volumes for internal and volumes for the internal networks.



No	DATE	AMENDMENT
E	27-07-2021	PROPOSED LAYOUT AMENDED
D	23-04-2020	PROPOSED LAYOUT AMENDED
C	08-04-2020	PROPOSED LAYOUT AMENDED
B	12-04-2018	PROPOSED LAYOUT AMENDED

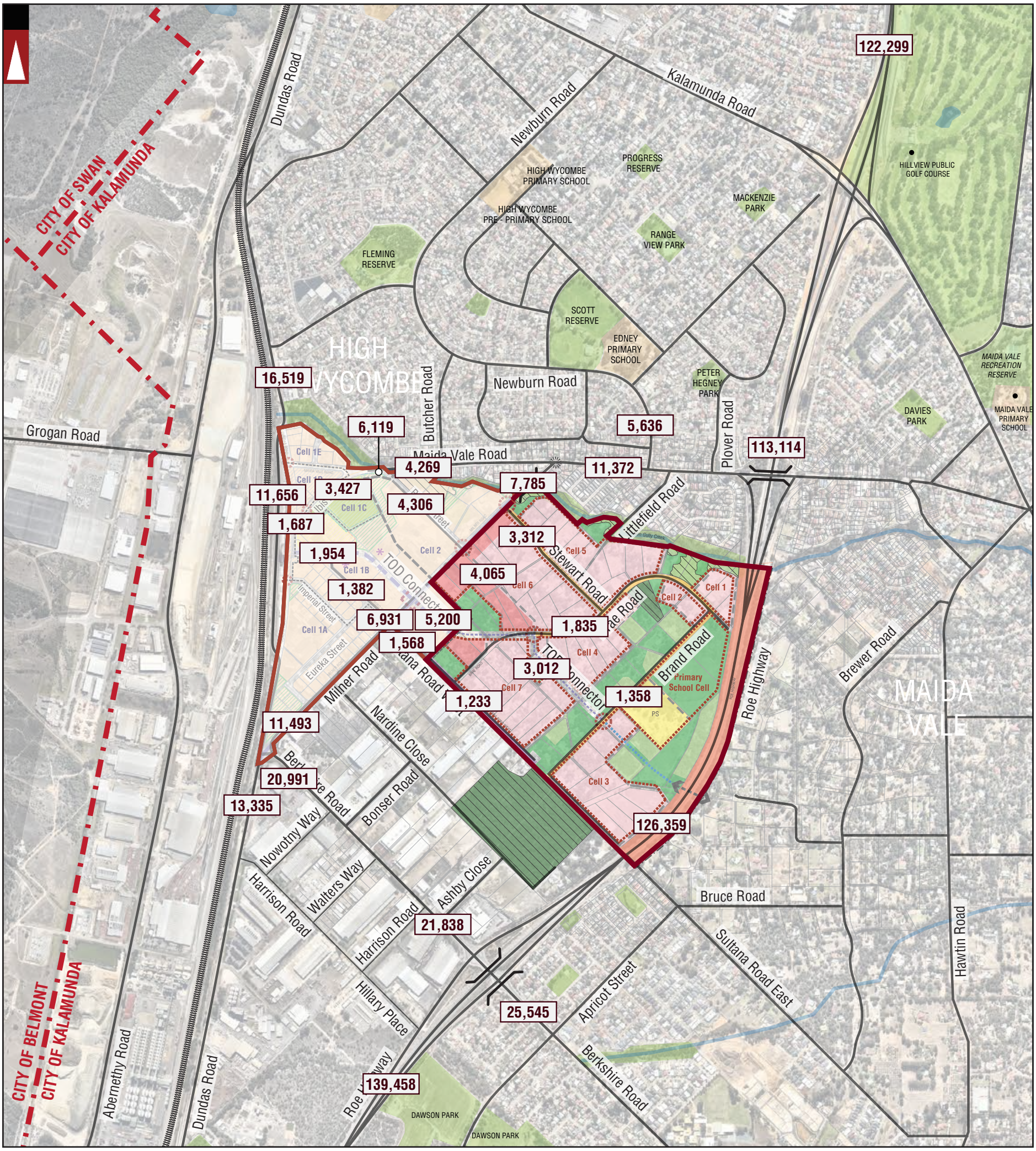
PROJECT: FORRESTFIELD NORTH - RESIDENTIAL PRECINCT

TITLE: TOTAL DAILY ANTICIPATED TRAFFIC - 2031

DRAWING NUMBER: KC00604.000_ S06A

DRAWN BY: J.S.

Civil & Traffic Engineering Consultants
Suite 7 No 10 Whipple Street Balcatta WA 6021
PH: 08 9441 2700
WEB: www.kctt.com.au
kctt



No	DATE	AMENDMENT
E	27-07-2021	PROPOSED LAYOUT AMENDED
D	23-04-2020	PROPOSED LAYOUT AMENDED
C	08-04-2020	PROPOSED LAYOUT AMENDED
B	12-04-2018	PROPOSED LAYOUT AMENDED

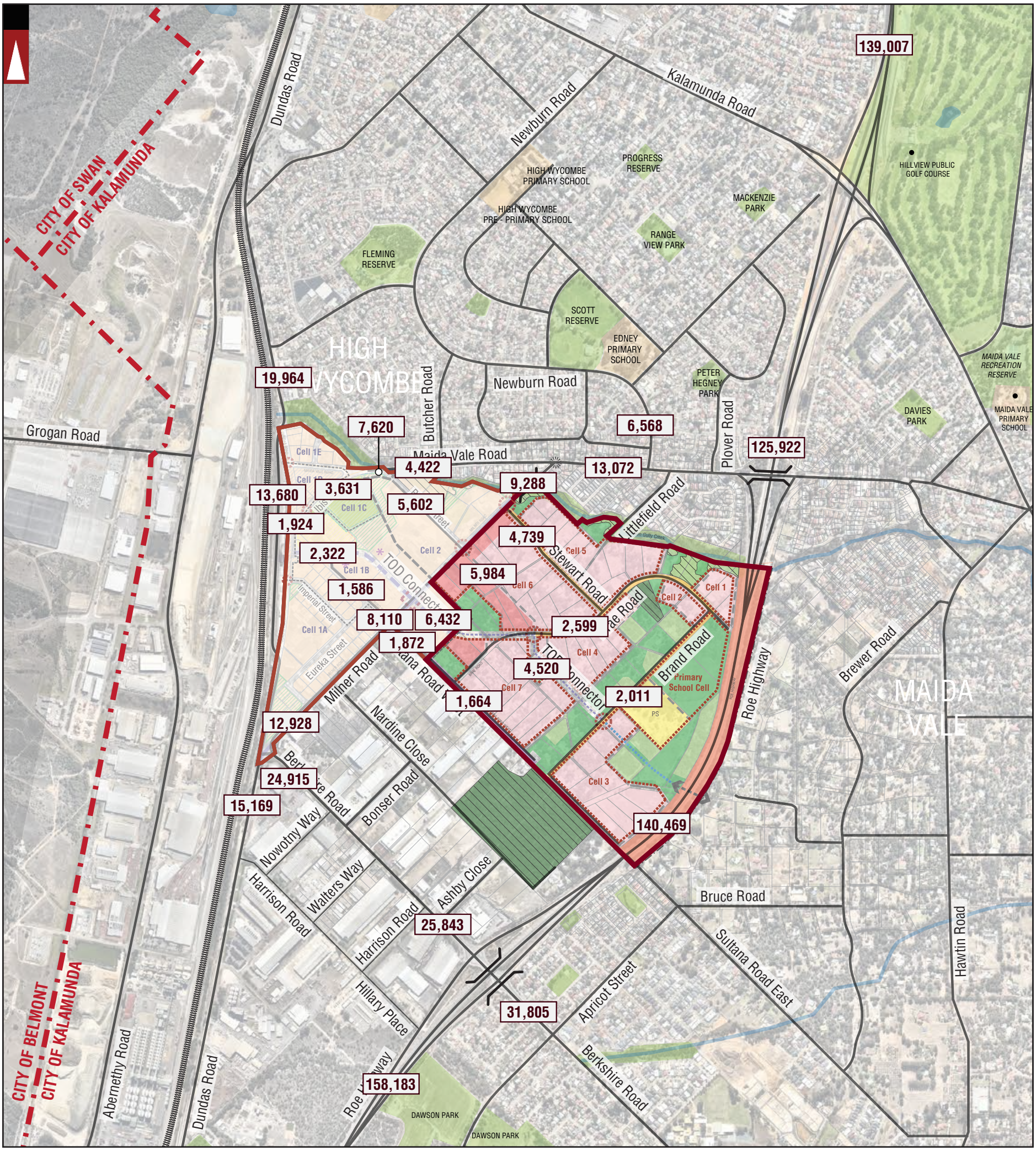
PROJECT: FORRESTFIELD NORTH - RESIDENTIAL PRECINCT

TITLE: TOTAL DAILY ANTICIPATED TRAFFIC - 2041

DRAWING NUMBER: KC00604.000_ S06B

DRAWN BY: J.S.

Civil & Traffic Engineering Consultants
Suite 7 No 10 Whipple Street Balcatta WA 6021
PH: 08 9441 2700
WEB: www.kctt.com.au
kctt



No	DATE	AMENDMENT
E	27-07-2021	PROPOSED LAYOUT AMENDED
D	23-04-2020	PROPOSED LAYOUT AMENDED
C	08-04-2020	PROPOSED LAYOUT AMENDED
B	12-04-2018	PROPOSED LAYOUT AMENDED

PROJECT: FORRESTFIELD NORTH - RESIDENTIAL PRECINCT

TITLE: TOTAL DAILY ANTICIPATED TRAFFIC - 2050+

DRAWING NUMBER: KC00604.000_ S06C

DRAWN BY: J.S.

Civil & Traffic Engineering Consultants
Suite 7 No 10 Whipple Street Balcatta WA 6021
PH: 08 9441 2700
WEB: www.kctt.com.au
kctt

element.

Technical Appendix F - KCTT Infrastructure Servicing Report

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INFRASTRUCTURE SERVICING REPORT

Forrestfield North – TOD and Activity Centre Precinct (ACP)

Forrestfield North, Western Australia

October 2021

Rev E

kctt

Infrastructure Servicing Report
KC00604.000 Forrestfield North TOD and Activity Centre Precinct

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Rev B	Jan 2020	C Kleyweg	P Varelis	Issued for Information.
Rev C	31/08/2021	C Kleyweg	P Varelis	Issued for Approval
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Prepared By: KCTT (Trading as Traffic and Transport Pty Ltd)
ABN 35 148 970 727 |
Postal address: PERTH: Unit 7, No 10 Whipple Street Balcatta WA 6021 | BELGRADE: 23 Hilandarska, Beograd 11000
Phone: 08 9441 2700 |
Website: www.kctt.com.au |

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KCTT Infrastructure Servicing Report

The Forrestfield North Transit Oriented Development (TOD) and Activity Centre (AC) Precinct Activity Centre Plan (ACP) has been carefully developed to promote equity between as many landowners as possible for the iterative development of landholdings under varied ownership in accordance with the intent of the ACP. This Infrastructure Servicing Report focuses on a wide range of infrastructure development strategies inclusive of: -

- major road network upgrade requirements including upgrades to: -
    - Dundas Road realignment, between Berkshire Road and Maida Vale Road.
    - Berkshire Road (upgrade to 4-lanes).
    - Milner Road (upgrade to 2 lane divided carriageway, south of Sultana Road West).
    - Milner Road (upgrade to 2 lane divided carriageway between Sultana Road West and Maida Vale Road with parking and widened pedestrian environments plus cycling facilities).
    - Sultana Road West (improved industrial access road east of Milner Road).
    - Maida Vale Road (upgrade to 2-lane divided carriageway east of Milner Road).
    - Maida Vale Road (upgrade to 2 lane divided carriageway west of Milner Road).
    - Provision of new TOD Connector Boulevard linking the Forrestfield North Railway Station, through the Forrestfield North TOD and AC Precinct, the Forrestfield North Residential Precinct LSP Area to the Maida Vale South LSP Area.
- key intersection upgrades at the following locations: -
    - Ibis Place / Maida Vale Road.
    - Berkshire Road / Dundas Road / Milner Road (dual lane roundabout).
    - Milner Road / Sultana Road West.
    - Milner Road / Maida Vale Road.
    - Milner Road / TOD Connector Boulevard (new intersection).
- conceptual bridge and interchange upgrades at the Roe Highway / Maida Vale Road interchange. These existing t-intersections have been upgraded to roundabout intersections in accordance with current MRWA requirements.
- infrastructure for alternative transportation modes, inclusive of public transport, consideration of future transport modes in design, pedestrian, and cyclist requirements.
- provision of on-street car-parking on all major internal roads suitable for high-density development for interim transportation modal choice requirements in Perth, but allowing for future modal shifts.
- incorporation of streetscaping, place-making and urban design requirements in the public realm as part of the overall infrastructure planning and civil concept designs.
- upgrade requirements to existing wastewater infrastructure to cater for the full development of Forrestfield North Residential, Forrestfield North TOD and AC Precinct and the Maida Vale South Structure Plan areas, with an up-front infrastructure provision to allow for development across both the Forrestfield North Residential LSP and Forrestfield North TOD and AC ACP Areas considering the number of different landholders within these areas.
- upgrade requirements for the reticulation of potable water.
- development of stormwater drainage strategies and the incorporation of Water Sensitive Urban Design strategies that will make Forrestfield North a strong proponent of WSUD, including: -
    - provision of infiltration drainage cells and storage cells in all major road networks.
    - at-source treatment at drainage pits.
    - infiltration / underground storage / WSUD measures at drainage collection points minimising the wastage of open space at ground levels in all POS / environmental conservation areas.

- future potential upgrade requirements for the reticulation of power assets into the Forrestfield North TOD and AC Precinct, to be determined iteratively as development proceeds and to be managed by developers on all roads abutting land to be developed as a standard WAPC condition of subdivision.
- future potential upgrade and relocation requirements for the connection of telecommunications and broadband internet, to be determined iteratively as development, to be managed by developers on all roads abutting land to be developed as a standard WAPC condition of subdivision; and
- future upgrade and relocation requirements for the provision of gas services, to be managed by ATCO Gas in coordination with developers where an open trench is provided by the developer for the reticulation of gas infrastructure.

Road Network

The road network has been designed to accommodate an interface between industrial land uses to the south and residential and activity centre uses north of Sultana Road West, with design to suit turning movements for RAV 4 and RAV 7 vehicles with key upgrades in Berkshire Road, Dundas Road, Milner Road (south of Sultana Road West) and in Sultana Road West. The TOD Precinct and AC Precinct are expected to be high attractors of traffic in the short to medium term, therefore robust designs need to be accommodated for the expected traffic attraction and generation which will be applied to key road networks inclusive of Maida Vale Road, Milner Road north of Sultana Road West and the TOD Connector Boulevard and understanding the impacts of the trip generation on external road networks. This is examined in detail in KCTT's Transport Impact Assessments for the TOD and AC Precincts and the Residential Precinct.

Designing an interface between local residential, commercial, retail and traffic attracted to / generated by the proposed land-uses within the TOD and AC Precincts, and the regional traffic that will be attracted to the parking for the Forrestfield North Railway Station is a balancing act in the development of a strong TOD / Activity Centre outcome.

Therefore, within the TOD and AC Precincts boulevard-style pedestrian amenity and promotion of on-road cycling are strong themes in this ACP. The design for the roads within the TOD and AC Precincts focuses on the mix of transportation usages and should not be designed with the car dominating all other forms of transportation. We recommend through the implementation phases of this project that consideration is continually given to designing road cross-sections in conjunction with the Integrated Transport Branch of the Department of Transport. These focuses include constraining vehicular speeds under 40kph and promoting mixed usage within these road reservations that allow for vehicular movement while providing safer environments for pedestrians, cyclists, and vulnerable road users. We believe that for Forrestfield North to become a vibrant centre, it is important that the streetscape environment is welcoming to more than just through vehicle movements and this is the ethos of the ACP.

Intersection Upgrades

There are a series of main intersections that require upgrade to accommodate the expected vehicular trip generations and attractions within the ACP and Residential LSP Areas, including the Berkshire Road / Dundas Road / Milner Road intersection which is designed to be a reconfigured 4-lane, 4-way roundabout with capability for RAV 7 vehicles. The intersection of Milner Road and Sultana Road West is of high importance to the overall form and function of the residential and TOD and AC precincts because it forms the boundary between the industrial land-uses to the south and the urban land-uses to the north. This intersection is designed to accommodate left and right turn manoeuvres for RAV 4 vehicles. Additional upgrades include the intersection of Maida Vale Road and Milner Road, which will need to be upgraded to a roundabout featuring 2-lane approaches (one circulating lane) and the

on-off ramps for Roe Highway / Maida Vale Road which will be reconfigured as roundabouts in accordance with current Main Roads WA policies.

Within the TOD and AC Precincts, the intersection of the TOD Connector Boulevard and Milner Road is an intersection of high importance and is proposed to be a signalised intersection to give greater priority to pedestrians in the future town centre. The road treatments within the TOD Connector Boulevard to Ibis Place are also very important. The intersection of Ibis Place and Maida Vale Road requires careful consideration to deal with future volumes of traffic under the Main Roads WA ROM model and is shown as a t-intersection with priority moving between Ibis Place and Maida Vale Road to the east. This is discussed in greater detail in the KCTT Transport Impact Assessments for the ACP and LSP Precincts.

Bridge Structures

KCTT have developed a concept design for the required widening of the Maida Vale Road flyover. Due to increased traffic on Maida Vale Road, the bridge is maintained at 2-lanes. The connectivity between Maida Vale South and the Forrestfield North Railway Station is of future importance as a longer-term investment, however the earlier proposed TOD Connector bridge has been removed from this ACP due to uncertainties in development timeframes.

Alternative Transportation Infrastructure

The Forrestfield North TOD and AC Precinct has a series of major pedestrian and cycling path networks providing full inter-connectivity within, and between the TOD and AC Precinct, the Residential Precinct, Maida Vale South, and High Wycombe to the north. All roads have minimum 1.8 metre width pedestrian paths, with on-road cycling in Milner Road (north of Sultana Road West), TOD Connector Boulevard, Ibis Place, Maida Vale Road, Dundas Road realignment, Raven Street, and shared paths on roads with higher order function.

The function of Maida Vale Road and Ibis Place is very important in the short to medium term as a main route of connectivity for local bus networks to access the Forrestfield North Railway Station. In the future, the TOD Connector Boulevard may provide direct connectivity between Forrestfield North and Maida Vale South, should the Maida Vale South LSP progress.

Wastewater Infrastructure

KCTT have held detailed discussions with the Water Corporation with relation to the upgrade and extension of wastewater infrastructure to service Forrestfield North and Maida Vale South as an interim measure, while the Water Corporation completes detailed planning for the Gooseberry Hill Planning Area. The interim plan involves the extension of wastewater infrastructure along Dundas Road for the TOD and AC Precincts and along Milner Road with 3 branch systems along Sultana Road West (picking up the proposed AC Precinct and the Industrial Composite land-uses in Sultana Road West, and into Raven Street and Stewart Road to service the majority of the Residential Precinct. Not all properties will be able to service directly off these extensions, however the premise is that the potential early delivery of wastewater infrastructure for the area will accelerate development potential across a greater number of landholdings than presently exists due to the disparate nature of land ownership in the Precinct Areas.

Water Infrastructure

As for the wastewater above, KCTT have held detailed discussions with the Water Corporation in relation to the future requirements for water. There are no impediments to short-term development in the TOD and AC Precincts, with existing services capable of servicing initial developments. The Water Corporation are awaiting further confirmation on likely development timeframes and yields of development per annum to finalise their conceptual

planning and the timing of required upgrades. Our understanding is that the area has suitable major infrastructure which can accommodate future expansion.

Power

KCTT have held initial informal discussions with respected electrical consultants UPD in relation to current Western Power policies when considering large-scale / long-timeframe developments of this nature. The development of power infrastructure to suit developments of this type is an iterative process, as additional power capacity cannot be "stored". We understand that Western Power is currently not utilising new 1-hectare transformer / substation sites but is preferring to increase the capacity of existing major substations. Some lead-in work may be required after the first 5 to 10 years of development, but this will be contingent on the rate of development in Forrestfield North and the rate of expansion of other land-use assets connected to local substations and feeders. The main infrastructure requirement will be the undergrounding of existing HV and LV assets in roads to be widened and in road reserves to be closed. Power requirements for the area will need to allow for the relocation of many existing assets due to road widening / road reservation closure and may need to allow for either the relocation of HV assets, or the selected undergrounding of such assets. The requirements for undergrounding of existing overhead power assets are developer-funded requirements which form part of each developer's WAPC Conditions of Subdivision, where the existing roads directly fronts the proposed development landholdings.

Undergrounding of power should only be considered a DCP item where infrastructure identified in the DCP necessitates undergrounding, and the requirement is not otherwise required through the subdivision process.

Telecommunications / Gas

Telecommunications and Gas infrastructure are available in the Structure Plan areas for immediate development and as per the discussions above in relation to power and water, telecommunications and gas infrastructure are upgraded as development commences and concept planning for headworks infrastructure is finalised as development rates per annum are known with greater certainty.

Summary

In summary, all infrastructure items have been considered in detail in the Infrastructure Servicing Reporting conducted for this project, with appropriate items costed for consideration as part of the future DCP for the Forrestfield North project area.

1. Location

Table 1 - Location Information

Structure Plan	Activity Centre Plan Precinct	Suburb	Locality
Forrestfield North District Structure Plan	TOD and AC Precincts	Forrestfield North / High Wycombe	City of Kalamunda

Brief Description of the Site:

The Transit Oriented Development (TOD) and Activity Centre (AC) is shown in Figure 1 on the following page and is bounded by Dundas Road (West), Milner Road (East), Imperial Street (South), and Maida Vale Road to the north, with the Activity Centre presently located on the north-eastern corner of the intersection of Milner Road and Sultana Road West.

The Forrestfield North TOD and AC Precincts have developed iteratively through the DSP and ACP processes. The proposed yields in the TOD and AC Precincts have been heavily influenced by the provision of the Forrestfield North Railway Station.

Historically, the land-uses in the Forrestfield North TOD and AC Precinct have been low density residential / low-intensity commercial uses. Retention of natural vegetation in this area has generally been sparse, except for the Poison Gully Creek alignment between Milner Road and Dundas Road where vegetation is heavily populated. Properties in the triangle bounded by Poison Gully Creek, Milner Road and Raven Street also feature reasonable quantities of vegetation.

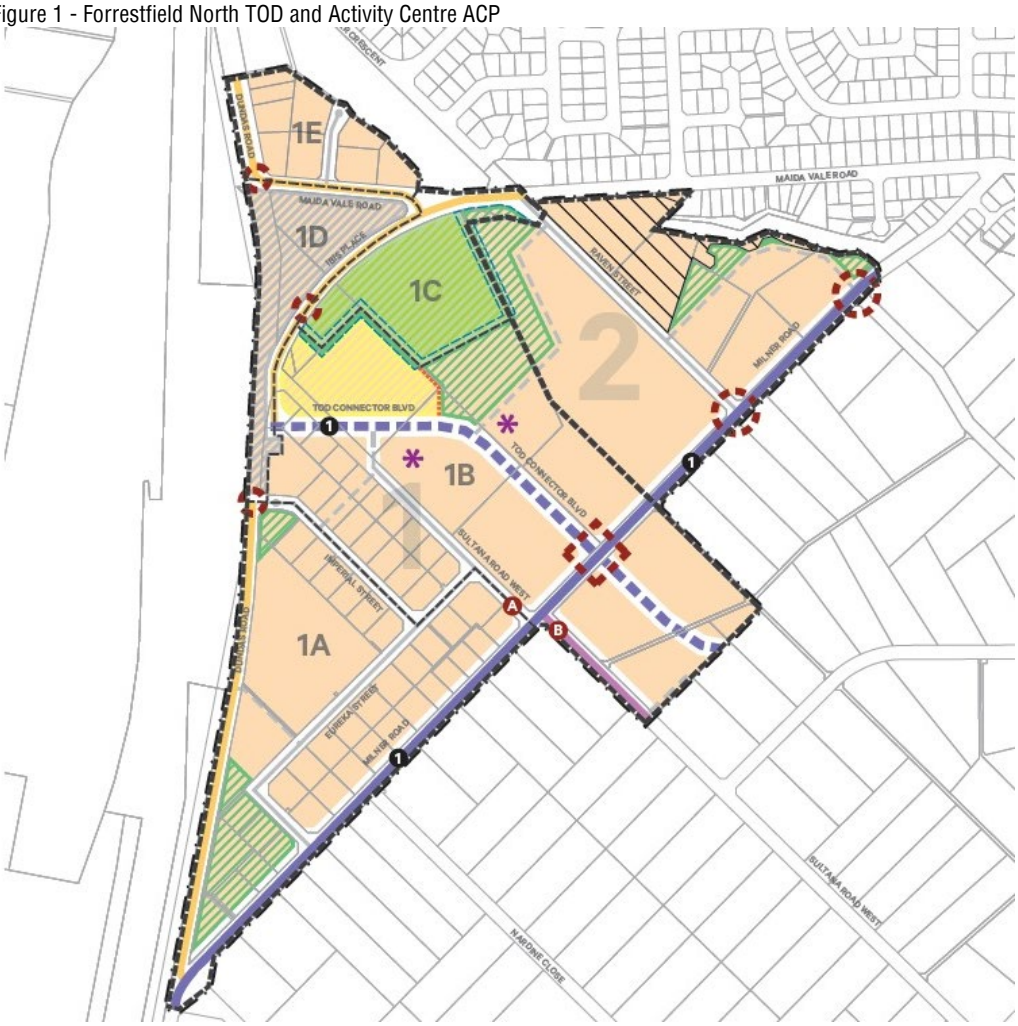
The land-uses surrounding the proposed ACP area include recreational and residential uses to the north (High Wycombe); to the east (Forrestfield North Residential Precinct); and industrial land-uses to the south (Forrestfield / Hazelmere) and industrial / bulky goods to the west (Perth Airport).

The ACP area is relatively flat with a high point in the north-eastern portion at 35 AHD, and a low point in the south-west section at 30 AHD. While the Perth Groundwater Atlas shows groundwater at around 12.5 metres below the natural surface, (Refer Section 1.5) recent testing suggests that the maximum groundwater levels may be much higher than this. Further, the soil conditions in this area may be variable, with expectancies of higher percentages of clays than is the case in the Residential Precinct to the east. The design and construction for the Forrestfield North Railway Station however changes the natural topography with the future concourse approximately 6 metres above the intersection of Milner Road and Sultana Road West. This has the impact of changing substantially the catchments from a drainage perspective and changing the strategies for localised infiltration with the TOD Precinct now exhibiting greater clearances to clays and groundwater than was exhibited with the existing topography. This is explained in greater detail in the sections on stormwater drainage management.

The Forrestfield North TOD and Activity Centre ACP is broken up into 6 sub-precincts as follows: -

- Precincts 1A to 1E
- Precinct 2

Figure 1 - Forrestfield North TOD and Activity Centre ACP



The following table shows the yields for the TOD and Activity Centre Precinct: -

Table 2 - Forrestfield North TOD and Activity Centre Precinct ACP Yields

Development Type	Yields (2050)
Residential	508 dwellings
Commercial	1,200m^2
Retail	5,160m^2
Showroom Retail	2,500m^2
Medical Centre	1,200m^2
Childcare Centre	1,250m^2
Community Facilities	Potential Aquatic Centre for 1,256 persons per day
Forrestfield Station - Parking	1,200 parking bays
Forrestfield Station – Kiss 'N' Ride Parking	100 parking bays

2. Proposed Development

The proposed development entails a TOD and AC Precinct of approximately 60 hectares as part of the Forrestfield North District Structure Plan (DSP). The TOD and AC Precincts propose a vibrant mix of land-uses, including office, residential, retail, showroom retail, childcare, community facilities and medical. The yields for each property type are shown in Table 2, with the ACP layout also shown on Page 10.

These yields form the basis of calculations for wastewater requirements as detailed in this report.

3. Existing Buildings

Does the ACP Area feature existing buildings? If YES nominate.	YES, the TOD precinct has sparse commercial / industrial buildings and a low number of residential dwellings (5). The residential dwellings are in the north-eastern corner of the Precinct Area in Raven Street and Milner Road. The AC precinct has existing residential dwellings, mainly populated in the central and southern portion of the precinct.
Number of buildings?	Approximately 40 buildings.
Approximate age of buildings?	Estimated age of buildings from a general inspection is between 10 and 35 years, depending on the asset. Some buildings within the ACP Area are potentially older.
Is asbestos likely to be an issue? YES / NO	YES Desktop studies show that there are some existing buildings, but more likely that there are some remnant structures and / or fencing types that may contain asbestos. Further investigation is required to confirm at the Subdivision Application / Development Application phase.
Are septic tanks present? YES / NO	YES The TOD and AC precincts are currently using septic tanks or other forms onsite sewerage disposal units as there are no sewer services connected. Thus, the presence of septic tanks within the Precincts should be considered on each private landholding.
Likely issues associated with remediation?	The key issues associated with the Precinct Areas are likely to include the presence of buried asbestos and uncontrolled fill; the presence of asbestos and asbestos materials in remnant fencing and older out-structures and buildings; the presence of leach drains, septic tanks and other localised wastewater systems. These issues are localised in nature and will be the responsibility of the landowner / developer in the future.

4. Earthworks

Describe the general levels across the site	The general levels across the site through desktop studies from the Slip Landgate portal shows a high point in the north-eastern portion of the site particularly east of Milner Road and south of Raven Street at 36.0m AHD. The site then has a gradual fall towards the south-western section which has a series of low points at RL 30.0m AHD in the following locations: - • Corner of Dundas Road and Imperial Street (basin located on the southern side of Imperial Street); • Dundas Road (midway between Imperial Street and Milner Road). This location coincides with a major drainage crossing from the TOD and AC Precinct to the Airport landholdings to the west; and • The corner of Milner Road and Dundas Road on the northern side of the intersection. **The proposed construction levels for the Forrestfield North Railway Station however change considerably the existing topography and provides better opportunities for the management of stormwater drainage by concentrating flows away from the railway station and areas of higher density and provides increased potential for en-route infiltration of localised runoff.**
Describe MGL levels across the site	JBSG's LWMS presently shows the depth to MGL ranges between 0.5 metres in the central area of the TOD Precinct generally north of Sultana Road West and east of Ibis Place, up to 7.5 metres along the western interfaces of the AC Precinct and along Milner Road. As per the comments above with relation to the changed topography due to the proposed levels of the Forrestfield North Railway Station, issues with MGL in TOD Precinct are minimised. This will be explained further below.
Is Groundwater an issue on this site? If YES, is fill likely to be required? If YES, what is the quantum of fill?	YES – in the area bounded by Ibis Place, Sultana Road West, Raven Street to an offset distance approximately 250 metres northwest of Milner Road. In our opinion this is the main location that requires substantial importation of fill. The Forrestfield North Railway Station has been designed with a concourse sitting approximately 6 metres above the existing level of the intersection of Milner Road and Sultana Road West. KCTT have ran an earthworks and drainage design utilising this change in topography and we have run average longitudinal grades from these locations at a maximum of 3%. Under this design much of the TOD Precinct has suitable clearance to groundwater to utilise localised drainage infiltration. Consideration should be given to provision of subsoil drainage to control the groundwater in this western interface zone where MGL may be likely to rise to within 1.5 metres of the natural surface to control all groundwater in these Precincts.
Can cut to fill techniques be utilised?	NOT REQUIRED Due to the proposed Forrestfield North Railway Station concourse requiring substantial importation of fill, cut to fill within these precincts is not expected to be required.

What is the likely depth of topsoil (mm)?	The expected range of topsoil depths is 100mm to 200mm. This should be confirmed by a geotechnical engineer.
Is the topsoil suitable for re-use? If YES, is there a suitable location to bury?	YES Further geotechnical investigations are required to investigate the depth of Bassendean Sands and Yoganup Sands – to be confirmed.
Describe the natural vegetation on the site? Dense / Moderate / Sparse / Cleared	Moderate to Sparse The natural vegetation is currently sparse across the site with a small portion in the north consisting of medium to large sized trees, particularly northeast of Raven Street and along the Poison Gully Creek alignment.
Are there significant trees that need to be kept?	YES The Poison Gully Creek alignment will have protection. Consideration should be given to the replacement of all trees lost in the landholdings in Raven Street, with each of these trees to be replaced by at least 1 street tree. The preliminary plans for street trees in the Forrestfield North DSP show a street tree regime that is likely to be in the order of up to 1 new tree per 20 linear metres of road verge and / or median in this area.
What is the likely soil profile?	Perth Groundwater Atlas indicated that the Precinct area is comprised of Bassendean Sands: quartz sand (dunes). Further geotechnical investigations are required to determine and confirm the in-situ soil however the responsibility rests with the developer to complete the appropriate geotechnical tests to the satisfaction of the City of Kalamunda.
Risk of acid sulphate soils? High / Moderate / Low	Low - Moderate risk of acid sulphate soils according to the desktop study through the Landgate Slip Portal.
Is there peat or other unsuitable materials? If YES nominate the likely unsuitable material.	A geotechnical investigation as part of the developer's submission for Subdivision or Development Approval and would need to be conducted to determine if any materials exist and or require removal. Through our desktop investigations there is a possibility there may be some unsuitable materials throughout the Precincts on sites that have been utilised for the storage of transportation equipment; general industrial land-uses; stock and equipment from the construction of the Forrestfield North – Airport railway link, or any other land-uses that create or store goods or by-products. The potential for peat / unsuitable materials in the soils is highest and likeliest in the areas where MGL's are highest.

5. Roadworks

Do existing roads require upgrade?	YES The ACP currently consists of low density commercial and residential dwellings, with road alignments and cross-sections developed to suit. Major upgrades are required to amend the road layout and to develop the road system such that it will work for a dense TOD community.
Do existing roads require upgrade?	YES With the new proposed residential development, all existing roads will need to be upgraded to compliment the new medium-high residential density development. Existing conditions of the roads are not adequate to accommodate the volume of traffic that will be generated by the new development. Current road widths are very narrow. This ACP proposes significant upgrading of key internal road networks.

In this section of the ISR, KCTT review in detail the existing road reservations and pavement composition, the proposed cross sections across the Residential and TOD and AC Precincts and provide analysis of the future road requirements for the Forrestfield North Residential and TOD and AC Precincts.

5.0.1 Existing Road Reservations and Pavements

The table below provides details on the existing roads that form a key part of the future road spine within the Forrestfield North Residential Precinct LSP area.

Table 3 - Data on the Existing Road Network

Road Name	Length (m)	Road Width (m)	Road Reserve Width (m)	Description
Sultana Road West	515	6.5	20	Two-way single road with an approximate width of 6.5m and road reserve width of 20m, according to desktop studies. With no road marking or kerbing therefore, the road will require upgrading. 2-way, 2-lane road.
Milner Road	1,630	7.5	20	Rural type road with no road markings or significant kerbing. 2-way, 2-lane road.
Dundas Road	1,150	8	20	Existing semi-mountable kerbing throughout. Major realignment works are required due to the Forrestfield North Railway Station works. 2-way, 2-lane road.
Imperial Street	310	6.5	20	Industrial type road with flush kerbing allowing drainage to flow overland onto verges. 2-way, 2-lane road.

Ibis Place	320	6.5	20	Existing coarse asphalt, rural residential road. Requires major upgrades, kerbing etc. 2-way, 2-lane road.
Raven Street	360	6.5	20	Existing coarse asphalt, rural residential road. Requires major road upgrades, kerbing etc. 2-way, 2-lane road.
Eureka Street	550	10.0	20	Industrial standard road. Fit for purposes. 2-way, 2-lane road.
Berkshire Road	1,085	9.0	20	Industrial Connector Road with 2 x 4.5m wide lanes and painted centreline.

5.0.2 LSP Road Cross Sections

The proposed road network will be discussed in detail in the Transport Impact Assessment (TIA), however from a civil infrastructure perspective the following roads have been identified as part of the road networks to be funded that are suitable for consideration as part of a future DCP, with each cross section showing the road name identifier.

Figure 2 – RD01 Berkshire Road Cross Section

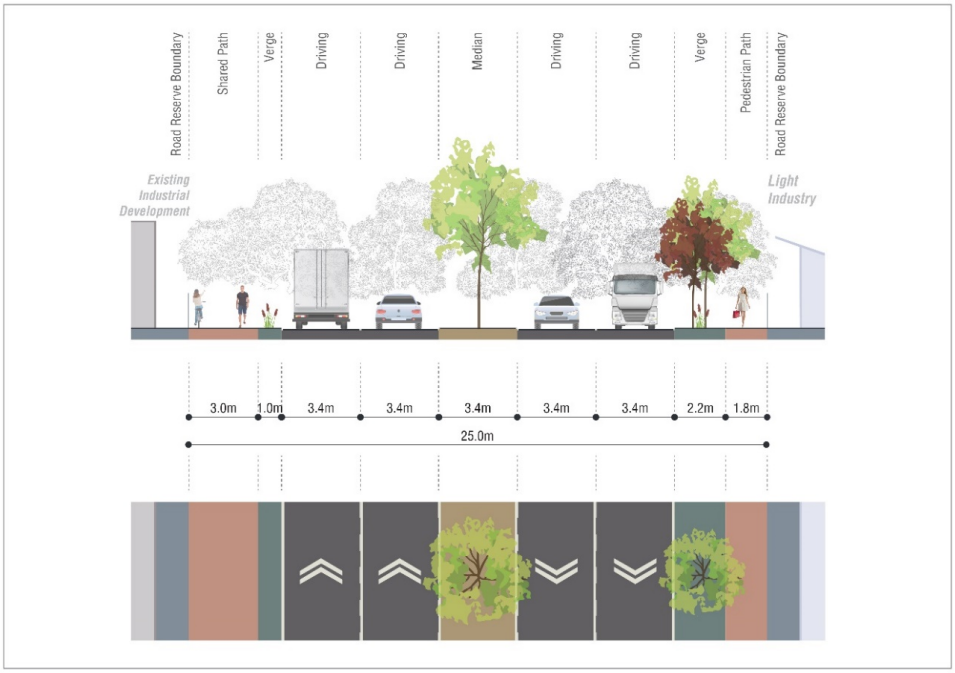


Figure 3 – RD02 Milner Road Cross Section (Sultana Road West to Stewart Road)

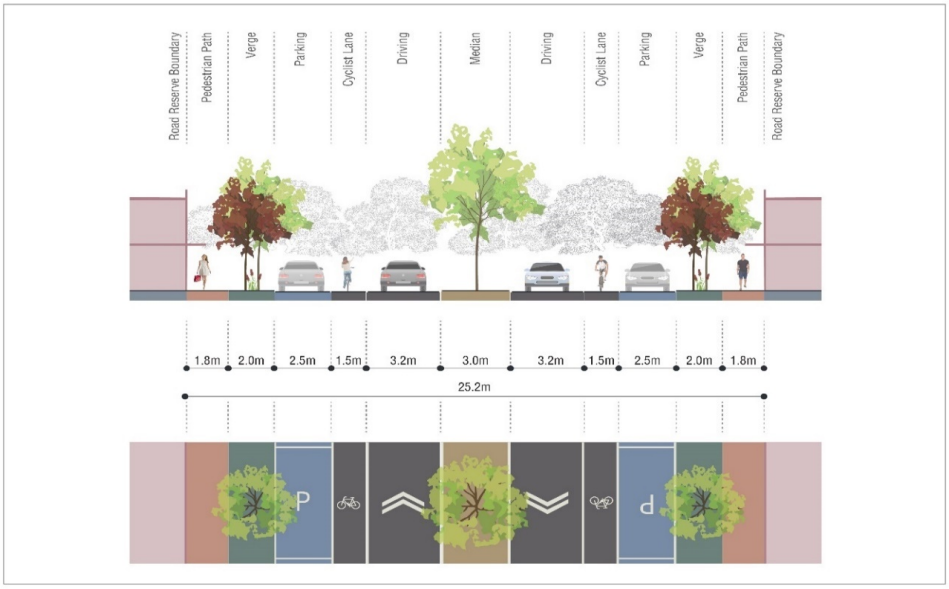


Figure 4 – RD03 Maida Vale Road (East of Milner Road), RD04 Maida Vale Road (Milner Road to Raven Street) and RD05 Maida Vale Road (West of Ibis Place)

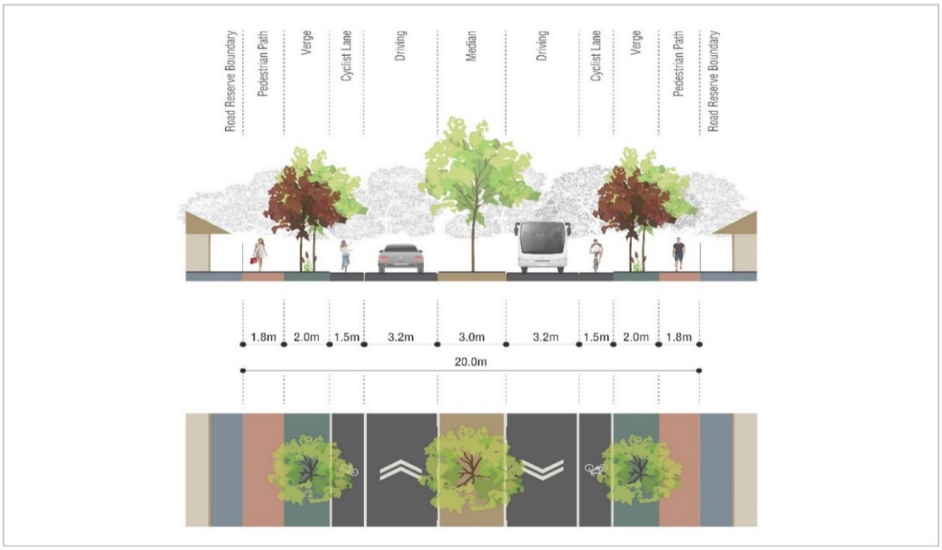


Figure 5 – RD06 Dundas Road (South of Berkshire Road), RD07 Dundas Road (Berkshire Road to Dundas Road Old) and RD08 Dundas Road (North of Deviation) Cross Section

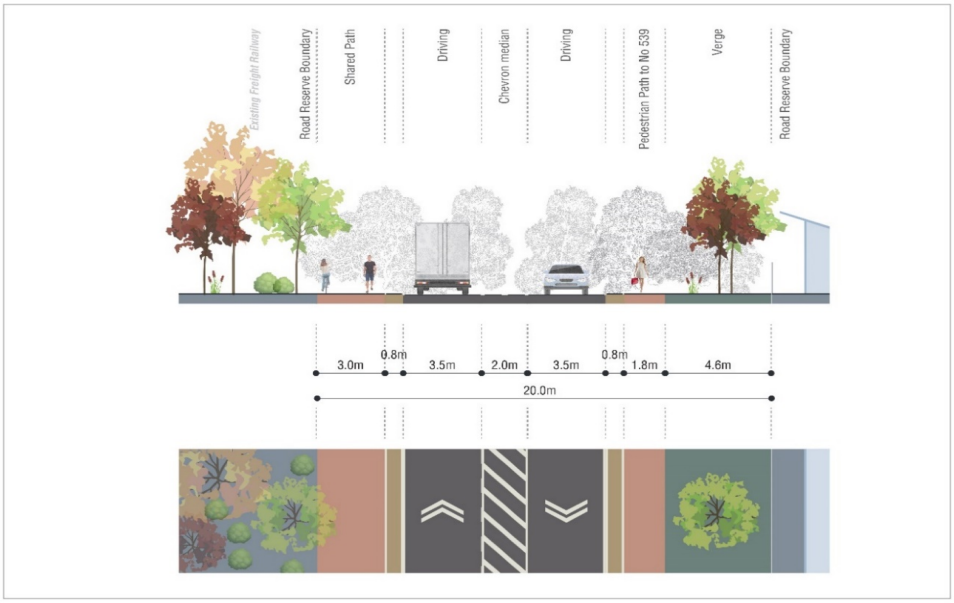


Figure 6 – RD09 Milner Road (Berkshire Road to Sultana Road West) and RD10 Milner Road (Stewart Road to Maida Vale Road) Cross Section

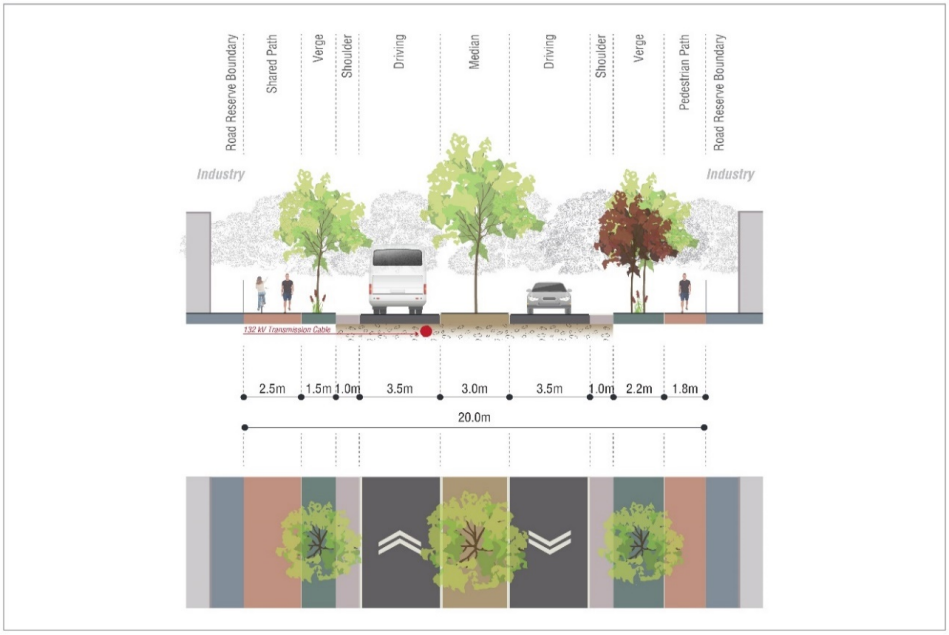


Figure 7 – RD11 Ibis Place / Enterprise Boulevard Cross Section

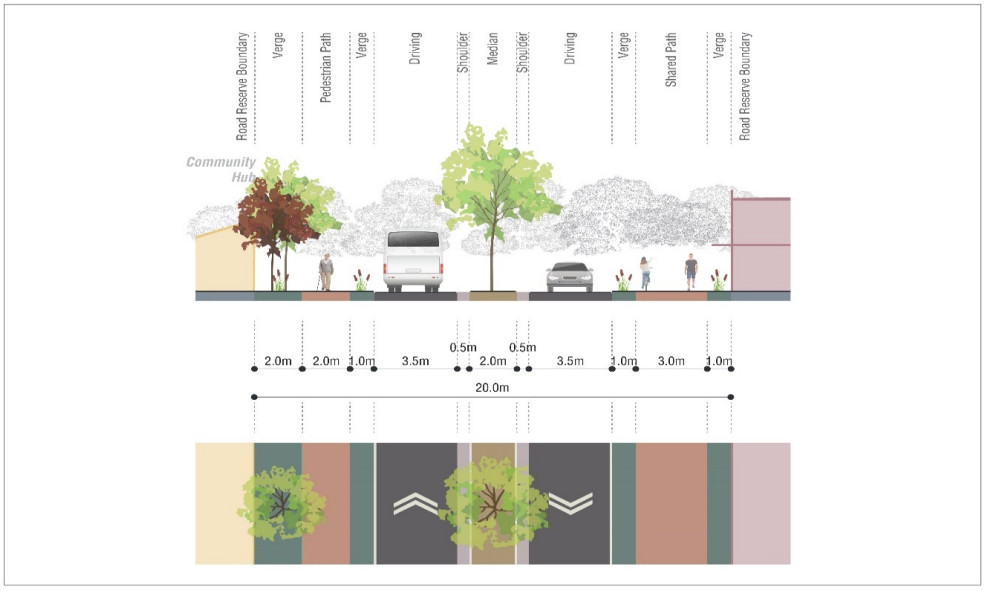


Figure 8 – RD12 Raven Street (Milner Road to Maida Vale Road) and RD13 Raven Street (Milner Road to Brae Road) Cross Section

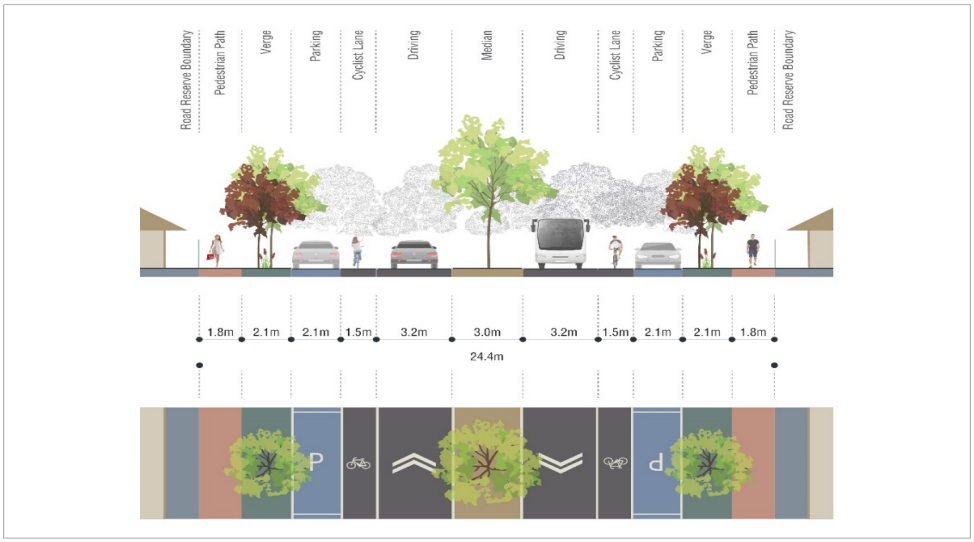


Figure 9 – RD14 TOD Connector Boulevard (Milner Road to Enterprise / Ibis), RD15 TOD Connector Boulevard (Milner Road to Brae Road) and RD16 TOD Connector Boulevard (East of Brae Road) Cross Section

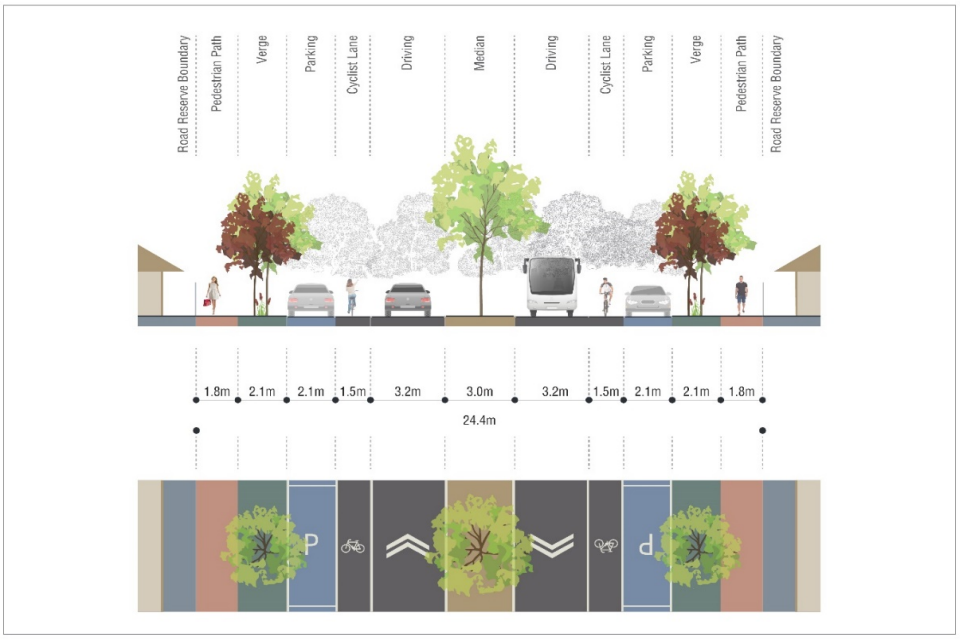


Figure 10 – RD17 Stewart Road (East of Milner Street) Cross Section

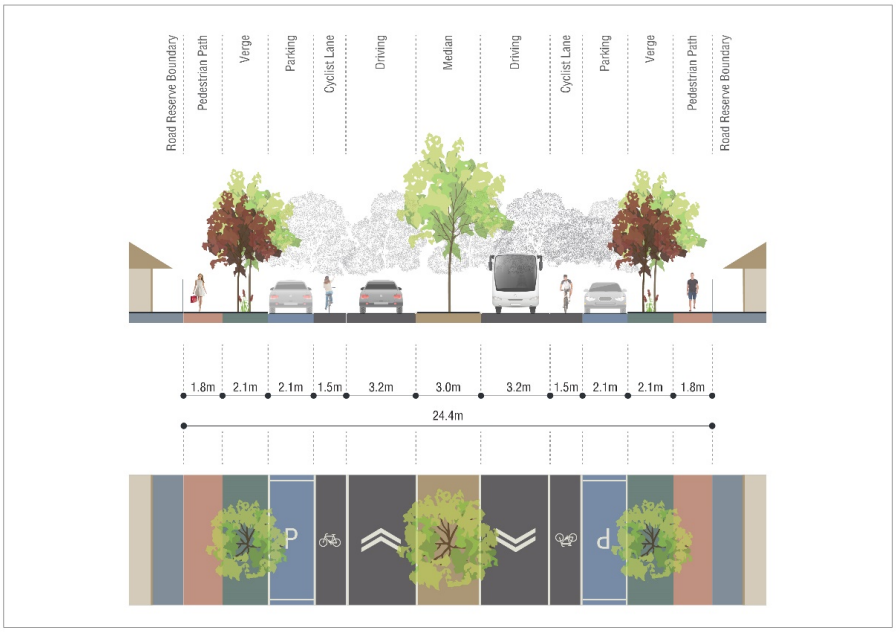
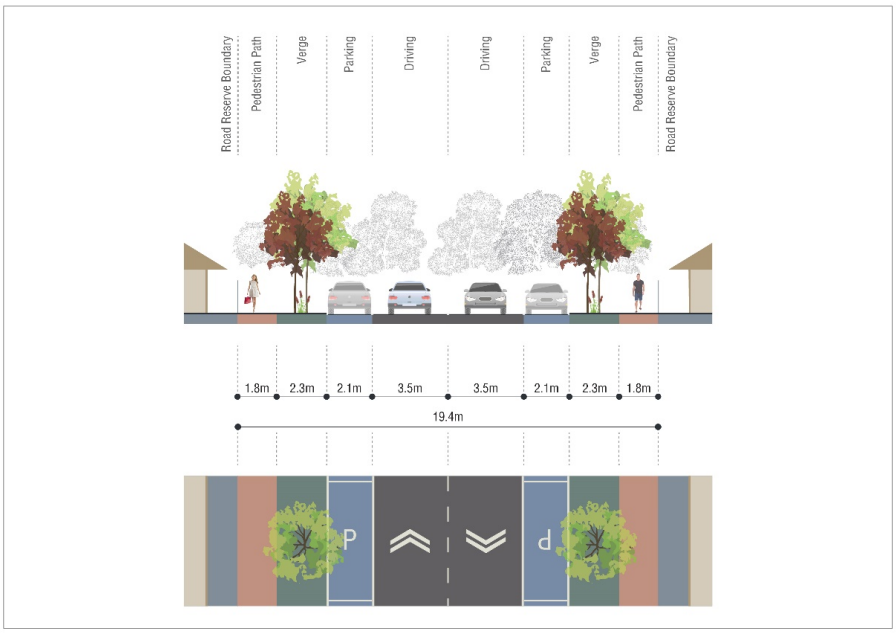


Figure 11 – RD18 Brae Road (East of TOD Connector Boulevard) and RD18A Brae Road (South of TOD Connector Boulevard) Cross Section



Note: RD18A is the same cross section in a 20.0 metre width road reserve.

Figure 12 – RD19 Brand Road (At District Open Space) Cross Section

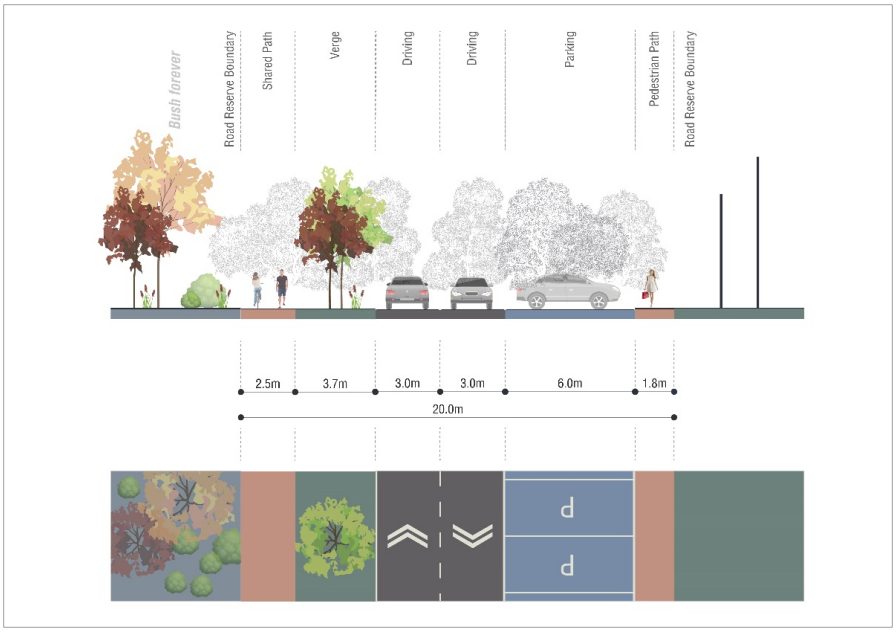


Figure 13 – RD19 Brand Road (South of District Open Space) Cross Section

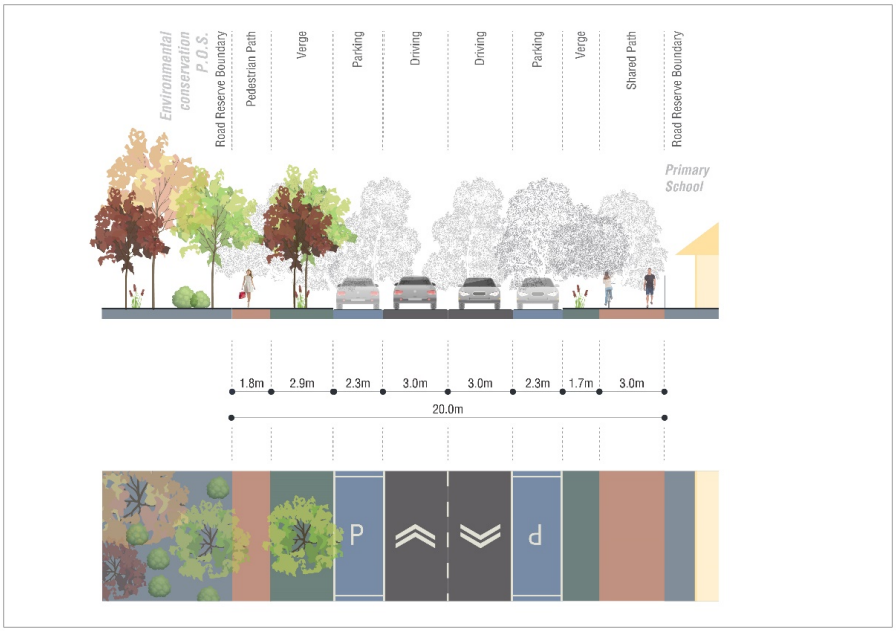
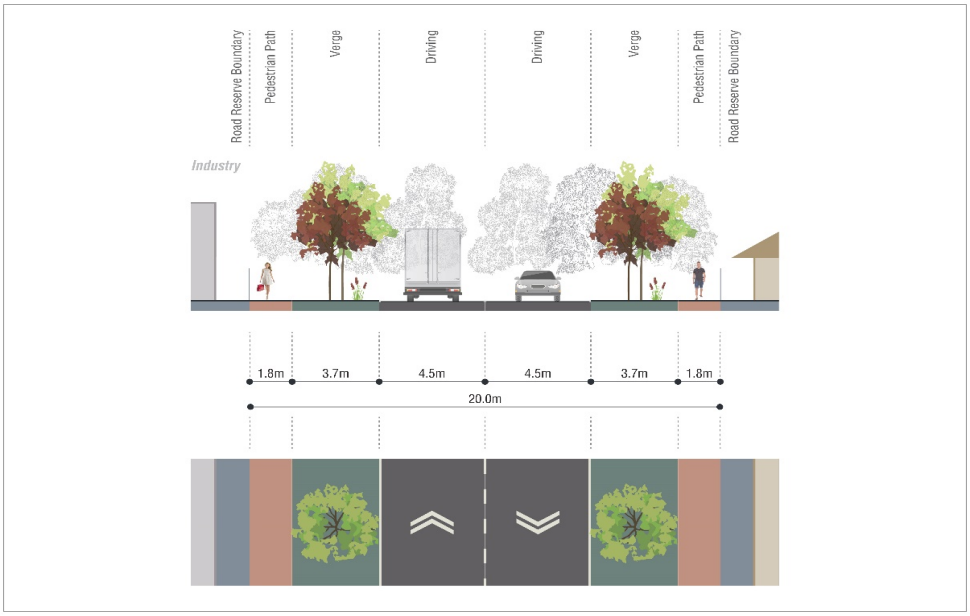


Figure 14 – Sultana Road West (East of Milner Road) Cross Section



Note: This cross section is existing.

5.0.3 Land Resumption Requirements for Roads

The proposed road network will be discussed in detail in the Transport Impact Assessment (TIA), however from a civil infrastructure perspective the following roads have been identified as part of the key road networks to be funded that are suitable for consideration as part of a future DCP. The following table shows the existing road reservation, the new road reservation, the length of the road, (not including intersections) and any applicable areas of land acquisition.

Table 4 - Proposed Road Network

Road ID	Road Name	Existing Road Reserve (m)	New Road Reserve (m)	Length (m)	Land Acquisition
RD01	Berkshire Road (Between Roe Hwy and Milner Road)	20.0	25.0	976.9	(976.9 x 5) = 4,884.6m^2
RD02	Milner Road (Sultana Road West to Stewart Road)	20.0	25.2	547	(547 x 5.2) = 2,844.4m^2
RD03	Maida Vale Road (East of Milner Road)	20.0	20.0	690	No widening required.
RD04	Maida Vale Road (Milner to Raven)	20.0	20.0	643	No widening required.
RD05	Maida Vale Road (West of Ibis Place)	20.0	20.0	257	No widening required.
RD06	Dundas Road (south of Berkshire Road)	20.0	20.0	150	No widening required.

RD07	Dundas Road (Berkshire Road to Dundas Road Old)	0	20.0	363	No widening required. Created as condition of the Railway Station Development.
RD08	Dundas Road (North of Deviation)	0	20.0	116	No widening required. As above for RD07.
RD09	Milner Road (Berkshire to Sultana)	20.0	20.0	641	No widening required.
RD10	Milner Road (Stewart Road to Maida Vale Road)	20.0	20.0	230	No widening required.
RD11	Ibis Place (new road to be designated as Enterprise Boulevard)	20.0	20.0	280	No widening required. As above for RD07.
RD12	Raven Street (Milner Road to Maida Vale Road)	20.0	24.5	418	418 x 4.5 = 1,881m^2
RD13	Raven Street (Milner Road to Brae Road)	0.0	24.5	427	427 x 24.5 = 10,461.5m^2
RD14	TOD Connector Boulevard (Milner Road to Enterprise / Ibis)	0.0	24.5	488	488 x 24.5 = 11,956m^2
RD15	TOD Connector Boulevard (Milner Road to Brae Road)	0.0	24.5	495	495 x 24.5 = 12,127.5m^2
RD16	TOD Connector Boulevard (East of Brae Road)	0.0	24.5	677	677 x 24.5 = 16,586.5m^2
RD17	Stewart Road (East of Milner Road)	20.0	24.5	552	552 x 4.5 = 2,484m^2
RD18	Brae Road (East of TOD Connector Boulevard)	19.4	19.4	875	No widening required.
RD18A	Brae Road (South of TOD Connector)	20.0	20.0	218	No widening required.
RD19	Brand Road	20.0	20.0	873	No widening required.

5.0.4 Land Resumption Requirements for LSP Intersections

The proposed road network will be discussed in detail in the Transport Impact Assessment (TIA), however from a civil infrastructure perspective the following intersections have been identified as part of the key road intersections to be funded that are suitable for consideration as part of a future DCP. The following table shows the proposed intersection, the proposed intervention or intersection type, the length of the road, and any applicable areas of land acquisition, measured from AutoCAD.

Table 5 - Key Intersections

Intersection ID	Intersection	Proposed Intervention	Land Acquisition
INT01	Milner Road / Maida Vale Road	2031: Give Way Sign-controlled Intersection, T-Intersection	n.a.
		2041: Roundabout, T-Intersection	420m^2

INT02A	Maida Vale Road / Roe Hwy Off Ramp	2031: Roundabout, 3-way	n.a. (within reservation)
INT02B	Maida Vale Road / Roe Hwy On Ramp	2031: Roundabout, 3-way	n.a. (within reservation)
INT03	Maida Vale Road / Newburn Road	2031 Provision of left and right turn deceleration lanes on Newburn Road	n.a.
		2041: Roundabout; T-Intersection	420m^2
INT04	Maida Vale Road / Dundas Road	2031: Roundabout, 3-way	375m^2
INT05	TOD Connector / Milner Road	2031-2050: Signalised Intersection, 4-way	n.a.
INT06	Dundas Road Old / Dundas Road New	2031: Signalised Intersection, 3-way	n.a.
INT07	Milner Road / Sultana Road West	2031: LILORI Configuration	n.a.
INT08	Milner Road / Nardine Close	2031: LILORI Configuration	n.a.
INT09	Brae Road / Stewart Road	2031: Give Way Sign-controlled Intersection, T-Intersection.	n.a.
		2041: Median addition to Stewart Road as a part of road upgrade.	n.a.
INT10	TOD Connector / Brae Road / Raven Street	2031: Give Way Sign-Controlled Intersection, 4-way.	n.a.
		2041: Medians added to Raven Street and TOD Connector Approaches as a part of road upgrades	n.a.
INT11	Brae Road / Brand Road	2031: Give Way Sign-Controlled Intersection, T-Intersection.	n.a.
INT12	Maida Vale Road / Ibis Place	Constructed.	n.a.
INT13	Maida Vale Road / Raven Street	2031: Give Way Sign-Controlled Intersection, 3-way.	
		2041: Medians added to all approaches as a part of road upgrades.	
		2050: Roundabout, 3-way.	420m^2
INT14	TOD Connector / Raven Street / Brae Road	2031: Roundabout, 4-way.	550m^2
INT15	Berkshire Road / Milner Road / Dundas Road	2031: 2-lane Roundabout, 4-way.	900m^2
INT16	TOD Connector / Brand Road	2031-2050:	420m^2

		Give Way Sign-Controlled Intersection, T-Intersection (Potential future requirement for a 4-way intersection).	
INT17	Milner Road / Raven Street	2031-2050: Roundabout, 4-way.	500m^2
	Milner Road / Sultana Road West	2031: Channelized T-Intersection with Priority for Heavy Vehicles; LILORI (No Right-Out movement).	n.a.
		2041: Median to be added to Milner Road as a part of road upgrade.	n.a.
INT18	Milner Road / Stewart Road	2031: Give Way Sign-Controlled Intersection, 4-way.	n.a.
		2041: Medians added to 3 out of 4 approaches as a part of road upgrades.	n.a.
		2050: Roundabout, 4-way.	500m^2

5.0.5 Maida Vale Road / Roe Hwy Bridge Reconstruction

Significant works are proposed for the Roe Highway and Maida Vale Road interchanges under the Forrestfield North Residential LSP and TOD and AC Precinct ACP. This section of Maida Vale Road is RD03A and comprises of a 95-metre section of road between the two proposed roundabout intersections with the Roe Highway On and Off Ramps. The proposed length of bridge span is approximately 50 metres between these two intersections and will involve a complete reconstruction of the bridge to suit the proposed road cross section.

5.0.6 Road Pavement Designs

In this section, KCTT have calculated ultimate pavement thicknesses based on the ultimate traffic volumes, with a CBR of 8% and assuming a 40-year pavement design life in accordance with Main Roads WA requirements.

Table 6 – Road Pavement Designs

Road ID	Road Name	Traffic Volumes (VPD)	Min Base Course Thickness (mm)	Total Flexible Pavement Thickness (mm)	Standard Pavement
RD01	Berkshire Road (Between Roe Hwy and Milner Road)	25,843	185	410	230mm sub-base 200mm base course
RD02	Milner Road (Sultana Road West to Stewart Road)	5,984	165	370	200mm sub-base 170mm base course
RD03	Maida Vale Road (East of Milner Road)	13,072	175	395	220mm sub-base 175mm base course
RD04	Maida Vale Road (Milner to Raven)	4,422	160	360	200mm sub-base 160mm base course
RD05	Maida Vale Road (West of Ibis Place)	7,620	170	375	200mm sub-base 175mm base course

RD06	Dundas Road (south of Berkshire Road)	15,169	180	395	200mm sub-base 200mm base course
RD07	Dundas Road (Berkshire Road to Dundas Road Old)	13,680	175	395	200mm sub-base 200mm base course
RD08	Dundas Road (North of Deviation)	13,680	175	395	200mm sub-base 200mm base course
RD09	Milner Road (Berkshire to Sultana)	12,928	175	395	200mm sub-base 200mm base course
RD10	Milner Road (Stewart Road to Maida Vale Road)	9,288	170	380	200mm sub-base 180mm base course
RD11	Ibis Place	3,631	160	350	200mm sub-base 160mm base course
RD12	Raven Street (Milner Road to Maida Vale Road)	5,602	165	365	200mm sub-base 165mm base course
RD13	Raven Street (Milner Road to Brae Road)	5,984	165	370	200mm sub-base 170mm base course
RD14	TOD Connector Boulevard (Milner Road to Enterprise / Ibis)	2,322	150	335	200mm sub-base 150mm base course
RD15	TOD Connector Boulevard (Milner Road to Brae Road)	6,432	165	370	200mm sub-base 170mm base course
RD16	TOD Connector Boulevard (East of Brae Road)	4,520	160	360	200mm sub-base 160mm base course
RD17	Stewart Road (East of Milner Road)	4,739	160	360	200mm sub-base 160mm base course
RD18	Brae Road (East of TOD Connector Boulevard)	2,599	155	340	180mm sub-base 160mm base course
RD18A	Brae Road (South of TOD Connector)	2,500	150	340	180mm sub-base 160mm base course
RD19	Brand Road	2,011	150	330	180mm sub-base 150mm base course

The table below provides details on the existing roads that form a part of the future road spine within the Forrestfield North TOD and AC Precinct ACP areas: -

Table 7 - Existing Road and Road Reserve Widths

Road Name	Length (m)	Road Width (m)	Road Reserve Width (m)	Description
Sultana Road West	515	6.5	20	Two-way single road with an approximate width of 6.5m and road reserve width of 20m, according to desktop studies. With no road marking or kerbing therefore, the road will require upgrading. 2-way, 2-lane road.
Milner Road	1,630	7.5	20	Rural type road with no road markings or significant kerbing.

				2-way, 2-lane road.
Dundas Road	1,150	8	20	Existing semi-mountable kerbing throughout. Major realignment works are required due to the Forrestfield North Railway Station works. 2-way, 2-lane road.
Imperial Street	310	6.5	20	Industrial type road with flush kerbing allowing drainage to flow overland onto verges. 2-way, 2-lane road.
Ibis Place	320	6.5	20	Existing coarse asphalt, rural residential road. Requires major upgrades, kerbing etc. 2-way, 2-lane road.
Raven Street	360	6.5	20	Existing coarse asphalt, rural residential road. Requires major road upgrades, kerbing etc. 2-way, 2-lane road.
Eureka Street	550	10.0	20	Industrial standard road. Fit for purposes. 2-way, 2-lane road.
Berkshire Road	1,085	9.0	20	Industrial Connector Road with 2 x 4.5m wide lanes and painted centreline.

6. Stormwater Drainage

Does the development plan show future on-site POS?	The Forrestfield North Activity Centre Plan shows multiple POS locations throughout the TOD and AC Precincts as depicted in the TOD and AC Precinct ACP. Some of the predominant locations regarding stormwater drainage are in POS areas adjacent to the Poison Gully Creek to the north, the southern corner of the intersection of Imperial Street and Dundas Road and the triangular section of land bounded by Dundas Road, Milner Street and Berkshire Road at the southern point of the TOD Precinct. There is also TEC POS-12 in the northern section of the TOD precinct, which requires protection under the ACP.
Is there a quantifiable impact on Earthworks Strategy?	YES. Refer below.
If YES, describe the impact:	The existing topography generally slopes towards the south-west corner of the ACP area as defined in the Earthworks section of this report. Combined with the proposed elevation of the Forrestfield North Railway Station, the catchments in the TOD and AC Precincts are now easily defined and deliver direct overland flows to the south for approximately 85% of the precinct with the remaining portion of the precinct draining towards the Poison Gully Creek.
Can infiltration drainage techniques be used?	Due to the increased fill provision in the TOD Precinct, particularly around the Forrestfield North Railway Station there will be opportunities to locally infiltrate stormwater drainage runoff. In other locations where the MGL approaches less than 2 metres clearance to finished levels, subsoil drainage systems should be considered with a controlling subsoil system designed to ensure MGL is maintained at a minimum clearance of 1.5 metres. The areas for subsoil are shown in this report and coordinated with the updated Urbaqua LWMS for the TOD and AC Precinct ACP.
Is subsoil drainage required?	As noted above in the section for infiltration drainage, the subsoil drainage should be used as a groundwater control methodology to ensure a minimum clearance of 1.5 metres is always held. The design of the subsoil system should ensure that the outlet is free draining with the most logical place to free drain the outlet into a future basin required in the southern POS.
Can bubble-up pits be used in the POS?	Bubble-up pits can be utilised where there are insufficient grades to free-drain to the POS and basins, although a combination of headwalls where there are sufficient grades in the catchment to allow free draining systems will most likely be used to discharge into the respective drainage basins, with bubble-up pits used for sections of lower lying catchment that cannot free drain into a POS basin.

KCTT's ISR for the TOD and AC Precinct references the work completed by Urbaqua for the LWMS. The LWMS is included in Appendix B of this ISR. Stormwater drainage calculations can vary considerably based on variations in coeeficients of runoff, final vertical design influencing slopes and times of concentration and densities of development. KCTT have run check calculations on the storage requirements for the various storm events. The City of Kalamunda will require developers within the TOD and AC Precinct to provide detailed calculations in UWMP's for specific development over the TOD and AC Precinct. The UWMP calculations should be referenced against the Urbaqua LWMS and the volumes and storages presented within this ISR, with commentary on how the UWMP baalnces the objectives of the LWMS while accommodating the proposed storage volumes based on their selection of appropriate design criterion.

The following Figure highlights the allowed runoff into wetlands from the Urbaqua LWMS.

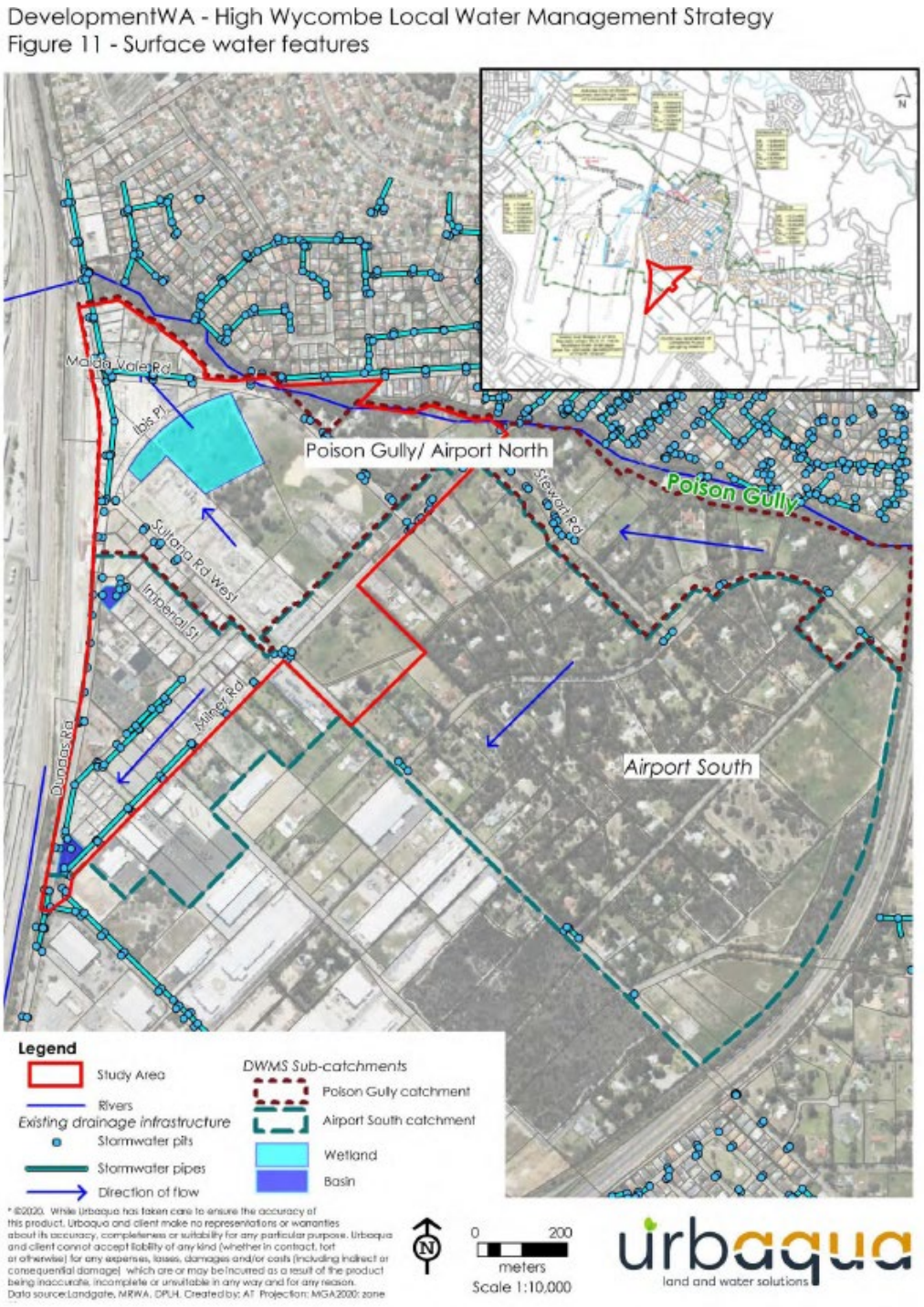
Figure 15 - Runoff into Wetland Calculations (Urbaqua LWMS)

Table 12: Runoff into wetland from local catchment

Duration	63.2% AEP (1yr ARI)	20% AEP (5yr ARI)	10% AEP (10yr ARI)	1% AEP (100yr ARI)
1 hr	374 m^3	489 m^3	519 m^3	752 m^3
3 hr	456 m^3	638 m^3	745 m^3	1,250 m^3
6 hr	389 m^3	711 m^3	868 m^3	1,513 m^3
9 hr	466 m^3	666 m^3	915 m^3	1,651 m^3
12 hr	485 m^3	816 m^3	869 m^3	1,731 m^3
18 hr	469 m^3	738 m^3	944 m^3	1,858 m^3
24 hr	355 m^3	742 m^3	841 m^3	1,790 m^3
48 hr	205 m^3	223 m^3	460 m^3	908 m^3
72 hr	36 m^3	147 m^3	305 m^3	490 m^3

The following Figure from the Urbaqua LWMS shows the direction of surface flows in the TOD and AC Precinct, with the existing City of Kalamunda underground stormwater drainage assets and pit locations shown in light blue.

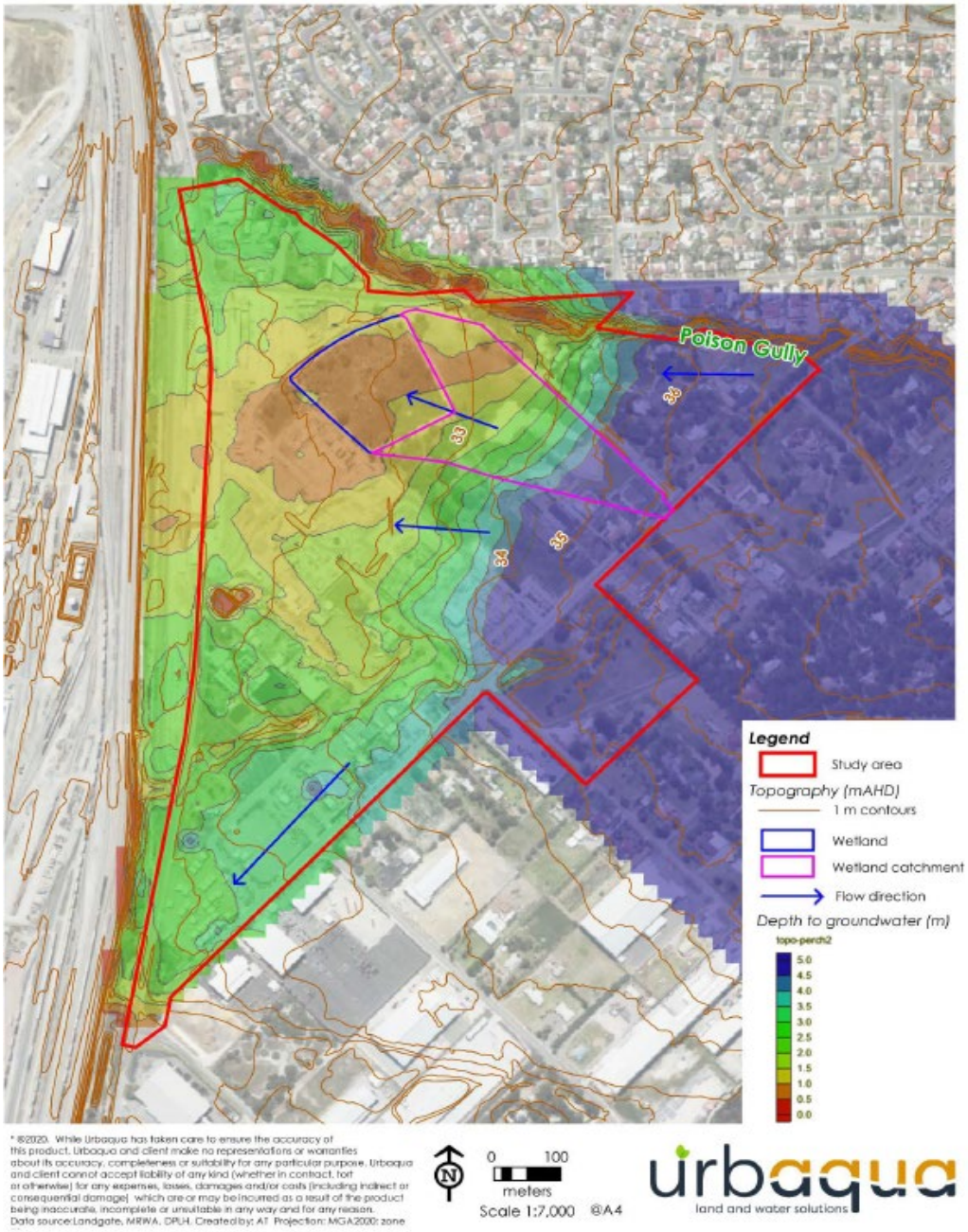
Figure 16 - Surface Water Features and Directions of Flow (Urbaqua LWMS)



The following Figure from the Urbaqua LWMS shows the direction of pre-development flows to the existing TEC and wetlands in the TOD and AC Precinct.

Figure 17 - Wetland Catchments and Direction of Overland Flows (Urbaqua LWMS)

Figure 12 - Wetland catchment



The Urbaqua LWMS specifies requirements for separation to groundwater MGL levels at the base of soakwells and the base of all stormwater infiltration devices.

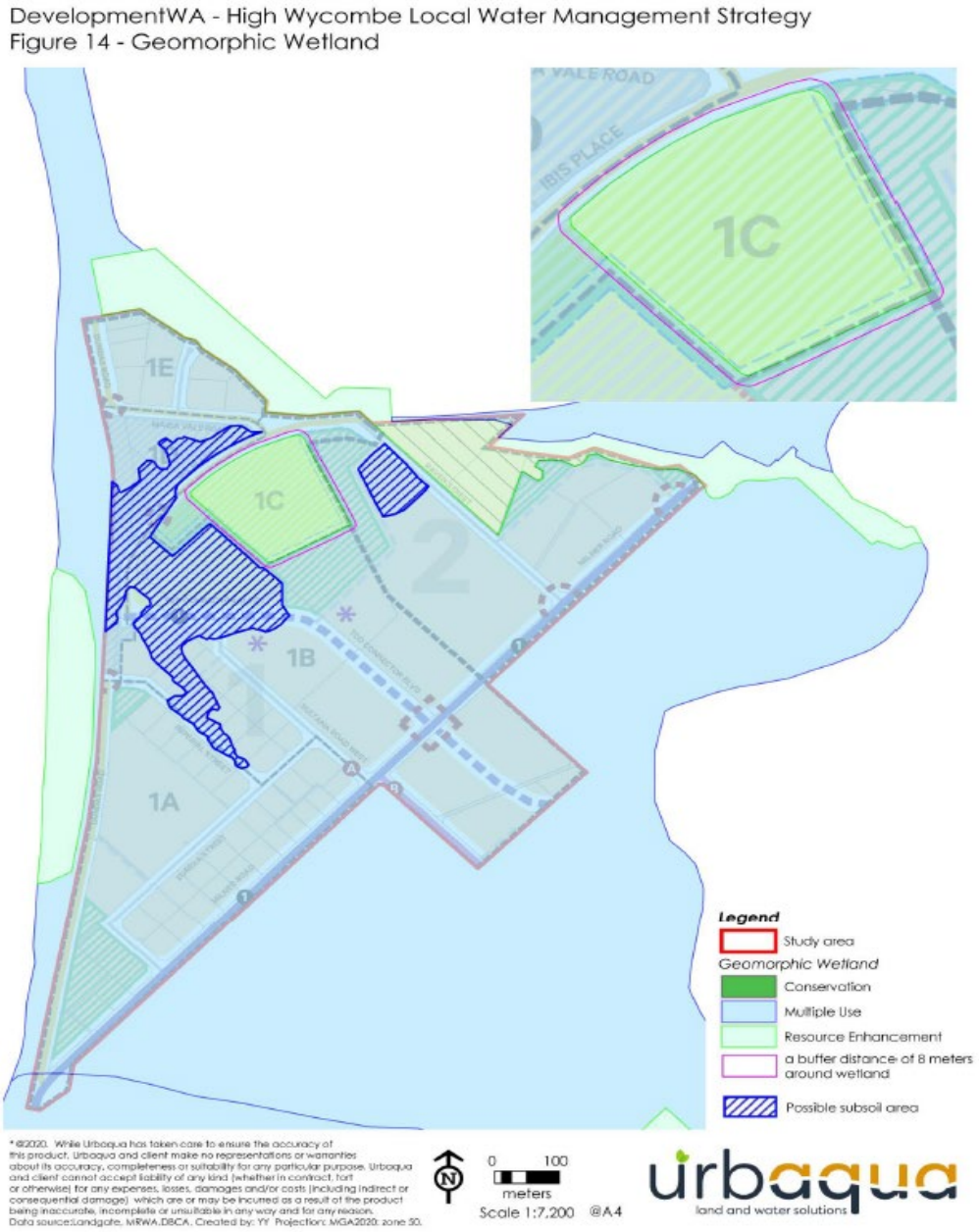
Figure 18 - Groundwater Separation Design Requirements in the TOD and AC Precinct (Urbaqua LWMS)

Table 13: Groundwater separation

Location	Specification	Comment
Soakwells	500 mm	From base of soakwell to MGL
Stormwater infiltration	300 mm	From invert of vegetated systems such as raingardens and swales to MGL

The following Figure from the Urbaqua LWMS highlights the existing low clearance between existing surface levels and the MGL groundwater levels in the TOD and AC Precinct. The dark blue shaded section provides potential guidance on areas for subsoil drainage however these areas should be compared against proposed design levels within the TOD and AC Precinct with reference to the raising of ground levels around the High Wycombe Railway Station precinct. The proposed basin in Catchment 2 (North Central Basin) should have a subsoil system at the base of the basin to improve drainage infiltration capacities in periods of inundation. The figure is provided on the following page.

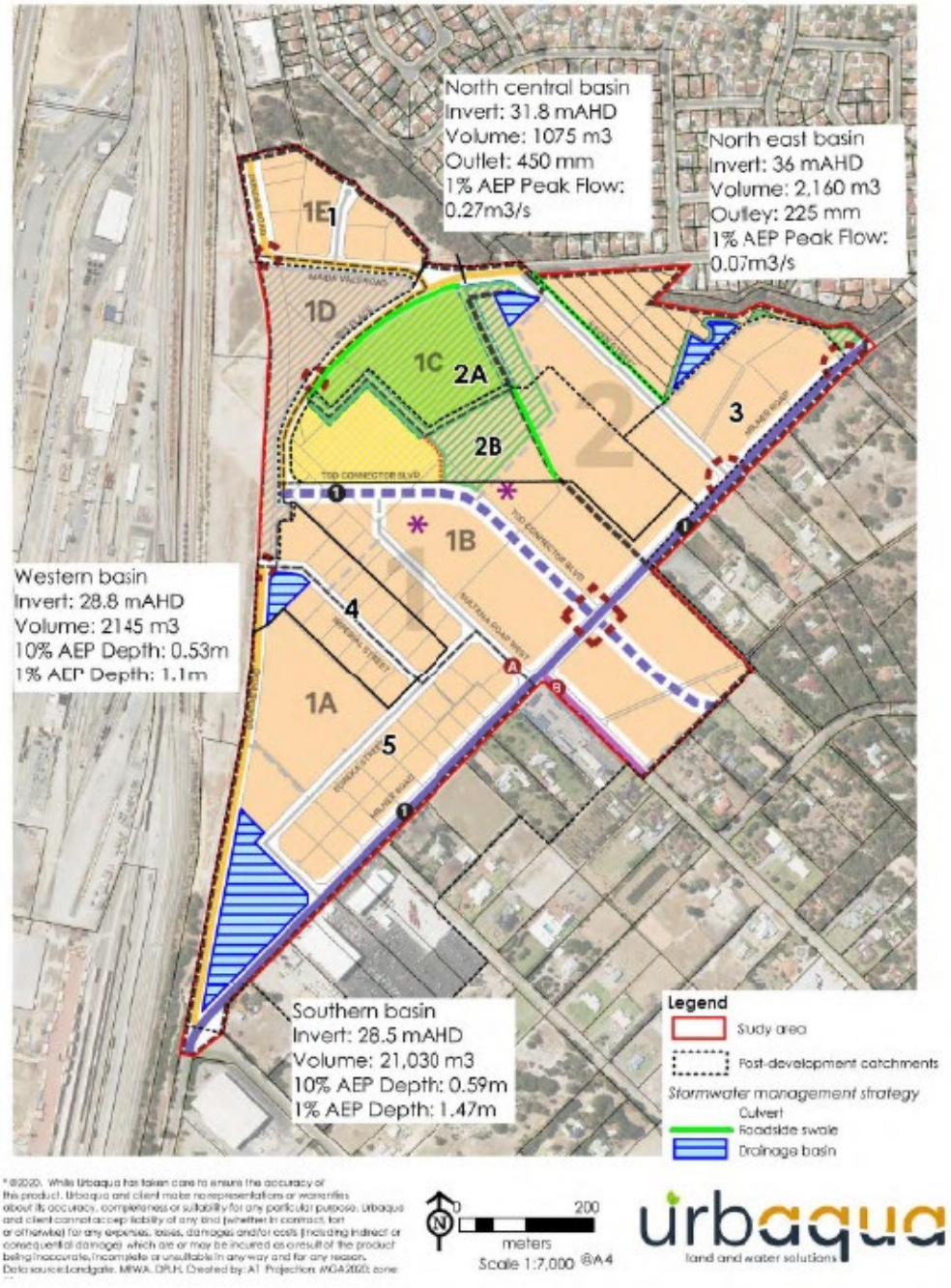
Figure 19 - Forrestfield North TOD and AC Precinct Areas of Low Clearance Between MGL Groundwater and Existing Surface Levels, With Reference to Areas for Subsoil Drainage in the ACP



The proposed stormwater management system in the TOD and AC Precincts provides a series of basins with allowable outflows into downstream natural environments in accordance with the Urbaqua LWMS. The proposed locations, sizes and invert levels are shown in the Figure on the following page.

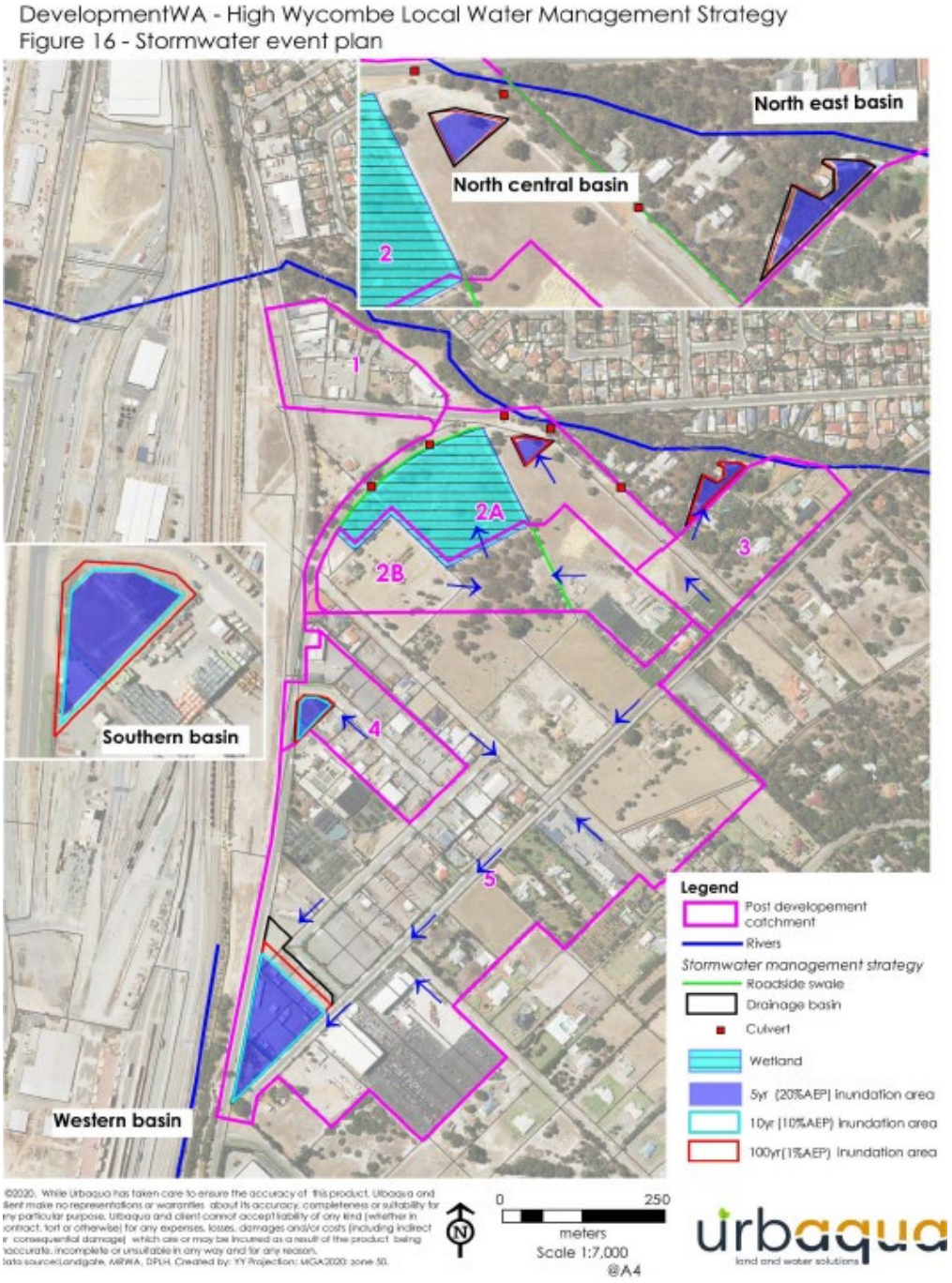
Figure 20 - Forrestfield North TOD and AC Precinct Stormwater Management System Including Basin Locations and Sizes

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 15 - Stormwater management system



The following Figure shows the location of basins with areas for the stormwater events as modelled in the Urbaqua LWMS. KCTT obtain slightly different storages using our modelling parameters as discussed in further detail in this ISR.

Figure 21 - Stormwater Event Plan (Urbaqua LWMS)



In consultation with the City of Kalamunda, the following methodology is agreed for the ACP area: -

- Roads are to be designed to 20% AEP with Maida Vale Road 10% AEP as per the TOD LWMS.
- Development areas are to be designed to the 63% AEP (15mm) as per the TOD LWMS (page 34), and light industry etc lots are "to be encouraged" (and so should be designed) for 10% AEP.

KCTT have modelled 5 catchments in the TOD and AC Precincts based on the proposed level of the Forrestfield North Railway Station concourse. The catchments are in accordance with the Urbaqua LWMS as follows: -

Table 8 - TOD and AC Precinct Catchment Area Breakdown (Urbaqua LWMS)

Table 14: Catchment area breakdown

Land use (ha)	Catchment 1 (North West)	Catchment 2 (North Central)	Catchment 3 (North East)	Catchment 4 (West)	Catchment 5 (South)
High Density Residential	0	3.48	3.56	0	0.51
Mixed Use / Activity Centre	0	2.23	0	0	0
Light Industrial	2.24	0.23	0	2.77	29
Road Reserve	1.21	2.48	0.75	0.8	7.67
POS / Wetland	0	6	0	0.25	1.89
Total Area (ha)	**3.45**	**14.43**	**4.31**	**3.82**	**39.07**

The detailed modelling and locations of swales will be an important consideration in the detailed design outcomes for the north central basin catchment. The Urbaqua figures are taken from Table 16: North Central Basin Concept in their LWMS.

Table 9 - Comparison of Storm Event Storages Catchment 2 North Central Basin (KCTT ISR / Urbaqua LWMS)

Storm Event	Urbaqua LWMS	KCTT ISR Review
20% AEP Area	1,078m^2	1,496m^2
20% AEP Depth	0.82m	0.82m
20% AEP Volume	595m^3	1,227m^3
1% AEP Area	1,302m^2	2,207m^2
1% AEP Depth	1.05m	1.05m
1% AEP Volume	868m^3	2,291m^3

KCTT recommend that the UWMP for proposed development in this area review the requirement for 1% AEP volumes as KCTT have modelled the allowable outflow of 0.27m^3/sec for Catchment 2, North Central Basin.

The following table shows the comparison for catchment 3 storages in the northeast basin.

Table 10 - Comparison of Storm Event Storages Catchment 3 North East Basin (KCTT ISR / Urbaqua LWMS)

Storm Event	Urbaqua LWMS	KCTT ISR Review
20% AEP Area	1,613m^2	729m^2
20% AEP Depth	0.43m	0.43m
20% AEP Volume	606m^3	314m^3
1% AEP Area	2,200m^2	1,118m^2
1% AEP Depth	0.98m	0.98m
1% AEP Volume	1,650m^3	1,072m^3

The following table shows the comparison for catchment 4 storages in the western basin in Imperial Street.

Table 11 - Comparison of Storm Event Storages Catchment 4 Western Basin (KCTT ISR / Urbaqua LWMS)

Storm Event	Urbaqua LWMS	KCTT ISR Review
20% AEP Area	1,544m^2	904m^2
20% AEP Depth	0.39m	0.39m
20% AEP Volume	525m^3	353m^3
1% AEP Area	2,360m^2	979m^2
1% AEP Depth	1.1m	1.1m
1% AEP Volume	1,903m^3	1,048m^3

The following table shows the comparison of catchment 5 storages in the southern basin at the corner of Milner Street, Berkshire Road and Dundas Road.

Table 12 - Comparison of Catchment Flows to Catchment 5 Southern Basin (KCTT ISR / Urbaqua LWMS)

Storm Event	Urbaqua LWMS	KCTT ISR Review
20% AEP Area	13,109m^2	12,152.4m^2
20% AEP Depth	0.42m	0.42m
20% AEP Volume	5,239m^3	5,104m^3
1% AEP Area	15,637m^2	10,377m^2
1% AEP Depth	1.47m	1.47m
1% AEP Volume	20,405m^3	15,202m^3

The following notes represent the design assumptions in the stormwater drainage calculations noted in this report and attached in Appendix A: -

- The calculations allow for outflows capped at pre-development rates and assume throttled outflow pipes at each basin location in accordance with the Urbaqua LWMS.
- The models are based on controlling groundwater to an absolute minimum of 1.5 metres clearance beneath surface.
- The earthworks strategies have modelled the Forrestfield North Railway Station at approximately 6 metres above the level at the intersection of Milner Road and Sultana Road West as per advice given by the City of Kalamunda.

KCTT confirm that the stormwater drainage strategy is robust based on the above storages being provided in accordance with the modelling and the attached plans.

7. Water

Are suitable water services located adjacent to the ACP area?	YES, the table below lists the existing services that will provide early points of connection for first developers, but that will require augmentation then upgrade under the direction of the Water Corporation. As explained previously, the Water Corporation manages the requirements for headworks based on the rates of development within the applicable areas, and the forecasted rates of development provided by developers. Therefore, it is not a requirement of the ISR to outline the timelines or the scope of upgrade requirements.

The existing network is noted in the table below: -

Road Name	Pipe Diameter	Location
Sultana Road West	150P-12	West of Milner Road
Sultana Road West	150CI	East of Milner Road
Imperial Street	150P-12	Eureka Street to Dundas Road
Milner Road	150P-12 150CI 100P-12	Milner Road South Milner Road North Milner Road South
Eureka Street	150P-12	Milner Road to Sultana Road West
Raven Street	100P-12	Raven Street East to Milner Road
Dundas Road	180PE 150P-12	Dundas Road South Dundas Road North

The following table provides a summary of the water reticulation strategy, the majority of which will be confirmed post ACP phase.

Are any assets to be constructed considered trunk infrastructure (i.e. 300mm-diameter and greater?)	Likely, particularly if development occurs at the higher ranges of the densities proposed under this ACP. The Water Corporation will determine final requirements for the provision of upgrades to headworks as development commences. Upon completion of this ACP submission the Water Corporation will complete their conceptual planning.
If YES, are the assets on the Water Corporations' Capital Works Programme?	Water assets greater than 300mm-diameter are likely to be placed on the Water Corporation's Capital Works Programme based on current policies. No headworks assets of this size are presently planned.
If NO, can the developer enter a funding arrangement?	Will need to be confirmed with the Water Corporation at the time of subdivision application.

8. Wastewater

KCTT have actively liaised with the Water Corporation through the various phases of the Forrestfield North Residential LSP and TOD and AC Precinct ACP submissions. The information is provided in detail in the below tables: -

Is the project in an existing Water Corporation Planning Area?	Yes. The ACP and LSP Areas are within the Gooseberry Hill Concept Plan.
Are amendments to Water Corporation Planning required?	Yes. Continued liaison with the Water Corporation is required with regards to future planning of infrastructure in the area. During the Forrestfield North Residential Precinct LSP submissions, KCTT provided full modelling of the estimated sewer flows based on the envisaged maximum dwelling yields likely under the planned R-codes for each area. This included review of future planning for: - • Maida Vale South LSP area on the eastern side of the Roe Highway, which is a total land area of over 1,200 hectares. • Forrestfield North Residential LSP Area. • Forrestfield North TOD and AC Precincts ACP Area. This information is shown below the current wastewater network below.

Current Wastewater Network

Location	Pipe Diameter	Lowest I.L.	Location
Sultana Road	225PVC-U	28.25	Intersection of Dundas / Sultana
Milner Road	225PVC-U	26.24	Near Intersection of Milner / Dundas / Berkshire
Dundas Road	600PVC	21.17	Near Intersection of Milner / Dundas / Berkshire
Imperial Street	225PVC-U	27.78	Intersection of Dundas / Imperial
Eureka Street	225PVC-U	26.95	Intersection of Milner / Eureka

The table below shows the breakdown of commercial, industrial, and residential zonings for the Forrestfield North Residential Precinct, with the TOD and AC Precinct and Maida Vale South combined to enable review by the Water Corporation within this document for the final sizing of sewer into the Forrestfield North Residential Precinct. The yields for the Residential Precinct can be viewed in the Forrestfield North Residential LSP documents.

The yields for the Maida Vale South area are from 2017 and have not been updated further as the area is undergoing long-term planning and is not expected to be finalised in time for these ACP and LSP documents to be finalised

Table 13 - Wastewater Flows (Residential Precinct, TOD and AC Precinct and Maida Vale South Catchments)

Catchment - Location	Total Area (ha)	Population Density / Persons Net Ha	Flow (Dry Ground)	Total Design Flow (L/s)
Residential Precinct				
Residential R30	12.0	105	180	2.8

Residential R40	14.45	120	180	3.8
Residential R60	17.6	180	180	6.9
Residential R80	5.3	200	180	2.3
Primary School	4.0	n/a	14976	0.5
Total (Residential Precinct)	**53.35**			**16.3**
TOD and AC Precinct				
Catchment 1A - Industrial	10.1	n/a	14976	1.3
Catchment 1B - Residential	8.5	180	195	3.3
Catchment 1B - Commercial	20.6	n.a.	21600	3.9
Catchment 1C – TEC	3.2	n.a.	n.a.	n.a.
Catchment 1D – Station Precinct	2.5	n.a.	9450	0.3
Catchment 1E – Commercial	2.7	n.a.	21600	0.5
Catchment 2 - Residential	14.2	200	195	6.2
Total (TOD and AC Precinct)	**73.9**			**15.5**
Maida Vale South				
Catchment 1	75	20	180	11.5
Catchment 2	83.3	30	180	19.1
Total (Maida Vale South)	**158.3**			**30.6**
Total (All Catchments)	**285.55**			**62.4**

The total ultimate wastewater flow to be accommodated into the existing Water Corporation system at the intersection of Dundas Road, Milner Road and Berkshire Road is therefore 62.4 l/sec.

Are any assets to be constructed considered trunk infrastructure (i.e. 300mm-diameter and greater?)	YES, major upgrades to the planned wastewater infrastructure are required. The Water Corporation has completed the updates to this planning. This report will be provided to the Water Corporation to enable further modification to the wastewater planning, as prior calculations were completed on higher densities within the Forrestfield North Residential and TOD and AC Precincts.
Are wastewater pumping stations required?	No.

Can the development be serviced adequately without the need for import fill? If NO, describe the fill requirements;	Yes. The planned fill around the Forrestfield North Railway Station requires consideration in the detailed design phases by each developer and the consulting engineer in coordination with the Water Corporation.
Is groundwater likely to be an issue?	Groundwater will be an issue for management throughout the TOD and AC Precincts as the main sewer for the whole of the Forrestfield North TOD, AC and Residential Precincts, plus the Maida Vale South LSP areas is fed from the intersection of Milner Road, Dundas Road and Berkshire Road and is approximately 6.5 metres depth. This depth will encounter groundwater throughout the TOD and AC Precincts. Where the works are pre-funded under instruction from the Water Corporation, the design team will need to provide full details on management of groundwater and potentially acid sulphate soils to the satisfaction of the Water Corporation, the City of Kalamunda and other agencies as is appropriate. This is a standard requirement of major infrastructure provision with the Water Corporation.

9. Gas

Are suitable gas services located adjacent to the site?	YES, with the increase in infrastructure the gas requirements are likely to change therefore the below listed services may need an upgrade. Liaison with ATCO Gas and or relevant authorities will be required.	
Location	**Pipe Diameter**	**Depth (*approximate)**
Dundas Road	110 PE 1.5 MP 70kPa	750mm below road level
Sultana Road West	110 PE 1.5 MP 70kPa	750mm below road level
Imperial Street	63 PE 1.5 MP 70kPa	750mm below road level
Milner Road	100 PVC 1.5 MP 70kPa	750mm below road level
Nardine Close	100 PVC 1.2 MP 70kPa	750mm below road level
Brae Road	100 PVC 1.5 MP 70kPa	750mm below road level

Appendix A

Drainage Calculations

Catchment Areas

Date
1/09/2021
KCTT Reference
KC00604.000
Client
City of Kalamunda

Project
Forrestfield North TOD and AC Precinct LSP

	Areas (m2)					
Catchment	Total	Lots	POS	Road	Easement / Other	Total (1 in 100yr) (Ha)
1 (Northern Commercial)	34,500	22,400	0	12,100	0	3.45
2 (North Central Basin)	144,200	59,400	60,000	24,800	0	14.42
3 (North East Basin)	43,100	35,600	0	7,500	0	4.31
4 (Western Basin)	38,200	27,700	2,500	8,000	0	3.82
5 (Southern Basin)	390,700	295,100	18,900	76,700	0	39.07
6	0					0.00
7	0					0.00
8	0					0.00
9	0					0.00
10	0					0.00
11	0					0.00
12	0					0.00
13	0					0.00
14	0					0.00

Percentage of Lots less than 300m2 (direct connections)	100.0%
Percentage of POS contributing to 1 in 1 and 1 in 5 yr events	100.0%
Road catchment C coefficient for direct pickup off roads	0.8

	Assumptions Made
1	Areas are as calculated from drawings and coordinated against Urbaqua LWMS.
2	Areas under 'Easement/Other', include Industrial Land Uses.
3	Areas termed 'Road' is the full road reserve area.
4	Areas termed 'POS' are all POS areas as shown on the plans.
5	All lots deemed to be small lots requiring their own drainage for the 1 in 5 yr event.
6	Assume 100% of the POS contributes overland flow into the 1 in 1 yr and 1 in 5 yr basins
7	All calculations allow for outflow of pre-development flows into downstream environments.

Pre-Development Flows

Date
1/09/2021

KCTT Reference
KC00604.000

Client
City of Kalamunda

Project
Forrestfield North TOD and AC Precinct LSP

	Pre-Development Runoff											
	1 in 5 ARI						1 in 100 ARI					
Catchment	time (hrs)	i, (mm/hr)	C	Area (ha)	Q(m3/s)	Q(m3)	time (hrs)	i, (mm/hr)	C	Area (ha)	Q(m3/s)	Q(m3)
1 (Northern Commercial)	4.48	9.3	0.23	3.45	0.020	329.41	4.48	17.2	0.3	3.45	0.049	798
2 (North Central Basin)	5.02	8.6	0.23	14.42	0.079	1424.67	5.02	15.9	0.3	14.42	0.191	3451
3 (North East Basin)	5.18	8.4	0.23	4.31	0.023	429.74	5.18	15.6	0.3	4.31	0.056	1046
4 (Western Basin)	5.17	8.4	0.23	3.82	0.020	379.65	5.17	15.6	0.3	3.82	0.050	924
5 (Southern Basin)	11.53	5	0.23	39.07	0.124	5159.45	11.53	9.1	0.3	39.07	0.296	12302

	1 in 10 ARI					
Catchment	time (hrs)	i, (mm/hr)	C	Area (ha)	Q(m3/s)	Q(m3)
1 (Northern Commercial)	4.48	10.6	0.24	3.45	0.024	393
2 (North Central Basin)	5.02	9.8	0.24	14.42	0.094	1701
3 (North East Basin)	5.18	9.6	0.24	4.31	0.028	515
4 (Western Basin)	5.17	9.6	0.24	3.82	0.024	455
5 (Southern Basin)	11.53	5.6	0.24	39.07	0.146	6056

	Assumptions Made
1	ToC as calculated in "ToC" Sheet for each individual catchment.
2	Runoff Coefficients Calculated as per CoSwan Drainage Criteria Handbook.
3	Catchment Area is full catchment areas as per "Catchment" sheet.
4	
5	
6	
7	

Runoff Calculations: 1-year ARI Post-Development

Date
1/09/2021

Project
Forrestfield North TOD and AC Precinct LSP

KCTT Reference
KC00604.000

Client
City of Kalamunda

kctt

Post-Development Runoff

1 in 1 ARI

	Inflow						Outflow									
Catchment	time (hrs)	i, (mm/hr)	C	Area (ha)	Peak Flow Rate, Q(m3/s)	Inflow, V(m3)	Infiltration Rate, mm/hr	Length of Road in Catchment	Soakage Pit Infiltration Outflow, m3	Soakage Pit Storage, m3	POS Bioretention Storage Required, m2	Base Perimeter (length)	Base Perimeter (width)	Designed Bioretention Storage m2	Designed POS Bioretention Volume m3	Depth
1 (Northern Commercial)	0.1	32.4	0.3	3.45	0.093	33.53	45	150	0.00	0.52	110.0	15.0	5.0	114.2	34	0.30
2 (North Central Basin)	0.1	28.4	0.3	14.42	0.341	122.86	45	1200	0.01	4.18	395.6	32.0	10.0	398.8	120	0.30
3 (North East Basin)	0.1	25.4	0.3	4.31	0.091	32.84	45	500	0.00	1.74	103.7	14.0	5.0	107.4	32	0.30
4 (Western Basin)	0.1	19.8	0.3	3.82	0.063	22.69	45	290	0.00	1.01	72.3	9.0	5.0	73.4	22	0.30
5 (Southern Basin)	0.1	19.8	0.3	39.07	0.645	232.08	45	2250	0.02	7.83	747.4	43.0	15.0	752.6	226	0.30
Total										**15.28**	**1429.0**			**1447**	**434**	

Soakage Pits	10%	refer note 7 below
Infiltration Rate	45	mm/hr
Side Slopes	3	

Assumptions Made

1	Assumed Tc of 6mins for 1year ARI Event
2	Infiltration Rate of Insitu Soil - Assumed infiltration rate of 45mm/hr approx 25% of the typicaly expected for sand.
3	Capture Pits (GP/SEP/CSEP) - 1 x 1050mm liner pit assumed per 60m of road, with soakage base 600m dia weephole with filter media for nutrient stripping.
4	Assumed capture pits have minimum trapped section of depth 600mm, and total overall depth of 2m.
5	Bioretention Area only Required in POS where Soakage Pits do not provide sufficent Storage, final size to be determined at detail design stage.
6	Catchment Area is as per the road reserve area calculated from dwg and provided in the "Catchments" sheet.
7	Allow soakage pits in 10% of the catchment area where clearance to MGL is at or exceeds 2.5 metres only

Runoff Calculations: 5-year ARI Post-Development

Date
1/09/2021
KCTT Reference
KC00604.000
Client
City of Kalamunda

Project
Forrestfield North TOD and AC Precinct LSP

Post-Development Runoff

1 in 5 ARI

	Inflow						Outflow											
Catchment	Storm Duration, T (hrs)	Rainfall Intensity i, (mm/hr)	Runoff Coefficient, C	Area (ha)	Peak Flow Rate, Q(m3/s)	Inflow, V(m3)	Infiltration Rate, mm/hr	Length of Road in Catachment	Soakage Pit Infiltration Outflow, m3	Basin Infiltration Outflow, m3	Soakage Pit Storage, m3	Required Basin Storage Area, m2	Base Perimeter (length)	Base Perimeter (width)	Designed Bioretention Storage m2	Designed POS Bioretention Volume m3	Design Depth (m)	check volume m3
1 (Northern Commercial)	0.17	74.4	0.55	3.45	0.392	235.29	41.67	150	0.00	1.44	0.52	391.3	15.6	10.0	392.2	235	0.60	
2 (North Central Basin)	0.28	55.8	0.54	14.42	1.207	1231.09	41.67	1200	0.02	9.11	4.18	1496.2	40.3	20.0	1496.2	1227	0.82	595
3 (North East Basin)	0.20	67.7	0.54	4.31	0.438	315.13	41.67	500	0.01	3.17	1.74	728.8	35.0	13.0	729.3	314	0.43	606
4 (Western Basin)	0.27	57.7	0.6	3.82	0.367	352.66	41.67	290	0.01	5.21	1.01	901.7	35.0	18.1	903.9	353	0.39	525
5 (Southern Basin)	0.82	29.1	0.55	39.07	1.737	5106.74	41.67	2250	0.13	208.81	7.83	12139.9	146.6	75.1	12152.4	5104	0.42	5239
Total											**15.3**	**15658**			**15674**	**7232**		

Soakage Pits 10% refer note 7 below
Infiltration Rate 41.67 mm/hr Note: 41.67mm/hr = 1m/day
Side Slopes 6

Assumptions Made

1	ToC as calculated in "ToC" Sheet for each individual catchment
2	Infiltration Rate of Insitu Soil - Assumed infiltration rate of 35mm/hr approx 25% of the typicaly expected for sand.
3	Capture Pits (GP/SEP/CSEP) - 1 x 1050mm liner pit assumed per 60m of road, with soakage base 600m dia weephole with filter media for nutrient stripping.
4	Assumed capture pits have minimum trapped section of depth 600mm, and total overall depth of 2m.
5	Basin Depths calculated using 5% Catchment Area as basin area/size, final sizing and depths to be calcualted at detailed design.
6	Catchment Area is as per the road reserve area calculate from dwg and provided in the "Catchments" sheet.
7	Allow soakage pits in 10% of the catchment area where MGL is 2.5 metres of greater clearance

Runoff Calculations: 100-year ARI Post-Development

Date
1/09/2021

KCTT Reference
KC00604.000

Client
City of Kalamunda

Project
Forrestfield North TOD and AC Precinct LSP

Post-Development Runoff

1 in 100 ARI

	Inflow						Outflow										
Catchment	time (hrs)	i, (mm/hr)	C	Area (ha)	Q(m3/s)	Inflow, Qi(m3)	POS Area, m2	Infiltration Rate, mm/hr	Pre Dev Allowed Outflow, m3	Length of Road in Catchment	Soakage Pit Infiltration Outflow, m3	Soakage Pit Storage, m3	Infiltration Basin Outflow, m3	Total Outflow, m3	Storage Required, m3	Storage Area Required, m2	Depth of Storage, m	Volume Storage Check
1 (Northern Commercial)	0.18	147	0.8	3.45	1.124	742	0	41.67	33	150	0.00	0.52	0.00	33	709	809	0.900	
2 (North Central Basin)	0.27	117	0.55	14.42	2.582	2479	60000	41.67	183	1200	0.02	4.18	0.02	183	2291	2207	1.050	868
3 (North East Basin)	0.22	133	0.9	4.31	1.433	1118	0	41.67	44	500	0.01	1.74	0.01	44	1072	1118	0.980	1650
4 (Western Basin)	0.28	113	0.9	3.82	1.078	1100	2500	41.67	51	290	0.01	1.01	0.01	51	1048	979	1.100	1903
5 (Southern Basin)	0.82	56	0.9	39.07	5.470	16081	18900	41.67	871	2250	0.13	7.83	0.11	871	15202	10377	1.470	20405
Total															**20322**	**15490**		24826

Soakage Pits	10%	refer note 8 below
Infiltration Rate	41.67	mm/hr
Side Slopes	6	

Assumptions Made

1	ToC as calculated in ToC Sheet for each individual catchment
2	Infiltration Rate of Insitu Soil - Assumed Sandy soil infiltration rate of 15mm/hr approx 25% of the typicaly expected for sand.
3	Capture Pits (GP/SEP/CSEP) - 1 x 1050mm liner pit assumed per 60m of road, with soakage base 600m dia weephole with filter media for nutrient
4	Assumed capture pits have minimum trapped section of depth 600mm, and total overall depth of 2m.
5	
6	
7	
8	No soakage allowed in pits for in the 1 in 100 year event

Peak Flow Runoff

Date
1/09/2021

Project
Forrestfield North TOD and AC Precinct LSP

KCTT Reference
KC00604.010

Client
City of Kalamunda

	Pre-Development		Post-Development			Difference	
Catchment	5 year ARI	100 year ARI	1 year ARI	5 year ARI	100 year ARI	5 year ARI	100 year ARI
1 (Northern Commercial)	0.020	0.049	0.093	0.392	1.124	0.073	1.074
2 (North Central Basin)	0.079	0.191	0.341	1.207	2.582	0.262	2.391
3 (North East Basin)	0.023	0.056	0.091	0.438	1.433	0.068	1.377
4 (Western Basin)	0.020	0.050	0.063	0.367	1.078	0.043	1.029
5 (Southern Basin)	0.124	0.296	0.645	1.737	5.470	0.520	5.174

* Peak Flow is in m3/s

IFD Table

Date
1/09/2021
KCTT Reference
KC00604.010
Client
City of Kalamunda

Project
Forrestfield North TOD and AC Precinct LSP

		Average Reccurrence Interval						
	DURATION	1	2	5	10	20	50	100
Mins	1 min	100	111	146	171	197	233	262
	2 min	87.6	96.2	124	144	165	195	219
	3 min	78.3	86.1	112	130	149	176	199
	4 min	71.1	78.3	102	119	137	162	183
	5 min	65.2	72	94.4	111	127	150	169
	10 min	47.5	52.6	69.4	81.6	93.9	111	125
	15 min	38.2	42.4	55.9	65.6	75.5	89.2	100
	20 min	32.4	35.9	47.3	55.5	63.8	75.3	84.5
	25 min	28.4	31.4	41.4	48.4	55.7	65.7	73.7
	30 min	25.4	28.1	36.9	43.2	49.7	58.6	65.8
	45 min	19.8	21.8	28.5	33.3	38.3	45.3	50.9
Hours	1 hour	16.5	18.2	23.6	27.6	31.8	37.7	42.5
	1.5 hour	12.8	14	18.2	21.3	24.5	29.2	33.1
	2 hour	10.6	11.6	15.1	17.7	20.4	24.5	27.9
	3 hour	8.23	8.99	11.6	13.7	15.9	19.2	22
	4.5 hour	6.37	6.95	9	10.6	12.4	15.1	17.4
	6 hour	5.32	5.8	7.52	8.88	10.4	12.7	14.7
	9 hour	4.11	4.49	5.83	6.9	8.09	9.91	11.5
	12 hour	3.43	3.74	4.86	5.75	6.74	8.26	9.59
	18 hour	2.64	2.88	3.75	4.42	5.16	6.29	7.27
	24 hour	2.18	2.39	3.1	3.65	4.23	5.12	5.88
	30 hour	1.88	2.06	2.67	3.13	3.61	4.33	4.94
	36 hour	1.67	1.83	2.36	2.75	3.15	3.76	4.27
	48 hour	1.37	1.51	1.94	2.24	2.54	3	3.36
	72 hour	1.05	1.15	1.47	1.68	1.88	2.17	2.4
	96 hour	0.867	0.952	1.21	1.37	1.53	1.75	1.92
	120 hour	0.753	0.826	1.05	1.19	1.32	1.5	1.64
	144 hour	0.675	0.74	0.936	1.06	1.18	1.35	1.47
	168 hour	0.619	0.678	0.858	0.977	1.09	1.25	1.37

RETURN PERIOD	A	B	C	D	E	F	G
1	2.737122	-6.37E-01	-2.52E-02	7.36E-03	9.94E-05	-3.48E-05	-3.40E-05
2	3.000591	-6.43E-01	-2.36E-02	7.35E-03	1.38E-05	-2.00E-05	-3.45E-05
5	3.237841	-6.56E-01	-2.00E-02	5.96E-03	7.56E-06	1.91E-04	-7.20E-05
10	3.374643	-6.66E-01	-1.72E-02	6.46E-03	-2.93E-04	1.61E-04	-5.86E-05
20	3.540845	-6.74E-01	-1.47E-02	5.99E-03	-4.38E-04	2.45E-04	-6.85E-05
50	3.742371	-6.84E-01	-1.15E-02	6.19E-03	-7.26E-04	2.56E-04	-6.26E-05
100	3.885548	-6.91E-01	-9.99E-03	5.86E-03	-7.61E-04	3.14E-04	-7.20E-05

Time of Concentration Calculations

Date
1/09/2021
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Project
Forrestfield North TOD and AC Precinct LSP

kctt

Post Development Peak Flow
ToC - 1in5 ARI - Manning Kinematic Equation

Catchment	Flow Path Length, m (L)	Change in Height, m (H)	Slope (So)	Catchment Roughness, n*	2year 24hr Rainfall Depth, mm (P)	ToC, mins	Rainfall Intensity, i (mm/hr)
1 (Northern Commercial)	155	1	0.006452	0.013	57.36	10	74.4
2 (North Central Basin)	420	5	0.011905	0.013	57.36	17	55.8
3 (North East Basin)	225	1.75	0.007778	0.013	57.36	12	67.7
4 (Western Basin)	275	1.5	0.005455	0.013	57.36	16	57.7
5 (Southern Basin)	1075	5.5	0.005116	0.013	57.36	49	29.1
6			#DIV/0!	0.013	57.36	#DIV/0!	#DIV/0!
7			#DIV/0!	0.013	57.36	#DIV/0!	#DIV/0!
8			#DIV/0!	0.013	57.36	#DIV/0!	#DIV/0!
9			#DIV/0!	0.013	57.36	#DIV/0!	#DIV/0!
10			#DIV/0!	0.013	57.36	#DIV/0!	#DIV/0!
11			#DIV/0!	0.013	57.36	#DIV/0!	#DIV/0!
12			#DIV/0!	0.013	57.36	#DIV/0!	#DIV/0!
13			#DIV/0!	0.013	57.36	#DIV/0!	#DIV/0!
14			#DIV/0!	0.013	57.36	#DIV/0!	#DIV/0!

Post Development Peak Flow
ToC - 1in100 ARI - Manning Kinematic Equation

Catchment	Flow Path Length, m (L)	Change in Height, m (H)	Slope (So)	Catchment Roughness, n	2year 24 Rainfall Depth, mm (P)	ToC, mins	Rainfall Intensity, i (mm/hr)
1	155	0.7	0.004516	0.013	57.36	11	146.6
2	420	5.5	0.013095	0.013	57.36	16	117.2
3	225	1.5	0.006667	0.013	57.36	13	133
4	275	1.25	0.004545	0.013	57.36	17	112.9
5	1075	5.5	0.005116	0.013	57.36	49	56
6			#DIV/0!	0.013	57.36	#DIV/0!	#DIV/0!
7	0		0.00375	0.013	57.36	0	#NUM!
8	0		0.003704	0.013	57.36	0	#NUM!
9	0		0.001818	0.013	57.36	0	#NUM!
10	0		0.00339	0.013	57.36	0	#NUM!
11	0		0.004918	0.013	57.36	0	#NUM!
12	0		0.004167	0.013	57.36	0	#NUM!
13	0		0.00303	0.013	57.36	0	#NUM!
14	0		0.00303	0.013	57.36	0	#NUM!

Pre Development Runoff
ToC - 1in5 ARI - Manning Kinematic Equation

Catchment	Flow Path Length, m (L)	Change in Height, m (H)	Slope (So)	Catchment Roughness, n	2year 24 Rainfall Depth, mm (P)	ToC, mins	Rainfall Intensity, i (mm/hr)
1	275	0.7	0.003	0.3	57.36	269	9.3
2	600	5.5	0.009	0.3	57.36	301	8.6
3	400	1.5	0.004	0.3	57.36	311	8.4
4	375	1.25	0.003	0.3	57.36	310	8.4
5	1200	5.5	0.005	0.3	57.36	692	5
6	0	0	#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
7			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
8			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
9			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
10			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
11			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
12			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!

13	#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
14	#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!

Pre Development Runoff							
ToC - 1in10 ARI - Manning Kinematic Equation							
Catchment	Flow Path Length, m (L)	Change in Height, m (H)	Slope (So)	Catchment Roughness, n	2year 24 Rainfall Depth, mm (P)	ToC, mins	Rainfall Intensity, i (mm/hr)
1	275	0.7	0.003	0.3	57.36	269	10.6
2	600	5.5	0.009	0.3	57.36	301	9.8
3	400	1.5	0.004	0.3	57.36	311	9.6
4	375	1.25	0.003	0.3	57.36	310	9.6
5	1200	5.5	0.005	0.3	57.36	692	5.6
6	0	0	#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
7			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
8			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
9			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
10			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
11			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
12			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
13			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
14			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!

Pre Development Runoff							
ToC - 1in100 ARI - Manning Kinematic Equation							
Catchment	Flow Path Length, m (L)	Change in Height, m (H)	Slope (So)	Catchment Roughness, n	2year 24 Rainfall Depth, mm (P)	ToC, mins	Rainfall Intensity, i (mm/hr)
1	275	0.7	0.003	0.3	57.36	269	17.2
2	600	5.5	0.009	0.3	57.36	301	15.9
3	400	1.5	0.004	0.3	57.36	311	15.6
4	375	1.25	0.003	0.3	57.36	310	15.6
5	1200	5.5	0.005	0.3	57.36	692	9.1
6			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
7			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
8			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
9			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
10			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
11			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
12			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
13			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!
14			#DIV/0!	0.3	57.36	#DIV/0!	#DIV/0!

Assumptions Made	
1	Roughness Coefficient, n, for Post Development flows assumed as 0.013 (Asphalt).
2	Roughness Coefficient, n, for Pre Development flows assumed as 0.3
3	Runoff lengths and difference in heights measured from plans.
4	
5	
6	
7	

Appendix B

Urbaqua LWMS

Transit Oriented Development, High Wycombe

Local Water Management Strategy

Prepared for Development WA

By Urbaqua

May 2021

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land and water solutions

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1 INTRODUCTION

Urbaqua has been commissioned by Development WA to prepare this Local Water Management Strategy (LWMS) in support of the Local Structure Plan for the Transit Orientated Development precinct (herein referred to as Study Area). The Study Area is located approximately 12 km east of the Perth CBD. The site is 57 hectares in size and is bound by:

- The future above-ground High Wycombe train station to the west;
- Proposed residential precinct to the east (subject to a separate structure plan);
- Poison Gully Creek to the north; and
- Light industry to the south.

The development is aimed at providing an activity centre around the train station, including higher density residential and commercial areas owing to the improved transport connections to the Perth CBD. The LWMS is prepared to support the Local Structure Plan that is being prepared by Development WA in conjunction with the City of Kalamunda. The document will guide sustainable water management during redevelopment.

1.1 Principles and design objectives

Consistent with *State Planning Policy 2.9: Water Resources* (WAPC, 2006) and *Better urban water management* (WAPC, 2008) a local water management strategy (LWMS) is required to be submitted to support any rezoning of land in a Local Planning Scheme or adoption of a Local Structure Plan to ensure that appropriate water management strategies are identified. The position of this strategy within the state government planning framework is defined in *Better urban water management* (WAPC, 2008) and is outlined in Figure 1.

The LWMS should be prepared in accordance with the Department of Water and Environmental Regulation's (DWER) *Interim: Developing a local water management strategy* (DoW, 2008a) and should demonstrate to the satisfaction of the WAPC in accordance with any approved DWMS:

- How the key principles and strategies of this plan have been addressed;
- How the urban structure will address water use and management;
- Existing and required water management infrastructure; and,
- Detailed land requirements for water management.

The principles and strategies contained within Section 5 of this LWMS will be implemented as part of detailed land use planning and development requirements and are consistent with the framework and requirements in *Better urban water management* (WAPC, 2008) as demonstrated in Appendix 1.

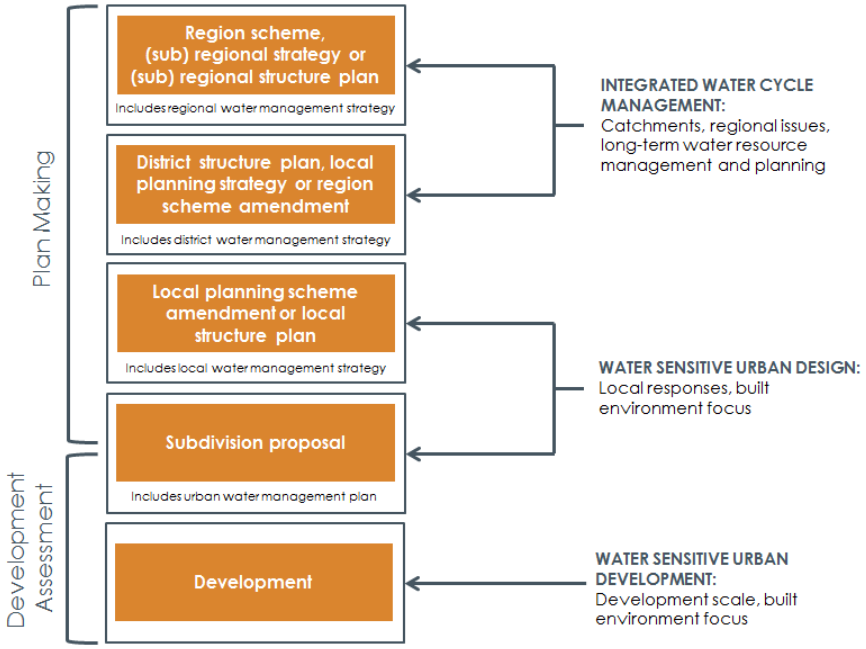

Figure 1: Integrating water planning with land planning processes (WAPC, 2008)

1.2 Planning background

The site is currently zoned as 'Urban' under the Metropolitan Regional Scheme and as 'Light Industry' (south), 'Industrial Development' (north) and 'Rural Composite' (east) under the City of Kalamunda's Local Planning Scheme No. 3.

A District Structure Plan (DSP) was prepared by the City of Kalamunda to facilitate rezoning a portion of the land from 'Rural' to the current 'Urban' zoning. The DSP addressed the Study Area and adjacent residential precinct to the east and was supported by the *Forrestfield North District Water Management Strategy* (Strategen, 2015) and subsequently the *Forrestfield North Residential Precinct LWMS* (Strategen, 2020).

1.3 Guiding documents

A number of guiding documents have been considered that are relevant to this strategy area. In addition to State Planning Policy 2.9, these documents inform the strategies and management principles contained within this Local Water Management Strategy:

- Developing a Local Water Management Strategy (DoW, 2008a);
- Stormwater Management Manual for Western Australia (DoW, 2004-09);
- Better Urban Water Management (WAPC, 2008); and,
- Decision Process for Stormwater Management in Western Australia (DWER, 2017).

2 PROPOSED DEVELOPMENT

Development of the Study Area to create an urban precinct provides opportunities to incorporate water sensitive urban design. This section outlines the key elements of the current land use and proposed re-development that influence water management.

2.1 Previous land use

The Study Area is predominantly composed of light industrial and industrial uses, with areas of remnant vegetation and rural houses. The site was largely cleared by the 1960s, with the major road layout established in the 1970s, along with the Forrestfield Marshalling Yard. Until the late 2000s, the Study Area was generally used for rural and agricultural purposes which have since been replaced by light industrial and laydown areas.

Throughout the previous development in the Study Area, pockets of vegetation have been retained, particularly along Poison Creek Gully and the wetland south of Maida Vale Road (Figure 2). The latter is mapped as a Resource Enhancement Wetland (REW) (UFI:15875) and has Threatened Ecological Communities (TEC). The hydrological function of this wetland is discussed further in Section 4.5.3, and protection is required through the proposed water management strategies.

2.2 Development

The local structure plan will guide the future urban development of the Study Area. Proposed development will feature:

- Retention and protection of the wetland and Poison Gully Creek foreshore;
- Revised major road layouts;
- Higher density residential development (in place of rural uses);
- A community hub adjacent to the railway precinct; and,
- Increased areas of public open space.

The concept design is presented in Figure 3 and included in Appendix 2.

Development will increase impervious areas and increase stormwater runoff. There is, however, an opportunity to integrate water sensitive urban design into the development and provide improved water quality and sustainability outcomes.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 2 - Study area



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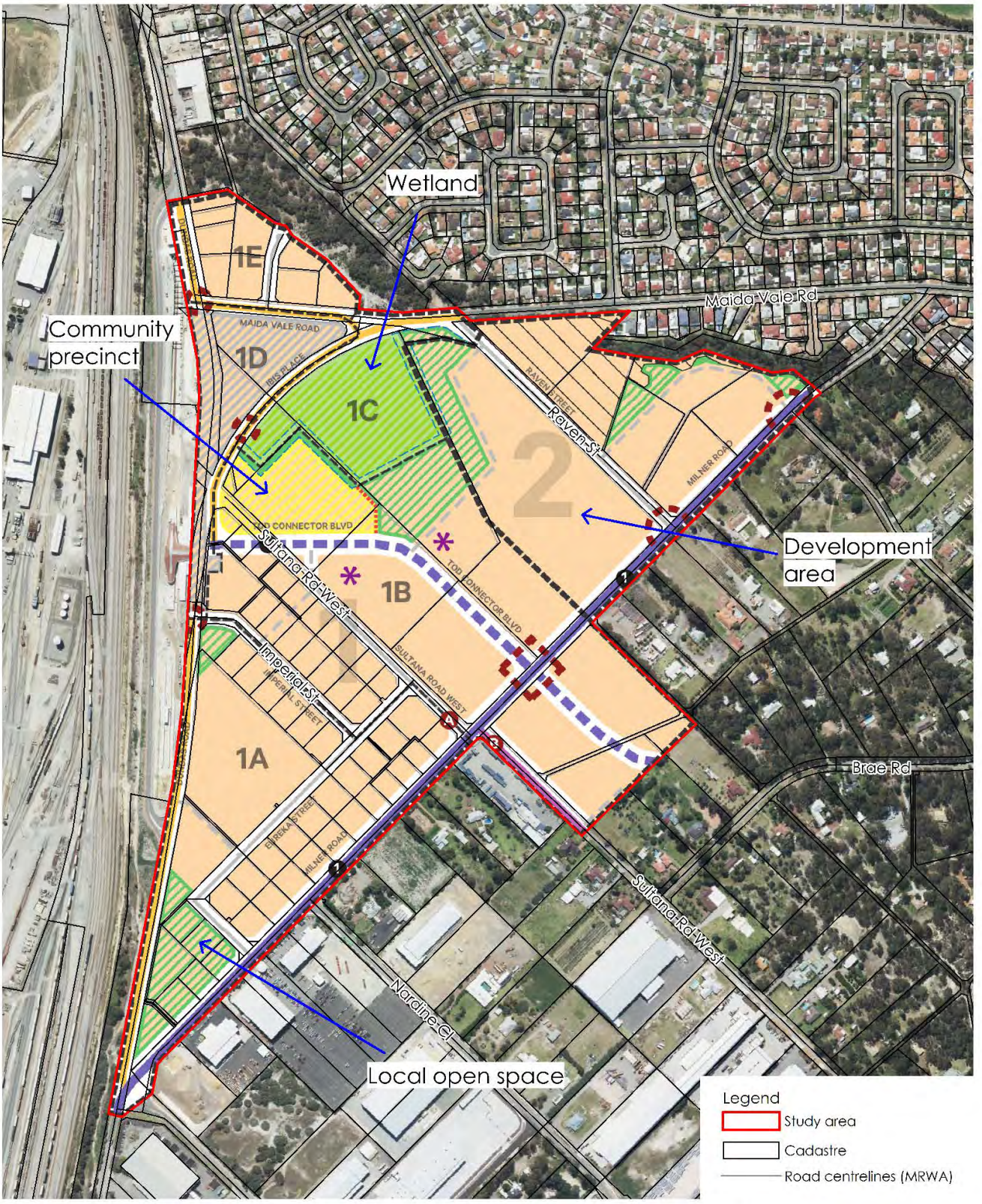
DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 3 - Concept plan



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3 DESIGN OBJECTIVES

Table 1 outlines the key design objectives to be achieved within the Study Area.

Table 1: Design objectives

Design Element	Objectives
Water sustainability	• Maximise local infiltration to replenish surface groundwater aquifers; and, • Provide alternative water sources for domestic irrigation and in-house domestic non-drinking water demands.
Protection of important environments	• Protect, and enhance where possible, Bush forever sites (Poison Gully and TEC wetland) from the potentially harmful effects of development.
Surface water management	• Maintain or restore desirable environmental flows and/or hydrological cycles; • For flood management, manage up to the 1% AEP (100-year ARI event) within the development area to pre-development flows and Water Corporation requirements (as per the Limestone Creek Capacity Review); • Ensure that the first 15 mm of rainfall receives treatment prior to discharging to a receiving environment (DWER, 2017); • Minor events should be managed to provide serviceability requirements; • Roads and public open spaces are to be designed to cater for the surface overflow for more severe storm events; and, • Habitable floors at least 300 mm above the 1% AEP flood level in the local drainage system and 500 mm above the 1% AEP flood level in basins/sumps (where there is no overflow relief) and Poison Gully Creek.
Groundwater management	• Provide appropriate separation between finished lot levels and groundwater to provide the expected level of amenity; • Ensure soakwells devices are designed at or above maximum groundwater levels; and, • Ensure that any subsoil drainage is treated prior to discharge to waterways and sensitive environments.
Management of disease vectors and nuisance insects	• Limit the creation of new sites for breeding of nuisance insects; • Prevent standing water in drainage infrastructure (infiltration within 96hrs); and, • Improve water quality throughout the development.
Implementation	• Provide a framework to implement water management strategies outlined in the LWMS; and, • Water management measures during construction to prevent damage to existing infrastructure and receiving environments.

4 EXISTING SITE CHARACTERISTICS

A summary of the existing environmental conditions in the Study Area are provided in this section, including determination of the opportunities and constraints for water management.

4.1 Climate

The climate is typical of the south western region of Western Australia and is characterised by the Koppen Climate Classification as Dry Subtropical featuring mild winters and hot to very hot summers. The dominate rainfall mechanisms are frontal systems caused by cold fronts associated with low pressure systems that extend across southern Australian between May and October. During the summer months, thunderstorms and ex-tropical cyclones can bring intense rainfall.

Average annual rainfall recorded at the nearest long-term Bureau of Meteorology (BoM) weather station (Perth Airport (no. 9021) west of the Study Area) since 1944 is 762 mm (Figure 5). Since 2000, the average annual rainfall is 667 mm, an approximately 12.5% decrease.

The significant decrease in annual rainfall is associated with a decrease in winter rainfall. Rainfall in the June and July has decreased by approximately 20%. There has been an increase in late summer rainfall associated with thunderstorms and ex-tropical cyclones, but this rainfall is often intense occurs over a short duration and does not affect the loss of winter rainfall.

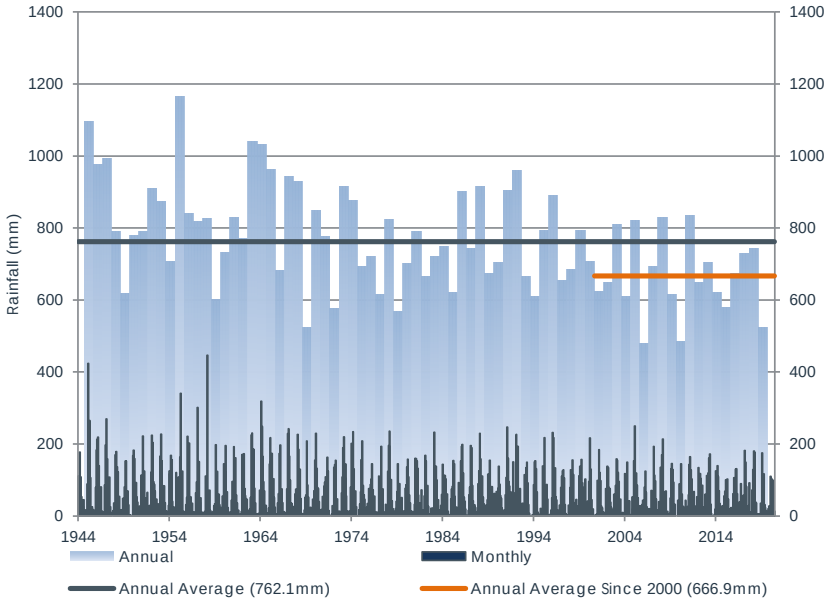


Figure 4: Rainfall summary data (Perth Airport, BoM, 2020)

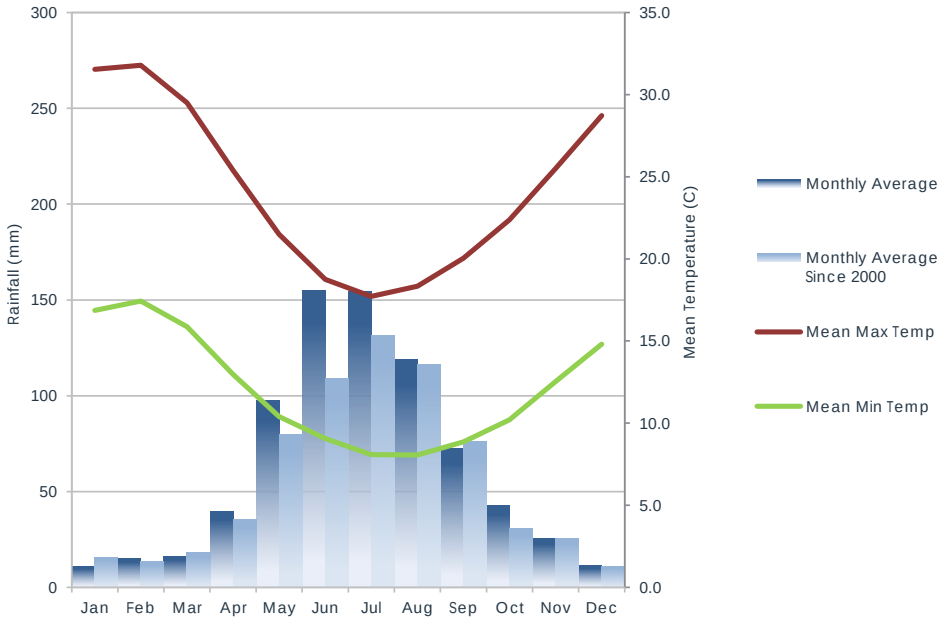


Figure 5: Climate summary data (Perth Airport, BoM, 2020)

Temperatures recorded at the Perth Metro WA (BoM station 9225) show that the average monthly maximum temperatures range between 18.0°C in July and 32.0°C in February, while average monthly minimum temperatures range between 8.0°C in July and 17.5°C in February. Prior to the year 2000, the average monthly maximum temperatures were 17.7°C in July and 31.8°C in February, while average monthly minimum temperatures range between 8.1°C in July and 17.4°C in February.

4.2 Topography

Topography at the Site varies from 37 mAHD in the west and from there it slopes downwards to the east reaching a minimum of 28.8 mAHD. The existing elevation contours are presented in Figure 6.

4.3 Geology and soils

Stormwater management is influenced by the soils of the Study Area and their ability to retain and infiltrate runoff.

4.3.1 Surface geology and soils

Regional surface geological mapping indicates the site is predominantly Sand S10, with a small portion of Mgs1in the southern corner (Figure 6).

Several bores were drilled by Strategen in 2011 and the geotechnical conditions were outlined in the DWMS (Strategen, 2015). The soils identified in these bores were described as predominantly sand with occasional gravelly sand present at surface. Low permeability

material (clayey sand) was identified in several locations, between 2.5 m and 5 m below the surface, including the wetland area. This is discussed further in the below sections.

4.3.2 Soil *Permeability*

4.3.2.1 *Forrestfield Airport Link Geotechnical Investigation (Golder Associates, 2015)*

This geotechnical investigation was conducted in 2015 as part of the Forrestfield Airport Link (FAL) Project, with test sites located within a portion of this Site Area. Infiltration pit PW1-01 is located just north of the Maida Vale Road and Dundas Road intersection. Permeability test pit locations I1-01 to I1-03 are located along Sultana Road West and Dundas Road (Figure 6).

The pumping test at PW1-01 indicated the test pit capacity was reached within 5 minutes and resulted in a negligible change in water level for 6 hours. This indicates the infiltration capacity in this location is negligible.

The permeability test results at I1-01 to I1-03 have been summarised in Table 2.

Table 2: Permeability Test Summary

Location	Test 1 K (m/day)	Test 2 K (m/day)	Test 3 K (m/day)	Average K (m/day)
I 1-01	18.6	12.9	11.7	14.40
I 1-02	5.0	3.7	3.1	3.93
I 1-03	11.4	10.5	9.8	10.57

(Golder, 2015)

The full FAL geotechnical report including test pit results has been provided in Appendix 3.

4.3.2.2 *Forrestfield North Residential Precinct Geotechnical Investigations*

Sands are present at surface level across the majority of this site, with clayey sand present from 2.25 to 3.5 mbGL as indicated by borehole logs and test pit logs (Appendix 3), and as discussed in Section 4.3.1. Hydraulic conductivity of sands and clayey sands were examined as part of the adjacent residential precinct LWMS (Strategen 2020). The findings indicate:

- Conductivity of fine to medium sand (the majority of the Study Area) was measured between 2.7 and >10 m/day (with a recommended design conductivity of 1.3 to 3.0 m/day).
- Clayey sand, the impermeable material at depth within the Study Area, had a measured hydraulic conductivity between 0.7 and 1.2 m/day (with a recommended design conductivity of 0.3 to 0.6 m/day).

It is noted that the City of Kalamunda have indicated that soakwells will not be permitted where the design hydraulic conductivity is less than 1m/day. This will be investigated further in the UWMP.

4.3.2.3 *Depth to Low Permeability Soils*

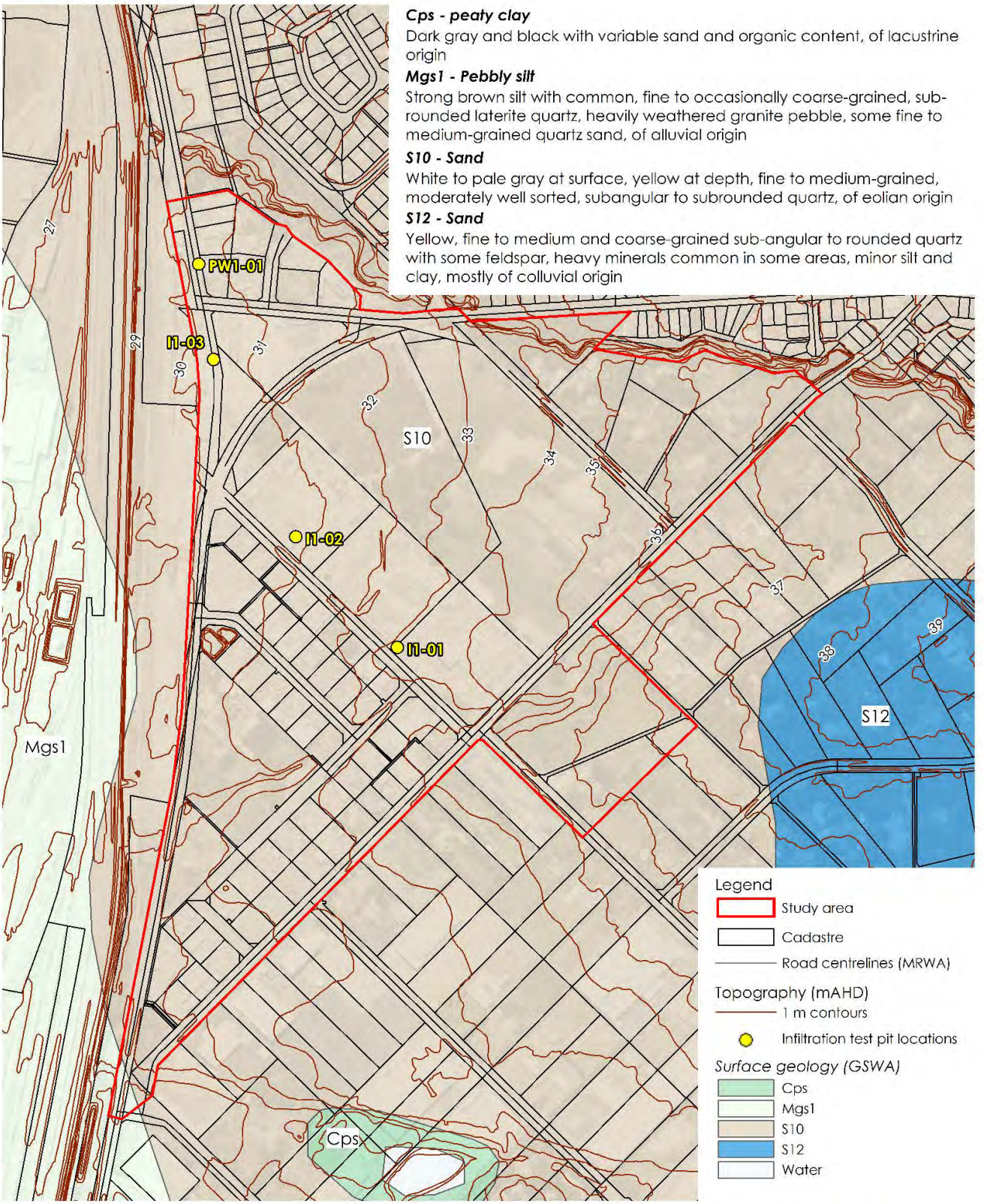
The DWMS recorded borehole logs for bores located in the northern portion of the Site Area installed in 2011. Depths to clayey sand at each borehole located within the Site Area boundaries are as follows:

- MW1: Clayey Sand from 3 mbGL to the total bore depth at 11 mbGL.
- MW7: Clayey Sand from 2.5 mbGL to the total bore depth at 13 mbGL
- MW8: Clayey Sand from 3.5 mbGL to the total bore depth at 9 mbGL
- MW9: Clayey Sand from 2.25 mbGL to the total bore depth at 5.5 mbGL

Figure 7 outlines the depth to the impermeable layers across the site, with interpolated mapping of the depths taken from the LWMS (Strategen, 2020). Further investigation will be undertaken for future UWMPs, including hydraulic conductivity.

Borehole logs have been provided in Appendix 3.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 6 - Soils and topography



Cps - peaty clay
Dark gray and black with variable sand and organic content, of lacustrine origin

Mgs1 - Pebbly silt
Strong brown silt with common, fine to occasionally coarse-grained, sub-rounded laterite quartz, heavily weathered granite pebble, some fine to medium-grained quartz sand, of alluvial origin

S10 - Sand
White to pale gray at surface, yellow at depth, fine to medium-grained, moderately well sorted, subangular to subrounded quartz, of eolian origin

S12 - Sand
Yellow, fine to medium and coarse-grained sub-angular to rounded quartz with some feldspar, heavy minerals common in some areas, minor silt and clay, mostly of colluvial origin

Legend
- Study area
- Cadastre
- Road centrelines (MRWA)

Topography (mAHD)
- 1 m contours
- Infiltration test pit locations

Surface geology (GSWA)
- Cps
- Mgs1
- S10
- S12
- Water

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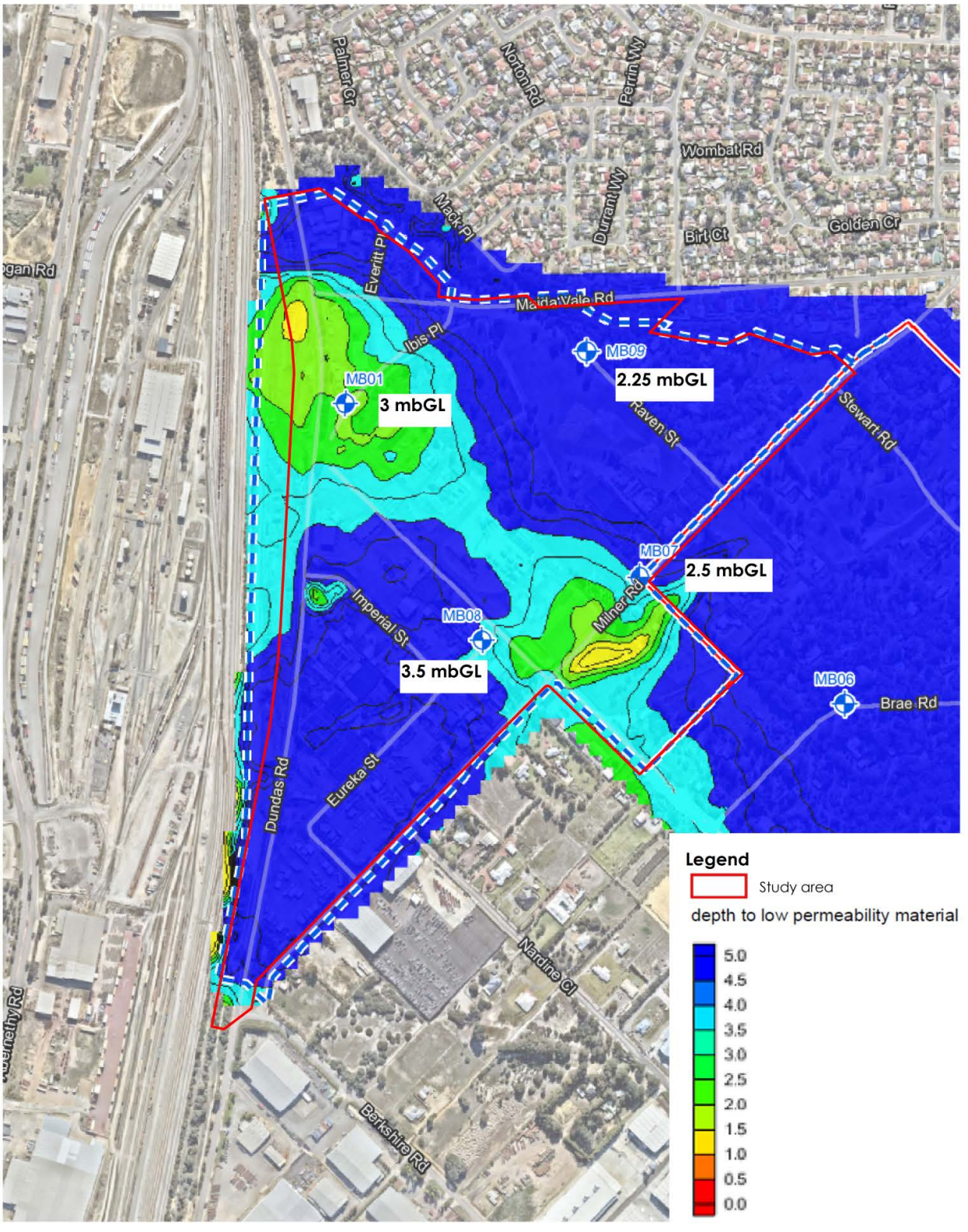
Figure 7 - Depth to low permeability material



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4.3.3 Acid sulfate soils

A review of the Department of Water and Environment Regulation Acid Sulfate Soils (ASS) risk mapping (DWER, 2020a) identifies the Study Area as having low to medium risk of ASS occurring within 3 m of the surface.

4.4 Groundwater resources

Determining the groundwater depth, quality and availability are crucial for forming total cycle water management strategies. These components are outlined below.

4.4.1 Public Drinking Water Source Area

There are no Public Drinking Water Source Areas within the Study Area.

4.4.2 Groundwater allocation

The Study Area is located within the Shire of Kalamunda groundwater management subarea. The Department of Water and Environmental Regulation's Water Register (DWER, 2020b) shows that there is limited groundwater available for allocation from the Superficial Aquifer within the Study Area. An Aquifer Allocation Report was requested from DWER in December 2020 and the details are shown in Table 3.

Table 3: Groundwater resource allocation availability (as of December 2020)

Management Area	Management Sub Area	Resource	Allocation Limit	Allocated Volume	Remaining Volume
Perth	Shire of Kalamunda	Superficial Swan	2,986,500 kL	2,891,178 kL	89,222 kL

There is currently a combination of temporary and standard licences within the Study Area. The temporary licences are associated with construction of the Airport rail line (160,000 kL/yr). The standard licences are summarised in Table 4.

Table 4: Existing groundwater licences within the Study Area

Licence Number	Licence Allocation	Groundwater Area	Groundwater Subarea	Issue Date	Expiry Date
GWL63807	89,390kL	Perth	Shire of Kalamunda	25/11/2019	25/11/2029
GWL152215	23,120kL	Perth	Shire of Kalamunda	23/10/2013	22/10/2023

4.4.3 Groundwater Levels

Site Specific Data

As outlined in the above sections, the presence of low permeability material (clayey sand) may result in perched groundwater at levels of 2.25+mbgl in parts of the site. Groundwater monitoring bores were installed by Strategen in 2011 prior to preparation of the DWMS for the

City of Kalamunda. Monitoring was undertaken for 15 months including two winters between September 2011 and December 2012, with two peak periods recorded in October 2011 and in September 2012.

Maximum Groundwater Levels (MGLs) varied between approximately 23 (MB7) and 33mAHD (MB9) (Figure 8). This equated to a depth to groundwater of approximately 0.5m to 10mbgl (Figure 9). The complete monitoring data set is provided in Appendix 4.

Perched Groundwater Assessment

A desktop review of geotechnical, groundwater and surface water data across the Site Area was conducted by Strategen in 2016. This review assessed the presence of perched groundwater within the Site Area, and specifically the Poison Gully Creek TEC and Ibis Place TEC areas. The results indicated if perched water were present, this would not be permanent.

There was no evidence of sub-surface seepage into Poison Gully from perched groundwater, even in areas where the Creek is deeply incised. The review concluded that any perched groundwater which may occur is likely to be ephemeral and limited to isolated areas, however further monitoring was proposed to assess seasonal behaviour and localised perched groundwater.

Data from the review was used to inform Strategen's calculation of perched MGLs at all DWMS bores as part of the Forrestfield North Residential Precinct LWMS (Strategen, 2020) investigations, including the bores located on this site MB1, MB7, MB8, and MB9. The desktop review has been provided in Appendix 4, with calculated MGLs discussed below and presented in Table 6. Perched groundwater was noted near MB9 as shown in Table 7.

Correlation to Historical MGLs

Rainfall in 2011 was approximately 10 % greater than the long-term average, indicating that it was a wet year and representative of higher groundwater conditions. Rainfall from September to December was considerably more than long term averages (Table 5), providing confidence that the measured levels were representative of recent maximum conditions.

Table 5: 2011 Rainfall Totals

Month	Long-term Average (mm)	2011 Rainfall Total (mm)	% Change
January	10.9	43.2	294.8%
February	15.0	0.4	-97.3%
March	16.2	0.0	-100.0%
April	39.7	26.2	-34.0%
May	97.6	58.6	-39.9%
June	155.1	143.2	-7.7%
July	154.2	164.6	6.7%
August	118.7	127.8	7.7%
September	72.3	102.4	41.6%
October	42.6	63.4	48.8%
November	25.4	38.6	51.8%

December	11.2	67.4	504.3%

Typically, maximum groundwater levels are determined through a comparison of onsite measurement with nearby long-term DWER monitoring bores. Comparisons were made to historic groundwater levels from the nearest DWER bore 61610508 and other available historical groundwater level data to provide an estimate of short-term perched MGL following high rainfall periods as part of the Forrestfield North Residential Precinct LWMS (Strategen, 2020) investigations.

The groundwater levels for 2011 and 2012 were compared to the groundwater levels at DWER bore (61610508) for an assessment of the perching that occurs where the clay layer is known to be present west of Milner Rd (and predominantly within the Site Area) (Strategen, 2020). The estimated maximum groundwater levels for each bore location have been provided in Table 6. The bores within this Site Area have been highlighted.

Figure 8 and Figure 9 display the groundwater level contours, and depth to groundwater across the site (Strategen, 2020). This indicates groundwater is shallow around MB1 and MB9, at or within 1.0m of the surface. A localised area of shallow groundwater is also located along Imperial Street.

Table 6: Groundwater levels (Strategen, 2020)

Bore	Easting	Northing	Ground level (mAHD)	Screened depth (mbgl)	2011 Peak (mbgl)	2011 Peak (mAHD)	2012 Peak (mAHD)	Difference Between 2011 and 2012	Calculated max groundwater level (mAHD)
MB01	404861	6464332	30.04	6 - 9	3.79	26.427	24.690	-1.737	30.027
MB02	405922	6464054	38.45	14 - 18	14.22	24.235	23.990	- 0.245	31.235
MB04	406211	6463643	39.73	15 - 19	15.73	23.876	23.800	- 0.076	30.876
MB05	406081	6463047	40.77	15 - 19	16.99	23.776	23.570	- 0.206	30.776
MB06	405683	6463843	36.24	13 - 16	11.92	24.319	23.970	- 0.349	31.319
MB07	405347	6464050	34.49	9 - 13	9.61	24.876	24.230	- 0.646	31.876
MB08	405086	6463948	31.84	6 - 9	3.97	27.873	25.750	- 2.123	30.873
MB09	405255	6464415	33.36	2 - 5.5	0.95	32.411	28.350	- 4.061	33.411

Table 7: Rainfall, DWER Bore (61610508) and groundwater levels data summary (Strategen, 2020)

Month	MGL 2011	MGL 2012	Difference (m)	Comment
DoW bore (61610508)	~ 14.5 mAHD	~ 14 mAHD	~ 0.5	2017 (Oct) levels ~15.2 mAHD (0.7 m higher than 2011, 1.2 m higher than 2012).
Rainfall (Jan-Sept)	666.4 mm	526 mm	0.14	Higher rainfall for Jan-Sept 2011.
Rainfall (Oct)	63.4 mm	13.8 mm	0.496	High Oct 2011 rainfall periods immediately prior to groundwater level measurement.
Oct groundwater levels East of Milner Rd	23.776 - 24.319 mAHD	23.570 - 23.990 mAHD	0.076 - 0.349 m	no significant perching is evident east of Milner Rd (absence of clay layer).
Oct groundwater levels West of Milner Rd	26.427 - 32.411 mAHD	24.690 - 28.350 mAHD	1.737 - 4.061 m	Significant perching to the northwest near MB09 due to clay layer west of Milner Rd.

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Figure 8 - Groundwater levels

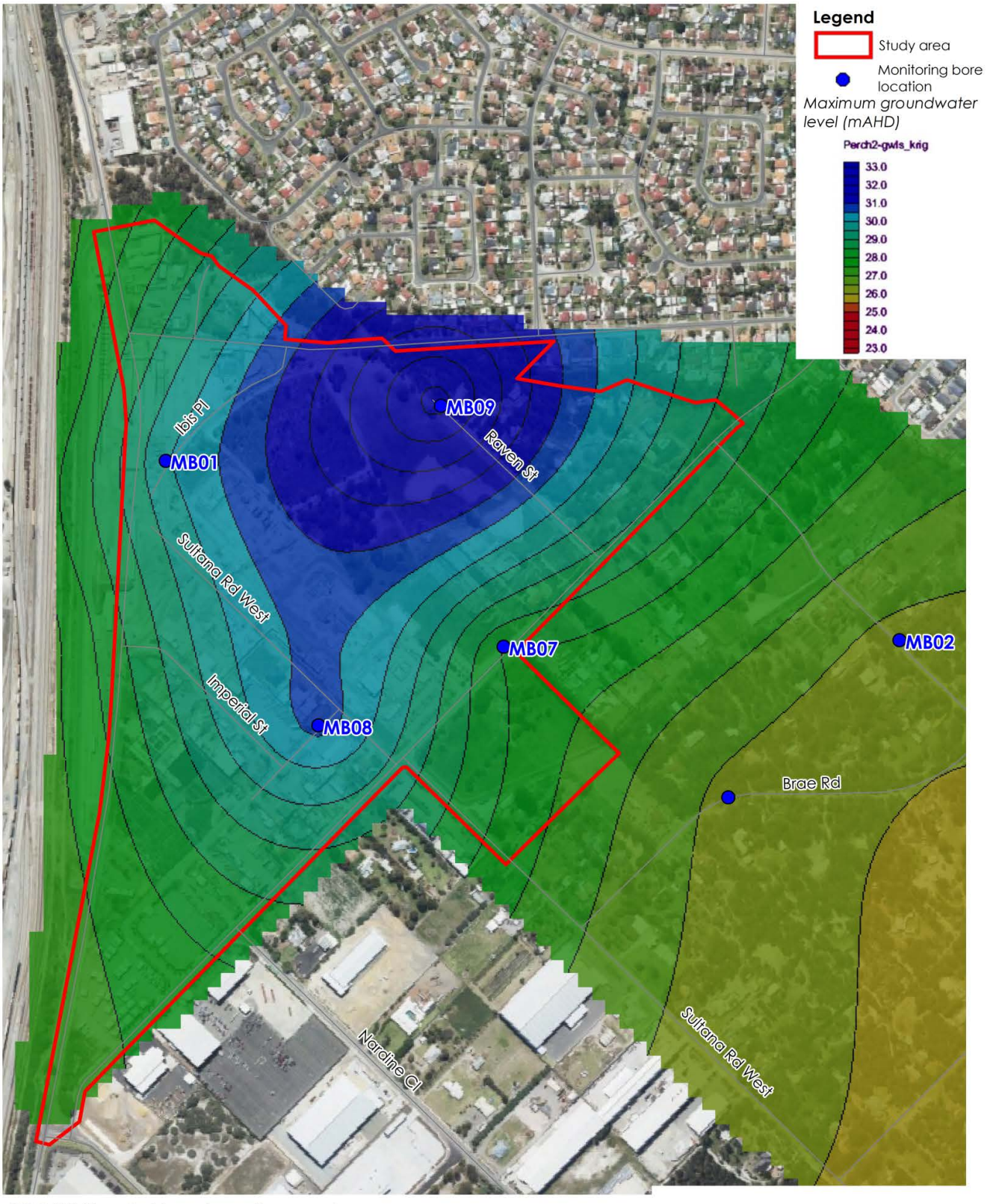

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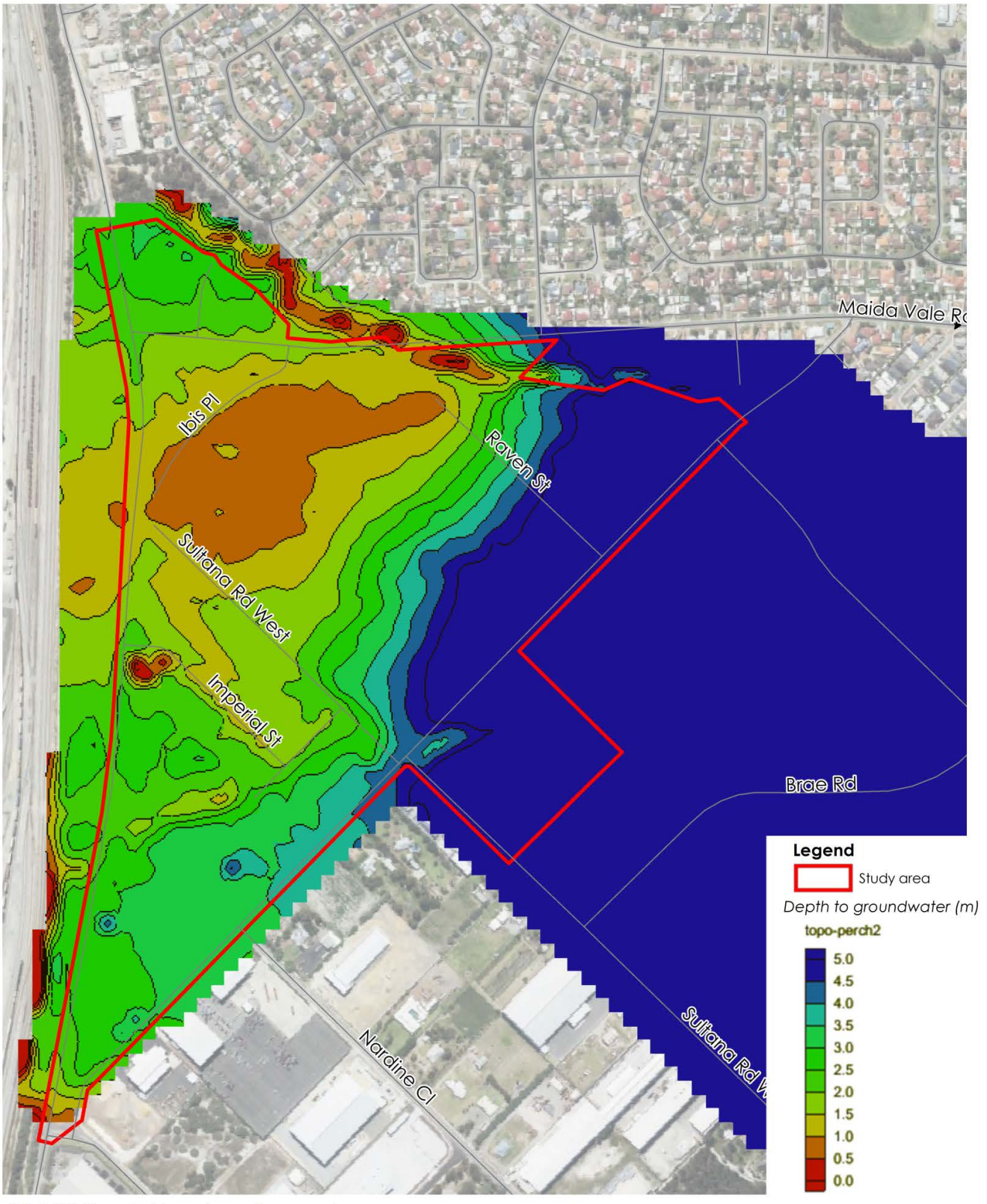
Figure 9 - Depth to groundwater



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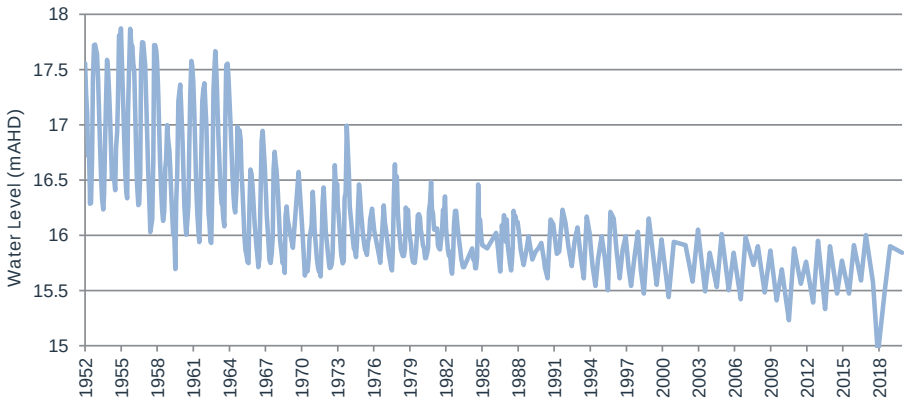


Figure 10: Groundwater levels in DWER Bore 675B

4.4.4 Groundwater Quality

Groundwater quality monitoring was undertaken by Strategen between September 2011 and December 2012. A summary of the key results is provided in Table 8, and all results are provided in Appendix 4.

Table 8: Groundwater quality monitoring results

Parameter	Results	MB01	MB07	MB08	MB09
pH	Median	5.97	6.18	6.09	6.35
	Range	5.68 – 6.62	5.84 – 6.34	5.83 – 6.58	6.02 – 6.68
Electrical Conductivity	Median	0.25	0.25	0.70	0.18
	Range	0.23 – 0.45	0.14 – 0.31	0.01 – 0.92	0.17 – 0.19
Dissolved Oxygen (%)	Median	32	69	76	53
	Range	26 – 87	49 – 134	73 – 271	44 – 62
Total Nitrogen (mg/L)	Median	1.80	8.00	21.50	1.30*
	Range	0.93 – 5.90	7.10 – 10.00	18.00 – 25.00	-
Total Phosphorus (mg/L)	Median	0.61	0.77	0.62	0.08*
	Range	0.21 – 1.70	0.20 – 1.60	0.18 – 1.20	-

** only one sample collected at MB09*

Exceeds Swan Canning WQIP Short term limits or Long-term limits (Swan River Trust, 2009)

Exceeds Australian and NZ Guidelines for Fresh and Marine Water Quality (ANZG, 2018)

Consistent with the former rural and agricultural land uses on the site (Section 2.1) nutrients generally exceed short- and long-term limits in the Swan Canning Water Quality Improvement Plan (Swan River Trust, 2009). Limited sampling of metals indicates concentrations of Copper, Chromium, Nickel, Lead and Zinc that exceed Australian and NZ Guidelines for Fresh and Marine Water Quality guidelines (ANZG, 2018). Results for all sampling is provided in Appendix 4 with reference to relevant guidelines. Groundwater mobilised through development (e.g. subsoil drainage) will require treatment prior to discharge to downstream waterways and wetlands.

4.5 Surface water resources

Existing surface water features within and adjacent to the Study Area will require protection from redevelopment or provide opportunities for modification to deliver the community an asset with social and ecological benefits. The surface water features are outlined below.

4.5.1 Natural water resources

Within and adjacent to the Study Area are two significant water features; Poison Gully Creek and a resource enhance wetland south of Maida Vale Road. Poison Gully Creek is an ephemeral creek that flows on the northern boundary of the Study Area and originates in Gooseberry Hill. Stormwater from the northern portion of the Study Area and residential areas in Maida Vale discharge to the creek. Downstream, Poison Gully Creek discharges into airport land via culverts under Dundas and Abernathy Roads. As shown in Figure 11, the creek eventually flows to Limestone Creek and the Swan River.

The capacity of Poison Gully Creek was assessed in the *Limestone Creek (Perth Airport Northern Main Drain) Stage 1 Capacity Review* (Water Corporation, 2010), particularly with consideration of the future development within the catchment. A hydraulic and hydrological model was established for the catchment to simulate the 10 year and 100-year ARI events. Results of the modelling indicated that there are no flooding concerns, either in the existing (2010) system or the ultimate development near the Study Area.

The hydrological model established by Water Corporation accounted for the runoff from sub-catchments in the Study Area: la_po166 and La_po016 (sub-catchments boundaries are provided in Appendix 5). Advice from Water Corporation for the Forrestfield North Residential Precinct is that post-development flows into Poison Gully Creek should not exceed pre-development flow. This is discussed further in Section 5.3. Proposed parameters for the residential precinct are provided in Table 9 and Table 10.

Table 9: Hydrological pervious and roughness parameters (Strategen, 2020)

Land Use	% Impervious	Manning's n Pervious	Manning's n Impervious
Urban roads	80%	0.04	0.025
R5-R12 lots	10%	0.05	0.025
R60/R80 lots*	90%	0.04	0.025
Rural lots	5%	0.04	0.025
Industrial	90%	0.04	0.025
POS	0%	0.04	0.025

**Initial 16 mm managed on lot, depending on soil conditions*

Table 10: Hydrological loss parameters (Strategen, 2020)

Soil Type	Initial	Continuing	Comment
Impervious	1.5 mm	0 mm/hr	
BoM – Forrestfield	26.0 mm	6 mm/hr	
Rural road impervious	16.0 mm	6 mm/hr	Orange sandy surface soils
Silty sand	16.0 mm	6 mm/hr	

Soil Type	Initial	Continuing	Comment
Guildford	10.0 mm	3 mm/hr	With some vegetation

The assumed parameters for pre-development within the Perth Airport North catchment were the Silty Sand conditions. These parameters have been adopted for the pre-development modelling outlined in Section 4.5.3, along with a description of the wetland south of Maida Vale road.

Poison Gully Creek has significant Aboriginal Heritage significance that is discussed further in Section 4.6. The foreshore area south of Maida Vale Road was previously defined in the DWMS (Strategen, 2015) which noted:

- The creek is deeply incised, with steep banks;
- Interpreted flood widths vary between 10 m and 40 m for the 100-year ARI event;
- Owing to the steep banks, typical wetland vegetation such as sedges and rushes was not abundant;
- Vegetation condition varied between completely degraded and good to very good, with the best vegetation located near Maida Vale Road; and,
- This area is recognised as a Bush Forever site (Section 4.6).

4.5.2 Existing Drainage Infrastructure

Drainage infrastructure through the site is a combination of formal and informal systems. Existing pit and pipe drainage is identified on Figure 11 and is generally restricted to Maida Vale and Dundas Roads in the north (discharging to Poison Gully Creek), and Eureka Street and Milner Road in the south, discharging to an existing drainage sump. This southern sump (between Milner Road and Dundas Road) and another sump on the corner of Imperial Street and Dundas Road are fenced-off systems with no amenity value. These systems ultimately discharge to the Perth Airport Drain (Airport South Catchment).

Minor roads feature roadside drains that are generally less than 500 mm deep and do not contain maintained vegetation. These drains convey runoff to Poison Gully Creek, the wetland or the existing sumps.

4.5.3 Wetlands

The site contains a Resource Enhancement wetland that is likely to be upgraded to a Conservation Category wetland (UFI: 15875) on Lot 12 Ibis Place, on the corner of Ibis Place and Maida Vale Road. The vegetation within the wetland has been assessed on numerous occasions. The most recent assessment by PGV Environmental confirmed the presence of a Threatened Ecological Community (TEC), which is why the wetland will likely be upgraded to and managed as a Conservation Category Wetland (CCW).

Wetland Vegetation

The vegetation on the main part of the wetland on Lot 12 is described generally as scattered Marri (*Corymbia calophylla*) trees over a low heath with common species being *Xanthorrhoea preissii, Verticordia densiflora, Stirlingia latifolia* and *Hypocalymma angustifolium*. The species composition of the vegetation, and absence of Paperbark and wetland sedge species, is consistent with the mapping of the wetland as a Dampland, which is a waterlogged basin that does not have above-ground water at any time of the year. The vegetation on the small extension of the wetland on Lot 9 is also considered the same TEC and contains Sheoak trees (*Allocasuarina fraseriana*) over a low heath understorey.

Wetland Buffers

The Environmental Protection Authority (EPA) indicate in Guidance Statement 33 *Environmental Guidance for Planning and Development* that 50m buffers should be applied to Conservation Category wetlands (EPA, 2008). Therefore, consideration has been given to include a minimum 50m buffer to the vegetation on the site (with the Local Open Space (LOS) providing a buffer greater than 50m). However, some existing and proposed land uses will make provision of a 50m buffer not possible on two of the boundaries (PGV Environmental, 2021).

Development of the High Wycombe train station and associated infrastructure, including a Western Power transformer site, is already in place to the north-west of the wetland. Accordingly, no buffer is possible to provide on most of the north-west boundary. The northern tip of the site will have a small buffer to existing Maida Vale Road (PGV Environmental, 2021).

A Community Hub is proposed to be constructed to the south-west of the wetland. The Community Hub will replace the existing industrial works that are currently on the site. No buffer is able to be provided to the south-west side unless the current concreted hardstand is dug up. This is considered impractical. The Community Hub is likely to include a library, public pool and carparking which will have a car parking component and will be sited along the boundary of the site with the buildings and pool located further away.

Urban development is proposed on the north-east side of the wetland. A 50m buffer will be provided on the north-east boundary. This area is currently cleared rural land with a few trees at the northern end. As the passive recreational needs of the community will be provided in the LOS on the south-eastern side, the north-east buffer does not need to include any additional areas for public use (PGV Environmental, 2021). It is therefore proposed to revegetate the buffer in this area. The form of the revegetation will need to consider any adverse impacts of revegetation on the bushfire hazard rating for the proposed urban development that will be constructed adjacent to the buffer (PGV Environmental, 2021).

The area to the south-east of the wetland will be 'Local Open Space' (LOS). The LOS is proposed to include grassland areas for recreational purposes as well as areas containing areas for passive recreation. In effect the buffer to the bushland on the south-east side will be well over 50m, extending up to 150m to the nearest area of urban development (PGV Environmental, 2021).

The buffer will not cater for stormwater drainage features such as infiltration and detention basins (See Section 5.3).

Wetland Hydrology

The hydrology of the wetland contrasts with the regional groundwater information, however local groundwater data (Section 4.4.3) demonstrates the presence of shallow, perched groundwater in the area around the wetland. Within the wetland and immediately to the east, the depth to groundwater is less than 1.0 m indicating that perched groundwater may result in seasonal water logging or inundation of the wetland (Figure 9).

Runoff from surrounding areas also contributes to the wetland hydrology and vegetation. A local catchment, determined from the available topographical data, is shown in Figure 12 indicating that approximately 5.5 ha drains into the wetland system. This catchment consists of cleared rural land and remnant vegetation.

Predevelopment Wetland Modelling

A hydrological model (XP-Storm) was established to determine the frequency and volume of water that discharges from this catchment into the wetland. Loss parameters for the model are based on Water Corporation suggestions for the residential precinct (Table 10). These parameters don't consider the relatively shallow depth to groundwater within the wetland catchment (Figure 9). To determine inflows, the catchment was separated into areas of deep and shallow groundwater, with different loss parameters (Type I and Type II respectively in Table 11). The estimated volumes of runoff entering the wetland for typical design storm events are provided in Table 12.

Table 11: Wetland catchment loss parameters (Strategen, 2020)

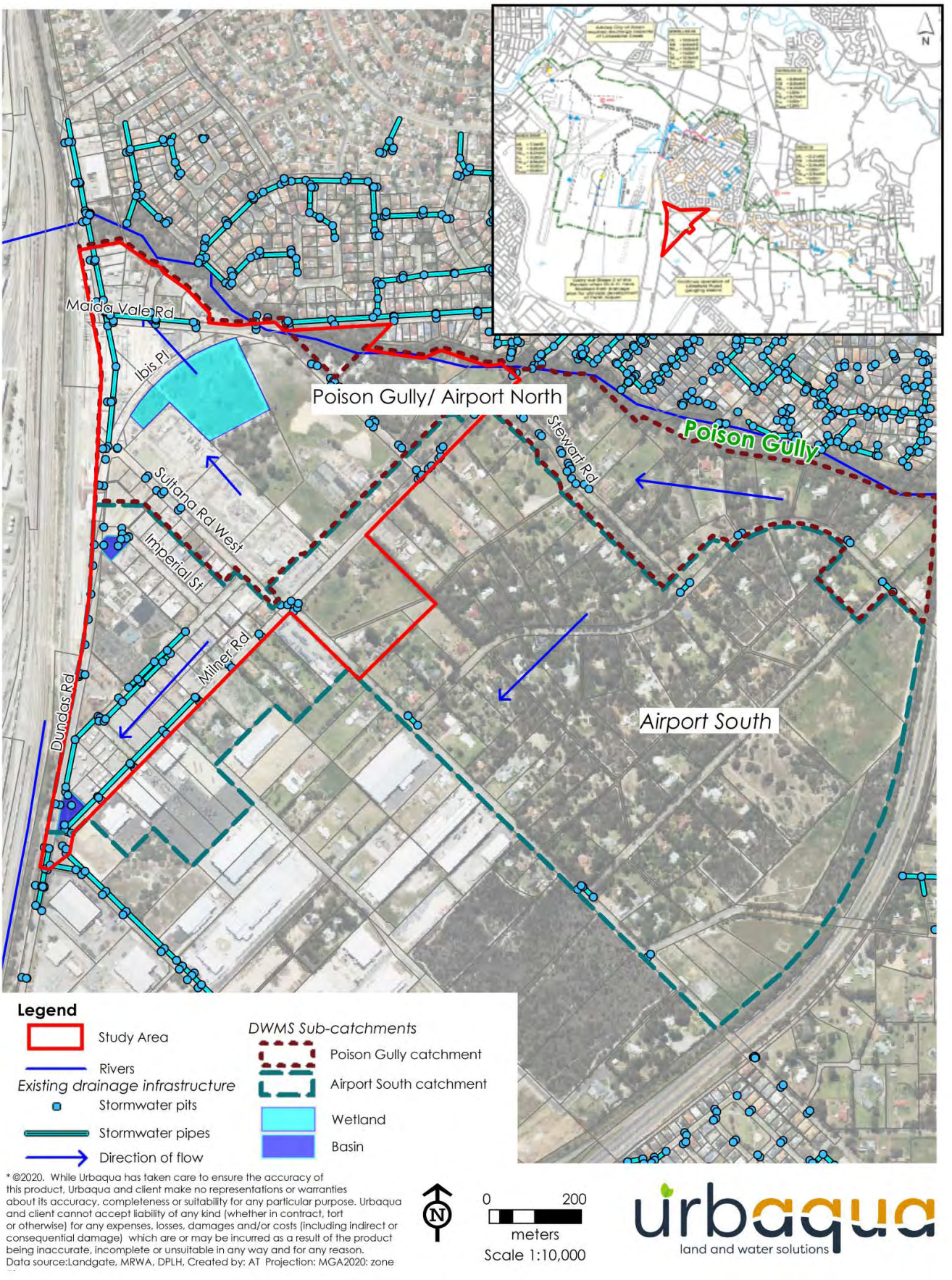
Soil Type	Groundwater Depth	Initial	Continuing	Type
Silty Sand	Deep	16 mm	6 mm/hr	I
Guildford with some vegetation	Shallow	10 mm	3 mm/hr	II

Table 12: Runoff into wetland from local catchment

Duration	63.2% AEP (1yr ARI)	20% AEP (5yr ARI)	10% AEP (10yr ARI)	1% AEP (100yr ARI)
1 hr	374 m^3	489 m^3	519 m^3	752 m^3
3 hr	456 m^3	638 m^3	745 m^3	1,250 m^3
6 hr	389 m^3	711 m^3	868 m^3	1,513 m^3
9 hr	466 m^3	666 m^3	915 m^3	1,651 m^3
12 hr	485 m^3	816 m^3	869 m^3	1,731 m^3
18 hr	469 m^3	738 m^3	944 m^3	1,858 m^3
24 hr	355 m^3	742 m^3	841 m^3	1,790 m^3
48 hr	205 m^3	223 m^3	460 m^3	908 m^3
72 hr	36 m^3	147 m^3	305 m^3	490 m^3

The modelling results suggest that regular runoff occurs into the wetland for all storm events. The shallow groundwater and impermeable material favours runoff from the local catchment and runoff occur for higher intensity, shorter duration events and lower intensity, higher duration. The peak runoff into the catchment occurs in the 1% AEP 18-hour event (1,858 m^3). Directing a portion of the stormwater runoff from the development into the wetland is therefore considered appropriate to maintain the hydrology and associated ecology.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 11 - Surface water features



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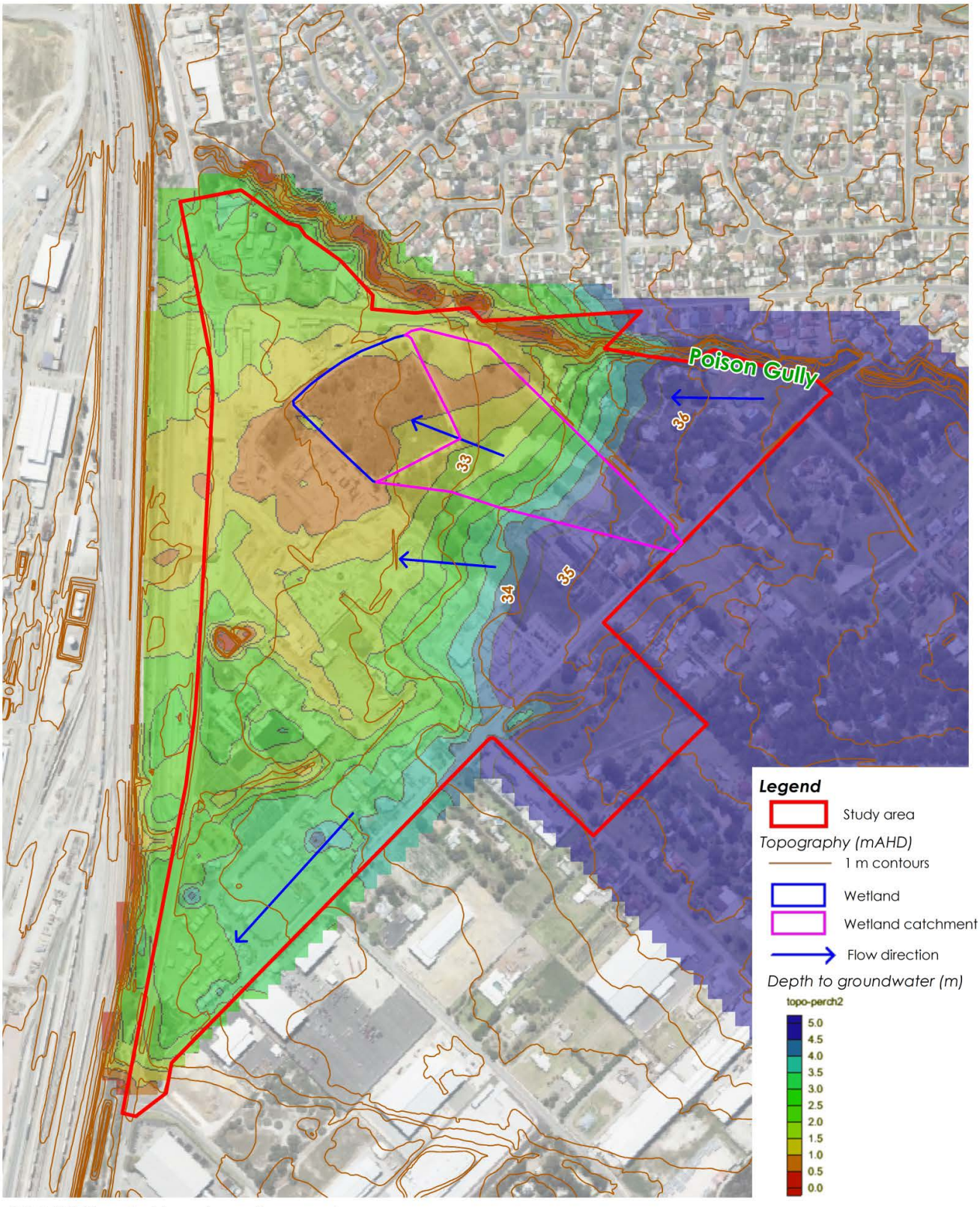
DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 12 - Wetland catchment



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4.6 Environmental and cultural

A search of the DWER Contaminated Sites Database (DWER, 2020d) indicates that there are no contaminated sites located within the Study Area.

The Poison Gully Creek foreshore area and a portion of private land north of Raven Street are mapped as Bush Forever (Site No. 45) as shown in Figure 13. This Bush Forever Site is inferred to contain TEC 3a: *Corymbia calophylla* (marri) – *Kingia australis* woodlands on heavy soils, Swan Coastal Plain (Government of Western Australia, 2000). The Bush Forever site will be maintained as part of the planning process. There are also areas of scattered remnant vegetation that have been included in future open space. The future open space adjacent to the wetland is shown in Figure 3 and the scattered vegetation in that open space can be seen in Figure 2.

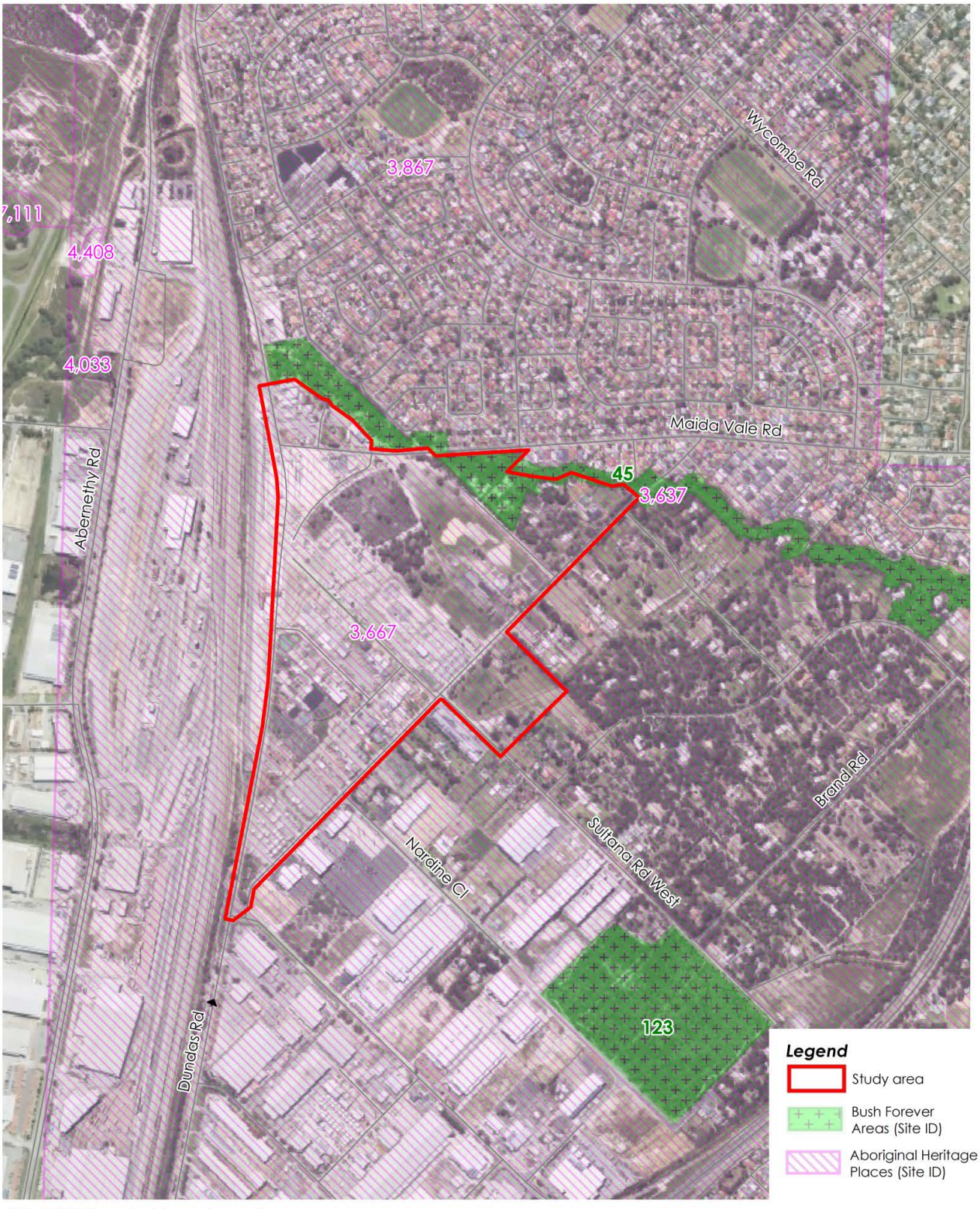
The Aboriginal Heritage Sites database (Department of Aboriginal Affairs, 2020) indicates that the Study Area is part of the Poison Gully Creek registered site (No. 25023) (Figure 13).

4.7 Summary

Based on the review of geological, hydrological and environmental information for the Study Area, the key considerations for water management are as follows:

- There is an existing wetland (REW) within the northern portion of the Study Area that features a TEC;
- Variable depth to groundwater across the site, with shallow perched groundwater identified surrounding the existing wetland;
- Poison Gully Creek flows on the northern boundary of the site and ultimately discharges to Limestone Creek and the Swan River;
- The southern portion of the site drains to fenced sumps that discharge to the Perth Airport Drain (Airport South Catchment);
- Drainage infrastructure through the site contains a combination of formal (sumps, pipes and pits) and informal (roadside drains);
- The existing wetland receives runoff from the local catchment;
- There is a Bush Forever site along Poison Gully Creek, inclusive of the foreshore area and pockets of remnant vegetation near the wetland; and,
- Groundwater monitoring onsite did not indicate any significant issues to prevent development.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 13 - Environmental and cultural features



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5 WATER MANAGEMENT STRATEGY

Water management strategies for the Study Area have been prepared in accordance with the guiding documents, policies and strategies (Section1), the intended development concept (Section 2) and the site considerations (Section 4). Strategies for water sustainability, stormwater, groundwater, and water quality improvement are outlined in this Section.

5.1 Water sustainability initiatives

The key objectives for water sustainability are:

- Maximise local infiltration to replenish surface groundwater aquifers; and,
- Provide alternative water sources for domestic irrigation and in-house domestic non-drinking water demands.

5.1.1 Water supply

The Study Area is located in an area served by the Water Corporation's integrated water supply scheme. All buildings will be connected reticulated drinking water distribution network.

There are approximately 4.97 ha of Public Open Space (POS) in the proposed design concept, including 2.03 ha of space that will not require ongoing irrigation (remnant vegetation). Based on the DWER standard irrigation rate of 7,500 kL/ha/yr approximately 22,065 kL/yr of allocation is required.

There is allocation available within the groundwater management sub-area, though current licences within the Study Area will become available following development. Transfer of the current licences would require negotiation with respective private licence holders. The City current holds a licence (GWL No. 158077) for irrigation of POS through the wider groundwater management sub-area for 936,725 kL/yr. The City has indicated a portion of this licence is available for irrigation within the Study Area and is the favoured source.

Confirmation of the final irrigation arrangement will be provided in the Urban Water Management Plan.

5.1.2 Wastewater treatment and disposal

The Study Area is located in an area served by the Water Corporation's integrated sewerage scheme and will be connected to a reticulated sewerage network.

5.1.3 Water conservation and efficiency measures

To reduce the consumption of scheme water newly constructed houses will be recommended to meet the Water Corporation's Waterwise homes and gardens criteria. That is:

- All showerheads installed will be better than the minimum WELS 3 Star rating;
- All taps installed will be better than the minimum WELS 4 Star rating;
- All toilets will be duel flush and exceed the minimum WELS 4 Star rating; and
- All water using appliances installed are rated WELS 4 Star or above.

Considerable savings can be achieved in potable water demand by using alternative sources for non-potable demands such as toilets, washing machines and irrigation. The construction of new commercial and residential buildings provides the opportunity to integrate these systems during design and maximise the potential for efficient use of potable water.

Proposed commercial and residential buildings should be required to harvest roof runoff with plumbing to toilets and washing machines. The harvesting of roof runoff can contribute to a reduction in stormwater runoff downstream. The design and performance of these systems would vary with each building and the City should provide guidance on volume and water saving benefits to encourage uptake. However, it is not possible to estimate the level of uptake for stormwater harvesting at this stage of planning and design, therefore, stormwater modelling for this LWMS does not assume that harvesting of roof runoff will occur and is therefore conservative in this regard.

The majority of the saving in potable water is the use of non-potable sources for irrigation of gardens and open space. At the single lot/dwelling scale these systems can be unfeasible, however with higher density and larger open space areas these approaches become more effective.

Greywater is wastewater from showers, basins and taps from bathrooms and laundry that can be treated and used as a non-potable source either for irrigation of gardens and open space or toilet flushing in commercial or residential buildings. In the proposed built form in the Study Area, greywater collected from residential, commercial and mixed used buildings can be treated and stored within basements and used for irrigation of open space. Designing and implementing these systems during the initial building design prevents retrofitting which is typically unfeasible.

Greywater should also be considered for the irrigation of green roofs and green walls to assist in reducing urban heat. These systems reduce heat by providing shade and evapotranspiration, and green roofs can also reduce stormwater runoff.

To further reduce irrigation requirements, frequent runoff from building carparks and hardstand areas should be directed towards gardens, tree pits or open space, or collected and used for this irrigation. This approach can also be used in carparks for passive watering of trees and to prevent ongoing watering. The Activity Hub presents an opportunity to demonstrate efficient non-potable water use to the community.

5.2 Groundwater management

The key objectives for groundwater management are:

- Provide appropriate separation between finished lot levels and groundwater to provide the expected level of amenity;
- Ensure soakwells devices are designed above maximum groundwater levels; and,
- Ensure that any subsoil drainage is treated prior to discharge to waterways and sensitive environments.

The following planning measures are adopted to achieve the above objectives:

- Subsoil drainage and imported fill to ensure adequate separation to groundwater;
- Ensure infiltration of stormwater runoff, consistent with existing conditions; and,
- Use of bio-retention areas within raingardens, tree pits and swales to improve groundwater quality compared with the existing conditions.

5.2.1 Subsoil Drainage

As discussed in Section 4.4.3, groundwater near the wetland portions of the Study Area is shallow and generally assumed to be at or within 1 m of the surface for the design of the groundwater management system (Figure 9). Given the depth to groundwater, subsoil drainage (slotted PVC wrapped in geo-fabric) will be installed at the maximum groundwater levels (MGLs) to control groundwater. This drainage prevents groundwater from rising uncontrolled through imported fill material and avoids the potential for water logging on residential lots or within road reserves.

Extensive use of subsoil drainage is not anticipated across the site and will be limited to areas of shallow groundwater. Subsoils will be located as follows;

- In areas where subsoil is required, the subsoil drainage will be located within the road reserve.
- Subsoil drainage will be excluded from wetland and buffer areas and should be located on opposite verges from wetland buffers where possible.
- Elsewhere, localised subsoil drainage for retaining walls and roads will be considered during the construction phase to alleviate any problematic soil moisture.
    - Where these conditions are identified short length subsoil drains will be installed.

To be cost effective and free draining, subsoil drains will be connected to the nearest road drainage system, with water quality treatment provided downstream. The minimum separation criteria to stormwater infrastructure are provided in Table 13.

Table 13: Groundwater separation

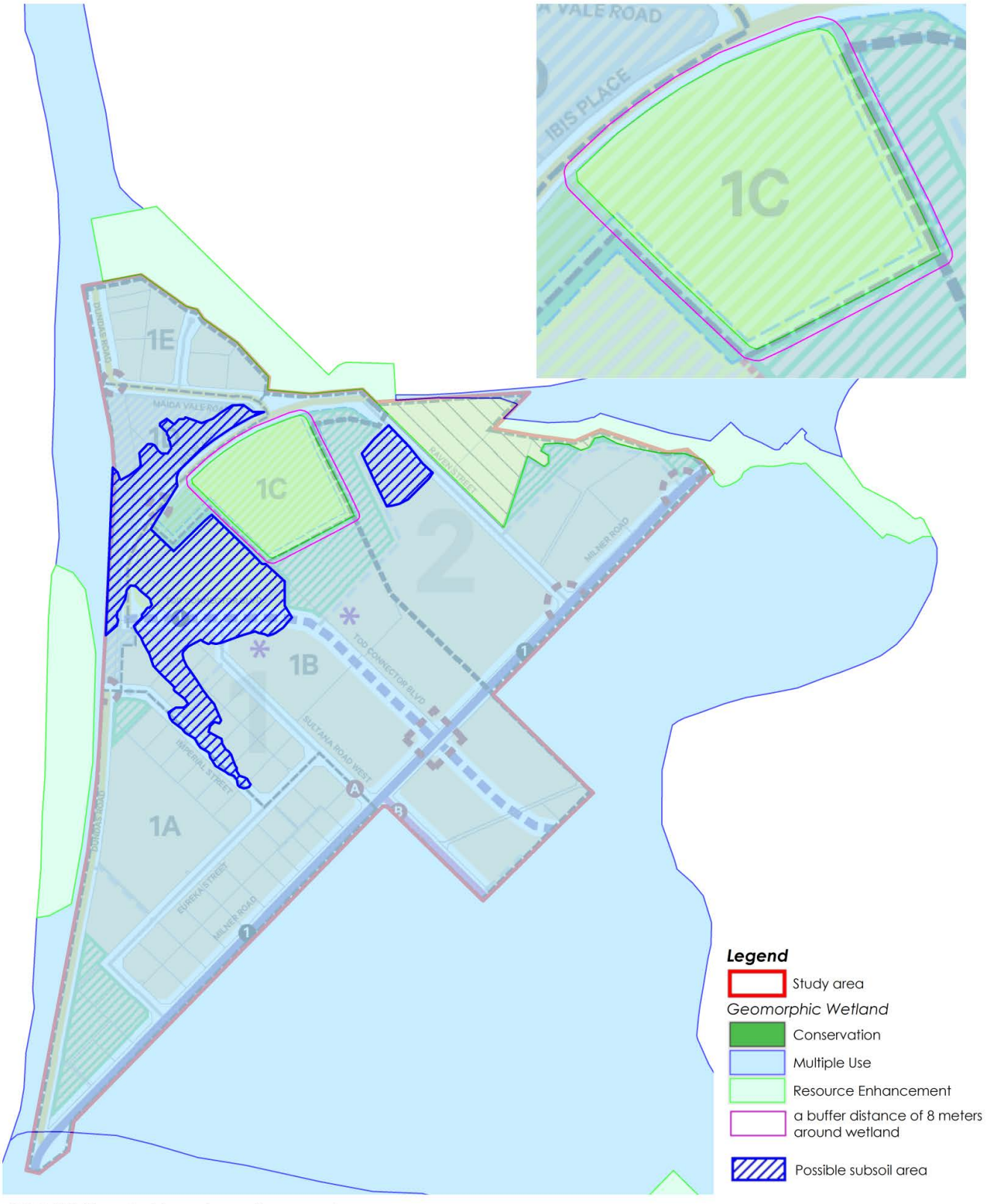
Location	Specification	Comment
Soakwells	500 mm	From base of soakwell to MGL
Stormwater infiltration	300 mm	From invert of vegetated systems such as raingardens and swales to MGL

Further modelling of subsoil spacing will be undertaken following confirmation of internal road layouts in the proposed development areas. That level of detail is not required for the LWMS however further information and a map showing the possible/ likely areas that may require subsoil (ie. Shallower depths to groundwater) are presented in Figure 14.

BUWM guidelines (WAPC, 2008) and the UWMP guidelines (DoW, 2012) require the following information to be supplied at UWMP stage with regards to subsoil drainage. This information will be provided in the UWMP;

- Subsoil criteria (locations, grade, outfalls, spacing).
- Outline subsoil drainage strategy, including avoidance and management of impacts on water dependent ecosystems and treatment of subsoil drainage water, prior to discharge to the surface water system. The subsoil drainage strategy should also address areas with nutrient-rich groundwater.
- Include subsoil drainage plan with invert levels at control points with likely lot levels.
- Permeability and compaction requirements for imported fill will be provided in the UWMP.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 14 - Geomorphic Wetland



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5.3 Stormwater management

5.3.1 Stormwater Management Objectives

The key objectives for surface water management on the Site are:

- Maintain or restore desirable environmental flows and/or hydrological cycles.
- For flood management, manage up to the 1% AEP (100-year ARI event) within the development area to pre-development flows and Water Corporation requirements (as per the Limestone Creek Capacity Review);
- Ensure that the first 15 mm of rainfall receives treatment prior to discharging to a receiving environment (DWER, 2017).
- Minor events should be managed to provide serviceability requirements which have been specified by the City of Kalamunda as:
  - 20%AEP for residential roads, and
  - 10%AEP for Maida Vale Road which is a district distributer road.
- Roads and public open spaces are to be designed to cater for the surface overflow for more severe storm events; and,
- Habitable floors at least 300 mm above the 1% AEP flood level in the local drainage system and 500 mm above the 1% AEP flood level in basins/sumps (where there is no overflow relief) and Poison Gully Creek.

Post development catchments are shown in Figure 15, with a breakdown of respective land uses in Table 14 below.

Table 14: Catchment area breakdown

Land use (ha)	Catchment 1 (North West)	Catchment 2 (North Central)	Catchment 3 (North East)	Catchment 4 (West)	Catchment 5 (South)
High Density Residential	0	3.48	3.56	0	0.51
Mixed Use / Activity Centre	0	2.23	0	0	0
Light Industrial	2.24	0.23	0	2.77	29
Road Reserve	1.21	2.48	0.75	0.8	7.67
POS / Wetland	0	6	0	0.25	1.89
Total Area (ha)	3.45	14.43	4.31	3.82	39.07

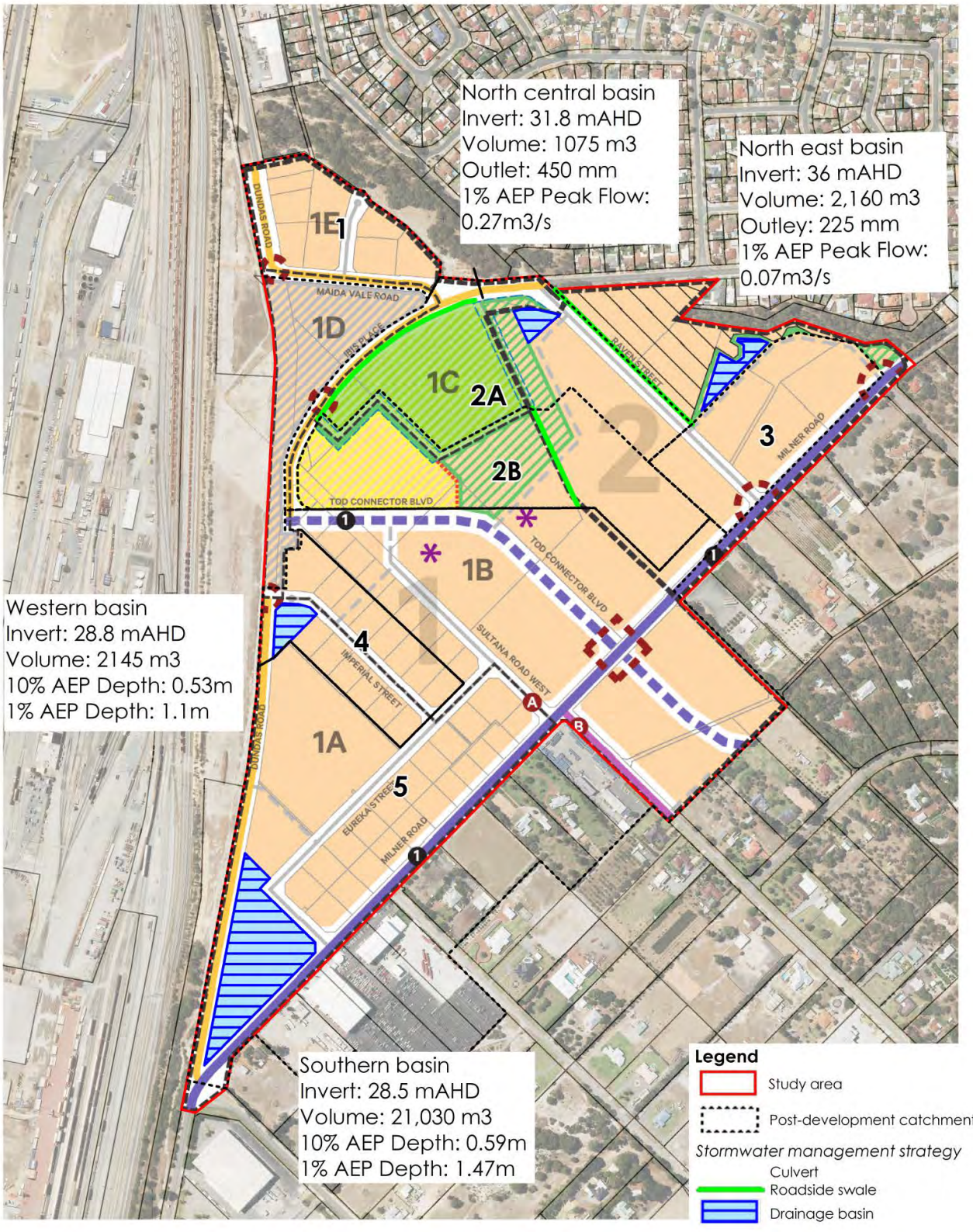
5.3.2 Small event management

Development within the Study Area will retain at least the first 15 mm of rainfall within lots and road reserves in accordance with the *Decision Process for Stormwater Management in WA* (DWER, 2017). Frequent events will be managed for the different lot typologies (indicated in Table 14 above) as per the following:

- Residential lots and strata properties to manage the 15 mm event through a combination of soakwells, rainwater tanks and raingardens. Residential development is proposed in Catchments 2, 3, and 5.
- Light Industry, mixed use and activity centres to retain at least the first 15 mm of rainfall within property boundaries. These developments should be encouraged to retain up to the 10% AEP 6 hr event (54.8 mm), using above ground or below ground storage for harvesting of roof runoff for internal re-use (toilets, washing machines) and irrigation of gardens. Light industrial development is proposed within catchments 1, 2, 4 and 5.
- Minor roads will manage the first 15 mm event through a combination of raingardens and swales; and,
- Major roads will manage the first 15 mm event using roadside and median swales. These are proposed for Dundas Road, Ibis Place, Raven Street, and the proposed along local open space within Catchment 2.

The first 15mm of rainfall will receive water quality treatment via the above drainage infrastructure mechanisms and is further discussed in Section 5.4.

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 15 - Stormwater management system



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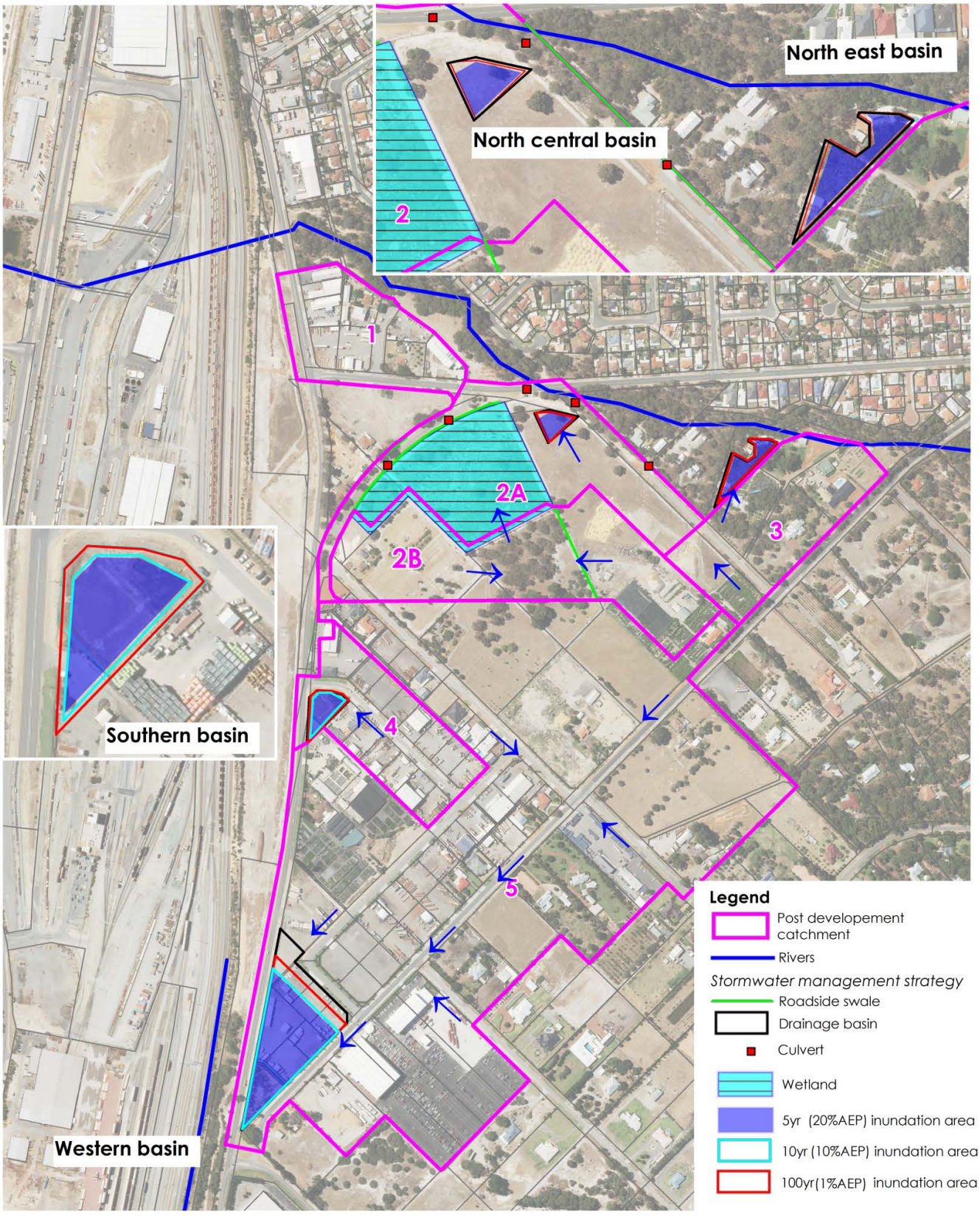
DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 16 - Stormwater event plan



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5.3.3 Minor event management

As detailed in Section 5.3.2, residential and industrial lots will infiltrate the small events onsite in soakwells. Runoff from residential and industrial lots above these events will enter the road drainage system. This runoff has been included in the hydrological modelling (Section 5.3.4) and basins have been sized accordingly to accommodate that runoff.

Conveyance System

Runoff from up to the 20% AEP (~5yr ARI) event which is in excess of the capacity of on-site lot retention systems (soakwells) will be conveyed through a combination of piped drainage and roadside swales into POS and drainage areas.

Surface conveyance (roadside swales) will be used wherever possible on major roads, with piped drains used only where swales cannot be reasonably accommodated in minor roads.

The use of soakage pits through any piped drainage system is recommended to further improve at-source infiltration through the site, particularly in the southern catchments (Catchments 4 and 5).

5.3.4 Major event management

Major flood runoff (1% AEP) will be conveyed via overland flow within the Study Area prior to discharge to drainage areas throughout the development. To size the various components of the system, XP-Storm modelling was completed and confirm discharge rates with respect to pre-development constraints. The requirement to maintain the wetland hydrology was also considered throughout modelling.

Basins and swales have been modelled with side slopes to allow for planting to provide water quality treatment functionality, as discussed further in Section 5.4

DWERs UNDO modelling has been undertaken to assess the water quality treatment effectiveness of the strategy. This is discussed further in Section 5.4.

XP Storm Modelling Method

The XP-Storm model has been developed to assess catchment flows and storages for the 63.2%, 20%, 10% and 1% AEP (approximately 1, 5, 10 and 100 years ARI) storm events based on temporal patterns and rainfall depths provided for the different storm events (BoM, 2019). The storm durations assessed for 63.2%, 20%, 10% and 1% AEPs was 1, 3, 6, 9, 12, 18, 24, 48 and 72 hours, including 10 temporal patterns for each storm event. Assumed losses for each land use are provided in Table 15. A description of each catchment is provided below.

Table 15: Post-development land use losses

Land use	% Impervious	Impervious IL	Impervious CL	Pervious IL	Pervious CL
Catchment 1 (North West)	87%	8	0	16	31.8
Catchment 2 (North Central)	52%	10	0	16	18.9
Catchment 3 (North East)	84%	10	0	16	26.7

Land use	% Impervious	Impervious IL	Impervious CL	Pervious IL	Pervious CL
Catchment 4 (West)	82%	10	0	16	34.8
Catchment 5 (South)	84%	10	0	16	35.8

IL: Initial Loss
CL: Continuing Loss

Catchment 1 – North West (3.45 ha)

Catchment 1 is currently developed (light industry) and is largely impervious. Minimal increases in the post-development runoff are anticipated. The proposed water management, both for minor/major roads upgrades and onsite retention within new light industry will reduce peak runoff and provide improved water quality protection. No additional drainage infrastructure is proposed for this area.

Catchment 2 – North Central (14.43 ha)

The proposed development in this catchment is anticipated to include high density residential areas, a community hub, public open space, and minor and major road reserves. Earthworks associated with the rail precinct and civil engineering design in the Study Area will alter the pre-development catchments.

Catchment 2 discharges to Poison Gully and provides runoff to the wetland. Post-development stormwater management therefore is required to meet the dual criteria of limiting discharge to the creek to pre-development rates and providing sufficient flows to the wetland to maintain the ecological function. The flows into the wetland have been established previously in Section 4.5.3, and is discussed in more detail in the section below.

Poison Gully

The pre-development discharge for Catchment 2 to Poison Gully has been modelled in XP-Storm. The total catchment is 19.1 ha, with approximately 3.1 ha of road reserve. Applying the recommended losses from Water Corporation (Strategen, 2018), a peak discharge rate of 0.27 m^3/s was adopted for the 1% AEP event to the Creek. This equates to 14 L/s/ha, which for comparison is lower than for catchments in the adjacent residential areas to the east (25 L/s/ha and 43 L/s/ha respectively). Erosion control measures are to be put in place at outflow points to prevent adverse impacts to Poison Gully during discharge.

The proposed development in this catchment is anticipated to include high density residential areas, a community hub, public open space, and minor and major road reserves. Earthworks associated with the rail precinct, and civil engineering design in the Study Area, will alter the pre-development catchments. The revised catchment discharging to the wetland is shown in Figure 16.

The opportunities for drainage within the development concept include POS and road reserves, as anticipated buffers for the wetland protection are not suitable for drainage infrastructure, particularly untreated runoff. Planted swales along major and minor roads provide suitable stormwater conveyance and attenuation of stormwater runoff. As shown in Figure 15, swales can be incorporated within the road reserves on non-active frontages for water quality treatment prior to runoff entering the wetland (generally located adjacent to open space, Bush Forever areas and wetlands).

Swales are assumed to be up to 5 m wide (1 m base width, 1:4 side slopes and 0.5 m deep), though a smaller footprint can be determined through detailed design and further site information. The swales will collect runoff from adjacent roads and development areas, providing water quality treatment (planting and amended soil media in some locations, discussed in Section 5.4) for at least the first 15 mm event. Small 375 mm culverts (as shown in Figure 15, allow for the detention of runoff in the swales prior to discharge to Poison Gully.

The other component of the stormwater management system in this catchment is a small detention basin in the local open space on Maida Vale Road. The proposed open space is already cleared and is in a low point in the catchment. Modelling identified that 1,075m^3 of storage is required, with a 450 mm outlet to Poison Gully. The post-development peak outflow from this basin was determined to be 0.27 m^3/s for the 1% AEP event, which is equal to the pre-development discharge rate (0.27 m^3/s). Details of the basin are provided in Table 16.

Table 16: North central basin concept

Parameter		Parameter	
Type	Basin	20% Pre-Dev Outflow	0.12 m^3/s
Side Slopes	1:6	20% AEP Depth	0.82 m
Base Elevation	31.8 mAHD	20% AEP Area	1,078 m^2
Base Area	405 m^2	20% AEP Volume	595 m^3
Top Area	1,456 m^2	20% AEP Outflow	0.12 m^3/s
Total Volume	1075 m^3	1% Pre-Dev Outflow	0.27 m^3/s
Total Depth	1.2 m	1% AEP Depth	1.05 m
Outlet size	450 mm	1% AEP Area	1,302m^2
Outlet U/S invert	32.4 mAHD	1% AEP Volume	868 m^3
Outlet D/S invert	31.9 mAHD	1% AEP Outflow	0.27 m^3/s
Infiltration Rate	1 m/day	Critical Duration	1 hr
Notes	On lot retention and swales within road reserved to convey 1% AEP		

Wetland Runoff

Post-development runoff into the wetland is required to maintain the ecological function and mitigate impacts of reduced rainfall (Section 4.1). The pre-development catchments identified for the wetland include areas of shallow, perched groundwater that contribute to runoff when the catchment is saturated. The proposed post-development catchment for the wetland based on predicted earthworks, includes the Community Hub, POS and light industrial residential area west of Raven Street (Figure 15). Roadside swales will treat the first 15mm from these areas (see Section 5.4 for water quality management) to reduce pollutants entering the wetland, with larger runoff events entering the wetland. The estimated runoff from various events is provided in Table 17, with a comparison to predevelopment runoff volume estimates for various design storm events.

Table 17: Pre- and Post-development runoff into wetland from local catchment

Duration	Scenario	63.2% AEP (1yr ARI)	20% AEP (5yr ARI)	10% AEP (10yr ARI)	1% AEP (100yr ARI)
1 hr	Post-dev	222 m^3	617 m^3	907 m^3	1,919 m^3
	Pre-dev	(374 m^3)	(489 m^3)	(519 m^3)	(752 m^3)
3 hr	Post-dev	216 m3	631 m3	966 m3	2,431 m3
	Pre-dev	(456 m^3)	(638 m^3)	(745 m^3)	(1,250 m^3)
6 hr	Post-dev	161 m3	447 m3	652 m3	2,491 m3
	Pre-dev	(389 m^3)	(711 m^3)	(868 m^3)	(1,513 m^3)
9 hr	Post-dev	92 m^3	347 m^3	518 m^3	2,190 m^3
	Pre-dev	(466 m^3)	(666 m^3)	(915 m^3)	(1,651 m^3)
12 hr	Post-dev	55 m^3	269 m^3	301 m^3	1,715 m^3
	Pre-dev	(485 m^3)	(816 m^3)	(869 m^3)	(1,731 m^3)
18 hr	Post-dev	87 m^3	306 m^3	357 m^3	1,710 m^3
	Pre-dev	(469 m^3)	(738 m^3)	(944 m^3)	(1,858 m^3)
24 hr	Post-dev	86 m^3	247 m^3	307 m^3	1,115 m^3
	Pre-dev	(355 m^3)	(742 m^3)	(841 m^3)	(1,790 m^3)
48 hr	Post-dev	98 m^3	64 m^3	102 m^3	417 m^3
	Pre-dev	(205 m^3)	(223 m^3)	(460 m^3)	(908 m^3)
72 hr	Post-dev	3.73 m^3	24 m^3	73 m^3	174 m^3
	Pre-dev	(36 m^3)	(147 m^3)	(305 m^3)	(490 m^3)

Highlighted are events predicted to contribute a higher volume of runoff to the wetland than predevelopment.

The results in Table 17 indicate there is a significant reduction in runoff volume for frequent events (63.2% AEP, 1 hour) as water is managed within the catchment to provide water quality improvement for this event. Although runoff inflows to the wetland have been reduced, it should be noted that the direct rainfall into the wetland (3.2 ha) remains unchanged following development. The wetland catchment features approximately 5 ha of impervious area with respective losses (Table 15), whereas the predevelopment catchment contributed runoff to the wetland only while soils were saturated. Increased runoff to the wetland is predicted to occur during less frequent events up to 9 hours (1% AEP).

Additional runoff in the wetland could be achieved by reducing the size of roadside swales along Ibis Place (northern boundary of the wetland) or permitting overflow from the north central basin into the wetland (and reconfiguration of the discharge to Poison Gully Creek). The existing swale and basin are designed to contain and convey the 1% AEP event but could be modified to manage smaller events with the larger events discharging into the wetland.

Catchment 3 – North East (4.31 ha)

It is anticipated that the predominant land use in this catchment will be residential. A new drainage basin will be installed adjacent to the river to restrict flows to pre-development discharge rates. Pre-development runoff rates determined using the loss parameters in Table

10, based on a catchment area of 3.1 ha. The post-development catchment is larger at 4.3 ha and includes additional roads and high-density residential areas. The proposed basin configuration is outlined in Table 18, including outflow rates with comparison to pre-development rates. The basin concept is also provided in Figure 15.

Table 18: North east basin concept

Parameter		Parameter	
Type	Basin	20% Pre-Dev Outflow	0.01 m^3/s
Side Slopes	1:6	20% AEP Depth	0.43 m
Base Elevation	36.0 mAHD	20% AEP Area	1,613 m^2
Base Area	1,215 m^2	20% AEP Volume	606 m^3
Top Area	2,460 m^2	20% AEP Outflow	N/A
Total Volume	2,160 m^3	1% Pre-Dev Outflow	0.07 m^3/s
Total Depth	1.2 m	1% AEP Depth	0.98 m
Outlet size	225 mm	1% AEP Area	2,200 m^2
Outlet U/S invert	36.6 mAHD	1% AEP Volume	1,650 m^3
Outlet D/S invert	36.2 mAHD	1% AEP Outflow	0.07 m^3/s
Infiltration Rate	1 m/day	Critical Duration	9 hrs
Notes	Harvesting of stormwater in residential areas may reduce storage requirements and should be confirmed in detailed design.		

Catchment 4 – West (3.82 ha)

The western catchment currently includes a small drainage sump that will be modified with development and be integrated into the proposed POS. The design will include fence removal and minimum 1:6 side slopes to allow access and use of the space by the community. The system currently discharges under Dundas Road to the downstream drainage system. The proposed basin concept is provided in Table 19.

Table 19: Western basin concept

Parameter		Parameter	
Type	Basin	20% AEP Depth	0.39 m
Side Slopes	1:6	20% AEP Area	1,544 m^2
Base Elevation	28.8 mAHD	20% AEP Volume	525 m^3
Base Area	1,158 m^2	10% AEP Depth	0.53 m
Top Area	2,487 m^2	10% AEP Area	1,694 m^2
Total Volume	2,145 m^3	10% AEP Volume	752 m^3
Total Depth	1.2 m	1% AEP Depth	1.1 m
Infiltration Rate	1 m/day	1% AEP Area	2,360 m^2
		1% AEP Volume	1,903 m^3

Parameter		Parameter	
		Critical Duration	9hrs
Notes	On lot retention and swales within road reserved to convey 1% AEP		

Catchment 5 – South (39.07ha)

The southern catchment is the largest within the Study Area and will feature a 1.89 ha POS area between Dundas Road and Milner Road (Figure 3). The current and proposed land use within the catchment is predominantly light industry. As with the western catchment, the existing sump will be modified to create a higher amenity landscape outcome.

The sump will be replaced with a basin that features varied side slopes that will allow some planting. On the western side (adjacent to the rail reserve) side slopes will be 1:3. On the eastern side (adjacent to development) the side slopes will be 1:6 at the bottom of the basin (the bottom metre) and 1:3 for the top 0.5m of the basin.

Road runoff from up to the 10% AEP event will be conveyed through a piped drainage system into the drainage basin, with light industry lots to contain to the 10% AEP onsite where possible. Major flood runoff (1% AEP) will be conveyed via overland flow prior to discharge to POS and drainage areas throughout the development. The basin will be vegetated with locally native, drought tolerant species but will be provided with irrigation to ensure healthy plant growth. Trees will be integrated with the planting to provide shade and to assist in maintaining good infiltration rates. The concept size for this basin is provided in Table 20.

Table 20: Southern basin concept

Parameter		Parameter	
Type	Basin	20% AEP Depth	0.42 m
Side Slopes	Varied (1:3 to 1:6)	20% AEP Area	13,109 m^2
Base Elevation	28.5 mAHD	20% AEP Volume	5,239 m^3
Base Area	11,850 m^2	10% AEP Depth	0.59 m
Top Area	15,700 m^2	10% AEP Area	13,636 m^2
Total Volume	21,030m^3	10% AEP Volume	7,512 m^3
Total Depth	1.5 m	1% AEP Depth	1.47 m
Infiltration Rate	1 m/day	1% AEP Area	15,637 m^2
		1% AEP Volume	20,405 m^3
		Critical Duration	9 hrs
Notes	- On lot retention and swales within road reserves to convey 1% AEP		

5.4 Water quality management

5.4.1 Biofiltration for Quality Treatment

Retention/infiltration of the first 15mm of runoff will limit the mobilisation of pollutants to downstream environments, providing improved water quality outcomes. The retention and

infiltration of runoff close to source through the various systems is supported by the existence of sandy soil and clearance to groundwater within the majority of the Study Area. Separation to shallow groundwater will be created through the importation of fill material in other areas and subsoil drainage may be required to prevent perched groundwater under the system. As discussed in Section 5.2, outflow from the subsoil drainage will be treated prior to discharge to existing waterways. It is also noted that the hydraulic conductivity of less than 1 m/day shall not be considered for soakwell design.

Roadside raingardens, swales and/or bio-retention areas are proposed for managing road runoff. The basins will be planted with nutrient stripping vegetation for water quality treatment where roadside swales are not available for treatment. These systems will be installed with at least 0.5 m of amended soil in accordance with *Adoption Guidelines for Stormwater Bio-filtration* (CRC, 2015). These systems are expected to provide for treatment and infiltration of the first 15 mm of rainfall and provide the opportunity to plant trees and mitigate urban heat impacts.

5.4.2 UNDO Water Quality Modelling

Site specific targets have been proposed for estuarine catchments of the Swan Coastal Plain as a part of background work undertaken by the Department of Water and Environmental Regulation during the development of the Urban Nutrient Decision Outcomes (UNDO) water quality modelling tool. The targets were developed based on consideration of the sensitivity of the receiving water body and the proximity of the development site. Applying this approach, the Study Area would lie outside the suggested 'proximity zones' of the estuarine portion (>1000 m). This would indicate that the following targets could be applied:

- 3.6 kg/ hectare / year of Nitrogen
- 0.2 kg/ hectare / year of Phosphorus

UNDO is a simple empirical decision support model with a flexible framework that evaluates nutrient reduction decisions for new urban developments on the Swan Coastal Plain. It has been developed by the Department of Water to provide urban development proponents with an easy-to-use tool for assessment by local and state government authorities.

An UNDO model has been developed for the Study Area and was run for existing conditions and the proposed redevelopment scenarios. For both scenarios, the Study Area is assumed to be located within Pinjarra soil type with at least 2 m clearance to groundwater.

Results of UNDO modelling, in the form of a report which is generated by the software containing details of all assumptions and inputs, are provided in Appendix 6 (existing conditions) and Appendix 7 (proposed development).

The outcomes of UNDO modelling indicate that on-site retention of the frequent rainfall event within raingardens and soakwells on lots and roadside raingardens will provide discharge loads of:

- 1.15 kg/ hectare / year of total nitrogen
- 0.06 kg/ hectare / year of total phosphorus

These loads are well within the recommended targets for developments within the Swan Coastal Plain that have been developed for a discussion paper as a part of supporting information for the UNDO modelling tool by the Department of Water and Environmental Regulation.

5.5 Management of disease vectors and nuisance insects

The key objectives for management of disease vectors and nuisance insects are:

- Limit the creation of new sites for breeding of nuisance insects.
- Prevent standing water in drainage infrastructure.
- Improve water quality throughout the development.

The construction of above ground water quality treatment systems (raingardens and roadside swales) is proposed within the Study Area. These systems will drain by infiltration through sandy soil with relatively high infiltration rate which minimise standing water times. Subsoil drainage will be installed underneath drainage basins in the northern portion of the Study Area to prevent standing water.

Physical, chemical and biological control methods can be used to manage mosquito populations. Methods which may to be employed (and their order of priority) include:

- Improving water quality, minimising nutrient loads and thereby reducing potential for algal blooms and fish kills; and,
- Should Mosquitos and Chironomid Midges become a nuisance, pesticides (larvicides and/or adulticides) will be used as required to kill mosquito larvae in breeding sites.

Table 21 below displays the approximate calculated emptying time for each basin, following the calculation method outlined by the *Stormwater Management Manual for Western Australia* (based on Engineers Australia (2009) and Argue (2004)). These are all below the maximum 3.5 day (84 hour) emptying time recommended for a 100-year ARI event (1%AEP). A conservative estimate on the infiltration rate has been used to demonstrate a worst-case scenario. For the Northern basins, there is a discharge to Poison Gully at predevelopment flow rates however the calculations below just assume loss through infiltration, so the emptying time is also conservative in that regard as well.

Table 21: Approximate Emptying Times

Basin	Area (m^2) [1]	1% AEP depth	Infiltration rate (m/day) [2]	Emptying time (hours)
North Central	1,456	1.00	1	48
North East	1,450	0.89	1	43
Southern Basin	17,900	1.46	1	70
Western Basin	2,487	1.10	1	53

1 Areas provided are the 1% AEP TWLs.

2 These infiltration rates are the same infiltration rates used to size each basin.

6 IMPLEMENTATION

The success of the water management strategies outlined in this document depends on their implementation through further planning, detailed design, construction and maintenance.

6.1 Urban Water Management Plans

Urban Water Management Plans (UWMPs) are the final water management documents within the state government planning framework outlined in Section 1.1. These documents are prepared as a condition of the subdivision (in support of local development plans) to demonstrate that designs achieve the objectives, strategies and design criteria outlined in this LWMS.

UWMPs should be prepared for consolidated development areas or sub-precincts (Appendix 2) in consultation with the City of Kalamunda and be based on local site investigations appropriate to the proposal and level of risk to water resources. The UWMP should be consistent with the requirements of the DWER's *Urban water management plans: Guidelines for preparing plans and for complying with subdivision conditions* (DoW, 2008b).

Specifically, the UWMP should include detailed engineering and landscaping designs and design of bio-retention systems and non-structural controls measures to manage impacts from construction.

6.1 Monitoring

Pre- and post-development monitoring is discussed below.

6.1.1 Pre-development monitoring

It is considered that there is no value in additional groundwater level monitoring prior to engineering design and further planning. Monitoring data has indicated the maximum groundwater is considered near surface surrounding the wetland, and water quality is reflective of the previous rural land uses. Further groundwater data collected during geotechnical investigation should be considered against monitoring data in this document during engineering design.

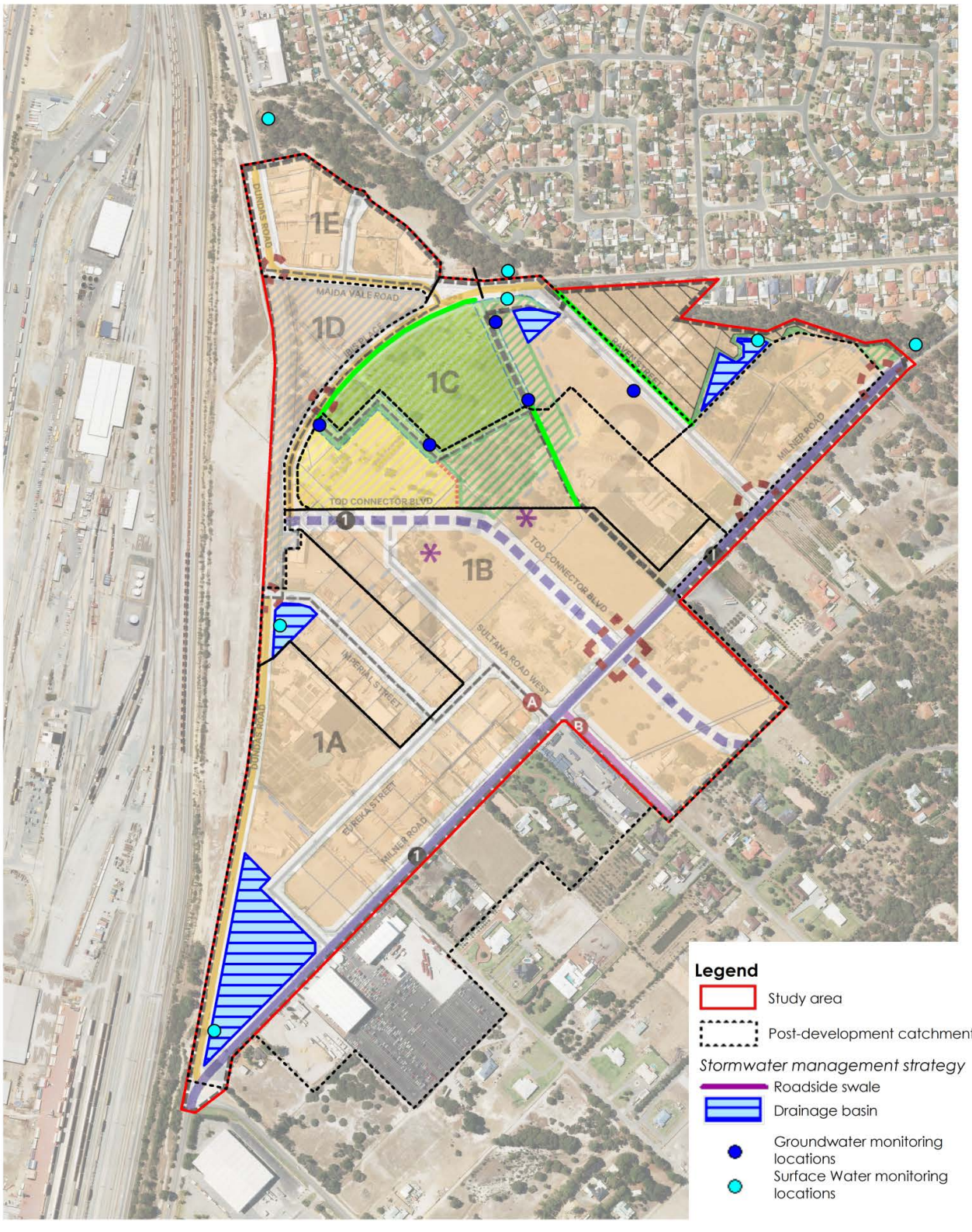
6.1.2 Post-development monitoring

A post-development monitoring program will be implemented and include the following:

Surface water

Surface water monitoring sites at the outlets of the drainage infrastructure and Poison Creek Gully at Milner Road, Maida Vale Road and Dundas Road are recommended to address the key objectives of site-specific post-development monitoring (Figure 17). Surface water sampling should be undertaken during August to October to capture peak winter flows, and in March to capture the first flush events post-summer. Surface water samples should be submitted to a NATA-accredited laboratory in accordance with Australian Standards and analysed for the following parameters:

DevelopmentWA - High Wycombe Local Water Management Strategy
Figure 17 - Water monitoring sites



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- In situ pH, electrical conductivity (EC), dissolved oxygen, temperature;
- Total suspended solids (TSS);
- Total nitrogen (TN) and total Kjeldahl nitrogen (TKN);
- Ammonium (NH4);
- Nitrate and nitrite (NOx-N);
- Total phosphorous (TP); and
- Filterable reactive phosphorus (FRP).

Groundwater

Post development groundwater monitoring should include both levels and quality. Monitoring of groundwater levels and the collection of groundwater samples should be undertaken on a monthly and quarterly basis respectively. Groundwater samples should be submitted to a NATA-accredited laboratory in accordance with Australian Standards and analysed for the following parameters:

- In situ pH, electrical conductivity (EC), dissolved oxygen, temperature;
- Total suspended solids (TSS);
- Total nitrogen (TN) and total Kjeldahl nitrogen (TKN);
- Ammonium (NH4);
- Nitrate and nitrite (NOx-N);
- Total phosphorous (TP); and
- Filterable reactive phosphorous (FRP).

The following additional parameters should be included in the laboratory analysis on an annual basis:

- Major anions (chloride, bromide and sulphate);
- Major cations (calcium, magnesium, sodium and potassium); and
- Iron (Fe) and aluminium (Al).

Post-development monitoring will be undertaken for a period of three (3) years. Commencement will be scheduled to include at least one (1) year following completion of the majority (80%) of development. The results of post-development monitoring will be reported to the City of Kalamunda on an annual basis. Indicative monitoring locations are provided in Figure 17, with the focus on shallow groundwater areas surrounding the wetland and in the northern portion of the Study Area.

Trigger values for the site, based on the pre-development monitoring are provided in Table 22. The trigger values are based on the 80th percentile of the pre-development monitoring results, which are considered representative of baseline conditions. Exceedance of these trigger values will require immediate resampling to confirm the results. The response to confirmed and regular exceedances will be to review maintenance practices to identify any shortcomings, followed by inspection and improvement to nearby water quality improvement structures.

Table 22: Trigger values for post-development monitoring

Parameter	Trigger Value	Parameter	Trigger Value
Total Nitrogen	8.00 mg/L	Ammonia	0.10 mg/L
Nitrite and Nitrate	7.50 mg/L	Total Phosphorus	1.12 mg/L
Total Kjeldahl Nitrogen	4.10 mg/L	Filterable Reactive Phosphorus	0.004 mg/L

Note that trigger values have excluded pre-development Nitrogen results from MB08. The median TN value was 21.50 mg/L and skews the trigger values for the remainder of the site. Monitoring in this area should consider the pre-development results in local analysis.

6.1 Construction

Construction activities have the potential to directly and indirectly impact local water resources and water management measures are required.

6.1.1 Licencing

Water will be required for construction activities such as dust suppression. Water for construction purposes could be sourced from groundwater allocation sought in Section 5.1.1

6.1.2 Construction Management

To ensure downstream waterways are protected, developers, builders and landscapers must implement best management practices to control erosion and sedimentation. Contractors and staff should be notified of specific construction management requirements including appropriate disposal of waste material, erosion control and dust suppression.

6.2 Roles and responsibilities

Key tasks, roles and responsibilities relating to delivery of local water management strategies are outlined in Table 23.

Table 23: Summary of roles and responsibilities

Task	Responsibility	Planning stage
Preparation of the UWMP	Landowner/ developer	Subdivision (UWMP)
Assessment / Approval of the UWMP	City of Kalamunda / DWER	Subdivision (UWMP)
Potable water supply planning and connection to main distribution network	Water Corporation	Subdivision (UWMP)
Design of water distribution networks	Landowner/ developer	Subdivision (UWMP)
Water and wastewater planning and connection to main distribution networks	Water Corporation	Subdivision (UWMP)
Design of wastewater reticulation networks	Landowner/ developer	Subdivision (UWMP)
Design of drainage networks including design of water quality treatment areas	Landowner/ developer	Subdivision (UWMP)
Development of detailed Landscaping plan incorporating stormwater management strategies	Landowner/ developer	Subdivision (UWMP)
Confirmation of ongoing management and maintenance requirements and agreement with the City for handover of responsibilities	Landowner/ developer/ City of Kalamunda	Subdivision (UWMP)
Wetland buffer planting/landscaping design	Landowner/ developer/ City of Kalamunda	Subdivision (UWMP)

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APPENDIX 1 LWMS CHECKLIST

Local water management strategy Item	Deliverable	☑	Comments
Executive summary			
Summary of the development design strategy, outlining how the design objectives are proposed to be met	Table 1: Design elements & requirements for BMPs and critical control points	☑	
Introduction			
Total water cycle management – principles & objectives Planning background Previous studies		☑	
Proposed development			
Structure plan, zoning and land use. Key landscape features Previous land use	Site context plan Structure plan	☑ ☑	
Landscape – proposed POS areas, POS credits, water source, bore(s), lake details (if applicable)	Landscape Plan	☑	
Design criteria			
Agreed design objectives		☑	
Pre-development environment			
Existing information and more detailed assessments (monitoring). How do the site characteristics affect the design?		☑	
Site Conditions – existing topography/ contours, aerial photo underlay, major physical features	Site condition plan	☑	
Geotechnical – topography, soils including acid sulphate soils and infiltration capacity, test pit locations	Geotechnical plan	☑	
Environmental – areas of significant vegetation, wetlands and buffers, waterways and buffers, contaminated sites	Environmental Plan plus supporting data where appropriate	☑	
Surface Water – topography, 100-year floodways and flood fringe areas, water quality of flows entering and leaving (if applicable)	Surface Water Plan	☑	
Groundwater – topography, predevelopment groundwater levels and water quality, test bore locations	Groundwater Plan plus details of groundwater monitoring and testing	☑	
Water sustainability initiatives			
Water supply & efficiency measures – private and public open spaces		☑	
Fit-for-purpose strategy and agreed actions. If non-potable supply, support with water balance		☑	
Wastewater management		☑	
Stormwater management strategy			
Flood protection – peak flow rates, volumes and top water levels at control points,100-year flow paths and100 year detentions storage areas	major event Plan Long section of critical points	☑	
Manage serviceability – storage and retention required for the critical 5-year ARI storm events Minor roads should be passable in the 5-year ARI event	minor event Plan	☑	

Local water management strategy Item	Deliverable	☑	Comments
Protect ecology – detention areas for the 1 yr 1 hr ARI event, areas for water quality treatment and types of (including indicative locations for) agreed structural and non-structural best management practices and treatment trains. Protection of waterways, wetlands (and their buffers), remnant vegetation and ecological linkages	small event Plan Typical cross sections	☑	
Groundwater management strategy			
Post development groundwater levels and fill requirements (including existing and likely final surface levels), outlet controls, and any subsoils	Groundwater/subsoil Plan	☑	
Actions to address acid sulfate soils or contamination		☑	
The next stage – subdivision and urban water management plans			
Content and coverage of future urban water management plans to be completed at subdivision. Include areas where further investigations are required prior to detailed design.		☑	
Monitoring			
Recommended future monitoring plan including timing, frequency, locations and parameters, together with arrangements for ongoing actions		☑	
Implementation			
Developer commitments		☑	
Roles, responsibilities, funding for implementation		☑	
Review		☑	

APPENDIX 2 CONCEPT PLAN

LEGEND

Activity Centre Plan Boundary

EXISTING PLANNING FRAMEWORK

Metronet East Redevelopment Area

Project Area: Forrestfield North (RAC-0)

Ecological Conservation (Conservation Category Wetland)

Bush Forever

PROPOSED PLANNING FRAMEWORK

Region Scheme Reserves

Railway

Local Scheme Reserves and Environmental Conservation

Local Open Space

Public Purposes: Community Purpose

Environmental Conservation

Other Categories

District Integrator A (Existing / Proposed)

District Integrator B (Existing / Proposed)

Neighbourhood Connector (Existing / Proposed)

Local Street (Existing / Proposed)

Proposed Roundabout

Proposed Signalised Intersection

A Left-in, Left-out Intersection

B No Right Out Intersection

Strategic Site

Hazard seperation required at subsequent planning stages to ensure that no residential land is affected by BAL-40 or BAL-FZ

Indicative Drainage Corridor

Notes

1 Final road widths and indicative cross sections to be determined following a traffic assessment

Precincts

1 Forrestfield Station Precinct

2 Residential Precinct

Sub-Precincts

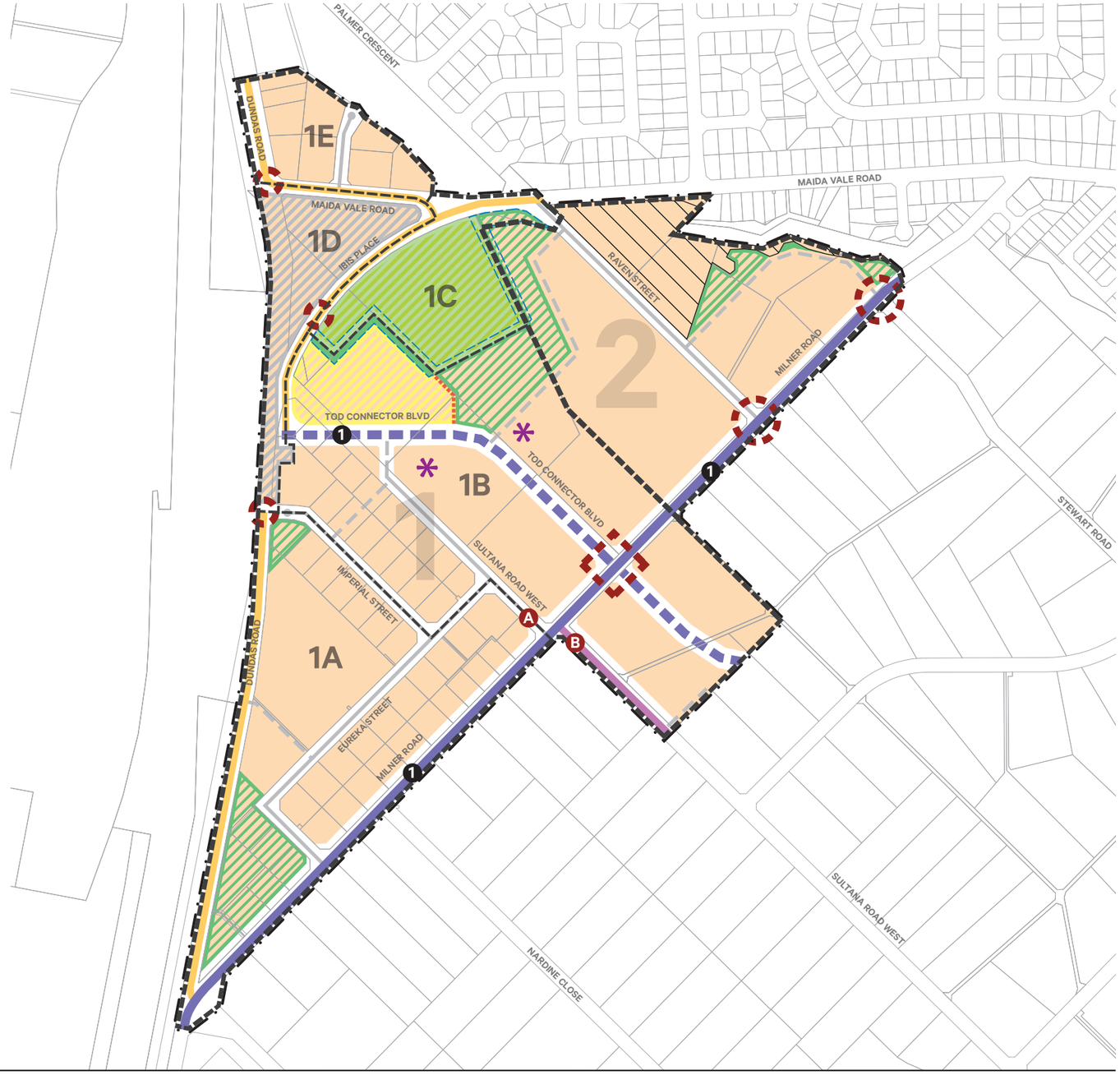
1A Eureka Sub-Precinct

1B Core Sub-Precinct

1C Conservation Sub-Precinct

1D Station Sub-Precinct

1E Maida Vale Sub-Precinct



Activity Centre Plan

Forrestfield North TOD Precinct

DRAFT

Date: 23 Mar 2020 Scale: 1:6000@ A3 1:3000 @ A1 File: **17-527 ST-1 A** Staff: MC GW Checked: MC

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element.

Level 18, 191 St Georges Terrace, Perth Western Australia 6000.
PO Box 7375 Cloisters Square, Perth Western Australia 6850.
T. +61 8 9289 8300 | **E.** hello@elementwa.com.au **elementwa.com.au**

APPENDIX 3 GEOTECHNICAL INFORMATION

Government of **Western Australia**
Public Transport Authority

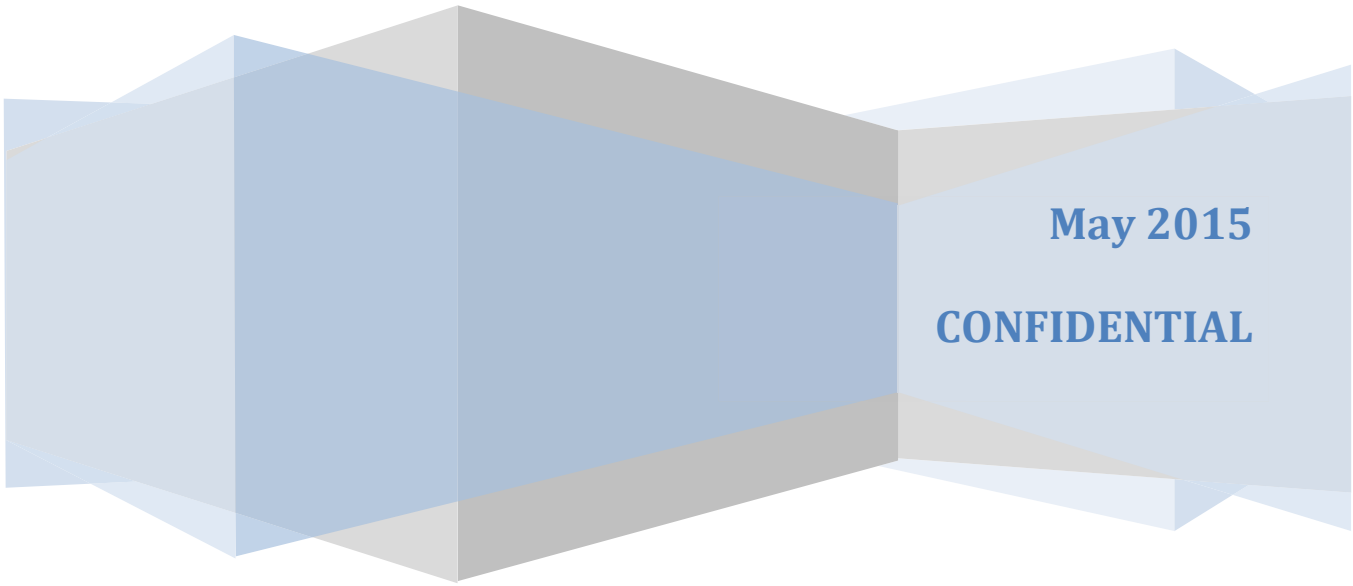
Forrestfield Airport Link Project

Stage 2 Geotechnical Investigation

Groundwater Conditions Report

Prepared by: Golder Associates Pty Ltd

FAL Document Number: FAL-PTAWA-GE-RPT-00009



May 2015

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Version control

Version No.	Date	Document Status	Prepared By	Endorsed By
A	17/04/2015	Draft	Golder Associates Pty Ltd	Eric Hudson-Smith
0	26/05/2015	Final	Golder Associates Pty Ltd	Eric Hudson-Smith

Approvals

The signatures below certify that this report has been reviewed, endorsed and approved, and that the signatories are aware of all the requirements contained herein and are committed to ensuring their provision.

Reviewed
……*Signature* *Date* Dave Thomas Delivery Manager

Approved
……*Signature* *Date* Ray Seman Project Director

CONFIDENTIAL

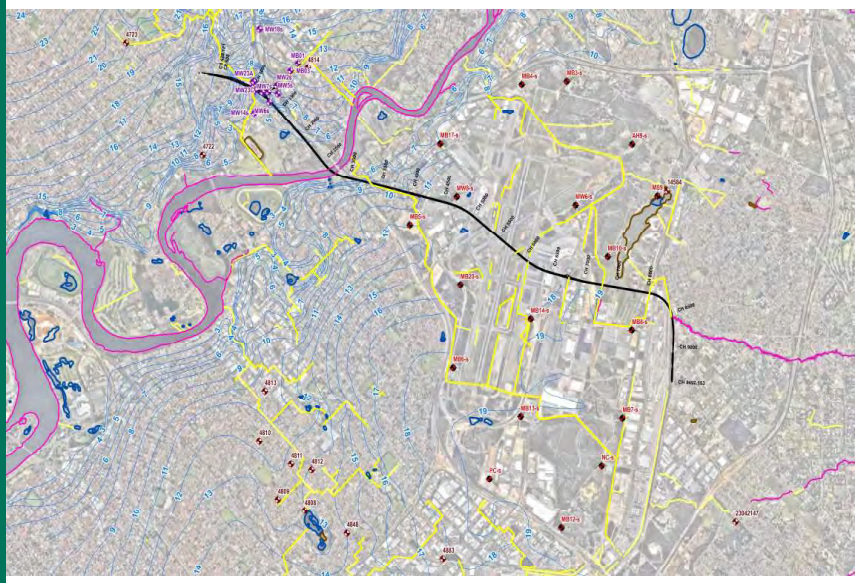
May 2015

FORRESTFIELD-AIRPORT LINK PROJECT, STAGE 2 GEOTECHNICAL INVESTIGATION

Groundwater Conditions Report

FOR INFORMATION

Submitted to:
Public Transport Authority
Public Transport Centre
West Parade
PERTH WA 6000



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REPORT

Golder Associates

Record of Revision

Company	Client Contact	Revision	Description	Date Issued
PTA	Mary Durkan	A	Draft issued for client review	17/04/2015
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List of Abbreviations, Acronyms, Definitions and Units

This list contains abbreviations, acronyms, definitions and units used on this project and in the complete Golder Associates Pty Ltd (Golder) Stage 2 geotechnical investigation report series.

AASS	Actual Acid Sulfate Soil
AF	Ascot Formation
AFa	Ascot Formation – Ascot Beds
AFj	Ascot Formation – Jandakot Beds
a_h	Peak horizontal ground acceleration
AHD	Australian Height Datum
AS	Australian Standard
ASS	Acid Sulfate Soil
ASTM	American Society for Testing and Materials
ATV	Acoustic Televiewer
AWS	Airport West Station
bgl	below ground level
BH	Borehole
BS	Bassendean Sand
$CaCO_3$	Calcium carbonate
CATS	Consolidated Airport Terminal Station
CBD	Central Business District
CBR	California Bearing Ratio
C_c	Compression index
CERCHAR	Laboratoire du Center d' Études et Recherches des Charbonnages de France, Abrasiveness Index
CH	Chainage
CIU	Consolidated, Isotropic and Undrained
CP	Cross Passage
CPT	Cone Penetration Test
CPTU	Piezocone Penetration Test
C_r	Recompression index
CRR	Cyclic Resistance Ratio
CRS	Chromium Reducible Sulfur
C_s	Swelling index
CSR	Cyclic Stress Ratio
c_u	Undrained shear strength (as shown on flat plate dilatometer test reports) = s_u
DBYD	Dial Before You Dig
DCP	Dynamic Cone Penetrometer
DDR	Dry Density Ratio
DIRD	Department of Infrastructure and Regional Development
DMT, D	Flat Plate Dilatometer
DPR	Drilling Parameter Recording

E'	Drained Young's modulus
EA	axial stiffness of tunnel lining
EE	Emergency Egress shaft
EI	bending stiffness of tunnel lining
EPB	Earth Pressure Balance
E_r	Equivalent rock Young's modulus
ERI	Electrical Resistivity Imaging
FAL	Forrestfield-Airport Link
FFS	Forrestfield Station
F_L	Liquefaction triggering factor
FR	Friction Ratio (cone penetration testing)
f_s	Sleeve friction (cone penetration testing)
FWS	Full Waveform Sonic
G_0	Small strain shear modulus
G	Working-strain shear modulus
GF	Guildford Formation
GPS	Global Positioning System
GS	Gnangara Sand
HA	Hand Augered borehole
HCl	Hydrochloric acid
IBC	Intermediate Bulk Containers
I_D	Material index (flat plate dilatometer testing)
I_r	Rigidity index = G/s_u
I_{s50}	Point Load Strength Index (PLI) corrected to 50 mm diameter
ISSMGE	International Society of Soil Mechanics and Geotechnical Engineering
k	hydraulic conductivity (in situ permeability)
k_h	horizontal hydraulic conductivity (permeability)
k_v	vertical hydraulic conductivity (permeability)
k_x	hydraulic conductivity (permeability) in the x direction in the model (= k_h)
k_y	hydraulic conductivity (permeability) in the y direction in the model (= k_v)
K_0	Coefficient of at-rest earth pressure
K_D	Horizontal stress index (flat plate dilatometer testing)
LCPC	Laboratoire des Ponts et Chaussées
LG	Gas monitoring well
M	Constrained modulus (flat plate dilatometer testing)
MASW	Multi-channel Analysis of Surface Waves
MG	Made Ground (Fill)
MIS	Marine Isotope Stage (palaeochannel reference)
MMDD	Maximum dry density/moisture content relationship using modified compactive effort
MRWA	Main Roads Western Australia
MW	Groundwater Monitoring Well
NATA	National Association of Testing Authorities, Australia
NQ	Drilling rod size with rod outside diameter = 69.9 mm
OF	Osborne Formation
OFf	Osborne Formation – Kardinya Shale Member fines dominated

OFm	Osborne Formation – Mirrabooka Member (including Molecap Greensand)
OFs	Osborne Formation – Kardinya Shale Member sand dominated
OLS	Obstacle Limitation Surfaces
P60	Piston sample (inner diameter 60 mm)
PAPL	Perth Airport Pty Ltd
PASS	Potential Acid Sulfate Soil
PCG94	Perth Coastal Grid 1994
PF	Perth Formation
PFc	Perth Formation clay
PFs	Perth Formation sand
PGA	Peak Ground Acceleration
Phi	Friction angle (as shown on flat plate dilatometer test reports) = ϕ'
PLI	Point Load Index = I_{s50}
PP	Pocket Penetrometer
PQ	Drilling core size with core diameter = 85 mm
PSD	Particle Size Distribution
PSP	Perth Sand Penetrometer
PTA	Public Transport Authority
PW	Pumping test Well
q_c	Cone resistance (cone penetration testing)
q_t	Cone resistance corrected for pore pressure at cone shoulder
R	Cone penetrometer radius
R	Return period factor
R_{int}	Interface strength parameter in Plaxis
RL	Reduced Level
RTK	Real Time Kinematic Global Positioning System
SAT	Soil Abrasion Test
SCPT	Seismic Cone Penetration Test (includes measurement of shear wave velocity)
SEM	Scanning Electron Microscopy
SF	Swan River Formation
SMDD	Maximum dry density/moisture content relationship using standard compactive effort
SPOCAS	Suspended Peroxide Oxidation Combined Acidity and Sulfur
SPT	Standard Penetration Test
SRA	Swan River Alluvium
SRB	Sulfate Reducing Bacteria
s_u	Undrained Shear Strength
SWTC	Scope of Works and Technical Criteria
T	Houlsby and Teh (1988) Modified Time Factor (for interpretation of piezocone dissipation tests)
T	Transmissivity
t_{50}	time to dissipate 50% of excess pore pressure (for interpretation of piezocone dissipation tests)
TBM	Tunnel Boring Machine
TDS	Total Dissolved Solids

TP	Test Pit
u	Pore pressure (piezocone penetration testing)
U63	Thin walled tube (inner diameter 63 mm)
UCS	Uniaxial Compressive Strength
Ud	Pore pressure index (flat plate dilatometer testing)
VS	Vane Shear
V_s	Measured shear wave velocity
VWP	Vibrating Wire Piezometer
XRD	X-Ray Diffraction
Z	seismic hazard factor
ν	Poisson's ratio
γ_{sat}	saturated unit weight
γ_{unsat}	unsaturated unit weight
ϕ'	effective friction angle
ϕ_g	geotechnical strength reduction factor
ρ	density
σ_{v0}	total vertical stress
σ'_{v0}	effective vertical stress
σ_{atm}	atmospheric pressure

Units:

°	degree
GPa	gigapascal
K	degrees Kelvin
ka	thousands of years ago
kg/m^3	kilograms per cubic metre
kg/t	kilograms per tonne
km	kilometre
kN/m^3	kilonewtons per cubic metre
kPa	kilopascal
L/s	litres per second
m	metre
m/d	metres per day
m^2/d	metres squared per day
m^2/yr	metres squared per year
Ma	millions of years ago
mg/L	milligrams per litre
mg/kg	milligrams per kilogram
ml/kg	millilitres per kilogram
mm	millimetre
µm	micrometre/micron
MPa	megapascal
ms	millisecond

ppm	parts per million
s	second
t/m^3	tonnes per cubic metre

Summary of soil particle size and soil description in accordance with Australian Standard AS1726-1993:

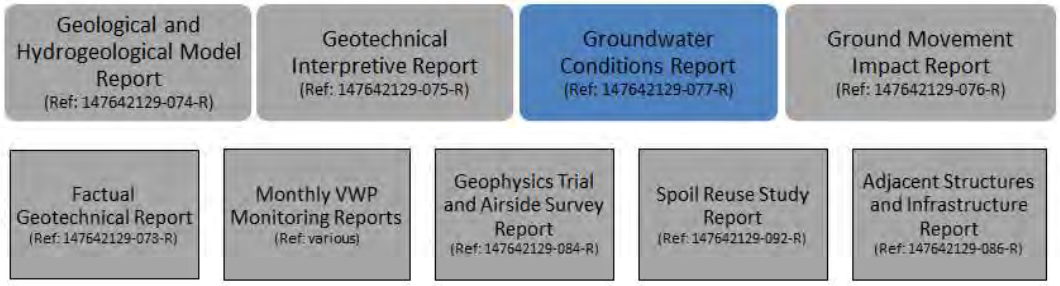
Major Division	Sub-Division	Particle Size	Generalised Material Description
Boulders		> 200 mm	Coarse grained soils with less than 50% smaller than 0.075 mm.
Cobbles		63 to 200 mm	
Gravel	Coarse	20 to 63 mm	
	Medium	6.0 to 20 mm	
	Fine	2.0 to 6.0 mm	
Sand	Coarse	0.6 to 2.0 mm	
	Medium	0.2 to 0.6 mm	
	Fine	0.075 to 0.2 mm	
Silt		0.002 to 0.075 mm	Fine grained soil with greater than 50% of material smaller than 0.075 mm. Material plots below the "A-Line" on plasticity charts.
Clay		< 0.002 mm	Fine grained soil with greater than 50% of material smaller than 0.075 mm. Material plots above the "A-Line" on plasticity charts.

1.0 INTRODUCTION

This report presents the results of hydrogeological interpretive studies carried out as part of Stage 2 Geotechnical Investigation for the proposed Forrestfield-Airport Link (FAL) project. The work was carried out for the Public Transport Authority (PTA) under contract number PTA2014009.

1.1 Related Reports

This report is one of a set of reports that was produced during the Stage 2 geotechnical investigation as shown in Sketch A.



Sketch A: Forrestfield-Airport Link Project - Golder Report Series Breakdown. This report is shown coloured and other reports in the series are shown in grey.

The reports are arranged in the following manner, with a brief overview of the report content provided below as an aid to the reader, although each report should be referred to for full details of the work presented:

1) Primarily interpretive reports:

- The **Geological and Hydrogeological Model Report** presents the geological and hydrogeological conceptual models, descriptions of the geological units and hydrogeological features and interpretation of historic and current investigation data in the form of geological sections.
- The **Geotechnical Interpretive Report** includes interpretation of the subsurface conditions, geotechnical design parameters and key geotechnical risks and mitigation measures.
- The **Groundwater Conditions Report** includes groundwater level data, analysis of hydraulic testing (pumping tests, slug testing and infiltration testing), groundwater modelling results and discussion regarding groundwater control during construction, stormwater disposal and drainage control.
- The **Ground Movement Impact Report** includes analysis and discussion of the effects of construction on adjacent infrastructure, including ground movements from construction activities including any dewatering-related ground movements.

2) Reports containing both factual and interpretive components:

- The **Factual Geotechnical Report** contains the data from the field and laboratory investigations and includes the current monthly Vibrating Wire Piezometer (VWP) Monitoring Report at the date of the factual report.
- The monthly **VWP Monitoring Reports** contain the progressive results of VWP monitoring along the alignment.

- The **Geophysics Trial and Airside Survey Report** contains the results and interpretation of a geophysical investigation trial and airside geophysical survey along a selected part of the alignment.
- The **Spoil Reuse Study Report** contains the results and interpretation of a laboratory testing program aimed at assessing the suitability of construction spoil for reuse as engineered fill.
- The **Adjacent Structures and Infrastructure Report** contains data on existing adjacent structures and major infrastructure.

2.0 PROJECT BACKGROUND

Forrestfield-Airport Link is a State Government project to deliver a rail service to the eastern suburbs. The proposed project alignment is about 8.5 km long, as presented on Figure 1, General Site Location Plan. At the time of reporting the project was planned to include two dive and portal structures, approximately 7.5 km of twin underground rail tunnels between the dive and portal structures, 13 cross passages between the two tunnels, with a permanent egress shaft to the surface at three of the cross passages, and three new railway stations at Airport West, Consolidated Airport Terminal and Forrestfield. Airport West and Consolidated Airport Terminal Stations are planned to be constructed underground. Forrestfield Station is planned to be constructed at-ground together with an associated at-grade parking area.

In preparing this report, reference was made to FAL Project chainages and cross passage and emergency egress shaft locations as shown on PTA Drawing No. 16-C-24-0044, Rev F, Reference Design Plan and Profile, dated 9 April 2015.

The proposed rail line spurs from near the existing at-ground Bayswater Station on the Midland Line, dives underground on the west side of Guildford Road, then generally follows adjacent to and on the south side of Tonkin Highway, passes under the Swan River, deviates under Tonkin Highway towards Perth Airport, runs beneath the airport estate, including both runways, and surfaces and terminates east of the freight rail main lines at Forrestfield.

The current proposed design tunnel configuration involves twin bored tunnels, each with internal diameter of 6.15 m. The separation between the two tunnel centrelines is planned to vary in the range 10 m to 15.5 m, but is typically about 13.5 m. The tunnels will be constructed using a Tunnel Boring Machine (TBM).

The Rev F Reference Design Plan and Profile includes ten cross passages between the twin bored tunnels (CP1, CP2, CP3, CP4, CP4a, CP5, CP5a, CP6, CP6a and CP6b), and a further three cross passages that are incorporated with emergency egress shafts (Emergency Egress Shaft Wright Crescent, Emergency Egress Shaft Domestic Airport Terminal and Emergency Egress Shaft Abernethy Road).

3.0 OBJECTIVES

The objectives of the groundwater conditions studies were to evaluate existing conditions and provide interpretive advice and numerical modelling of construction dewatering for various assumed construction methodologies and design assumptions, based on advice from PTA for possible scenarios, as follows:

- Report the results and interpretation of hydraulic testing carried out during the Stage 2 Geotechnical investigation.
- Provide recommended hydrogeological design parameters.
- Provide recommendations on the design parameters for disposal of stormwater by soakage at Bayswater dive structure, Airport West Station, Consolidated Airport Terminal Station, Forrestfield Station and associated car park.

- Provide advice on possible risks from seasonal and periodic variations and trends to groundwater conditions. Evaluate the maximum design groundwater level for the next 50 years and 120 years at each of the Bayswater dive structure, Airport West Station, Consolidated Airport Terminal Station, Forrestfield Station and Forrestfield car park based on review of historical records and new monitoring data.
- Provide advice on methods of groundwater control during construction, likely dewatering discharge rates, methods of pumped groundwater disposal and constraints or limitations on dewatering activities at each of the Bayswater dive structure, Airport West Station, Consolidated Terminal Station and Forrestfield dive.
- Conduct preliminary 3D groundwater drawdown and recharge modelling at each of the 4 main sites above and show steady state drawdown contours and cross-sections at each structure. Where settlement due to groundwater drawdown is predicted to cause damage that requires repair, suggest means of reducing effects of drawdown.
- Assess the requirements for drainage control measures to protect the integrity of excavations and rail formation and present design concepts, if appropriate. Any requirements for long-term drainage shall be included with appropriate advice on maintenance requirements to satisfy long term effective operation of the drainage system.

4.0 SITE DESCRIPTION

The following sections provide a brief overview of the site conditions including a brief discussion on the geology and hydrogeology. Full details and discussion on the geological model and hydrogeological model developed for the project is provided in the Geological and Hydrogeological Model report (Golder, 2015b).

4.1 Topography and Surface Features

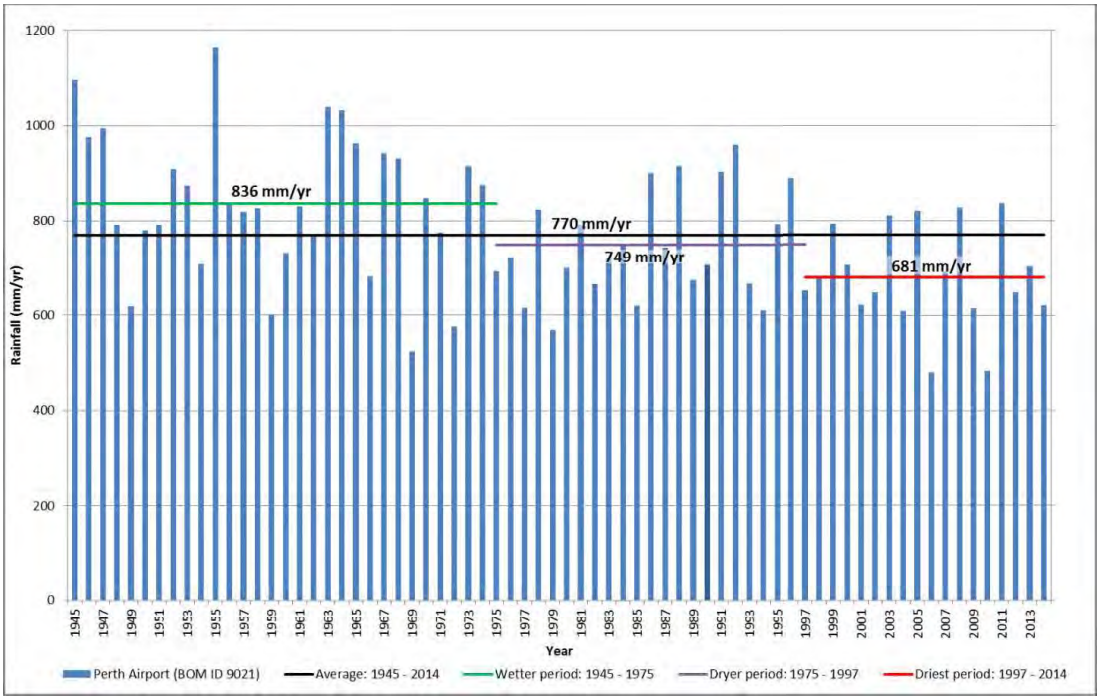
Local topography and surface features are discussed in further detail in (Golder, 2015b). In summary there are three main topographic areas across the proposed alignment as shown on Figure 2. From west to east these include:

- Bassendean Dune System and Swan River Terraces (CH428 to CH3200) – The western end of the proposed alignment has deflated sand dunes of the Bassendean Dune System set alongside alluvial terraces of small creeks and the ancient Swan River. The resulting topography is gently rolling between the crests and troughs of the deflated dune system and eroded river terraces. Overall the elevations are between RL 20 m AHD and RL 25 m AHD at CH428 and gently decreasing to RL 0 m AHD at the Swan River (CH2700). On the eastern side of the Swan River, the elevation quickly rises to between RL 5 m AHD and RL 11 m AHD.
- Bassendean Dune System and Sand Plain (CH3200 to CH7700) – The middle portion of the alignment is largely comprised of sand of the Bassendean Dune System. Over much of this area interdunal depressions previously containing swamps, wetlands and damplands have been infilled during development of the area. In some instances infilling has been completed by excavating local sand for fill. Anecdotal evidence suggests infilling will have modified the current surface level by less than 5 m. Elevation gently increases from approximately RL 8 m AHD at CH3200 to approximately RL 20 m AHD at CH7700.
- Pinjarra Plain (CH7700 to CH9492) – The eastern end of the alignment is located on the Pinjarra Plain which generally comprises alluvial fan deposits extending out from the Perth Hills. Across the alignment a thin layer (1 m to 2 m thickness) of Bassendean Sand covers most the alluvial fan deposits. The terrain begins to rise from approximately RL 20 m AHD at CH 7700 up to RL 30 m AHD by CH8600. As with many geomorphological features in the Perth area, the Pinjarra Plain tends to have an overall north to south alignment. This means the portion of the proposed alignment that runs approximately north to south between CH8600 and CH9492 has a relatively constant elevation of approximately RL 30 m AHD.

4.2 Climate

Sketch B shows the annual rainfall from the Bureau of Meteorology rainfall station (BOM ID 9021) located at the Perth Airport for the period from 1945 to 2014 and indicates the following:

- The average annual rainfall from 1945 to 2014 is 770 mm/yr.
- Perth experienced a wetter period between 1945 and 1975 with an average annual rainfall of 836 mm/yr.
- Since 1975 it has experienced a drying climate with an average from 1975 to 1997 of 749 mm/yr.
- The average annual from 1997 to present is only 681 mm/yr.



Sketch B: Annual Rainfall from 1945-2014 (BOM ID 9021 – Perth Airport)

4.3 Geology

Full details of the geological model and detailed description of geological units relevant to the project are provided in the Geological and Hydrogeological Model Report (Golder, 2015b). A summary of the identified relevant geological units is provided in Table 1. A conceptual geological model along the alignment (in long section) is provided on Figure 2.

Table 1: Summary of Project Specific Geological Units

Geological Unit		Typical Description	Depositional Environment	Approximate Age	Diagenesis
Name	**Unit Colour**				
Fill (MG)		Primarily fine to medium grained yellow to brown sand. Also road base and other types of fill	Controlled and uncontrolled sand fill in recent history.	<200 years	Some cementation of limestone road base may have occurred.
Swan River Formation (SF)		Silty clay, clayey silt, silty sand and sand, dark grey, dark brown to black with shells and organic material, low to high plasticity fines, very soft to firm, or very loose to loose.	Palaeochannel deposits in marine, estuarine and fresh water conditions.	0 to 30 ka	Natural consolidation of this unit may have been enhanced by construction of the Redcliffe Bridge approach embankments
Perth Formation (PF)	PF / PFs	PFs: Sand to silty sand, yellow, brown and grey, fine to coarse grained, medium dense to dense. Some fine to coarse gravel, generally rounded to sub-rounded quartz, may be present.	Palaeochannel deposits in marine, estuarine and fresh water conditions.	80 to 150 ka	Some local pockets (up to approximately 2 m thick) of very weakly cemented (non-carbonate) sand may be present at the contact with the overlying Bassendean Sand.
	PF / PFc	PFc: Clay, some silt and sand, blue-grey, mottled red, yellow, brown, generally low to medium plasticity and generally stiff to hard some soft to firm materials close to the Swan River.			
Bassendean Sand (BS)		Sand, light grey, yellow, dark brown, fine to medium grained, loose to dense, fining upwards where fluvial in origin, with thin (up to 1 to 2 m) localised iron cemented layers. May contain peaty sand, silty and clay associated with wetland or dampland interdunal deposits.	Mixed fluvial and aeolian origin.	80 to 750 ka	Localised iron-cementation has occurred in this unit creating pockets and layers that can be up to approximately 2 m thick of very weakly to moderately well cemented sandstone ("coffee rock").
		transitional boundary / facies change			
Guildford Formation (GF)		Clayey sand, silty sand, sand and clay, brown, pale grey, orange, fine grained layers stiff to hard and low plasticity, coarse grained layers are medium dense to very dense. Includes potential sand deposits of the Yoganup Formation east of the RAC driving centre (~ CH7600).	Fluvial origin generally deposited as part of alluvial fan system. Yoganup formation is a littoral (coastal) deposit.	170 ka to 2 Ma	Aside from natural consolidation no other extensive diagenetic processes have been noted.
		transitional boundary / facies change			
Gnangara Sand (GS)		Sand and silty sand, blue-green, dark green, fine grained, loose to dense.	Freshwater and marine deposit in nearshore coastal environment.	170 ka to 2 Ma	Alteration of materials at contact with the underlying unit may have led to reduction in strength at the contact (e.g. decomposition of organics).
Ascot Formation (AF)		Ascot Beds (AFa): Carbonate sandy gravel, gravelly sand and sand, fine to medium grained sand, grey, yellow, medium dense to dense, some siliceous calcarenite layers.	Inner shelf, nearshore marine environment, deposited during multiple marine transgressions.	750 ka to 3 Ma	This unit appears to have undergone carbonate cementation since deposition of the original sediments. Subsequent leaching may have created small voids and cavities that have now been infilled with sand. Recementation of previously leached sand and gravel layers may still be occurring.
		Jandakot Beds (AFj): Carbonate sand, sandy gravel and gravelly sand, fine to coarse grained, grey, dark grey and blue-grey, medium dense to very dense, some siliceous calcarenite. Polished rounded black phosphatic gravel often found at the base. High strength conglomerate boulders may also be present at the base of this unit but have only been noted in the vicinity of the Perth Airport air traffic control tower to date.			
		major unconformity			
Osborne Formation (OF)	OFm	Mirrabooka Member (OFm): Sand, silty sand and clayey sand, dark green to dark grey, medium to coarse grained, dense to very dense, siliceous and glauconitic. Includes the Molecap Greensand.	Shallow-marine origin.	100 to 120 Ma	Potential secondary formation of pyrite may have occurred.
		facies change			
		Kardinya Shale Member sand dominated (OFs): Silty sandstone, fine-grained, layered, dark green and black, moderately weathered to fresh, extremely low to medium strength.			
		facies change			
		Kardinya Shale Member fines dominated (OFf): Sandy mudstone and sandy siltstone, black and dark green, moderately weathered to fresh, extremely low to medium strength.			

Notes:

- This table is intended to represent a basis for broad geological subdivision of the geology and is not the basis of engineering characterisation. Indications of consistency and relative density in the table are generally typical for the units and do not necessarily represent the full range of values encountered. Not every permutation of material grading is provided. Rock materials may exist in units that are principally soils and vice versa and not all diagenesis processes may have been identified.
- The carbonate and non-carbonate soil and rock classification systems used for this study are defined in Appendix B of (Golder, 2015a).

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Ordinary Council Meeting 12 October 2021 Attachments
Attachment 10.5.2.3
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5.0 HYDROGEOLOGICAL CONDITIONS

5.1 Definitions

The following definitions have been adopted in this report:

- Groundwater level means the level at which the pressure in the pore water in the subsurface material is atmospheric. The terms groundwater level, phreatic surface, groundwater table and water table may be used interchangeably. If perched groundwater conditions exist there may be more than one groundwater level at any particular plan position.
- Piezometric pressure means pore water pressure measured at a point below ground level.
- Piezometric level means piezometric pressure referenced to the level in metres AHD that water would rise to in a standpipe installed with a porous tip registering the pressure at a point below ground.
- Perched groundwater means a saturated lens of water supported by a low permeability unit that occurs above the regional groundwater table (refer to Figure 3). The extent of the perched water table depends on the extent and continuity of the low permeability unit.

5.2 Assignment of Hydrogeological Units

A conceptual hydrogeological model showing the main aquifers, regional groundwater and piezometric levels and flow directions is shown on Figure 3. A summary of the hydrogeological units in the model is provided in Table 2. The hydrogeological units have been split into two zones; East and West, based on the regional geology. Figure 3 shows that the West Zone extends to east of the Swan River where the Perth Formation terminates and the East Zone extends east from this point.

Table 2: Summary of Project Specific Hydrogeological Units

Zone	Regional Unit	Hydrogeological Unit	Local Aquifer Type	Geological Units Included
West (CH428-CH3200)	Superficial Aquifer	Upper Aquifer	Unconfined	MG, BS, PFs
		Bayswater Aquitard	Aquitard	PFc
		Lower Aquifer	Confined (west of Swan River) Semi-confined (east of Swan River)	PFs
	Osborne Aquitard		Aquitard	OF (OFs)
East (CH3200-CH9493)	Superficial Aquifer	Bassendean Sand	Unconfined	MG, BS
		Guildford-Gnangara	Semi-confined	GF, GS
		Ascot Formation		AF
	Mirrabooka Aquifer		Semi-confined Aquifer	OF (OFm)
	Osborne Aquitard		Aquitard	OF (OFf)

Notes: This table is intended to represent a basis for broad hydrogeological subdivision of the geology and is not the basis of engineering characterisation.

The two main regional aquifers of relevance beneath the proposed alignment are the Superficial Aquifer (present in both the east and west zones) and the Mirrabooka Aquifer (present in the east zone only). Along the proposed alignment the Superficial Aquifer comprises all the materials overlying the Osborne Formation. In the west zone, the Superficial Aquifer comprises an Upper and Lower Aquifer separated by an aquitard (PFc). The Superficial Aquifer includes seven of the main geological units listed previously, which all have unique hydrogeological properties. The Superficial Aquifer and the Mirrabooka Aquifer is in direct hydraulic connection.

5.2.1 West Zone

Within the West Zone, groundwater of relevance to the project is largely contained within the sandy dominated materials of the Perth Formation and the overlying Bassendean Sand. However, the occurrence of fines dominated sediments within the Perth Formation (termed the Bayswater Aquitard in Table 2) separates the water bearing materials into two aquifers. Differences in groundwater levels measured within these two aquifers substantiate the presence of these two aquifers and this is discussed in more detail in Section 5.3.5. For the purpose of this report the two aquifers have been named the Upper Aquifer and Lower Aquifer. The Lower Aquifer is underlain by the Osborne Aquitard (KSc).

5.2.2 East Zone

Within the east hydrogeological zone all materials overlying the Osborne Formation are part of the regional Superficial Aquifer. However, the materials have been separated into distinct hydrogeological units (BS, GF, GS and AF) due to variations in their hydrogeological characteristics. Clayey/silty soils within the Guildford Formation and Gnangara Sands are expected to be responsible for creating semi-confined conditions. Underlying the Ascot Formation over part of the east zone is the semi-confined Mirrabooka Member (Mirrabooka Aquifer), which is in direct hydraulic connection with the Ascot Formation. All materials are underlain by the Osborne Aquitard (KSc).

5.3 Groundwater Levels

5.3.1 Published Information

The Perth Groundwater Atlas (WRC, 1997), which presents the inferred maximum historical groundwater level contours, covers the alignment between CH428 and CH4250 and at CATS (refer to Figure 4). Approximate groundwater levels from the 1997 Groundwater Atlas for each main structure are summarised below:

- Bayswater Dive Structure (at portal) – between RL 5 m AHD and RL 6 m AHD (contours suggest there has historically been a surface drain at the portal.
- Airport West Station – between RL 10 m AHD and RL 11 m AHD.
- Consolidated Airport Terminal Station – RL 19 m AHD.
- Forrestfield Dive Structure – No coverage.

It should be noted that the 1997 Groundwater Atlas in some areas presents the groundwater level prior to installation of some of the drains that are used to control groundwater levels in built-up areas. Therefore, the maximum historical groundwater levels shown in the 1997 Groundwater Atlas may not represent the possible maximum future groundwater levels.

5.3.2 Historical Trends

Historical groundwater level trends for the Superficial Aquifer were evaluated using data obtained from the Department of Water (DoW) WIN database, Perth Airport Pty Ltd (PAPL) and CSBP investigation (Parsons Brinckerhoff, 2013). Figure 4 shows the location of selected monitoring wells with available hydrographs while Appendix A presents the hydrographs. The historical groundwater level data indicates:

- A step-decline in groundwater levels occurred in the Belmont area in the late 1950's (e.g. WIN ID 4809) and 1960's (e.g. WIN ID 4883) which is due to installation of drains to lower the groundwater level to allow for land development.
- Steady groundwater levels between the mid 1960's and the present day.
- The Superficial Aquifer along the alignment is full and surface drains generally constrain the maximum groundwater level in the area.

- Although annual rainfall totals indicate a drying climate (Section 4.2), rainfall (and subsequent recharge) experienced in recent years is still sufficient to fill the aquifer resulting in flow into the surface drains that constrain groundwater levels to their maximum local height. A decline in groundwater levels due to change in climate is therefore not observed in this area yet.

5.3.3 Seasonal Fluctuations

Seasonal groundwater level fluctuations along the alignment were assessed by reviewing the following hydrographs (refer to Figure 4 for location of wells):

- DoW hydrographs (data from the 1950's to present).
- PAPL hydrographs (data from 1999 to present).
- CSBP hydrographs (Parsons Brinckerhoff, 2013) (data from 2003 to 2011)
- Vibrating wire piezometer (VWP) data (Golder, 2015i) from instruments installed by GHD (GHD, 2014c) during the Stage 1 geotechnical investigation (data from one season only 2014-2015).

A summary of the seasonal fluctuation data is provided below:

- West Zone:
  - Upper Aquifer – 0.7 m to 1.1 m
  - Lower Aquifer – 1.2 m to 1.6 m (based on 2014/2015 hydrographs from BH 0-02, BH 0-03 and BH 0-08 only).
- East Zone:
  - Superficial Aquifer – General range between 0.6 and 1.6 m with an average around 1.2 m, but at BH 0-07 in Forrestfield the seasonal variation is observed to be 3.1 m (Golder, 2015i). This larger seasonal fluctuation is expected to be due to the predominance of clayey and lower permeability soils close to the ground surface that make up the Guildford Formation in this area, which is also referenced in published information (Davidson, 1995). VWP readings from BH1-02, BH1-04 and BH1-07 show similar seasonal declines as BH0-07, suggesting that the 3 m seasonal variation could exist over the whole Forrestfield Area.
  - Mirrabooka Aquifer – no seasonal data is currently available, but based on the available readings from VWPs installed in the Mirrabooka Aquifer (BH1-22 and BH1-24), the piezometric levels show similar trends to piezometers installed in the Superficial Aquifer at CATS. The seasonal variation would therefore be expected to be similar to the seasonal variations in the Superficial Aquifer (the maximum seasonal variation observed in MB14s is 1.2 m).

5.3.4 March 2015 Levels

Groundwater levels were measured in 95 monitoring wells (installed as part of the geotechnical and environmental investigations for the FAL project) on 10 March 2015. Appendix B provides the groundwater and piezometric level measurements for the monitoring wells and the VWPs while Figures 5 shows the monitoring locations and the values measured during the March 2015 groundwater level survey. The inferred approximate groundwater and piezometric levels for each dive and station location are summarised below:

- Bayswater Dive Structure, extending from the shallowest to deepest section of the proposed dive structure, respectively:
  - Upper Aquifer – RL 10 m AHD to RL 5 m AHD
  - Lower Aquifer – RL 4 m AHD to RL 1.5 m AHD (east of the Bayswater Main Drain).
- Airport West Station – RL 8.2 m AHD to RL 8.8 m AHD

- Consolidated Airport Terminal Station – RL 16.8 m AHD to RL 17.0 m AHD
- Forrestfield Dive – RL 23 m AHD.

5.3.5 Multi Aquifer System (West Zone)

Groundwater levels measured in the Bayswater Dive area indicate a significant downward vertical hydraulic gradient between the Upper and Lower Aquifers (head difference of 3.7 m at the Bayswater Dive Portal). This difference in levels indicates that the Upper Aquifer in this area is "perched" above the Bayswater Aquitard which is therefore likely to have a very low hydraulic conductivity. It is possible that the head difference between the two aquifers could be further exacerbated as a result of potential pumping from the Lower Aquifer.

Based on the current monitoring locations the difference in groundwater level between the two aquifers is most distinct along the alignment between the Bayswater Dive Structure east toward CH1800. There is a change in groundwater level of 3.8 m between CH1800 and CH2200, after which there is insignificant difference in groundwater/piezometric levels between monitoring wells screened in the shallow and deep portions of the aquifer, suggesting that there is no longer a significant multi-layered aquifer system east of this position. The reason for an apparent merging of the multi-layer aquifer system is likely associated with both a thinning of the Bassendean Sand and intersection of the groundwater table with the Perth Formation east of approximately CH1800.

5.3.6 Perched Groundwater Levels

5.3.6.1 West Zone

Perched groundwater is unlikely to be present in this zone between CH428 and CH1800 because the groundwater level in the Upper Aquifer is already close to the ground surface in many places and the unsaturated soil mainly consists of Bassendean Sand. However, although not encountered during this investigation, the presence of localised coffee rock that may cause locally perched groundwater conditions cannot be ruled out.

Between CH1800 and CH2330 (west of Swan River) and between CH2700 and CH3200 (east of Swan River) the groundwater table is located in the Perth Formation, with overlying unsaturated Bassendean Sand. It is possible that perched groundwater conditions could form during the wet season above the contact of the Bassendean Sand with the underlying clay dominated Perth Formation (PFc).

5.3.6.2 East Zone

Groundwater levels near the proposed Forrestfield Dive structure and station indicate the presence of perched groundwater conditions in the Superficial Aquifer (Guildford Formation). Insufficient groundwater level data is currently available to identify whether these perched conditions are seasonal or persistent throughout the year. A summary of groundwater levels from March 2015, which indicated perched conditions, is provided below:

- MW 3-044 (s) (RL 23.2 m AHD) and MW 3-044 (d) (22.9 m AHD) located together adjacent to the dive structure – head difference of approximately 0.3 m.
- MW 3-060 (s) (RL 23.9 m AHD) and MW 3-060 (d) (RL 22.4 m AHD) located together adjacent to Abernethy Road at CH8100 – head difference of approximately 1.5 m.
- MW3-019 (RL 26.2 m AHD) located approximately 125 m east of MW3-026 (RL 22.8 m AHD) and MW3-012 (22.9 m AHD) – head difference of approximately 3.3 m.
- MW3-004 (RL 24.3 m AHD) and MW3-009 (RL 24.5 m AHD) located in between MW3-001 (RL 23.3 m AHD) and MW1-05 (RL 23.4 m AHD) – head difference of between approximately 1.0 m and 1.2 m.

Given that the level of perching is locally highly variable, it is likely that these perched conditions are caused by localised and discontinuous bands/layers of low permeability clay or coffee rock within the Guildford Formation soils.

5.4 Groundwater Recharge

Rainfall and stormwater runoff, via soakage basins and stormwater drains, are considered to be the major contributors to groundwater recharge for all water bearing hydrogeological units of relevance to the proposed alignment. This relationship is somewhat indirect for the Ascot Formation and Mirrabooka Member as these units receive recharge via downward leakage from the overlying Bassendean Sand and Guildford Formation where direct rainfall recharge occurs.

5.5 Groundwater Discharge

5.5.1 Surface Water

The Swan River crosses the alignment at around chainage CH2700 and ultimately acts as the main surficial discharge point for the majority of groundwater of relevance to the construction of the project. Discharge to the river is evident by the observed upward gradient in monitoring wells at the Swan River as well as the observed hydraulic gradient towards the river (Figure 4).

The following summarises other surface water bodies in the vicinity of the alignment:

- Gobba Lake is located on the western side of the Swan River approximately 130 m south-west of the alignment at about chainage CH1750. This lake has existed as a swamp/wetland since before development in the area (Golder, 2015b) and exists due to a local topographic low point (< RL 4 m AHD). This surface water body is understood to be continually inundated and therefore is expected to act as a groundwater discharge point with evaporative losses occurring predominantly during the summer months and potentially recharging the aquifer locally during the winter months.
- Munday Swamp is located approximately 150 m north of the alignment at approximate chainage CH7550. The area comprises a local topographic low point (< RL18 m AHD) explaining the presence of Munday Swamp and the seasonally flooded terrain north of the Manheim vehicle auction yard and the RAC Driving Track. This area is expected to act as a discharge point for groundwater, particularly through evaporation directly from the lake surface during summer months and where the groundwater level is high enough to reach the surface and cause inundation in the surrounding damp lands. The likely presence of low permeability soils below Munday Swamp is likely to limit any recharge to the aquifer from the swamp.
- Poison Gully Creek crosses the proposed alignment at CH8550 and was realigned in the late 1960's or early 1970's during construction of the Forrestfield Rail Yard. This creek is ephemeral and discharges into a constructed drain (part of the Perth Airport drain system) after crossing beneath Dundas Road.

5.5.2 Drains

Groundwater levels across the proposed alignment are affected by the drainage network constructed for various residential and commercial developments. The two main drains along the alignment include the Bayswater Main Drain and Airport Main Drain/Belmont Main Drain, both of which empty into the Swan River. However, there is a wide network of surface and subsurface drains present that feed into both of these drains which help control groundwater levels, particularly in former wetland areas present beneath the airport estate and over the eastern zone of the proposed alignment:

- The alignment crosses the Bayswater Main Drain at approximately CH1000. Review of historical aerial photography (Golder, 2015c) shows this drain was once a small creek and a topographical low for the area that drained wetlands located to the north, west and east. Although the drain still completes this function many of the creeks and wetlands have been infilled or piped underground to facilitate development.
- The Belmont Main Drain runs parallel (approximate offset of 25 m) from about CH3375 to CH4000
- The Perth Airport drain system crosses the alignment at about CH4700, CH5860, CH6080, CH7280, CH8030 and CH8400.

5.5.3 Local Groundwater Users

Groundwater is abstracted along the alignment from all aquifers identified in this report, which is likely to be mainly for reticulation (household and business) and industrial supply. The effect of pumped abstraction on groundwater can be seen in VWP data (Golder, 2015i) as summarised below:

- The majority of the VWPs show daily to twice daily groundwater level drawdown of up to 0.5 m along most of the alignment except in the Forrestfield area (up to CH8100 only). These drawdowns only last for a few hours, suggesting that these responses are due to reticulation bores being turned on.
- The greatest drawdown from pumping is observed between CH5000 and CH6000 at Perth Airport (BH0-01, BH0-05, BH0-09 and BH0-10) where drawdown ranges between 0.5 m and 1.5 m with greatest drawdown occurring in the Ascot Formation. The drawdown pattern is more irregular and can last for up to 2 weeks followed by no pumping for weeks or months. The drawdown pattern is more regular (weekly) during the dry season and less frequent during the wet season.
- None of the monitoring locations in the Upper Aquifer at Bayswater have to date shown an effect from pumping.

The observed effect from local groundwater abstraction (other than the pumping tests carried out as part of this investigation) in the vicinity of the four main structures is as follows:

- Bayswater Dive – VWP data (BH0-02, BH2-22 and BH2-19) indicates daily groundwater level drawdown responses between 0.1 m and 0.5 m in the Lower Aquifer with a maximum response of up to 1 m in BH2-22 at the end of December 2014.
- Airport West Station – VWP data (BH0-04, BH2-05 and BH2-11) indicates daily and in some cases twice-daily drawdown response of up to approximately 0.3 m in the Guildford, Ascot and Osborne Formations.
- Consolidated Airport Terminal Station – VWP data (BH0-06, BH1-22 and BH1-24) indicates daily drawdown responses of around 0.1 m in the Guildford Formation, 0.25 m in the Ascot Formation and less than 0.05 m in the Osborne Formation.
- Forrestfield Dive – VWP data (BH0-07, BH1-04 and BH1-02) show negligible effect in response to nearby abstraction of groundwater.

Given that groundwater level monitoring along the alignment is showing temporary (daily) effects of groundwater level drawdown from reticulation wells, any groundwater abstraction for the project (e.g. construction dewatering) would also affect nearby existing wells. It is not uncommon in Perth that reticulation bores used for individual household reticulation consist of large diameter shallow liners, which would only have up to a few metres of water column in the bores during summer. Therefore there is a potential risk that some of these reticulation bores could become dry during construction dewatering, if drawdown is not carefully managed.

5.6 Groundwater Flow

The groundwater levels measured in March 2015 are consistent with the groundwater flow direction shown in the Perth Groundwater Atlas (WRC, 1997) which indicates two main regional groundwater flow components relevant to the project.

To the north of the Swan River, the regional groundwater flow direction is south to south-east, discharging into the Swan River. Groundwater flowing through this part of the alignment originates from the southern part of the Gnangara Mound which is the dominant regional groundwater head (Davidson, 1995).

South of the Swan River, the regional groundwater flow direction is north-west, discharging into the Swan River. Groundwater flowing through this part of the alignment originates from the Darling Scarp area to the east and from recharge along the alignment (Davidson, 1995).

An assessment of groundwater flow velocities across the site was not carried out within the scope of this study. We note that for ground improvement processes such as ground freezing, the groundwater flow velocity in the treatment area is an important factor. Further evaluation of groundwater flow velocity may be required in areas where ground improvement is required, depending on the level of risk at each location.

5.7 Hydraulic Properties

The following methods and tests were used and undertaken to assess the hydraulic properties of the different hydrogeological units and aquifers:

- Analysis of particle size distribution (PSD) data from 207 samples (refer to Appendix C for results) to derive hydraulic conductivity through empirical relationships.
- Single well hydraulic (slug) testing in 31 monitoring wells along the alignment (refer to Appendix D for method description and test results).
- Test pumping in four pumping test wells, one at each of the dive structures and stations (refer to Appendix E for description of tests and results).

In addition, a desktop study was undertaken to summarise hydraulic properties from previous available investigations, (Parsons Brinckerhoff, 2013), (HydroSolutions, 2004).

It is important to note that the different tests reflect different scales of the aquifer being tested. The PSD is only carried out on small and discrete soil samples and therefore the results do not represent the entire aquifer, but only the tested sample. Similarly, a slug test only affects the immediate area around the well and does not necessarily represent the wider extent of the aquifer. The results from PSD samples and slug testing should therefore be used with caution and can generally only be used to provide an indication of the order of magnitude of the hydraulic conductivity and provide a better understanding of the potential variability within the soils and the overall homogeneity of the aquifer.

The results from properly conducted pumping tests are the most important tool in hydrogeological investigations and far superior to other methods of assessing hydraulic properties and aquifer performance as the results are representative of a much larger extent of the aquifer that will require dewatering. Therefore more weight is put on the pumping test results than results from the slug testing or by derivation from the PSD data.

5.7.1 Transmissivity

The analysis of pumping test data provides an estimate of the transmissivity of the aquifer being tested, which is equal to the average hydraulic conductivity times the saturated thickness of the aquifer.

Hydrosolutions carried out a hydrogeological investigation at Perth Airport in 2004 (HydroSolutions, 2004), which included undertaking five pumping tests, of which four were completed in the Guildford Formation and one in the Mirrabooka Aquifer. The reported results indicated transmissivity ranging from 30 m^2/d to 300 m^2/d in the Guildford Formation and 80 m^2/d in the Mirrabooka Aquifer.

Mackie Martin & Associates carried out two pumping tests in the Upper Aquifer in August 1990 at the CSBP site located 200 m northeast of the Bayswater Dive Structure. The reported results indicated a transmissivity of 100 m^2/d in the Upper Aquifer. With a reported saturated thickness of 4 m (Bassendean Sand), this would correspond to a hydraulic conductivity of 25 m/d.

Table 3 presents the results from the four pumping tests carried for the Stage 2 Geotechnical Investigation, which are based on different analytical solutions, distance drawdown curves and replication of each pumping test in a numerical groundwater model (see Appendix E). The results from the groundwater modelling analysis of the pumping test data is considered to be more representative than other approaches as it includes the effects from all the different hydrogeological units and boundary conditions and has therefore been given more weight compared to analytical solutions. The distance drawdown curves were generally found to give similar results as to the numerical modelling of the pumping test.

Table 3: Transmissivity Values from the Pumping Test Analysis

Location	Aquifer	Range (m^2/d) **	Chosen Value (m^2/d)
Bayswater Dive Structure *	Lower	20–300	113
Airport West Station	Superficial	40–140	76
Consolidated Airport Terminal Station	Superficial and Mirrabooka	130–700	274
Forrestfield Dive Structure	Superficial (GF)	100–320	180

Notes: *No pumping test was carried out in the Upper Aquifer during the Stage 2 Geotechnical Investigation
** The values have been rounded to nearest 10.

5.7.2 Hydraulic Conductivity

Table 4a and Table 4b present a summary of the interpreted and adopted values of the horizontal hydraulic conductivity (in m/d and m/s, respectively) for each of the hydrogeological units and presents the range and median from the different methods used to estimate the hydraulic conductivity. The Tunnel Proximity Study (Golder, 2014c) values were collated based on a review of both published and unpublished reports. The last two columns provide range of values adopted for this study for each of the geological units.

Given the sedimentary nature of the geological units, which have been deposited in fluvial environments, it would be expected that the ratio between the vertical and horizontal hydraulic conductivity (k_v/k_h) would be less than 1 (i.e. the vertical hydraulic conductivity is smaller than the horizontal hydraulic conductivity). The general literature suggests that k_v could be several orders of magnitude smaller than k_h and it is not uncommon to assume a k_v/k_h ratio of 0.1.

The ratio between the vertical and horizontal hydraulic conductivity (k_v/k_h) was assessed during the groundwater modelling of the pumping tests and was found to generally range between 0.1 and 1. The analysis at Bayswater Dive Structure, Airport West Station and Consolidated Airport Terminal Station indicated that the ratio in the units could be around 0.1 while at Forrestfield the results did not provide a definitive conclusion regarding the ratio (use of 0.1 to 1 gave almost similar results – best fit was 0.25). Based on the composition, heterogeneity and distinct layering observed in the materials encountered in the boreholes (e.g. in OF) we would expect that the k_v/k_h ratio is smallest in the PF, GS, GF and OF. The greatest ratio is likely to be observed in the BS, but also the sandy GF encountered in the Forrestfield Area could have a higher ratio as indicated by the pumping test analysis. There was no distinct layering observed in the AF and this unit has a low percentage of fines in most areas, which would suggest that the ratio in AF could approximately be somewhere in between the ratio for PF and BS.

Based on the above, Golder judge that it is reasonable to use a k_v/k_h ratio of 0.1. However, since the installation of D-walls would result in mainly vertical flow upwards into the excavations once dewatering begins, the vertical hydraulic conductivity of the hydrogeological units would significantly influence the dewatering rates and drawdown. A sensitivity analysis was therefore carried out on the k_v/k_h ratio to assess the effect it could have on dewatering rates and drawdown (refer to Section 6.0).

5.7.3 Storage Coefficient

The estimated storage coefficients from the analytical solutions for the four pumping tests were found to range between 1×10^{-5} and 5×10^{-2} suggesting confined to semi confined conditions. The highest storage coefficient was estimated for the pumping test at the Consolidated Airport Terminal Station while the lowest was estimated for the confined Lower Aquifer at Bayswater Dive Structure.

The storage coefficients were also estimated through the numerical modelling of the pumping tests and the results indicated that the best matched shape between the modelled drawdown and observed drawdown for the tests was obtained by using confined conditions with storage coefficients of 1.5×10^{-5} (Bayswater Dive Structure) and 2.5×10^{-3} (Forrestfield Dive Structure).

Table 4a: Summary of Horizontal Hydraulic Conductivities by Hydrogeological Unit (m/d)

Geological Unit – Stage 1 (2014)	Geological Unit – Stage 2 (this study)		Tunnel Proximity Study		Stage 2							
			Range	Suggested Value	Particle Size Distribution^		Slug Testing		Test Pumping		Adopted Values	
					Range	Median	Range	Median	Range	Median	Range	Value
			m/d									
Swan River Alluvium & Alluvial Sand	Swan River Formation		0.1-10	1	3-9	6	0.2	0.2	-	-	0.001-1.0	0.001
Bassendean Sand	Bassendean Sand		10-40	15	1-25	9	***	***	-	-	10-40	15
Guildford Formation	Perth Formation	Sand	0.1-70	12	1-23	5	0.5-28	11	1-7.5	4	1-30	7.5
		Clay			-	-	**	**	-	-	0.0001-0.1	0.0001**
	Guildford Formation				1-36	2	0.2-8	2.5	6-11	8	1-10	7
	Gnangara Sand		-	-	0.4-8	2	-	-			0.5-5	1
Ascot Formation	Ascot Formation		5-100	10	3-20	4	3-50	7.5	5-30#	11	5-50	8-14##
Osborne Formation	Osborne Formation	Kardinya Shale	0.01-1	0.01	-	-	0.0002 to 0.0006	0.0005	0.0005*	-	0.0002 to 0.001	0.0005
		Mirrabooka Member			0.3-20	9	3-6	4.5	1-3*	2	1-10	1-3^^

Notes: * Values have been estimated through numerical analysis in the groundwater model.

** No monitoring wells are installed in the Perth Formation Clay (Bayswater Aquitard) - Value is based on the unit acting as an aquitard.

*** No monitoring wells are screened entirely in Bassendean Sand.

Lower part of range is from the Airport West pumping test while the upper range is from the Consolidated Airport Terminal Station pumping test.

Lower part of range is for the Airport West area while the upper range is from the Consolidated Airport Terminal Station area.

^ Estimations exclude PSD samples that have fines content greater than 20% and the values therefore represent the more permeable samples.

^^ The higher range is for the top of the Mirrabooka Aquifer with hydraulic conductivity reducing with depth.

Parameters are intended to be used for groundwater seepage analysis. Consolidation analysis may require refined values for Swan River Formation and Perth Formation (clay), refer to the GIR (Golder, 2015c).

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Table 4b: Summary of Horizontal Hydraulic Conductivities by Hydrogeological Unit (x10^{-5} m/s)

Geological Unit – Stage 1 (2014)	Geological Unit – Stage 2 (this study)		Tunnel Proximity Study		Stage 2							
			Range	Suggested Value	Particle Size Distribution^		Slug Testing		Test Pumping		Adopted Values	
					Range	Median	Range	Median	Range	Median	Range	Value
			x10^{-5} m/s									
Swan River Alluvium & Alluvial Sand	Swan River Formation		0.12-12	1.2	3.5-10.4	6.9	0.23	0.23	-	-	0.0012-1.2	0.0012
Bassendean Sand	Bassendean Sand		11.6-46.3	17.4	1.2-28.9	10.4	***	***	-	-	11.6-46.3	17.4
Guildford Formation	Perth Formation	Sand	0.12-81	13.9	1.2-26.6	5.8	0.58-32.4	12.7	1.2-8.7	4.6	1.2-34.7	8.7
		Clay			-	-	**	**	-	-	0.00012-0.12	0.00012 **
	Guildford Formation				1.2-41.7	2.3	0.23-9.3	2.9	6.9-12.7	9.3	1.2-11.6	7
	Gnangara Sand		-	-	0.5-9.3	2.3	-	-			0.6-5.8	1
Ascot Formation	Ascot Formation		5.8-115.7	11.6	3.5-23.1	4.6	3.5-57.9	8.7	5.8-34.7$^{\#}$	12.7	5.8-57.9	9.3-16.2$^{\#\#}$
Osborne Formation	Osborne Formation	Kardinya Shale	0.012-1.2	0.012	-	-	0.0002 - 0.0007	0.0006	0.0006*	-	0.0002 to 0.0012	0.0006
		Mirrabooka Member			0.3-23.1	10.4	3.5-6.9	5.2	1.2-3.5*	2.3	1-10	1.2-3.5^^

Notes: * Values have been estimated through numerical analysis in the groundwater model.

** No monitoring wells are installed in the Perth Formation Clay (Bayswater Aquitard) - Value is based on the unit acting as an aquitard.

*** No monitoring wells are screened entirely in Bassendean Sand.

$^{\#}$ Lower part of range is from the Airport West pumping test while the upper range is from the Consolidated Airport Terminal Station pumping test.

$^{\#\#}$ Lower part of range is for the Airport West area while the upper range is from the Consolidated Airport Terminal Station area.

^ Estimations exclude PSD samples that have fines content greater than 20% and the values therefore represent the more permeable samples.

^^ The higher range is for the top of the Mirrabooka Aquifer with hydraulic conductivity reducing with depth.

Parameters are intended to be used for groundwater seepage analysis. Consolidation analysis may require refined values for Swan River Formation and Perth Formation (clay), refer to the GIR (Golder, 2015c).

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5.8 Groundwater Quality

This section only provides a general overview and high level discussion on groundwater quality to assist with the hydrogeological conceptualisation and general discussion on dewatering discharge disposal options. The overview relates to groundwater samples taken at the beginning and end of the pumping tests (Appendix E) as well groundwater quality results from the Environmental Site Investigation (Golder, 2015h). Given that the results from the pumping tests represent a more average value of the aquifer tested, more emphasis has been put on the pumping test results in this report. For specific and detailed groundwater quality information along the alignment, the Environmental Site Investigation (Golder, 2015h) should be referred to.

The following summarises the general groundwater quality data along the alignment:

- pH generally ranged between 5.5 and 8.1 indicating between slightly acidic and alkaline conditions. For the pumping tests the higher pH (alkaline) was observed at the AWS and CATS, which is not surprising given that the pumping wells were pumping for the carbonate rich Ascot Formation. The lowest pH (acidic) was observed in Forrestfield where the field average pH was 5.7 while the average pH in the Lower Aquifer during the pumping test was 7.1.
- Groundwater range between fresh and brackish with highest salinity observed in the Swan River Formation in the vicinity of the Swan River. The Total Dissolved Solids (TDS) values ranged between approximately 250 mg/L (Lower Aquifer at BDS) and 750 mg/L (AWS) during the pumping tests, indicating fresh water. The salinity was generally found to remain stable during the pumping tests.
- Some metal concentrations were generally identified above detection limits at all locations. Total iron, which is one of the key parameters to dewatering discharge disposal and effectiveness of recharge wells, was found to range between 0.1 mg/L and 22 mg/L. The highest iron concentration was measured in the Lower Aquifer at the end of the Bayswater pumping test and the concentrations was found to increase by almost an order of magnitude between the start and end of the pumping test. The total iron concentration during the three other pumping tests ranged between 0.03 mg/L (Forrestfield) and 2.8 mg/L (AWS) with similar concentrations at the beginning and end of the tests.
- Nutrient (as total nitrogen), another key parameter for dewatering discharge disposal options, was found to range between 0.1 mg/L and 66 mg/L, with the highest concentrations observed in the Guildford Formation at Emergency Egress Shaft Abernethy Road. Similarly, the highest nutrient concentrations during the pumping tests were observed in Forrestfield (33 mg/L to 44 mg/L), while at the remaining pumping test locations total nitrogen ranged between 0.1 mg/L (CATS) and 0.78 mg/L (AWS).
- Total alkalinity, which indicates the buffering capacity of the water, ranged between 20 mg/L and 390 mg/L, lowest in Forrestfield and highest at AWS. The higher alkalinity concentrations were not surprisingly found in areas where Ascot Formation is present, while the lower total alkalinity was found in the Guildford Formation. The results indicate that the water in the Guildford Formation could have insufficient buffering capacity to maintain a stable pH (DEC, 2011).
- The following Contaminants of Potential Concern (CoPC) were encountered during the environmental groundwater quality sampling (Golder, 2015h):
  - Hydrocarbons (at 26 sample locations), chloroform (at 3 sample locations) and cyanide (Bayswater only) were found to be above detection levels in some groundwater samples.
  - PFOS (a fire fighting chemical) was found in the groundwater (at 5 sample locations) between AWS and Emergency Egress Shaft Domestic Airport Terminal.

The above CoPC (chloroform was not analysed) were not detected in any of the samples collected during the pumping tests except for Cyanide, which was above the detection limit in the sample taken at the beginning of the pumping test, but was subsequent below the detection limit in the sample taken at the end of the pumping test.

The groundwater quality will have an effect on the dewatering discharge treatment requirements and disposal options as discussed in Section 6.6.

The contamination monitoring undertaken as part of the CBSP investigation indicate that the groundwater quality in the Upper and Lower Aquifers at Bayswater are different, with nutrient and metal concentrations in the Upper Aquifer being up to an order of magnitude higher than in the Lower Aquifer (Australian Environmental Auditors, 2013). This large difference in groundwater quality between the aquifers suggests that the Upper Aquifer and Lower Aquifer are disconnected, which is supported by the high head difference in groundwater level (Section 5.3.5).

6.0 DEWATERING

This section discusses the construction dewatering requirements for the four main structures; Bayswater Dive Structure, Airport West Station, Consolidated Airport Terminal Station and Forrestfield Dive Structure and outlines preliminary dewatering rate estimates, dewatering methodologies and groundwater disposal options and constraints. The section is based on PTA's Reference design Rev F, a document titled Reference Design Summary Table RevB provided by PTA and assumed construction methodologies and cut-off wall elevations as outlined in this section.

6.1 Dewatering Requirements

Table 5 presents the construction details and dewatering requirements for the structures considered in this study based on the Rev F alignment and plan and profile drawings.

Table 5: Construction Details and Dewatering Requirements

Structure	Approximate Excavation Dimensions (m)			Required GWL Drawdown** (m)	Dewatering Period (weeks) ***	Geological Unit in which D-wall Terminates
	Length	Width	Depth*			
Bayswater Dive West of BMD	270	10	0.0–7.8	0.0–4.0	16	BS and PF
Bayswater Dive East of BMD	230	11–20	7.8–12	2.9–7.1 #		BS and PF
Bayswater Retrieval Box	20	20	12	7.1 #		PF
Airport West Station	156	27	12	12.1	52	OF (OFs)
Consolidated Terminal Station	160	28–46	17	16.2	52	OF (OFm)
Forrestfield Retrieval Box	20	20	10	5.5	16	GF
Forrestfield Dive Structure	300	10–24	0-10	0.0–5.5		

Notes: * Depth is below current ground surface, GWL = Groundwater Level, BMD = Bayswater Main Drain
** Estimated drawdown from groundwater level to 1 m below the base of excavation based on an estimated average seasonal groundwater level.
*** Total period during which dewatering is required.
# Required drawdown in the Lower Aquifer

The Reference Design Revision F indicates that D-walls are planned to form the four main structures. This approach significantly reduces the dewatering requirements because horizontal groundwater inflow to the excavation (except for any leakage through the D-wall) is essentially cut off. Accordingly, the groundwater inflow into the excavation becomes predominantly vertical where groundwater flows under the D-walls and up into the floor of the excavation.

The dewatering requirements and effect of dewatering (rates and extent of groundwater level drawdown) is dependent upon the wall depth and the geological unit in which the D-wall terminates. The analysis reported here is based on PTA's Reference Design Summary Table RevB (which is based on the Rev F Reference design):

- The D-walls on all sides will terminate 3 m into the Osborne Formation at the Airport West Station and Consolidated Airport Terminal Station (including the concourse level box).
- At the Bayswater and Forrestfield Dive Structures the D-Walls will extend to a depth that is between about 1.9 and 2.3 times the depth of excavation, respectively.

Bayswater Dive Structure

At Bayswater Dive Structure dewatering will be required from both the Upper Aquifer and Lower Aquifer as follows:

- West of the Bayswater Main Drain excavation and dewatering will only be required from the Upper Aquifer.
- East of the Bayswater Main Drain dewatering will only be required from the Lower Aquifer, because the D-walls will extend into the PFc and Lower Aquifer and thereby cut-off any horizontal groundwater inflow into the Upper Aquifer (apart from any leakage) and any flow down the open decline between the walls.
- At the deepest part of the dive and tunnel portal the D-walls will extend into the Osborne Formation, which will significantly reduce the groundwater inflow from the Lower Aquifer into the excavation. Nevertheless, the greatest dewatering is expected to be required from the Lower Aquifer at the Bayswater Retrieval Box where the piezometric level would need to be lowered by up to 7.1 m.

Airport West Station

The D-Walls will extend into the Kardinya Shale (KS) of the Osborne Formation and thereby cut off direct groundwater inflow from the Superficial Aquifer. Given the low hydraulic conductivity (0.0005 m/d) of the KS, dewatering would be required to initially remove stored water within the soils inside the excavation and thereafter remove any upward seepage through the Kardinya Shale and leakage through the D-walls. The dewatering requirements for the Airport West Station should therefore be relatively low.

Consolidated Airport Terminal Station

The same concept as for Airport West Station applies to the Consolidated Airport Terminal Station where the Superficial Aquifer is cut-off and seepage into the excavation can only occur through upward seepage from the Osborne Formation and leakage through the D-walls. However, given that the Osborne Formation below the Consolidated Airport Terminal Station consists of the Mirrabooka Member which has a hydraulic conductivity that is 4 orders of magnitude higher than the Kardinya Shale, the groundwater inflow into the excavation will be significantly higher at this station.

Forrestfield Dive Structure

The aquifer at the Forrestfield Dive Structure is comprised of the Guildford Formation within which the D-walls will terminate. Groundwater inflow into the excavation will occur from the open end of the dive unless slurry wall cut-offs are deployed and vertically upward from below the excavation between the D-walls. The most dewatering for this structure will be required at the Forrestfield Launch Box where the groundwater would need to be lowered by up to 5.5 m.

Other Structures

Construction dewatering will be required for other structures such as the emergency egress shafts. These structures have not been assessed within the scope of this current study, but will need to be addressed in the detailed design.

6.2 Dewatering Methodology

The dewatering methodology for this project will depend on the specific site and dewatering requirements, but is likely to consist of a combination of:

- **Dewatering Wells** – drilled wells with screens and associated gravel packing placed around the internal perimeter of the excavation between the D-walls.
- **Sump Pumping –** strategically placed dewatering sumps or trenches (drains) within the excavation where groundwater is removed using sump pumps.
- **Dewatering Spears (Well Points) –** shallow spears could be used to lower the groundwater level locally or where the saturated thickness of the aquifer is limited.

The use of dewatering wells and sump pumping is considered the preferred option for all four structures. However, the use of dewatering spears could be advantageous for dewatering of the Upper Aquifer at the Bayswater Dive Structure where the saturated aquifer is of limited thickness and dewatering wells will be less effective. However, dewatering wells will still be required to dewater the Lower Aquifer.

The dewatering wells must be placed inside the excavations to minimise the dewatering rates and drawdown in the aquifers. The number and spacing of the dewatering wells have not been evaluated as part of this work; this will need to be done as part of a detailed dewatering design once the methodology and sequence for the overall construction of each structure is known.

Sump pumping from inside the D-wall supported excavations is likely to be required to remove residual shallow groundwater not drained by the dewatering wells or to control surface water from rainfall events. Some local shallow dewatering may be required in the Forrestfield car park area for site preparation and/or installation of services, etc. Due to the expected limited groundwater level drawdown requirements, the dewatering methodology for this area would likely consist of a combination of sump pumps and dewatering spears.

6.3 Groundwater Modelling

6.3.1 Groundwater Modelling

The 3D groundwater modelling software Visual MODFLOW 2011.1 Premium (numerical finite difference groundwater flow modelling software used extensively throughout the world) was used to simulate existing groundwater conditions along the alignment and evaluate preliminary dewatering rates and the effect the construction dewatering could have on the groundwater level and piezometric levels. Appendix F provides details on the construction and assumptions used in the 3D numerical groundwater model.

6.3.2 Model Scenario Runs

Table 6 presents a summary of the scenarios modelled.

Table 6: Model Scenarios.

Structure	Recharge of Dewatering Discharge	Scenario	Comment
Bayswater Dive Structure	No*	1A	k_v/k_h ratio of 0.1 in Upper and Lower Aquifer
		1B	k_v/k_h ratio of 1 in the Upper and Lower Aquifer
Airport West Station	No	2A	D-walls extend into Osborne Aquitard resulting in limited dewatering (i.e. recharge wells not likely to be required or preferred disposal option)
Consolidated Airport Terminal Station	Yes	3A	k_v/k_h ratio of 0.1 in all layers
		3B	k_v/k_h ratio of 1 in Mirrabooka Aquifer
Forrestfield Dive Structure	Yes	4A	k_v/k_h ratio of 0.1 in all layers
		4B	k_v/k_h ratio to 1 in in all layers

* Estimated average dewatering rates are low and there is minimal drawdown effect in the Upper Aquifer, suggesting that recharge wells may not be required.

Given that installation of D-walls would result in mainly vertical flow upwards into the excavations once dewatering begins, the vertical hydraulic conductivity of the hydrogeological units in which the D-walls terminate will have a significant influence on the dewatering rates and extent of groundwater level drawdown outside the excavation. Sensitivity runs (scenarios with the suffix "B") were therefore carried out where the k_v/k_h ratio has been increased 10 times (i.e. vertical hydraulic conductivity is the same as horizontal hydraulic conductivity) in the hydrogeological units into which the D-walls are installed. At Airport West Station the hydraulic conductivity of the KS is so low that a change k_v/k_h ratio or a change of one order of magnitude in hydraulic conductivity would not noticeably change the dewatering rates or the groundwater level drawdown. A sensitivity run was therefore not performed for Airport West Station.

A preliminary dewatering assessment carried out by Golder in 2014 (Golder 2014) indicated that on-site recharge of the abstracted groundwater would be required to minimise and manage the groundwater level caused by the groundwater abstraction. Recharge wells were therefore installed around the excavations in the models to conceptually show the effect of the wells. However, the actual number and location of the recharge wells and distance from the excavation area will need to be assessed during the detailed design phase. The recharge wells in the models were generally screened across the whole Superficial Aquifer to a similar depth as the D-walls (wells were not modelled deeper than the D-walls as this could further promote groundwater flow back into the aquifer, resulting in increased dewatering rates).

Based on recent experience from projects where recharge wells have been used in the Perth Formation for dewatering disposal, the following assumptions were included in the model:

- The long-term recharge rate of a recharge well would not exceed 1 L/s
- Approximately 85% of the abstracted groundwater can be introduced back into the aquifer, which is considered a more realistic scenario than assuming a perfect system with 100% recharge of the abstracted water. With 85% recharge the model takes into account maintenance of recharge wells, short-term temporary shutdown, etc. However, the amount of abstracted water that is returned back into the aquifer could have an effect on the extent of the groundwater level drawdown around each excavation and it is therefore recommended that the dewatering system design and the aim during operation should be to achieve as high a recharge percentage as possible.

At Airport West Station the removal of storage inside the excavation (main dewatering requirement) would not result in a lowering of the groundwater level outside of the excavation and recharging this water back into the aquifer would therefore not be required. Given the expected low seepage rates at Airport West Station after the initial pumping of stored water, recharge wells are not likely to be the most efficient or preferred disposal option and therefore the abstracted groundwater was not recharged back into the aquifer in the Airport West Station scenario run (Scenario 2A).

At Bayswater Dive Structure the D-walls will cut-off groundwater inflow into the Upper Aquifer except for limited inflow down the open decline between the walls. The D-walls will extend into the Osborne Formation at the tunnel portal where the deepest excavation is required and therefore dewatering from the Lower Aquifer will be limited. Given that the groundwater model results indicate that the average dewatering rates will be less than 3 L/s, the abstracted groundwater was not recharged back into the aquifer in the Bayswater Dive Structure scenario runs.

6.4 Dewatering Rates

Table 7 presents the estimated dewatering rates and volume for the four main structures for the different scenarios and indicates:

- The dewatering rate at Airport West Station will be limited if the D-walls extend into the Kardinya Shale (Osborne Formation). The model run indicated that of the estimated 3 L/s, less than 1 L/s was attributed to seepage through the Kardinya Shale while the rest was due to estimated leakage through the D-walls.
- The modelled low dewatering rate for the Bayswater Dive Structure is expected, given that the pumping test (at 12 L/s) was able to lower the piezometric pressure in the Lower Aquifer by up to 9 m in the vicinity of the well (only 7 m is required during construction) and by up to 1.5 m at a distance of 500 m from the well. Given that D-walls will be used to significantly limit groundwater inflow into the excavation, the construction dewatering rates would be expected to be significantly lower than during the pumping test.
- The modelled dewatering rates at the Consolidated Airport Terminal Station are based on the D-walls on all sides terminating 3 m into the Osborne Formation, including the concourse level box. Should the D-walls surrounding the concourse level box terminate in the Ascot Formation, the dewatering rates would increase.
- The vertical hydraulic conductivity has a significant effect on the dewatering rates with an increase of the average dewatering rate of 1.5 times to 2.6 times for the Consolidated Airport Terminal Station and Forrestfield Dive Structure, respectively when k_v was increased.

Table 7: Modelled Dewatering Rates and Volume

Location	Scenarios	Dewatering Rate (L/s)		Dewatering Volume (kL)
		Initial*	Average	
Bayswater Dive Structure	1A	8	3	39,000
	1B (sensitivity analysis)	8	4	42,000
Airport West Station	2A	9	3	91,000
Consolidated Airport Terminal Station	3A	40	30	948,000
	3B (sensitivity analysis)	54	44	1,399,000
Forrestfield Dive Structure	4A	18	14	138,000
	4B (sensitivity analysis)	46	35	354,000

Notes: *Initial rate for the first 2 weeks to lower the groundwater level to required dewatering level

The depth and distance of the recharge wells would also have an effect on the dewatering rates due to the "recycling" effect of water back into the excavation from the recharge wells. The closer and deeper the recharge wells are to the excavation and bottom of the D-wall the greater the "recycling" effect and thereby dewatering rates would be greater.

6.5 Effects of Dewatering

The abstraction of groundwater during the construction dewatering will result in a lowering of the groundwater table and the piezometric pressure in the hydrogeological units. The effect will depend on the groundwater management practices and methods adopted during the dewatering operation and on the chosen disposal option.

Figures 6 to 11 show a plan and cross-sectional view of each main structure with the modelled groundwater drawdown contours at the end of the dewatering period for the different scenarios while Table 8 presents the distance to the contours of groundwater table drawdown and change in piezometric level.

Table 8: Modelled Extent of Groundwater Level Drawdown (m)

Scenarios	Groundwater Table			Piezometric Level			
	0.1 m	0.5 m	1.0 m	0.1 m	0.5 m	1 m	Geological Unit
1A *	70	-	-	2,000	380	60	PFs
1B *							
2A *	300	40	-	300	50	-	AF/OFs
3A	430	-	-	450	150	100	OFm
3B	590	-	-	590	170	110	AF/OFm
4A	210	-	-	210	60	25	GF
4B	500	170	65	490	170	70	GF/BS

Notes: *No recharge wells.
The reported distances (in metres) are from the external wall face to the groundwater level/piezometric contour

The results in Table 8 and Figures 6 to 11 indicate:

- **Groundwater table:**
  - Groundwater table drawdown of greater than 1 m does not occur in any of the modelled scenarios indicating the recharge wells are capable of reducing the groundwater table drawdown to within the SWTC requirements, except for Scenario 4B in Forrestfield where the groundwater table drawdown of 1 m extends approximately 65 m from the D-walls.
  - The groundwater table drawdown of 0.5 m is predicted in Scenario 2A (AWS) and 4B (Forrestfield Dive Structure) at a distance of 40 m and 170 m, respectively.
  - The difference in drawdown between Scenarios A and B for all structures indicates that the drawdown will extend further if the k_v/k_h ratio is 1 (compared to 0.1).
  - Groundwater mounding of up to 0.2 m is predicted around the recharge wells.
  - At Airport West Station the difference in drawdown shape between north and south (Figure 7B) is due to the presence of the Belmont Main Drain which reduces the extent of the groundwater level drawdown toward the south (i.e. Belmont Drain acts a positive flow boundary at the water table).
- **Piezometric level:**
  - The drawdown in piezometric levels extends further than the drawdown in the groundwater table. The drawdown is expectedly greatest in the hydrogeological unit where the D-walls terminate as this is the unit where the greatest groundwater inflow into the excavation occurs.

- For the Consolidated Airport Terminal Station the modelled drawdown in piezometric levels in the Mirrabooka Aquifer do not propagate to the ground surface (Figures 8B and 9B), which is due to a combination of the effect of the recharge wells and the presence of the more permeable Ascot Formation above the Mirrabooka Aquifer. The effect that Ascot Formation alone has on the shape of the piezometric level contours can be seen for the Airport West Station scenario (Figure 7) where no recharge wells are modelled.
- The difference in drawdown between Scenarios A and B for all structures except Bayswater Dive Structure indicates that the drawdown will extend further if the k_v/k_h ratio is 1 (compared to 0.1). The reason this effect is not noticeable at the Bayswater Dive Structure is that the groundwater inflow into the excavation is mainly horizontal due to the D-walls extending into the Osborne Formation along three sides of the excavation.

The results from the preliminary groundwater modelling indicate that it should be possible to design a dewatering system incorporating recharge wells that would limit the drawdown in the groundwater table outside each excavation to less than 1 m in accordance with SWTC requirements, particularly at the two Airport Stations where the D-walls extend below the Ascot Formation. At Forrestfield, the modelling indicates that greater than 85% of pumped groundwater would need to be recharged to meet the SWTC requirements.

6.5.1 Groundwater Quality

Normally the greatest concerns of the effects that construction dewatering could have on groundwater quality is related to the presence of potential ASS within the groundwater table drawdown cone of depression and mobilisation of contaminated water through the change in groundwater flow direction.

Drawdown of the groundwater table surrounding a site can lead to oxidation of ASS resulting in sulfuric acid being formed which would result in release of metals, nutrients and acidity into the soil and groundwater system (DEC, 2011). This risk can be minimised by reducing the groundwater table drawdown. However, increasing oxygen concentrations through aeration of abstracted groundwater during dewatering could potentially also cause oxidation of ASS when the groundwater is recharged back into the aquifer. This would need to be considered during the design of the dewatering and recharge system.

The results from the preliminary groundwater model indicate that it would be possible to design a dewatering system that would limit changes in groundwater flow direction and thereby limit the risk of mobilisation of contaminated water.

6.5.2 Surface Water

The results from the preliminary groundwater model indicate that it would be possible to design a dewatering system that would not affect the surface water bodies discussed in Section 5.5.1.

The model results indicate that the Belmont Main Drain will act as a positive boundary (i.e. water will flow from the drain into the water table aquifer to keep the groundwater table steady at the drain). Given that this man-made drain was constructed to lower the groundwater table, this flow is currently not considered to be an issue. It is possible that some of the Perth Airport drains could also be affected during the dewatering of the Consolidated Airport Terminal Station.

6.5.3 Means of Reducing Effects

If the piezometric level drawdown would need to be further reduced below the drawdown modelled in the current scenarios, recharge wells could be installed to a greater depth (i.e. below the D-walls), which should reduce the piezometric level drawdown outside of the D-walls. However, this deepening of recharge wells is likely to result in an increase in dewatering rates as the "recycling" effect would become greater.

If groundwater monitoring during the dewatering operation shows a greater than expected effect on the groundwater level and piezometric level, additional recharge wells could be installed between the excavation and the area of concern (e.g. a row of recharge wells could be installed adjacent to the airport control tower, if this was required).

6.6 Dewatering Disposal Options

Dewatering discharge should be disposed of so as to not cause harm to the environment or allow flooding.

The SWTC specifies that the groundwater levels in all aquifers within 500 m of the dewatering activity, unless otherwise specified, must not be elevated/lowered by more than 1 m during the project works when compared to locally recorded seasonal highs/lows in that aquifer. Golder considers that to achieve these criteria the use of onsite recharge wells is likely to be the most feasible dewatering disposal option, where dewatering is required.

Any pumped groundwater that is in excess of that required to be recharged to meet the above SWTC criteria could be disposed of by alternative methods. The following dewatering discharge disposal options (in addition to recharge wells) have been identified:

- Reuse on site
- Infiltration
- Stormwater system
- Sewer.

These options would also serve as contingency disposal options to the recharge well scheme.

6.6.1 Recharge Wells

The advantage of using recharge wells is that the abstracted water will be reintroduced back into the aquifer within the vicinity of where it is abstracted, which will result in a much lower net withdrawal of groundwater and thereby minimise the effect the dewatering will have on changes to the groundwater level and flow direction. Recharge wells can also be placed strategically between the dewatering area and sensitive structures and environments to control the groundwater levels and thereby groundwater flow direction.

The performance of the recharge wells would depend on the geological units they are screened in and the water quality of the discharged water. From experience in the Perth Metropolitan Area it is not uncommon to assume that the long-term recharge rate of a recharge well installed in the PF and GF would not exceed 1 L/s. However, the recharge well capacity would depend on the aquifer transmissivity and it is expected that higher recharge rates may be achievable for wells installed in the AF and the sandy GF in the Forrestfield area.

The dewatering discharge quality, particularly high iron concentrations, could significantly limit the long-term performance of the recharge wells and therefore water treatment and ongoing maintenance (cleaning) of the dewatering discharge system is likely to be required to minimise the risk of iron clogging of screens.

The SWTC specifies specific recharge system requirements.

6.6.2 Reuse on Site

Reuse of abstracted groundwater as a means for disposal will be dependent on the demand of the various contractors on site and the dewatering discharge quality and may include use for dust suppression or water supply to the TBM. The groundwater quality would need to be assessed at each specific site location. Site specific groundwater quality is detailed in the Environmental Site Investigation (Golder, 2015h).

6.6.3 Infiltration

The infiltration capacity of the soils at the dive and station locations is discussed in detail in Section 8.0. The viability of shallow infiltration (as opposed to deeper recharge) as a disposal option depends on the following factors:

- The area available close to the excavation where an infiltration basin can be constructed. The greater the area, the higher the achievable infiltration rate.

- The depth to groundwater table. Infiltration capacity reduces significantly where the groundwater table rises to the base of the basin in response to infiltration.
- The permeability of the subsurface profile at the infiltration basin location. The presence of layers of clay or fines can significantly reduce the infiltration capacity.

The shallow depth to groundwater at all four major structure locations is considered a significant restriction on infiltration capacity and therefore on infiltration as a disposal option. Based on infiltration testing (refer to Section 8.0), infiltration may not be a feasible option at Forrestfield and limited infiltration capacity would also be expected at the Bayswater Dive structure, Airport West Station and Consolidated Station.

6.6.4 Stormwater System

Disposal of dewatering discharge into the local stormwater drainage system will ultimately result in the dewatering effluent discharging into either the Bayswater or Belmont Main Drains and ultimately into the Swan River. The groundwater quality results indicates that some treatment of the dewatering discharge would be required prior to disposal to the stormwater system, which could include adjustment of pH and removal of some metals and nutrients to meet the Swan River Trust (SRT) guideline criteria for disposal to the Swan River.

Prior to commencement of dewatering, approval to discharge the abstracted groundwater into the stormwater system would need to be obtained from the local council (various) and asset owners (Water Corporation operates the Belmont and Bayswater Main Drains while PAPL operates the Perth Airport drain system). The asset owner is unlikely to provide access to their system before necessary regulatory approval has been obtained for discharge of water into the end environment (likely to be Department of Environment Regulation (DER) and SRT). This option is likely to require rigorous water treatment and monitoring requirements.

During the pumping tests at Forrestfield, Consolidated Airport Terminal Station and Airport West Station the abstracted groundwater was discharged into the stormwater drains at rates of up to 12 L/s after receiving approvals from Shire of Kalamunda (for Forrestfield), PAPL (Consolidated Airport Terminal Station) and City of Belmont, the Water Corporation and SRT (for Airport West Station). Given the short-term disposal duration approval was obtained without having to the treat the water for metals and nutrients. However, additional water monitoring was undertaken from the Belmont Main Drain during the pumping test at the Airport West Station as requested by the SRT. This data could be utilised for future applications if regulatory approval to discharge into the stormwater system at the Airport West Station will be sought.

It is important to note that the regulators will assess the disposal options based on a site specific risk assessment, which would require site specific water quality data as well as information on the estimated flow rates, duration of dewatering and the time of year that the dewatering is to be carried out.

6.6.5 Sewer

For discharge to sewer the following should be considered:

- Approval would be required from the Water Corporation by lodging a “one-off discharge of industrial waste”.
- Proximity to current sewer locations suitable for discharge will need to be considered and discussed with the Water Corporation.
- Disposal is normally restricted by the sewer capacity, which would need to be discussed with the Water Corporation and would therefore depend on the expected discharge rates and volumes. This could, therefore, be a limitation as dewatering rates may be higher than the available capacity.
- This option is unlikely to require on-site treatment, though it may require pH adjustment (criteria is pH 6 to pH 10) and removal of Total Suspended Solids (TSS) by installing a settling trap (mandatory requirement). To prevent escape of sewer gases to the environment, a water trap would also need to be installed between the settling trap and the sewer connection point.

There is a unit cost (per kL) for disposal to sewer, which would depend on the water quality.

7.0 DESIGN GROUNDWATER LEVELS (50 AND 120 YEAR)

This section presents the estimated maximum design groundwater level along the alignment for 50 and 120 year design periods with particular emphasis on Bayswater Dive Structure, Airport West Station, Consolidated Airport Terminal Station and Forrestfield Station.

At Bayswater where there is a dual aquifer system, the estimated design groundwater level refers to the maximum groundwater level in the Upper Aquifer.

At Forrestfield the presence of low permeability material above the regional groundwater level has been found to cause perched groundwater conditions in some areas, with the perched groundwater levels locally found to be several metres higher than the regional groundwater level following rainfall. These perched conditions may be seasonal or may occur year-round (Section 5.3.6). For Forrestfield both regional and perched design groundwater levels have been discussed.

7.1 Methodology

We have taken a pragmatic approach to the estimation of the design groundwater levels, where the present and historic groundwater levels, hydrogeological features and key parameters that would affect the groundwater level are closely considered.

The installed groundwater monitoring network along the alignment provides a comprehensive record of present-day groundwater levels for developing current groundwater level contours and allowing direct comparison with historical records.

7.1.1 Key Parameters

The identified key parameters that could have an effect on the maximum groundwater levels are:

- Change in Climate
- Geology
- Drains
- Groundwater Abstraction.

Change in Climate

As outlined in Section 4.2, Perth has experienced a drying climate over the last 40 years, which has become even more severe over the last 20 years. The reduction in rainfall is generally known to have resulted in a decline in groundwater levels over large parts of the Perth Metropolitan Area. However, the currently available groundwater level hydrographs for various locations along the alignment do not indicate any significant decreasing trend in the groundwater level, even with the drying climate, which suggests that the current lower annual rainfall is still sufficient to fill the aquifer in the study area (Section 5.2.2).

A flood modelling study was carried for the FAL project (BG&E, 2015), which included an extensive review of available information on climate change scenarios. The main overarching conclusions from the reviewed information suggests that there will be an increase in sea level (between 0.5 m and 1.1 m) and that the rainfall intensities will increase even though the annual rainfall trend in the western region of Australia could decrease (i.e. the climate could become drier, but storm events could become more severe). The rise in sea level would result in a rise in the water level in the Swan River, which would result in an increase in groundwater level in the vicinity of the Swan River. The more severe storm events could potentially increase short-term rises in groundwater levels during the wet season, particularly in areas with thicker unsaturated zones.

The climate changes are currently not expected to have a major effect on the future groundwater levels along most of the alignment except for an increase in groundwater level at the Swan River due to global sea level rise. A lower future annual rainfall could result in decreasing groundwater levels, while an increase in the rainfall to pre-1960's rainfall volumes could result in increasing groundwater levels.

Geology

Over the majority of the alignment the top soil consists of sandy soils promoting rainfall recharge. However, the geotechnical and environmental investigation results indicate that clay or low permeability material is present within less than 1 m from the ground surface in the Forrestfield area (along the tunnel alignment and in the associated car park). This low permeability material has been found to cause perched groundwater conditions in some areas with groundwater level being close to the surface. The groundwater level could possibly even reach the ground surface or cause local inundation in some local areas after large rainfall events (which could be monitored in the future during the wet season). Therefore, in the Forrestfield area the geology is causing locally perched aquifer conditions.

If climate change would result in more severe storm events, this could result in more frequent and severe inundation in the Forrestfield area.

Drains

A large network of man-made drains exists over the study area and along the alignment. The main drains are described in Section 5.5.2. The main purpose of the drainage networks is to lower the groundwater level in the full aquifer to allow for land development in the area. The drains were installed progressively from the 1950's to the 1970's (see the aerial photography review presented in (Golder, 2015c) resulting in lowering of the groundwater level.

The existing drainage networks are considered to drain the groundwater table, particularly during the wet season and thereby control the maximum groundwater level in the area. The drains are therefore currently considered to be the main parameter that determines the maximum groundwater level in the area.

Groundwater Abstraction

The main groundwater abstraction along the tunnel alignment is considered to be for reticulation (residential, public parks and Perth Airport) and for industrial purposes. No large scale groundwater abstraction has been identified in the area. The VWPs are showing temporary (daily) effects of groundwater level drawdown from reticulation wells, but no long-term decreasing trends are observed in any wells with historical groundwater level data that have been reviewed during this study (Section 5.2.2). In addition, groundwater abstraction for reticulation purposes would be highest during the dry season which is when the groundwater levels are lowest.

Groundwater abstraction is therefore currently not expected to have a major effect on the design groundwater levels.

7.1.2 Design Groundwater Level Estimation

7.1.2.1 50 Year Design Groundwater Levels

The following data was utilised to generate the 50 year design groundwater levels:

- Groundwater levels from monitoring wells and VWP's installed along the alignment during the geotechnical and environmental investigation.
- Historical hydrographs available from DoW, PAPL and the former CSBP Bayswater site (Parsons Brinckerhoff, 2013).
- Drain and wetland elevations estimated from topographic contours (1 m interval) from Landgate.
- Information on global sea level rise.

Based on the analysis of the key parameters it is considered that the existing man-made drainage network will control the maximum groundwater level over a large part of the alignment in the future. It is therefore important that the drainage network is incorporated into the estimation of the design groundwater level.

The design groundwater level contours for the study area have been estimated by contouring groundwater level interpolation points created as follows:

- **Drainage system** – The invert levels for the drainage system were obtained from 1 m topographic contours. It was assumed that the groundwater level will be up to 1.0 m above the invert levels during the wet season (i.e. 1 m was added to invert levels) based on observations at Belmont Main Drain. It has been assumed that all existing drains will remain in operation throughout the design period.
- **Wetlands and lakes –** Aerial photography was utilised together with surface contours to estimate the water levels.
- **Swan River Level Rise –** A rise in average Swan River water level due to global sea level rise of 0.4 m (from RL 0.2 m AHD to RL 0.6 m AHD).
- **Wells with historical hydrographs** – The maximum historical groundwater level was estimated for each well based on the groundwater levels measured over the period of monitoring (generally up to 15 years for PAPL wells, 8 years for CSBP wells and up to 60 years in the DoW wells).
- **Wells with no historical hydrographs (new wells installed along the alignment) –** The March 2015 measured groundwater level was used as a starting point. The groundwater level difference between the estimated maximum groundwater level and the March groundwater levels in nearby wells with historical hydrographs was then added to the March 2015 groundwater levels. This process assumes that similar historical trends would be exhibited by nearby wells. The difference between the maximum and March 2015 levels was found to vary along the alignment between +0.8 m and +4.0 m.

The interpolation points resulting from the above approach were then manually contoured. The contours are based on currently available data. If additional information (e.g. if sub-soil drains exist beneath car parks or roads, or additional monitoring wells) becomes available or if existing drains are infilled, the design groundwater level would need to be reassessed and adjusted accordingly.

7.1.2.2 120 Year Design Groundwater Levels

The derived 50 year design groundwater level was utilised as the base for generation of the 120 year design groundwater level. The following assumptions were made to develop the 120 year design groundwater level:

- The Swan River average level will rise by another 0.4 m to RL 1 m AHD
- The groundwater level would rise by 0.5 m over the whole alignment except for:
  - Where the Bayswater and Perth Airport drains cross the alignment, the level was only increased by 0.25 m
  - Along CH3400 to CH4000 the level was only increased by 0.25 m because of the presence of the Belmont Main Drain
  - In the Forrestfield area where an increase of 1 m was adopted due to the available unsaturated thickness and high seasonal variation due to the presence of lower permeability soils.

7.2 Design Groundwater Level

Table 9 summarises the design groundwater level at the stations, dive structure and Forrestfield car park while Figure 12 shows the estimated design groundwater levels over a long-section along the tunnel alignment together with the current ground surface. The results indicate:

- The depth to the design groundwater levels from the current ground surface ranges between 0 m and 5 m with the design groundwater level being within 2 m of the ground surface over the majority of the alignment.
- The groundwater level reaches the surface at several locations, which typically corresponds to where drains were previously installed.

Table 9: 50 and 120 Year Design Groundwater Levels

Location	50 Year Design Groundwater Level			120 Year Design Groundwater Level		
	Range	Average	Average Depth to Groundwater Level	Range	Average	Average Depth to Groundwater Level
	m AHD		m	m AHD		m
Bayswater Dive Portal	5.9–6.2	6.1	<1	6.4–6.7	6.6	<0.5
Airport West Station	9.3–9.9	9.6	1.3	9.5–10.2	9.9	1.0
Consolidated Airport Terminal Station	18.2–18.4	18.3	<1	18.7–18.9	18.8	<0.5
Forrestfield Dive Portal	27*	27*	3	28*	28*	2
Forrestfield Station	27*	27*	2	28*	28*	1
Forrestfield Car Park	27–34 *	NA	2-3	28–35*	NA	1–2

Notes: *Regional Aquifer – Perched groundwater level conditions exist in this area (see Section 7.2.1).
**Range refers to change in groundwater level along the length of the structure.
NA = average is not applicable due to the large area of the car park.

It should be noted that the design groundwater levels over parts of the alignment are lower than the maximum inferred groundwater level contours in the 1997 Perth Groundwater Atlas (WRC, 1997). The main reason for this difference is that the inferred maximum groundwater level contours in the 1997 Atlas include data that dates back to before some of the drains were installed.

7.2.1 Perched Aquifer at Forrestfield

Figure 5 shows the area in Forrestfield where shallow monitoring wells indicate that perched groundwater is either present or is considered likely to occur during the wet season. As described in Section 5.3.6 the differences in groundwater levels measured in shallow and deep wells have been found to vary between 0.3 m and 3.3 m, suggesting that the level of perching is locally highly variable, which is likely because of localised and discontinuous bands/layers of low permeability clay within the Guildford Formation soils. Given that the differences are based on groundwater levels measured toward the end of the dry season, these differences are likely to further increase during the wet season.

With the currently available groundwater level information it is not possible to accurately establish a 50 or 120 year design groundwater level surface for the perched groundwater. It is considered possible that perched groundwater in some areas could reach the ground surface or cause local inundation after large rainfall events. Further groundwater level monitoring would be required to establish the extent of the perched groundwater level areas and the connectivity/separation of the perched groundwater levels and the regional aquifer.

Some of the shallow environmental monitoring wells were reported to be dry during installation. It is recommended that these wells should still be part of the monitoring program in the wet season and following rainfall events to assess the possible perched groundwater level conditions in the area.

7.3 Risks Associated with Design Groundwater Levels

The following risks have been identified for the estimated maximum design groundwater levels:

- Underground structures will need to be designed to resist forces due to buoyancy (uplift) and hydrostatic pressures as well as to resist the ingress of water through walls of the structure.
- The river flood level may cause local short-term changes in groundwater level within and immediately adjacent to the Swan River flood plain. The design of any structures within this zone must consider the river flood level.
- Due to the groundwater level being within a few metres from the ground surface, the infiltration capacity for disposal of stormwater is considered to be low over majority of the alignment and at Bayswater Dive, Airport West Station and Consolidated Airport Terminal Station. Though the depth to the regional groundwater level in the Forrestfield area ranges between 3 m and 6 m, the presence of perched groundwater level conditions could also result in reduced infiltration capacity in this area.
- The presence of shallow perched groundwater and potential for groundwater inundation in the Forrestfield area could affect pavements and rail track subgrades if not managed appropriately during design and construction.
- Any facilities that will be constructed on or under the ground surface may require to be designed for moist or wet conditions (due to capillary rise).

7.4 Drainage Control

It is understood that PTA will require that all underground structures be 'tanked' which should eliminate any requirements for under-drainage systems below the base slabs.

To facilitate excavation and construction of the dive structures and stations, retaining walls will be required to be constructed around the perimeter of the structures. Reference Design Revision F indicates that the retaining structures will comprise D-walls. Depending on the depth, location and orientation in relation to groundwater flow, the installation of the D-walls could act as groundwater flow barriers, which could result in groundwater mounding behind the D-walls and may result in an increase in hydrostatic pressure against the D-walls. Therefore, permanent groundwater control in areas where groundwater mounding would occur may be required:

- ***Bayswater Dive Structure -*** The D-walls will extend through the Upper Aquifer into the Lower Aquifer, resulting in a permanent groundwater flow barrier in the Upper Aquifer where the walls are aligned obliquely or perpendicular to the groundwater flow direction:
  - East of Bayswater Main Drain – The groundwater flow is in a westerly to south-westerly direction toward the Bayswater Main drain, which could result in some groundwater mounding behind the northern D-walls.
  - West of Bayswater Main Drain – The groundwater flow is mainly in a north-easterly to easterly direction toward the Bayswater Main Drain and limited groundwater mounding could occur behind the southern D-walls.

  At the Bayswater Main Drain, the D-walls may need to be extended to below the invert level of the Bayswater Main Drain, which would require diversion of the Bayswater Main Drain. This diversion may require additional groundwater control (dewatering) during construction.
- ***Airport West Station and Consolidated Airport Terminal Station -*** The station boxes are generally located parallel to the groundwater flow direction from east to west and are therefore unlikely to significantly impede groundwater flow. However, some groundwater mounding could occur behind the eastern D-walls.

- ***Forrestfield Area –*** The D-walls associated with the Forrestfield Dive Structure will be constructed perpendicular to the groundwater flow direction. However, according to Reference Design Summary Table RevB (for Reference design Rev F) the D-walls will not extend through the full thickness of the Superficial Aquifer and it is therefore considered unlikely that the D-walls will significantly impede groundwater flow and result in significant groundwater mounding behind the up-gradient D-wall.

  The presence of perched groundwater in Forrestfield could result in some groundwater drainage control requirements at the Forrestfield Station and car park area. The required groundwater control could consist of the installation of sub-soil drains or granular drainage blankets.

It is not uncommon to have issues in the Perth Metropolitan Area with clogging of sub-soil drains due to high iron concentrations in the groundwater which precipitates out in the drain pipes and thereby reduces the efficiency of the system over time. The groundwater quality results from along the alignment indicate a large range in iron concentrations ranging between 0.1 mg/L and 22 mg/L, which is lower than compared to groundwater in other parts of Perth (e.g. up to 100 mg/L in Perth CBD). The risk of clogging would therefore depend on the site specific groundwater quality. Other reductions in efficiency of the pipes could be caused by scaling of the pipes due to calcium carbonate or gypsum, but this is normally more easily managed than iron precipitation. The design of a drainage system must consider that access is provided to the drain system to allow for periodic cleaning of the pipes and maintenance/replacement of pumps.

8.0 INFILTRATION CAPACITY

This section presents the estimated infiltration capacity based on infiltration tests carried out at Bayswater Dive Structure, Airport West Station, Consolidated Airport Terminal Station and the Forrestfield area. The development of the pumping test wells at the dive structures and two airport stations gave the opportunity for larger scale infiltration tests (infiltration in pits) and at Bayswater Dive Structure the water was also infiltrated on site using an infiltration basin during the pumping test.

The following infiltration tests were therefore carried out:

- **Inverse auger hole method –** A 75 mm diameter hole was drilled using a hand auger to install a 72 mm diameter slotted PVC casing which was filled with water and then the decline in water level was measured over time.
- **Infiltration pits** – Infiltration pits with base areas of 3 m^2 to 30 m^2 were excavated to a depth of 1 m and then filled with water from the development of the pumping wells; the declining water level was then measured over time.
- **Infiltration basin** – At Bayswater an infiltration basin with an area of approximately 250 m^2 was excavated to allow for infiltration of the abstracted water during the pumping test; the rate of pumping into the basin was used to estimate the infiltration rate.

Given that the infiltration pits and basins have significantly larger footprints and that significantly larger volumes of water are used, these methods are considered to be more representative of likely infiltration rates and more weight should therefore be given to the results from these tests. The inverse auger hole method was therefore only undertaken in the Forrestfield area (three tests) and at Consolidated Airport Terminal Station (three tests) to obtain additional information.

The results from the Inverse Auger Hole method are presented in Appendix G while the results from the infiltration pits and basin are presented in Appendix H.

A summary of the infiltration test results is provided in Table 10 and indicates that infiltration rates range from negligible to 12 m/d.

Table 10: Infiltration Test Results

Location	Geological Unit	Approximate Depth to Groundwater (m bgl)	Test Type	Infiltration Rates (m/d)
Bayswater Dive Structure	Bassendean Sand	2.0	Inverse Auger	NT
			Infiltration Pit	6* - 11
			Infiltration Basin	~2
Airport West Station	Bassendean Sand	2.2	Inverse Auger	NT
			Infiltration Pit	3
Consolidated Airport Terminal Station	Bassendean Sand	2.0	Inverse Auger	4 – 10
			Infiltration Pit	1**
Forrestfield area	Bassendean Sand	Unknown	Inverse Auger	3 – 12
		6.2	Infiltration Pit	Negligible

Notes: *The lower range is after consecutive tests in the same infiltration pit.
**Some Coffee rock encountered in the infiltration pit,
NT = No Tests.

The infiltration capacity of the ground below an infiltration area such as a drainage swale or soak well will depend on a number of factors including:

- the soil profile below the base of the infiltration area, both within the unsaturated (above the groundwater level) and the saturated zone below the groundwater level
- the depth to groundwater level below the base of the infiltration area
- a reduction in permeability caused by the build-up of any fines or other contaminants at the base of the infiltration area carried in by the water that is discharged to the infiltration area
- any compaction of the soil below the base of the infiltration area undertaken during construction.

The inverse auger hole test indicates a high permeability of the tested Bassendean Sand and Fill, which is expected given that the sand is fine to coarse grained and that the tests were short-term and carried out at a shallow depth that would generally not be influenced by the proximity to groundwater level. However, the larger area and longer duration infiltration pit and basin tests indicate lower infiltration rates. These lower infiltration rates are considered to be more representative of conditions applicable to a drainage swale since these tests would have resulted in saturation of the zone between the base of the pit/basin and the groundwater level. Once the saturation front reaches the groundwater level, the hydraulic gradient decreases significantly, which leads to a proportional decrease in infiltration rate.

Given the shallow depth to the groundwater level it is therefore estimated that short-term (less than 1 hour) infiltration rates could be about 5 to 10 m/d in the Bassendean Sand while longer term (several hours to days) infiltration rates are likely to be around 1 to 5 m/d.

Based on the infiltration test results and the presence of low permeability Guildford Formation soils at shallow depth (which are found to cause locally perched groundwater conditions), the Forrestfield area is considered to have the poorest infiltration capacity of the four areas assessed along the alignment. Therefore even though the results from the inverse auger test indicate relatively high infiltration rates, the presence of clayey material within a few metres from the ground surface would significantly reduce the infiltration capacity in this area.

The shallow depth to groundwater level over the whole alignment and the lower permeability soils in the Forrestfield area must be considered in the design of stormwater infiltration/soakage.

9.0 KEY HYDROGEOLOGICAL CHARACTERISTICS AND CONSIDERATIONS

The following sections summarise the key hydrogeological characteristics observed during the Stage 2 Geotechnical Investigation, which will need to be considered during design and construction.

9.1 Bayswater Dive Structure

For the Bayswater Dive Structure the following key points have been identified:

- A two aquifer system exists in the area with a difference in groundwater level and piezometric level in March 2015 between the Upper and Lower Aquifers of about 3.7 m and with a downward vertical gradient (i.e. groundwater level is highest in the Upper Aquifer).
- The results from the pumping test, which was carried out in the Lower Aquifer, indicated that the two aquifers are isolated from each other. This isolation is clearly illustrated by the response in two monitoring wells that were installed 5 m from the pumping well, which during the test pumping indicated no drawdown in the Upper Aquifer and 9 m of drawdown in the Lower Aquifer.
- Pumping from the Lower Aquifer resulted in a large cone of depression with a reduction in piezometric level in the Lower Aquifer observed to be approximately 8.5 m at a distance of 40 m from the pumping well and up to 1.5 m approximately 500 m away from the pumping well towards the Swan River.
- The pumping test analysis indicates that heterogeneous conditions are present in the Lower Aquifer.
- Previous investigations indicate that the groundwater quality is significantly different in the Upper and Lower Aquifers and that the Upper Aquifer is impacted from groundwater contamination from a site located to the north-east of the Bayswater Dive Structure. Careful management will therefore be required to avoid cross-contamination between the two aquifers during and post-construction and the dewatering system for the Upper and Lower Aquifers will need to be designed so that each aquifer is dewatered separately.
- The installation of the D-walls to facilitate excavation and construction of the dive structure would create a permanent groundwater flow barrier in the Upper Aquifer where the walls are aligned obliquely or perpendicular to the groundwater flow direction, which could result in groundwater mounding against the up-gradient side of the D-walls. The groundwater mounding could potentially require some groundwater control in areas where the groundwater level is close to the ground surface and may result in an increase in hydrostatic pressure against the D-walls.
- Reference Design Summary Table RevB (for Reference design Rev F) indicates that at the deeper part of the dive structure and tunnel portal the D-walls will extend into the Osborne Formation. This would also significantly reduce the groundwater inflow from the Lower Aquifer into the excavation. The estimated low dewatering rate is therefore expected given that the pumping test well was able to dewater the Lower Aquifer to below the required dewatering level (without retaining walls) during the pumping test.
- At the Bayswater Main Drain, the D-walls may need to be extended to below the invert level of the Bayswater Main Drain, which would require diversion of the Bayswater Main Drain. This diversion could require additional groundwater control (dewatering) during construction.

9.2 Airport West Station

For the Airport West Station the following key points have been identified:

- Hydraulic testing at Airport West indicates that that the hydraulic conductivity of the Kardinya Shale of the Osborne Formation is very low. According to the Reference Design Revision F the D-walls will extend into the Kardinya Shale (OFs) of the Osborne Formation and thereby cut off direct inflow from the Superficial Aquifer. Dewatering requirements will therefore only consist of emptying out the storage

contained by the D-walls and any groundwater seepage through the Kardinya Shale as well as through the D-walls. Minimising seepage through the D-walls would reduce the inflow rates. Minimisation of seepage through the walls would require that water stops between panels are installed to the bottom of the D-walls (or at close as practicable).

- Given the expected low dewatering rates, a recharge well system for dewatering discharge disposal may not be required. Alternative options such as disposal to the stormwater system may be a suitable option.
- The pumping test indicates that the Belmont Main Drain will act as a positive boundary to drawdown of the groundwater table. However, a reduction in piezometric pressure was also observed in the monitoring wells on the southern side of the Belmont Drain, indicating that the cone of depression in the Ascot Formation will extend beyond the Belmont Drain to the south, should it be decided that groundwater abstraction from the Ascot Formation would be required.
- The groundwater and piezometric levels in the monitoring wells were found to be affected by local council and individual household groundwater bore reticulation systems, suggesting that any abstraction from the Superficial Aquifer could affect nearby existing wells if drawdown is not carefully managed.
- The tunnel alignment in this area and the Airport West Station are generally located parallel to the groundwater flow direction from east to west toward the Swan River. As such, construction of the tunnel and installation of D-walls for the Airport West Station is unlikely to significantly impede groundwater flow. Nevertheless, given the shallow depth to groundwater level, it is recommended that the effect of construction of the tunnel and station infrastructure be assessed during detailed design.

9.3 Consolidated Airport Terminal Station

For the Consolidated Airport Terminal Station the following key points have been identified:

- According to the Reference Design Revision F the D-walls will extend 3 m into the Mirrabooka Member of the Osborne Formation (OFm) and thereby cut off direct groundwater inflow from the Superficial Aquifer. The main groundwater inflow into the excavation will be from the Mirrabooka Aquifer. Understanding the depth, extent and hydrogeological properties of the Mirrabooka Aquifer therefore becomes the most important factor in estimating the dewatering requirements for the Consolidated Airport Terminal Station.
- The available borehole logs in the Mirrabooka Aquifer indicate that the hydraulic conductivity will reduce with depth due to the increasing influence of interbedded layers of sandy mudstone which are expected to have a significantly lower hydraulic conductivity. None of the boreholes drilled into the Mirrabooka Aquifer penetrated through it and the thickness of the Mirrabooka Aquifer is therefore not well defined.
- The analysis of the pumping test from the Consolidated Airport Terminal Station showed that the best fit when replicating the pumping test data in the groundwater model was obtained using a ratio of vertical to horizontal hydraulic conductivity of 0.1 for all the hydrogeological units. The dewatering scenarios indicate that the ratio would have a significant influence on the final dewatering rates.
- Based on the importance of the Mirrabooka Aquifer it is recommended that additional investigation is undertaken to determine the thickness and aquifer properties of the Mirrabooka Aquifer. The pumping test should include a monitoring well configuration that allows for determination of the vertical and horizontal hydraulic conductivity and to assess whether the hydraulic conductivity reduces with depth.
- The tunnel alignment in this area and the Consolidated Airport Terminal Station are generally located parallel to the groundwater flow direction from east to west toward the Swan River. As such, the construction of the tunnel and installation of D-walls for the Consolidated Airport Terminal Station is therefore unlikely to significantly impede groundwater flow. Nevertheless, given the shallow depth to

groundwater level, it is recommended that the effect of construction of the tunnel and station infrastructure be assessed during detailed design.

9.4 Forrestfield Area

For the Forrestfield area the following key points have been identified:

- The pumping test analysis at the Forrestfield Dive Structure does not provide definitive conclusion to the ratio between the vertical and horizontal hydraulic conductivity in this area (the use of 0.1 to 1 gave almost similar results – best fit was 0.25). The groundwater model scenario runs indicate that this ratio will influence the dewatering rates.
- The groundwater level measurements in the installed monitoring wells and VWPs indicate a large (approximately 3 m) seasonal variation in groundwater level as well as the presence of perched groundwater conditions in this area. Given that the Stage 2 investigation was carried out in the dry season and perched groundwater may dry out during summer in some areas, the extent and connectivity with the regional aquifer is currently unknown and can only be established through groundwater level monitoring over the wet season. It is therefore recommended that additional monitoring wells are installed and groundwater level monitoring undertaken during the wet season. It is also recommended that following a large rainfall event during the wet season a site walkover be undertaken to gain a better understanding of the potential extent of inundated areas.
- The groundwater quality during the pumping test was found to be acidic (average pH of 5.7) with limited buffering capacity, which would need to be considered during the dewatering design. The high nutrient concentrations may restrict construction dewatering and long-term drainage disposal to the stormwater system.
- Based on the infiltration test results and the presence of low permeability soils at shallow depth the Forrestfield area is considered to have the poorest infiltration capacity of the four areas assessed along the alignment.
- The D-walls associated with the Forrestfield Dive Structure will be constructed perpendicular to the groundwater flow direction. However, according to Reference Design Summary Table RevB (for Reference design Rev F) the D-walls do not extend through the full thickness of the Superficial Aquifer and it is therefore considered unlikely that the D-walls will significantly impede groundwater flow and result in significant groundwater mounding behind the up-gradient D-wall. Nevertheless, given the presence of the seasonally perched groundwater conditions, it is recommended that the effect the installation of D-walls could have on the groundwater level at the dive structure be assessed during the detailed design.

10.0 RECOMMENDATIONS FOR FURTHER WORK

Further hydrogeological investigation is recommended to allow detailed design of specific areas or to provide additional information to reduce uncertainty. In summary the following additional investigation work is currently recommended:

1) General:
   a) Establish a groundwater monitoring program along the alignment.
   b) Carry out single well hydraulic testing in future installed wells.
2) At Airport West Station:
   a) Install additional monitoring wells which are screened in the Osborne Formation at the new proposed Airport West Station location.
   b) Carry out single well hydraulic testing in the new wells to provide confirmation of the hydraulic conductivity of the Kardinya Shale Member.

3) At Consolidated Airport Terminal Station:

   a) Drill an additional geotechnical borehole to determine the thickness of the Mirrabooka Aquifer.

   b) Install a new pumping test well in the Mirrabooka Aquifer and carry out a pumping test. .

4) At Forrestfield Area:

   a) Undertake further investigation to assess the extent of the perched groundwater conditions.

   b) Undertake further assessment to enable a better understanding of the connectivity between the regional aquifer and the perched groundwater conditions.

   c) Carry out additional infiltration testing in areas where stormwater soakage may be considered.

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12.0 LIMITATIONS

Your attention is drawn to the document – "Limitations", which is included as Appendix I to this report. This document is intended to assist you in ensuring that your expectations of this report are realistic, and that you understand the inherent limitations of a report of this nature. If you are uncertain as to whether this report is appropriate for any particular purpose please discuss this issue with us.

Report Signature Page

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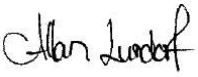

Michael Bartlett
Hydrogeologist

Allan Lundorf
Senior Water Resources Engineer

MJB,AL/AL,DPS/asu

A.B.N. 64 006 107 857

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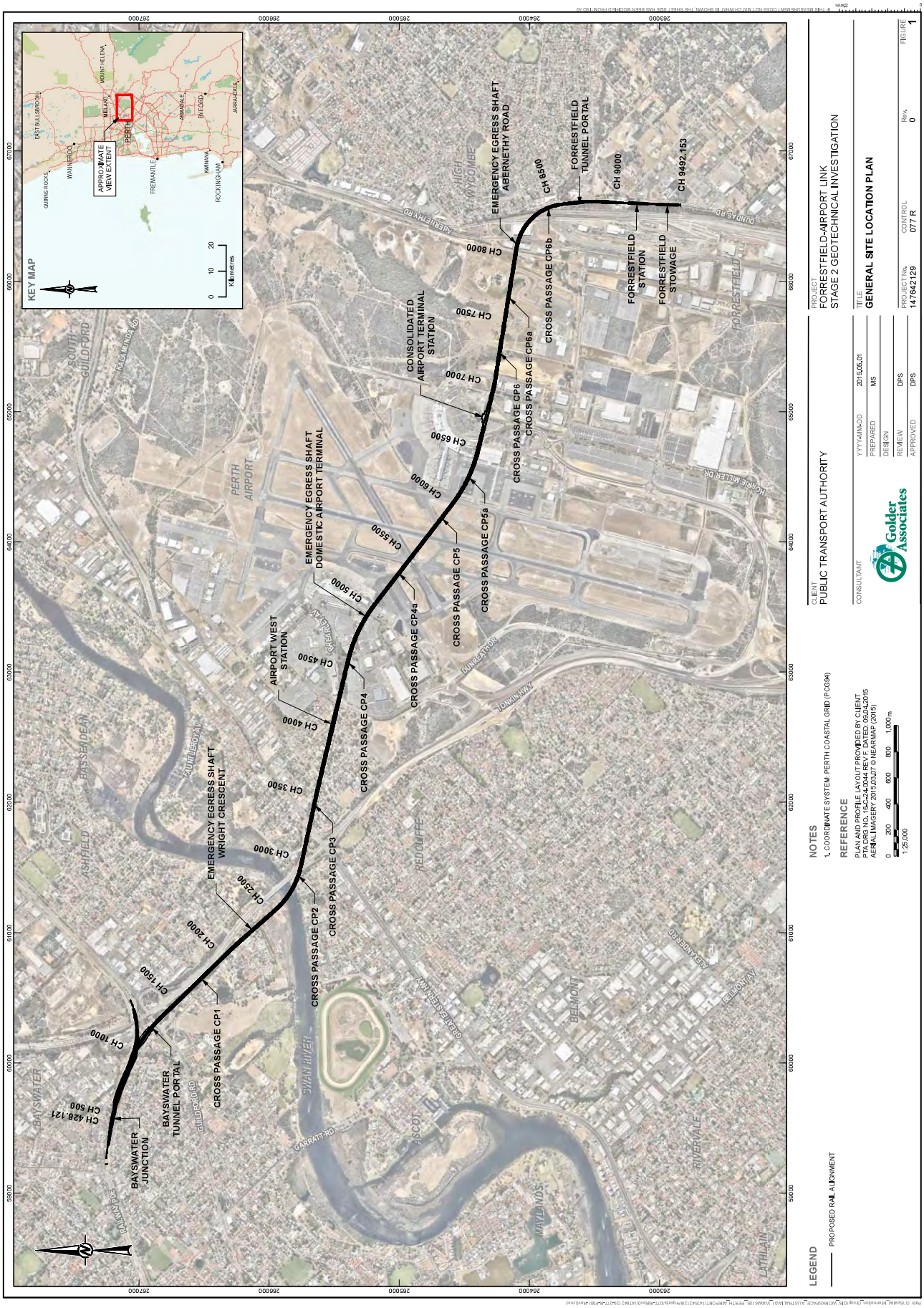
May 2015
Report No. 147642129-077-R-Rev0


KEY MAP
APPROXIMATE VIEW EXTENT
PERTH
MIDLAND
MOUNT HELENA
EAST BULLSBROOK
WANNEROO
QUINNS ROCKS
FREMANTLE
ARMADALE
BYFORD
KWINANA
ROCKINGHAM
JARRAHDALE
0 10 20 Kilometres
LEGEND
PROPOSED RAIL ALIGNMENT
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PERTH AIRPORT ALIGNMENT REV 2 DATED 29.04.2015
AERIAL IMAGERY 2015,2017 © NEARMAP (2015)
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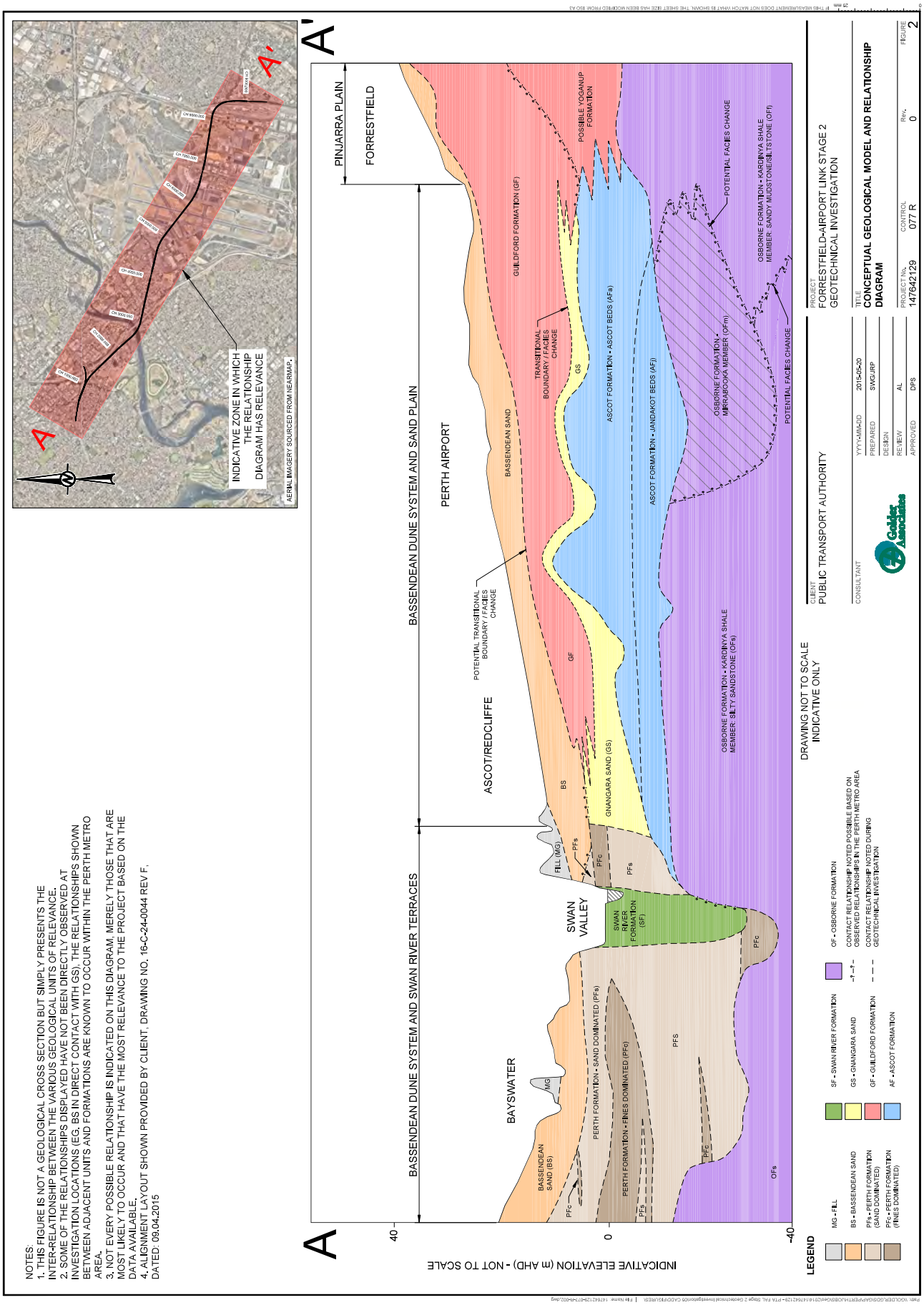
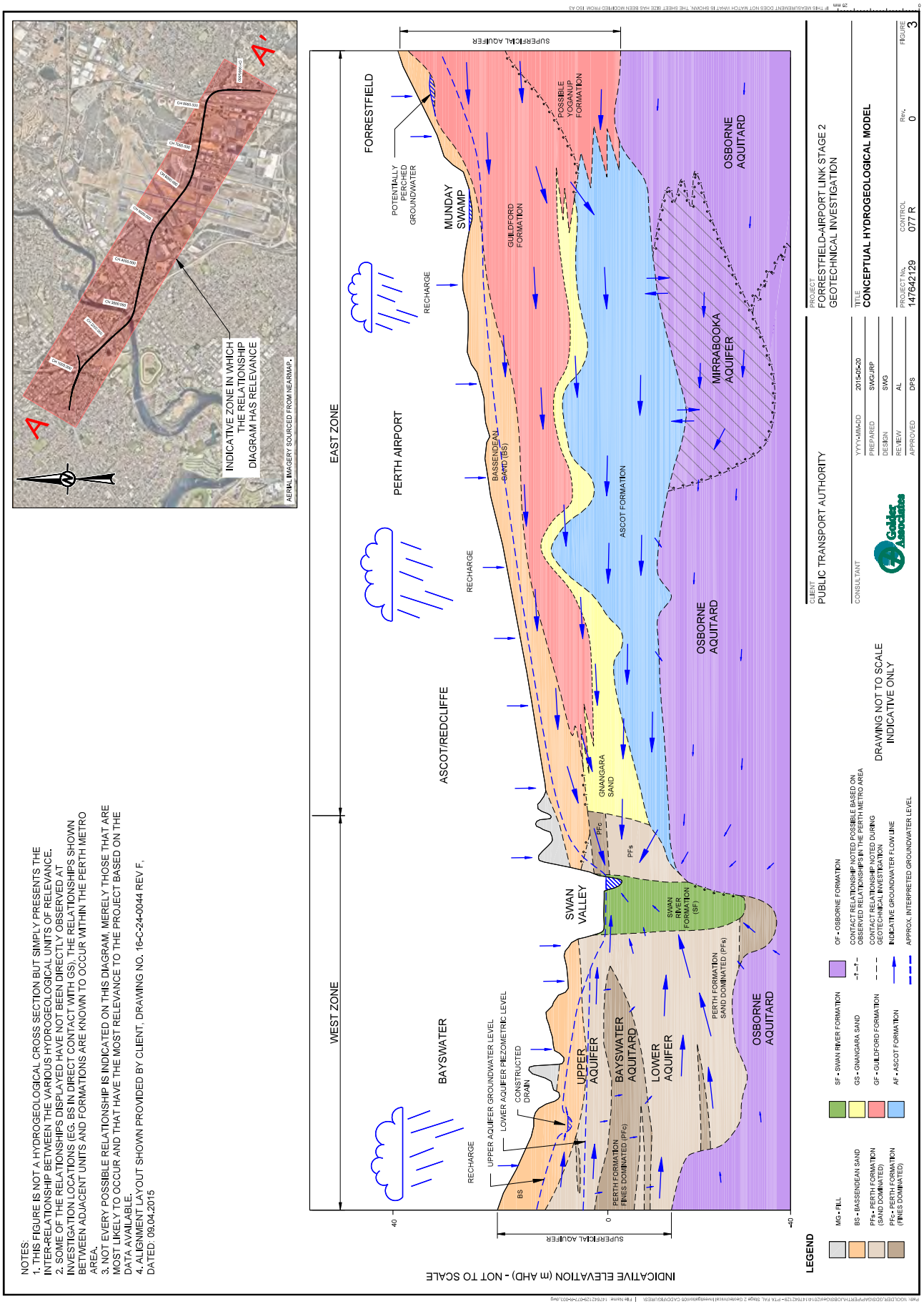
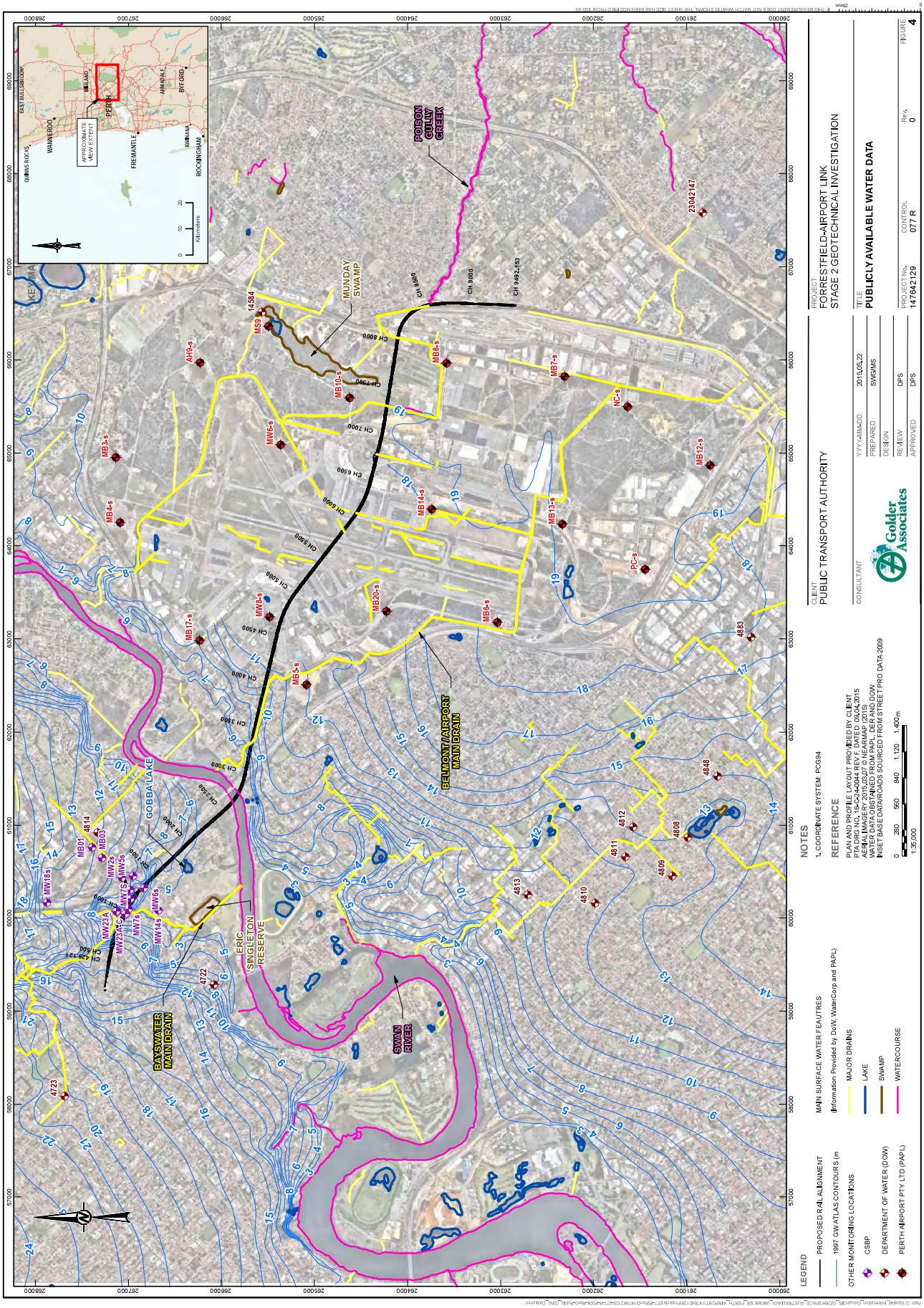
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.01
PREPARED MS
DESIGN
REVIEW DPS
APPROVED DPS
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GENERAL SITE LOCATION PLAN
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 1
BAYSWATER JUNCTION
BAYSWATER TUNNEL PORTAL
CH 428.121
CH 500
CH 1000
CH 1500
CH 2000
CH 2500
CH 3000
CH 3500
CH 4000
CH 4500
CH 5000
CH 5500
CH 6000
CH 6500
CH 7000
CH 7500
CH 8000
CH 8500
CH 9000
CH 9492.153
CROSS PASSAGE CP1
CROSS PASSAGE CP2
CROSS PASSAGE CP3
CROSS PASSAGE CP4
CROSS PASSAGE CP4a
CROSS PASSAGE CP5
CROSS PASSAGE CP5a
CROSS PASSAGE CP6
CROSS PASSAGE CP6a
CROSS PASSAGE CP6b
EMERGENCY EGRESS SHAFT WRIGHT CRESCENT
AIRPORT WEST STATION
EMERGENCY EGRESS SHAFT DOMESTIC AIRPORT TERMINAL
CONSOLIDATED AIRPORT TERMINAL STATION
EMERGENCY EGRESS SHAFT ABERNETHY ROAD
FORRESTFIELD TUNNEL PORTAL
FORRESTFIELD STATION
FORRESTFIELD STOWAGE
PERTH AIRPORT
BAYSWATER
BASSENDEAN
ASHFIELD
SOUTH GUILDFORD
GUILDFORD
KALAMUNDA RD
GUILDFORD RD
KAMBALDA RD
ABERNETHY RD
HIGH WYCOMBE
DUNDAS RD
FORRESTFIELD
HOWARD MILLER DR
DUNREATH DR
TONKIN HWY
GREAT EASTERN HWY
REDCLIFFE
BELMONT
ASCOT
RIVERVALE
GARRATT RD
SWAN RIVER
MAYLANDS
LATHLAIN
ALBANY HWY
OAKLANDS ESTATE WAY
KEANE RD



NOTES:
1. THIS FIGURE IS NOT A GEOLOGICAL CROSS SECTION BUT SIMPLY PRESENTS THE INTER-RELATIONSHIP BETWEEN THE VARIOUS GEOLOGICAL UNITS OF RELEVANCE.
2. SOME OF THE RELATIONSHIPS DISPLAYED HAVE NOT BEEN DIRECTLY OBSERVED AT INVESTIGATION LOCATIONS (EG, BS IN DIRECT CONTACT WITH GS), THE RELATIONSHIPS SHOWN BETWEEN ADJACENT UNITS AND FORMATIONS ARE KNOWN TO OCCUR WITHIN THE PERTH METRO AREA.
3. NOT EVERY POSSIBLE RELATIONSHIP IS INDICATED ON THIS DIAGRAM, MERELY THOSE THAT ARE MOST LIKELY TO OCCUR AND THAT HAVE THE MOST RELEVANCE TO THE PROJECT BASED ON THE DATA AVAILABLE.
4. ALIGNMENT LAYOUT SHOWN PROVIDED BY CLIENT, DRAWING NO. 16-C-24-0044 REV F, DATED: 09.04.2015



NOTES:
1. THIS FIGURE IS NOT A HYDROGEOLOGICAL CROSS SECTION BUT SIMPLY PRESENTS THE INTER-RELATIONSHIP BETWEEN THE VARIOUS HYDROGEOLOGICAL UNITS OF RELEVANCE.
2. SOME OF THE RELATIONSHIPS DISPLAYED HAVE NOT BEEN DIRECTLY OBSERVED AT INVESTIGATION LOCATIONS (EG. BS IN DIRECT CONTACT WITH GS), THE RELATIONSHIPS SHOWN BETWEEN ADJACENT UNITS AND FORMATIONS ARE KNOWN TO OCCUR WITHIN THE PERTH METRO AREA.
3. NOT EVERY POSSIBLE RELATIONSHIP IS INDICATED ON THIS DIAGRAM, MERELY THOSE THAT ARE MOST LIKELY TO OCCUR AND THAT HAVE THE MOST RELEVANCE TO THE PROJECT BASED ON THE DATA AVAILABLE.
4. ALIGNMENT LAYOUT SHOWN PROVIDED BY CLIENT, DRAWING NO. 16-C-24-0044 REV F, DATED: 09.04.2015


POISON GULLY CREEK
MUNDAY SWAMP
BELMONT / AIRPORT MAIN DRAIN
GOBBA LAKE
ERIC SINGLETON RESERVE
BAYSWATER MAIN DRAIN
SWAN RIVER
LEGEND
PROPOSED RAIL ALIGNMENT
1997 GW ATLAS CONTOURS (m
OTHER MONITORING LOCATIONS
CSBP
DEPARTMENT OF WATER (DOW)
PERTH AIRPORT PTY LTD (PAPL)
MAIN SURFACE WATER FEAUTRES
(Information Provided by DoW, WaterCorp and PAPL)
MAJOR DRAINS
LAKE
SWAMP
WATERCOURSE
NOTES
1. COORDINATE SYSTEM: PCG94
REFERENCE
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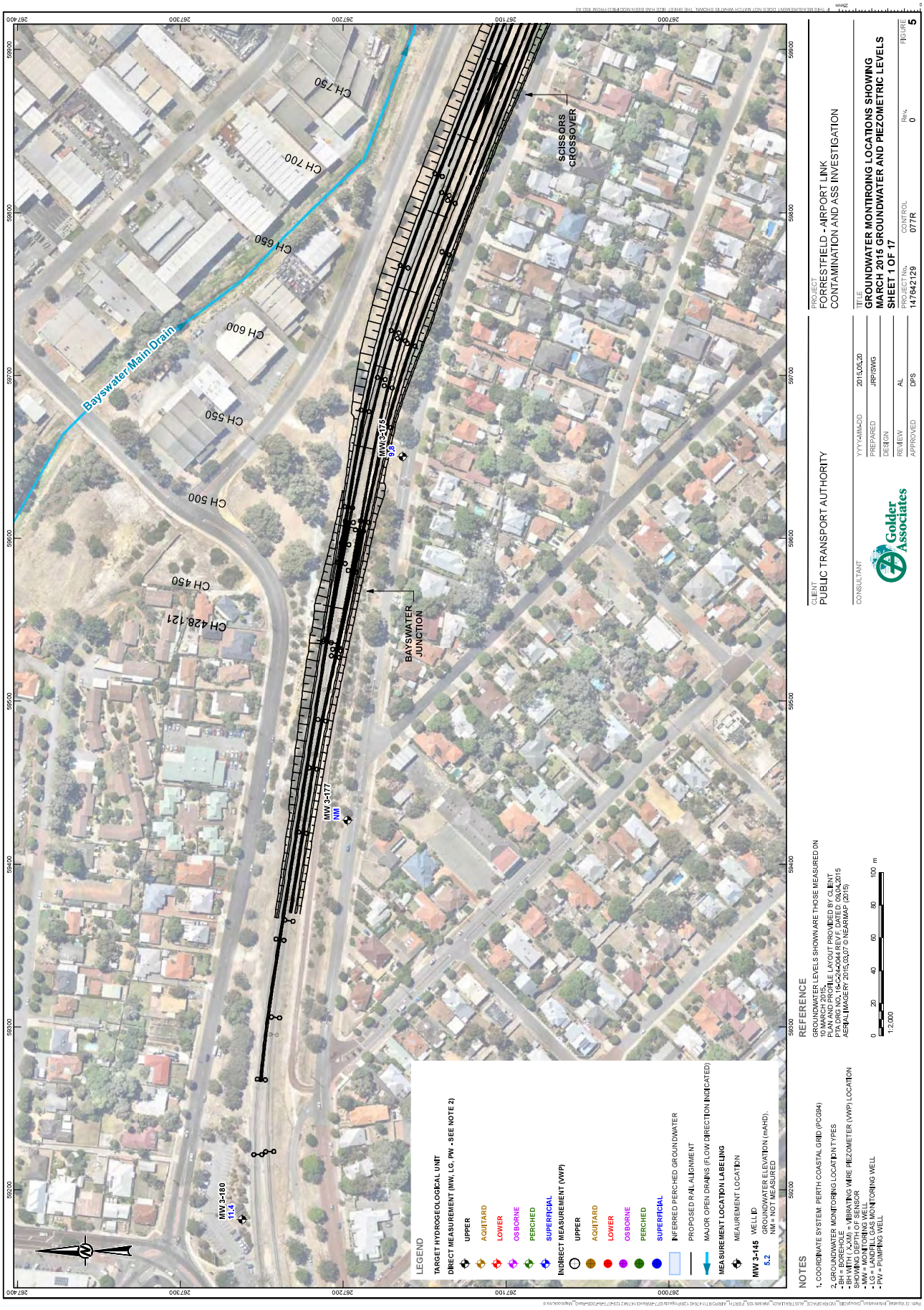
PTA DRG NO. 16-C-24-0044 REV F, DATED: 03.04.2015
AERIAL IMAGERY 2015.03.07 © NEARMAP (2015)
WATER DATA OBSTAINED FROM PAPL, DER AND DOW
INSET BASE DATA/ROADS SOURCED FROM STREET PRO DATA 2009
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.22
PREPARED SWG/MS
DESIGN
REVIEW DPS
APPROVED DPS
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
PUBLICLY AVAILABLE WATER DATA
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 4


Bayswater Main Drain
CH 750
CH 700
CH 650
CH 600
CH 550
CH 500
CH 450
CH 428.121
SCISSORS CROSSOVER
BAYSWATER JUNCTION
MW 3-175
9.8
MW 3-177
NM
MW 3-180
11.4
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELING
MEAUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD), NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (X.XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-24-0044 REV F, DATED: 09.04.2015
AERIAL IMAGERY 2015.03.07 © NEARMAP (2015)
0 20 40 60 80 100 m
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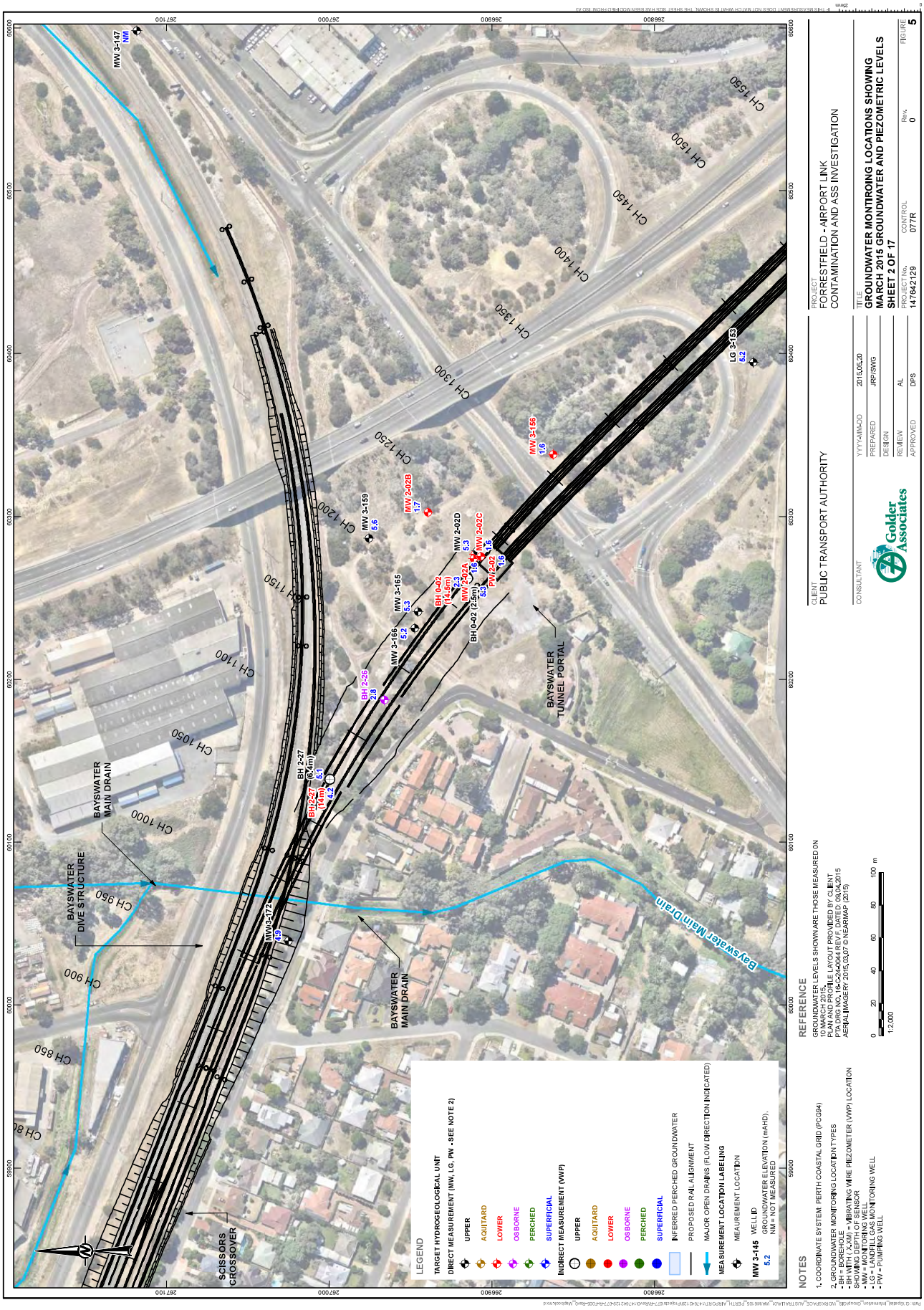
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONTIROING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 1 OF 17
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
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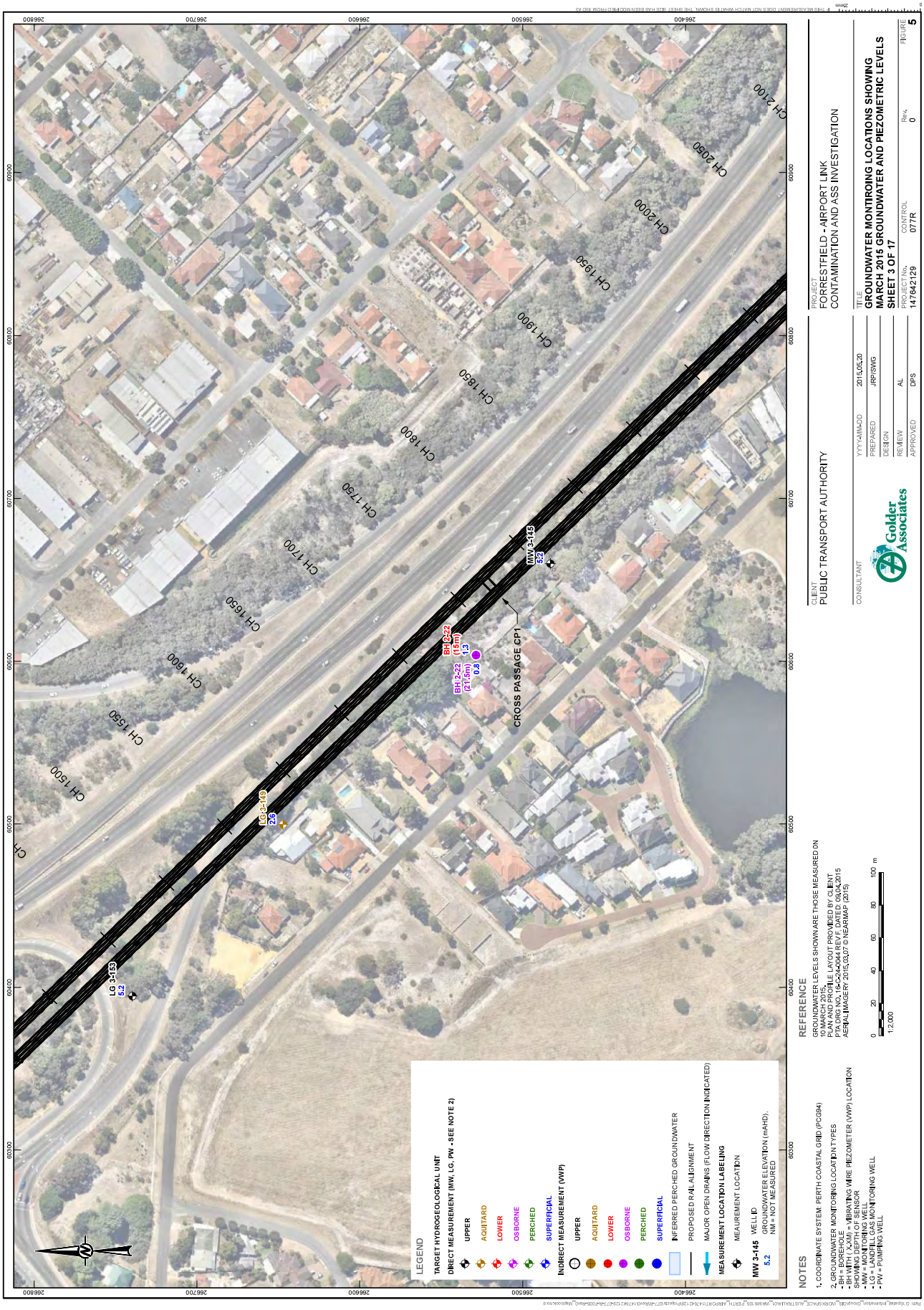
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
BAYSWATER DIVE STRUCTURE
BAYSWATER MAIN DRAIN
BAYSWATER TUNNEL PORTAL
SCISSORS CROSSOVER
Bayswater Main Drain
CH 850
CH 900
CH 950
CH 1000
CH 1050
CH 1100
CH 1150
CH 1200
CH 1250
CH 1300
CH 1350
CH 1400
CH 1450
CH 1500
CH 1550
MW 3-147 NM
MW 3-172 4.9
BH 2-27 (6.4m) 5.1
BH 2-27 (14.1m) 4.2
BH 2-25 2.3
MW 3-165 5.3
MW 3-166 2.4
BH 0-02 (14.5m) 2.3
BH 0-02 (2.5m) 5.3
MW 2-02A 1.6
MW 2-02C 1.6
MW 2-02D 5.3
PW 2-02 1.6
MW 2-02B 1.7
MW 3-159 5.6
MW 3-156 1.6
LG 3-153 5.2
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 2 OF 17
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244-0044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015, 03.07 © NEARMAP (2015)
0 20 40 60 80 100 m
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CROSS PASSAGE CP1
BH 2-22 (16m) 1.3
BH 2-22 (21.5m) 0.8
MW 3-145 5.2
LG 3-153 5.2
LG 3-149 2.6
CH 1500
CH 1550
CH 1600
CH 1650
CH 1700
CH 1750
CH 1800
CH 1850
CH 1900
CH 1950
CH 2000
CH 2050
CH 2100
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITROING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 3 OF 17
YYYY-MM-DD 2015.05.20
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DESIGN
REVIEW AL
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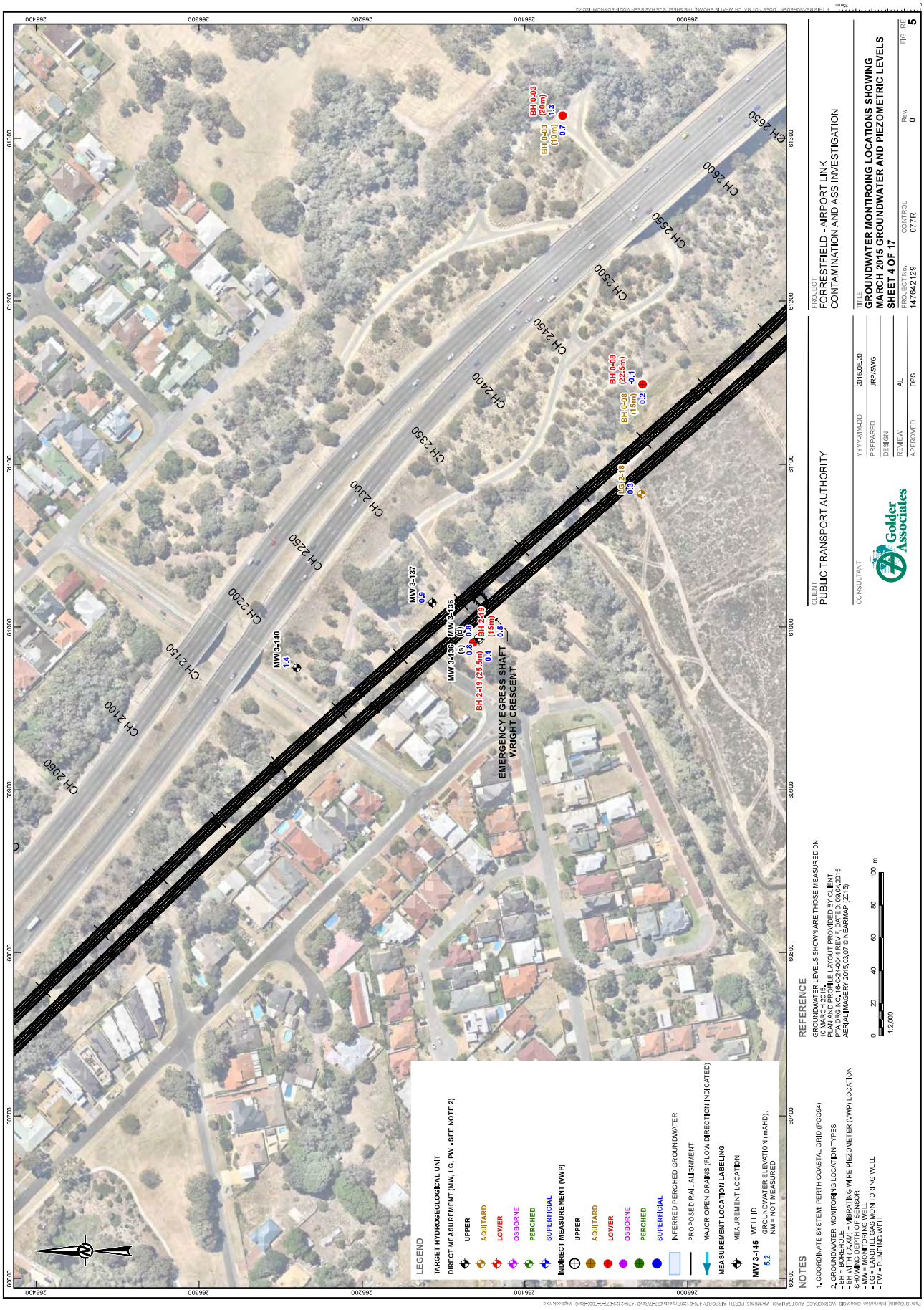
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5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AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELING
MEAUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD), NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (X,XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
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CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.20
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DESIGN
REVIEW AL
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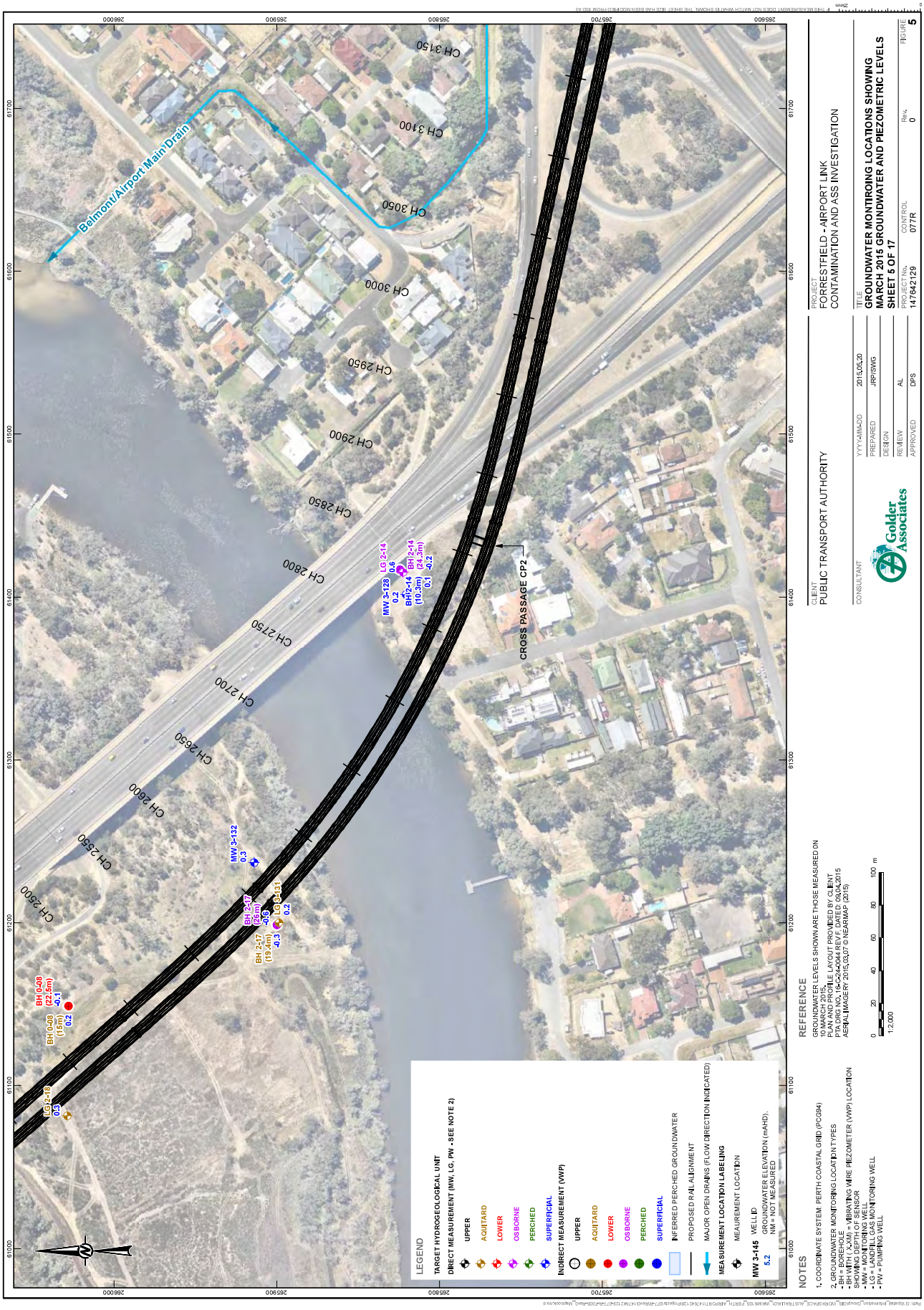
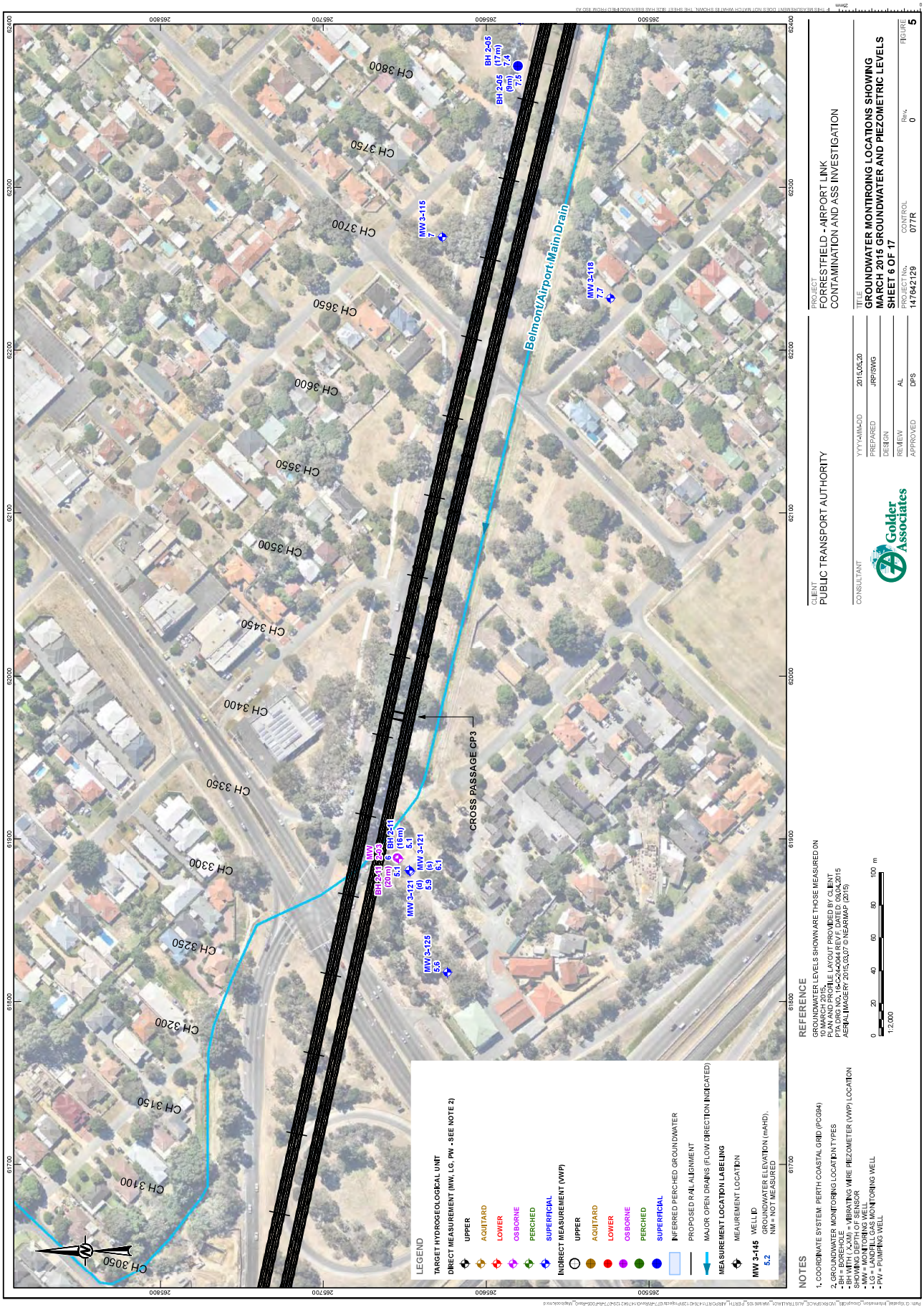
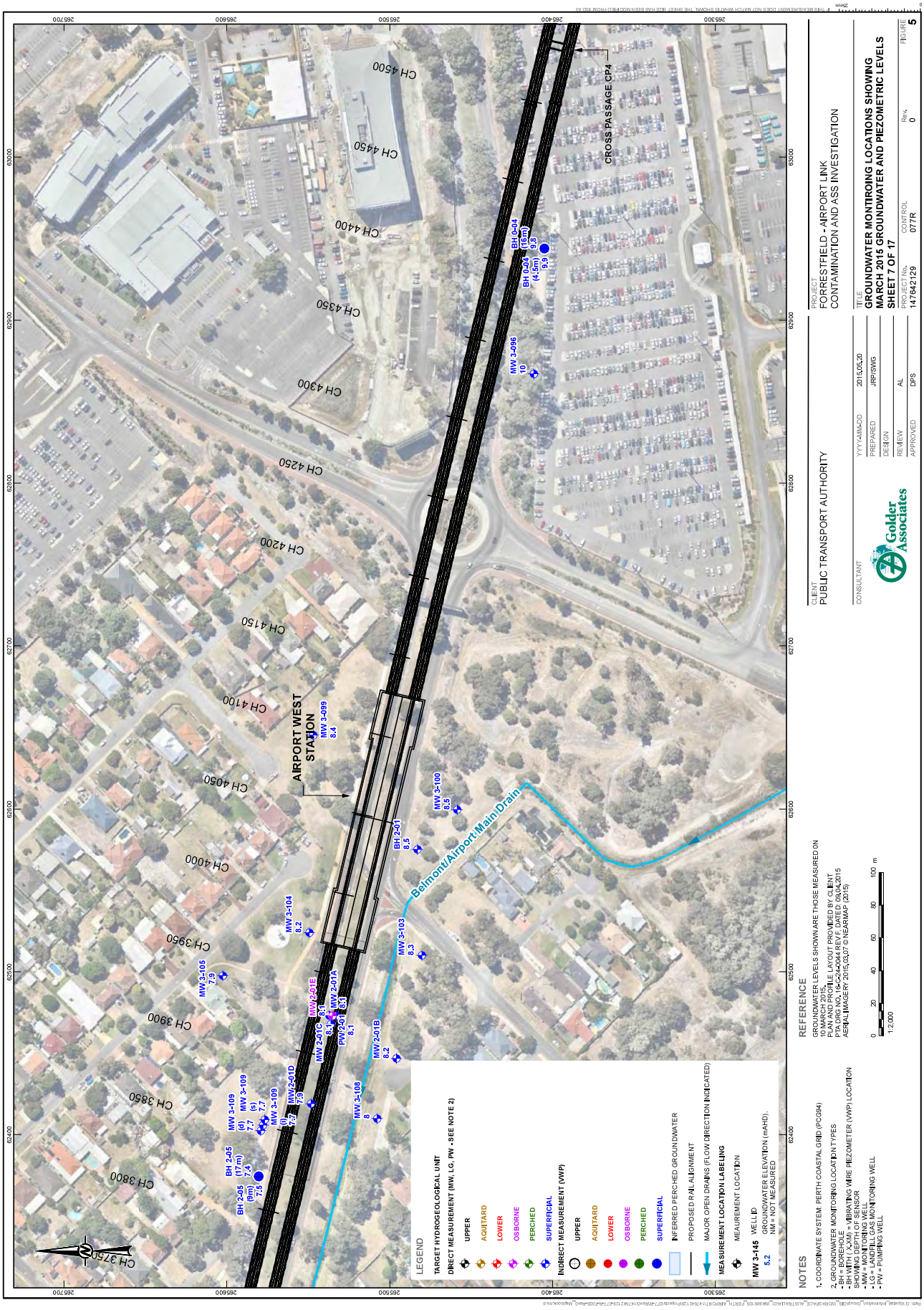
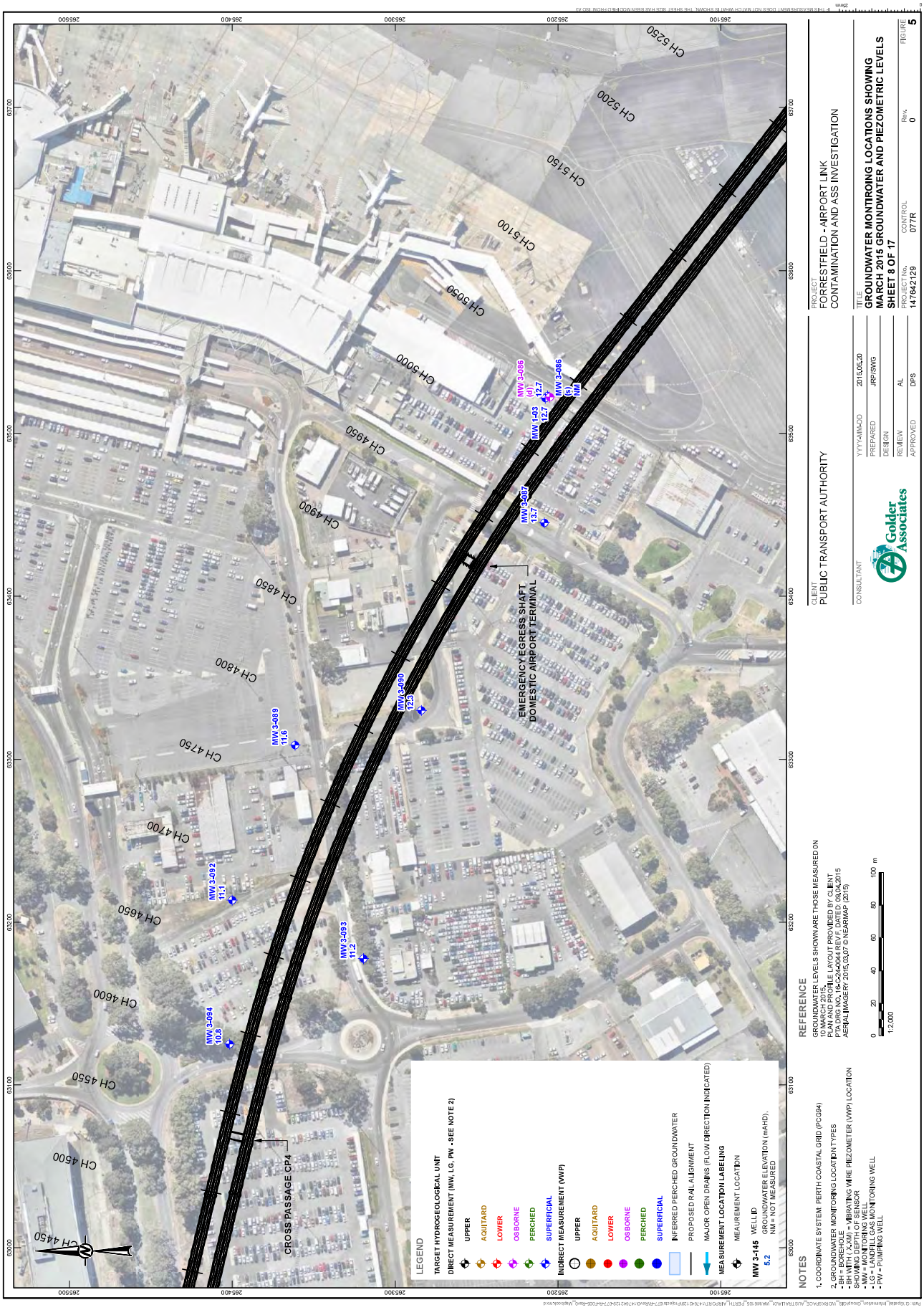
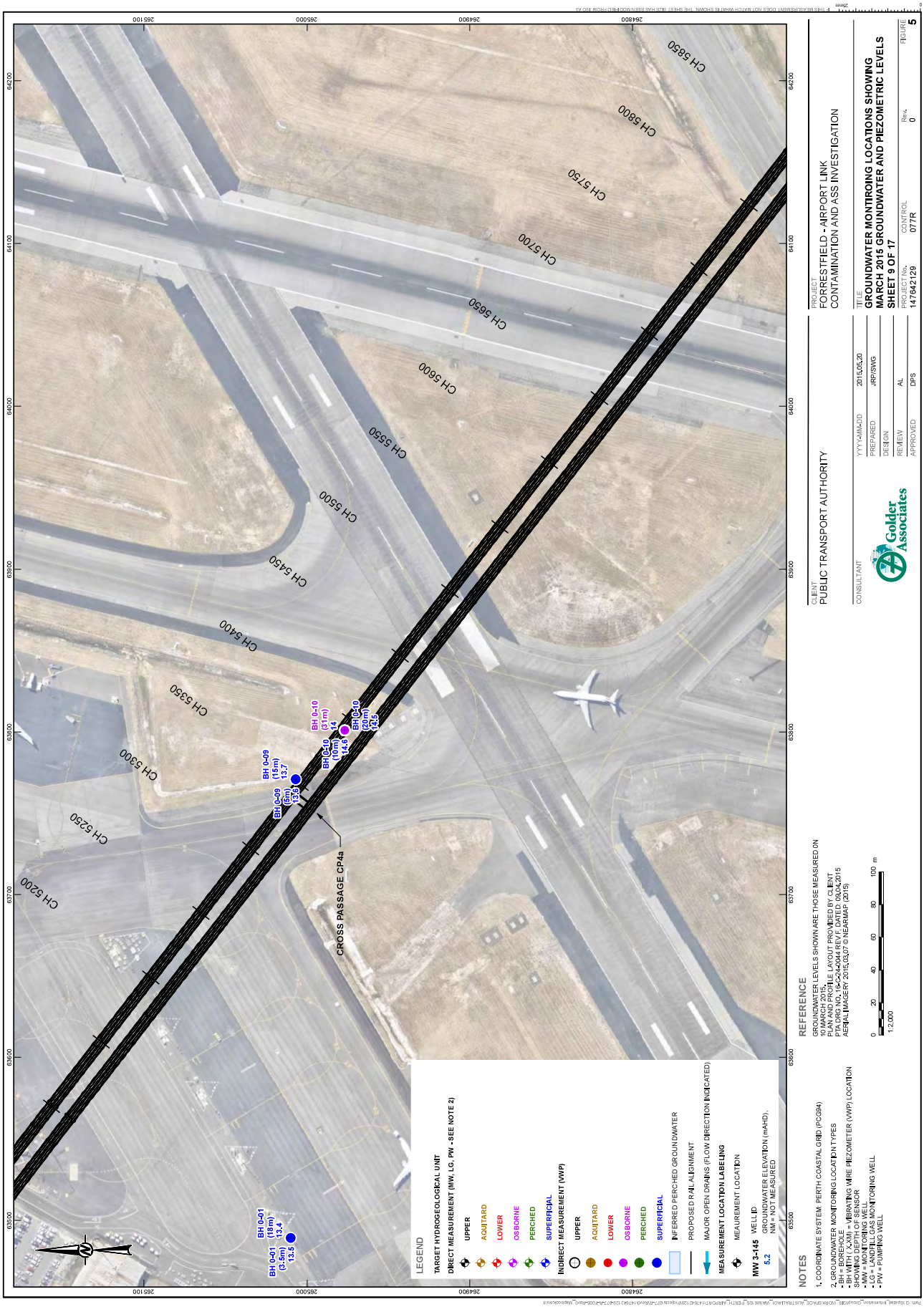
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONTIROING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 4 OF 17
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5
EMERGENCY EGRESS SHAFT
WRIGHT CRESCENT
MW 3-137 0.9
MW 3-136 (d) 0.8
MW 3-136 (s) 0.8
BH 2-19 (25.5m) 0.4
BH 2-19 (15m) 0.5
MW 3-140 1.4
LG 2-18 0.3
BH 0-08 (22.5m) -0.1
BH 0-08 (15m) 0.2
BH 0-03 (20m) 1.3
BH 0-03 (10m) 0.7
CH 2050
CH 2100
CH 2150
CH 2200
CH 2250
CH 2300
CH 2350
CH 2400
CH 2450
CH 2500
CH 2550
CH 2600
CH 2650

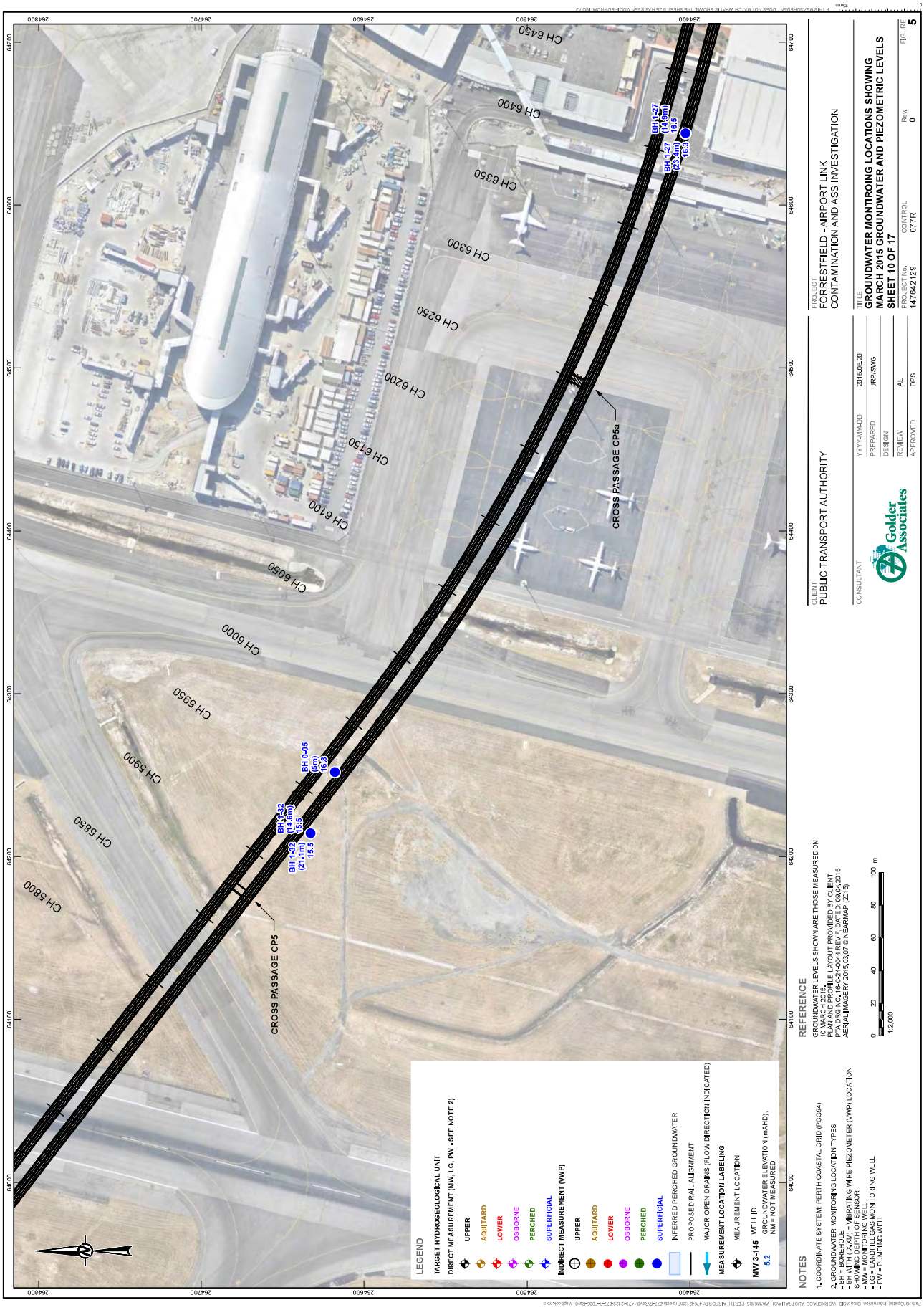

Belmont/Airport Main Drain
CH 3150
CH 3100
CH 3050
CH 3000
CH 2950
CH 2900
CH 2850
CH 2800
CH 2750
CH 2700
CH 2650
CH 2600
CH 2550
CH 2500
CROSS PASSAGE CP2
MW 3-129 0.2
LG 2-14 0.6
BH 2-14 (24.3m) -0.2
BH 2-14 (10.3m) 0.1
MW 3-132 0.3
BH 2-17 (26m) -0.5
BH 2-17 (19.4m) -0.3
LG 2-11 0.2
BH 0-08 (22.6m) -0.1
BH 0-08 (15m) 0.2
LG 2-10 0.3
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X.XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = GROUNDWATER MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244-0044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 5 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5

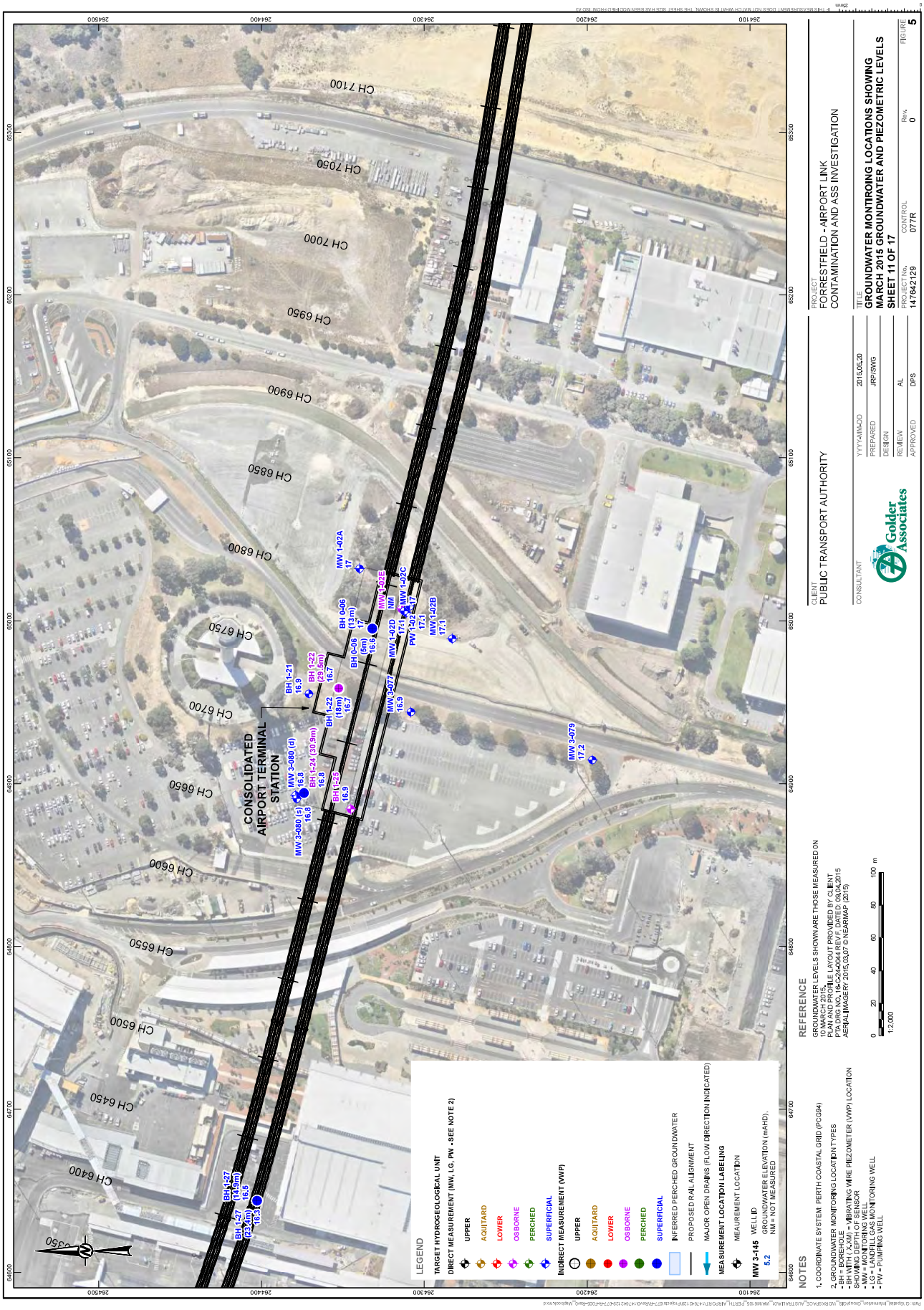

Belmont Airport Main Drain
CROSS PASSAGE CP3
CH 3050
CH 3100
CH 3150
CH 3200
CH 3250
CH 3300
CH 3350
CH 3400
CH 3450
CH 3500
CH 3550
CH 3600
CH 3650
CH 3700
CH 3750
CH 3800
MW 3-115 7
MW 3-118 7.7
BH 2-05 (17m) 7.4
BH 2-05 (9m) 7.5
MW BH2-11 2-03 (20m) 6
BH 2-11 (16m) 5.1
MW 3-121 (d) 5.8
MW 3-121 (s) 6.1
MW 3-125 6.6
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 6 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5

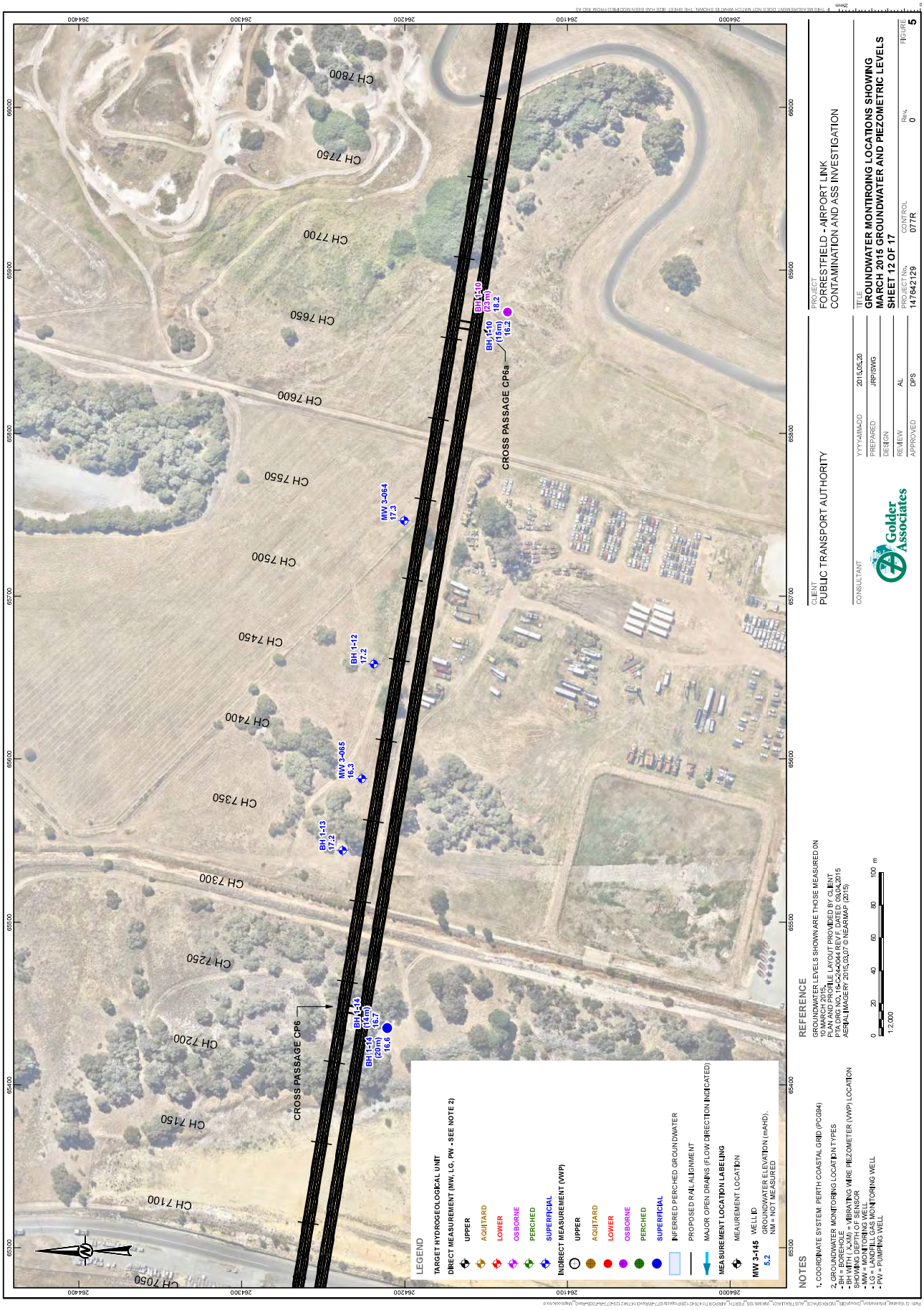

AIRPORT WEST STATION
CROSS PASSAGE CP4
Belmont/Airport Main Drain
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-24-2044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 7 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5

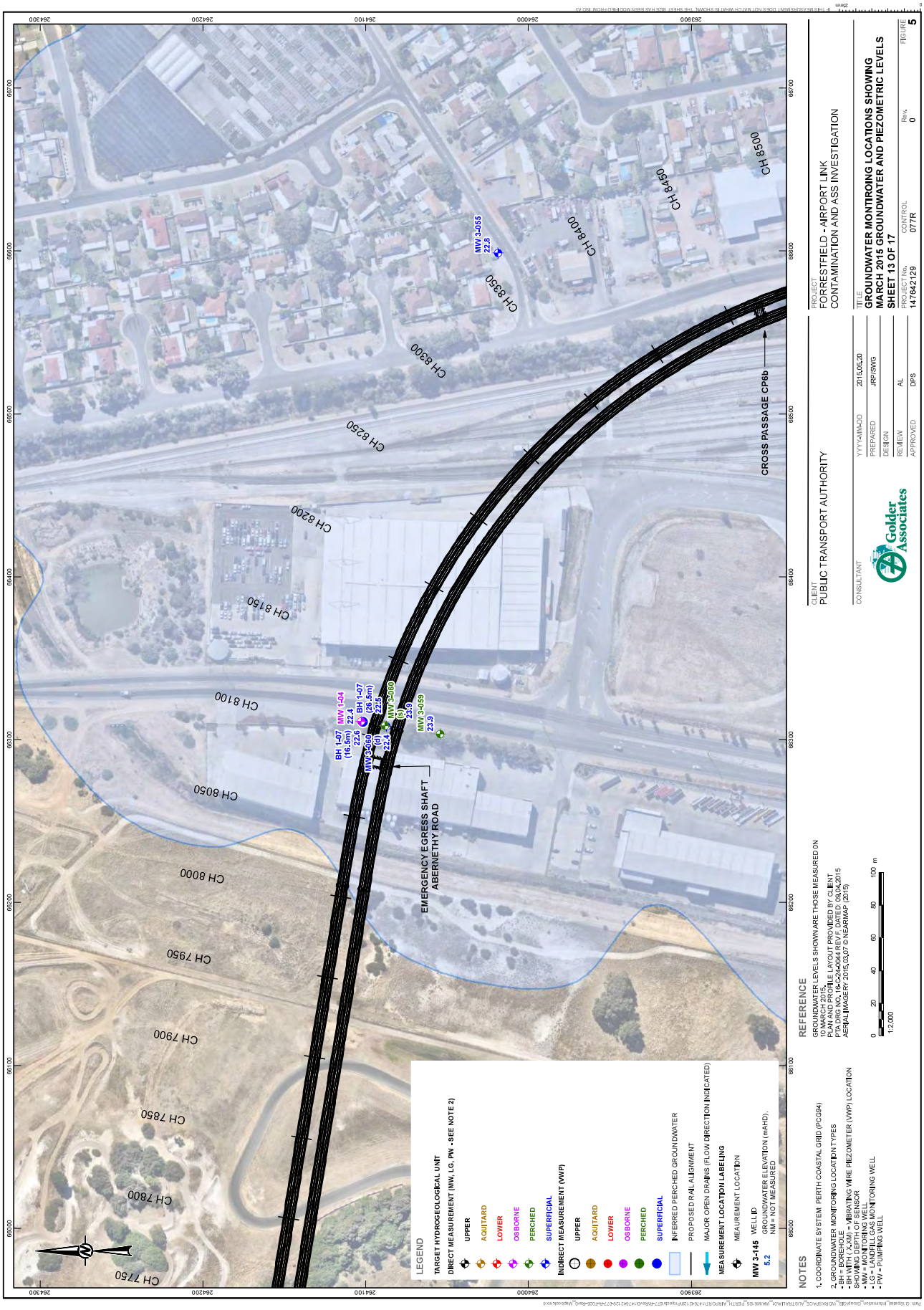

LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELING
MEAUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD), NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (X,XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-24-0044 REV F, DATED: 09.04.2015
AERIAL IMAGERY 2015.03.07 © NEARMAP (2015)
1:2,000
0 20 40 60 80 100 m
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONTIROING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 8 OF 17
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5
EMERGENCY EGRESS SHAFT
DOMESTIC AIRPORT TERMINAL
CROSS PASSAGE CP4
MW 3-086 (d) 12.7
MW 3-086 (s) NM
MW 1-03 12.7
MW 3-087 13.7
MW 3-090 12.3
MW 3-089 11.6
MW 3-092 11.1
MW 3-093 11.2
MW 3-094 10.8
CH 4450
CH 4500
CH 4550
CH 4600
CH 4650
CH 4700
CH 4750
CH 4800
CH 4850
CH 4900
CH 4950
CH 5000
CH 5050
CH 5100
CH 5150
CH 5200
CH 5250

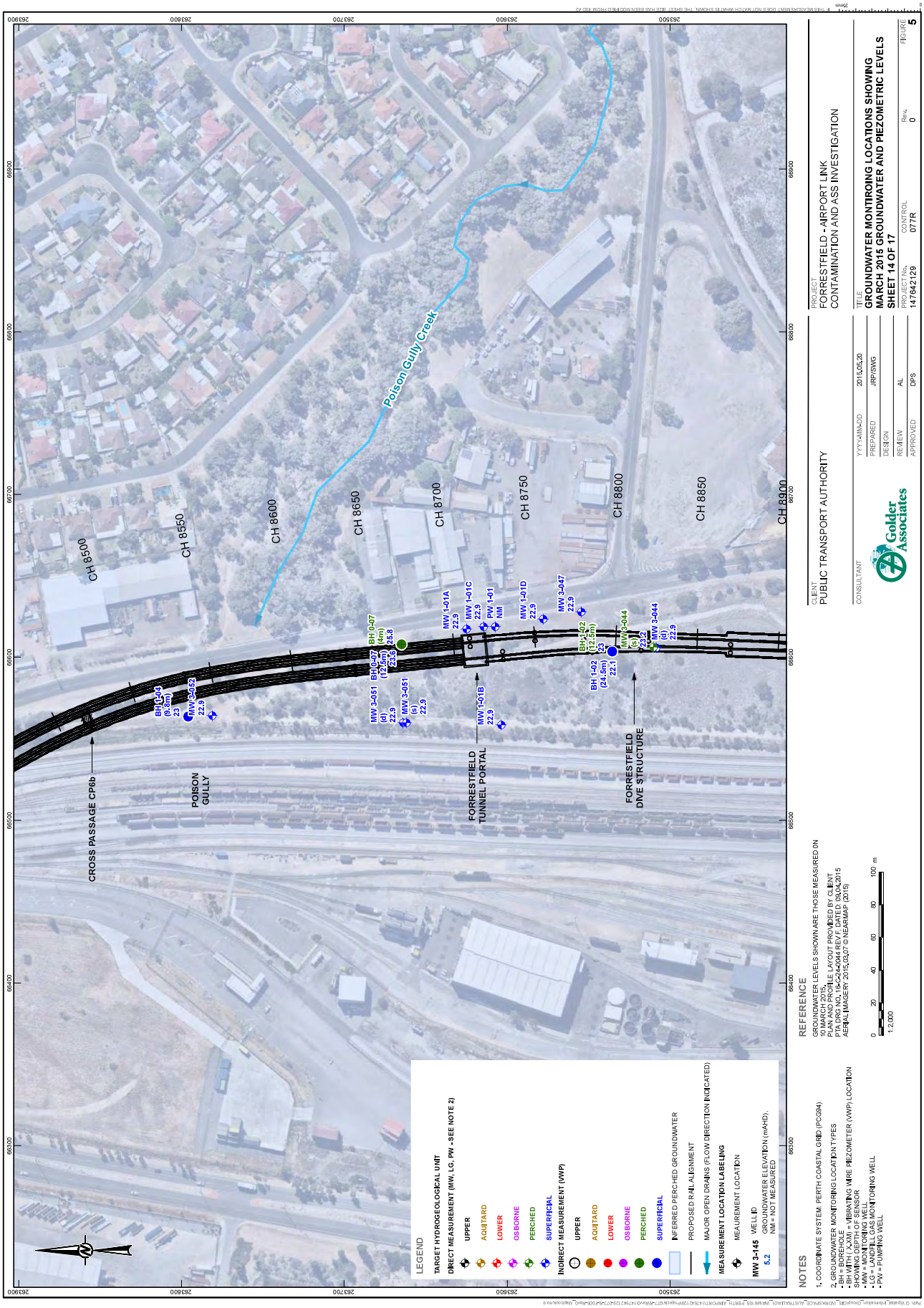

LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
BH 0-01 (18m) 13.4
BH 0-01 (3.5m) 13.5
BH 0-09 (15m) 13.7
BH 0-09 (5m) 13.9
BH 0-10 (17m) 14
BH 0-10 (10m) 14.6
BH 0-10 (2m) 14.5
CROSS PASSAGE CP4a
CH 5200
CH 5250
CH 5300
CH 5350
CH 5400
CH 5450
CH 5500
CH 5550
CH 5600
CH 5650
CH 5700
CH 5750
CH 5800
CH 5850
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 9 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


CROSS PASSAGE CP5
CROSS PASSAGE CP5a
BH 0-05 (5m) 16.5
BH 1-32 (14.6m) 16.5
BH 1-32 (21.1m) 15.5
BH 1-27 (12.4m) 16.5
BH 1-27 (23.2m) 16.5
CH 6450
CH 6400
CH 6350
CH 6300
CH 6250
CH 6200
CH 6150
CH 6100
CH 6050
CH 6000
CH 5950
CH 5900
CH 5850
CH 5800
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 10 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


CONSOLIDATED AIRPORT TERMINAL STATION
CH 7100
CH 7050
CH 7000
CH 6950
CH 6900
CH 6850
CH 6800
CH 6750
CH 6700
CH 6650
CH 6600
CH 6550
CH 6500
CH 6450
CH 6400
MW 1-02A 17
MW 1-02E NM
MW 1-02C 17.1
MW 1-02D 17.1
PW 1-02 17.1
MW 1-02B 17.1
BH 0-06 (13m) 17
BH 0-06 (6m) 16.9
BH 1-21 16.9
BH 1-22 (29.5m) 16.7
BH 1-22 (18m) 16.7
MW 3-077 16.9
BH 1-24 (30.9m) 16.8
MW 3-080 (d) 16.8
MW 3-080 (s) 16.8
BH 1-25 16.9
MW 3-079 17.2
BH 1-27 (14.5m) 16.5
BH 1-27 (23.4m) 16.3
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015 (03.07.15 NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 11 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


CH 7800
CH 7750
CH 7700
CH 7650
CH 7600
CH 7550
CH 7500
CH 7450
CH 7400
CH 7350
CH 7300
CH 7250
CH 7200
CH 7150
CH 7100
CH 7050
CROSS PASSAGE CP6
CROSS PASSAGE CP6a
BH 1-14 (14m) 16.7
BH 1-14 (20m) 16.6
BH 1-13 17.2
MW 3-065 16.3
BH 1-12 17.2
MW 3-064 17.3
BH 1-10 (23m) 18.2
BH 1-10 (15m) 16.2
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD). NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
• BH = BOREHOLE
• BH WITH (-XXM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
• MW = GROUNDWATER MONITORING WELL
• LG = LANDFILL GAS MONITORING WELL
• PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 12 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-M) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015, 03.07 © NEARMAP (2015)
1:2,000
EMERGENCY EGRESS SHAFT
ABERNETHY ROAD
CROSS PASSAGE CP6b
CH 7750
CH 7800
CH 7850
CH 7900
CH 7950
CH 8000
CH 8050
CH 8100
CH 8150
CH 8200
CH 8250
CH 8300
CH 8350
CH 8400
CH 8450
CH 8500
MW 1-04 22.4
BH 1-07 (16.5m) 22.6
BH 1-07 (26.5m) 22.5
MW 3-060 (d) 22.4
MW 3-060 (s) 23.9
MW 3-059 23.9
MW 3-055 22.8
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITROING LOCATIONS SHOWING
MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 13 OF 17
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS


CH 8500
CH 8550
CH 8600
CH 8650
CH 8700
CH 8750
CH 8800
CH 8850
CH 8900
Poison Gully Creek
CROSS PASSAGE CP6b
POISON GULLY
FORRESTFIELD TUNNEL PORTAL
FORRESTFIELD DIVE STRUCTURE
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-M) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = GROUNDWATER MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244-0044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015, 03.07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 14 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


CH 8900
CH 8950
CH 9000
CH 9050
CH 9100
CH 9150
CH 9200
CH 9250
CH 9300
FORRESTFIELD STATION
SCISSORS CROSSOVER
MW 3-038 22.9
MW 3-026 22.8
MW 3-019 26.2
MW 3-034 NM
MW 3-013 23.1
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-M) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244-0044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 15 OF 17
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5


FORRESTFIELD STOWAGE
CH 9300
CH 9350
CH 9400
CH 9450
CH 9492.153
MW 3-013
23.1
MW 3-012
22.9
MW1-105
23.4
MW 3-009
24.5
LEGEND
TARGET HYDROGEOLOGICAL UNIT
DIRECT MEASUREMENT (MW, LG, PW - SEE NOTE 2)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INDIRECT MEASUREMENT (VWP)
UPPER
AQUITARD
LOWER
OSBORNE
PERCHED
SUPERFICIAL
INFERRED PERCHED GROUNDWATER
PROPOSED RAIL ALIGNMENT
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
MEASUREMENT LOCATION LABELLING
MEASUREMENT LOCATION
MW 3-145 WELL ID
5.2 GROUNDWATER ELEVATION (mAHD).
NM = NOT MEASURED
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (-X-M) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = GROUNDWATER WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL
REFERENCE
GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015,03,07 © NEARMAP (2015)
0 20 40 60 80 100 m
1:2,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION
TITLE
GROUNDWATER MONITORING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS
SHEET 16 OF 17
YYYY-MM-DD 2015.05.20
PREPARED JRP/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077R
Rev. 0
FIGURE 5



NOTES

1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)

2. GROUNDWATER MONITORING LOCATION TYPES
- BH = BOREHOLE
- BH WITH (X.XM) = VIBRATING WIRE PIEZOMETER (VWP) LOCATION SHOWING DEPTH OF SENSOR
- MW = MONITORING WELL
- LG = LANDFILL GAS MONITORING WELL
- PW = PUMPING WELL

REFERENCE

GROUNDWATER LEVELS SHOWN ARE THOSE MEASURED ON 10 MARCH 2015.
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-24-0044 REV F, DATED: 09.04.2015
AERIAL IMAGERY 2015.03.07 © NEARMAP (2015)

0 20 40 60 80 100 m
1:2,000

CLIENT
PUBLIC TRANSPORT AUTHORITY

CONSULTANT
Golder Associates

YYYY-MM-DD	2015.05.20
PREPARED	JRP/SWG
DESIGN	
REVIEW	AL
APPROVED	DPS

PROJECT
FORRESTFIELD - AIRPORT LINK
CONTAMINATION AND ASS INVESTIGATION

TITLE
GROUNDWATER MONTIROING LOCATIONS SHOWING MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVELS SHEET 17 OF 17

PROJECT No.	CONTROL	Rev.	FIGURE
147642129	077R	0	5

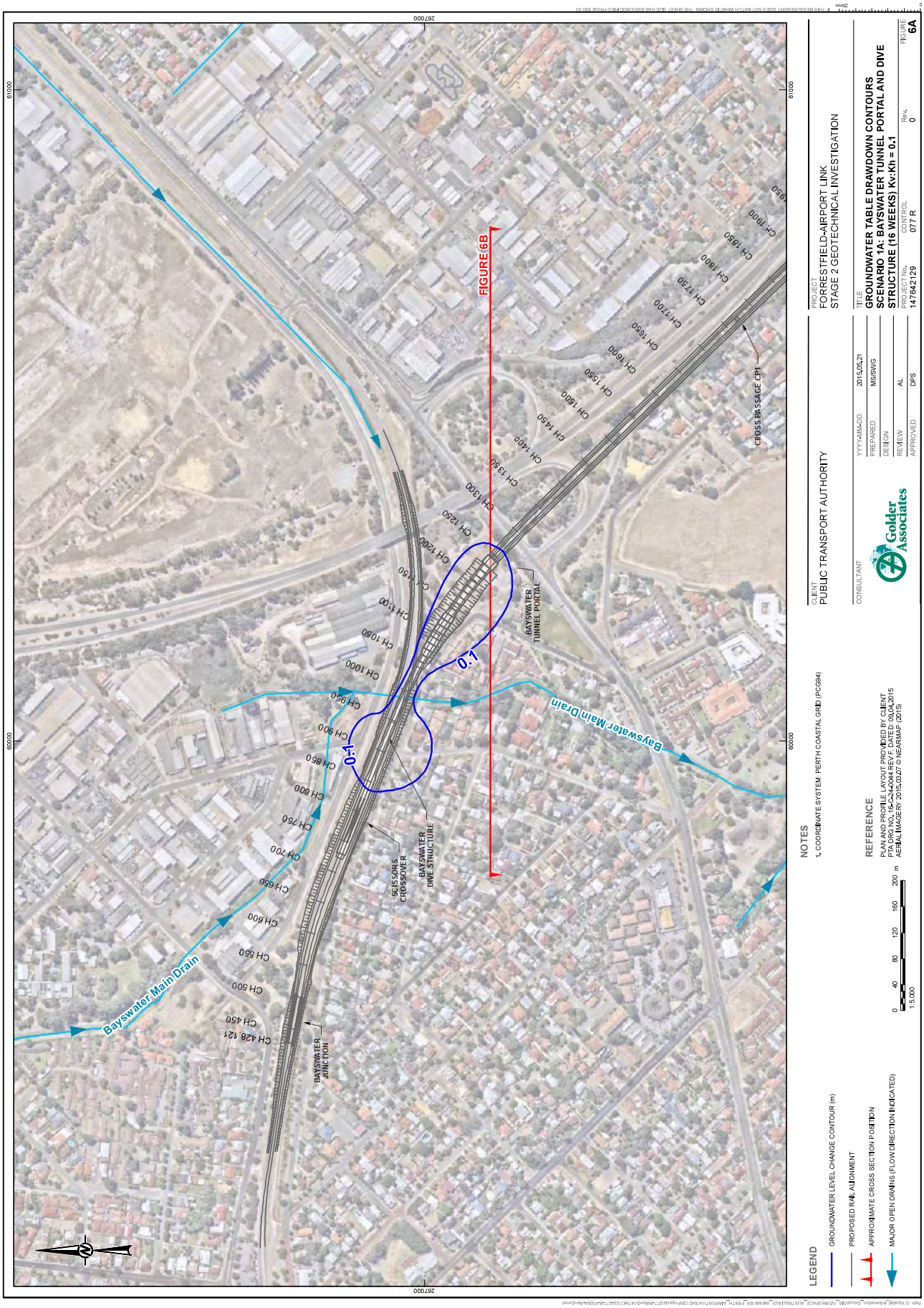
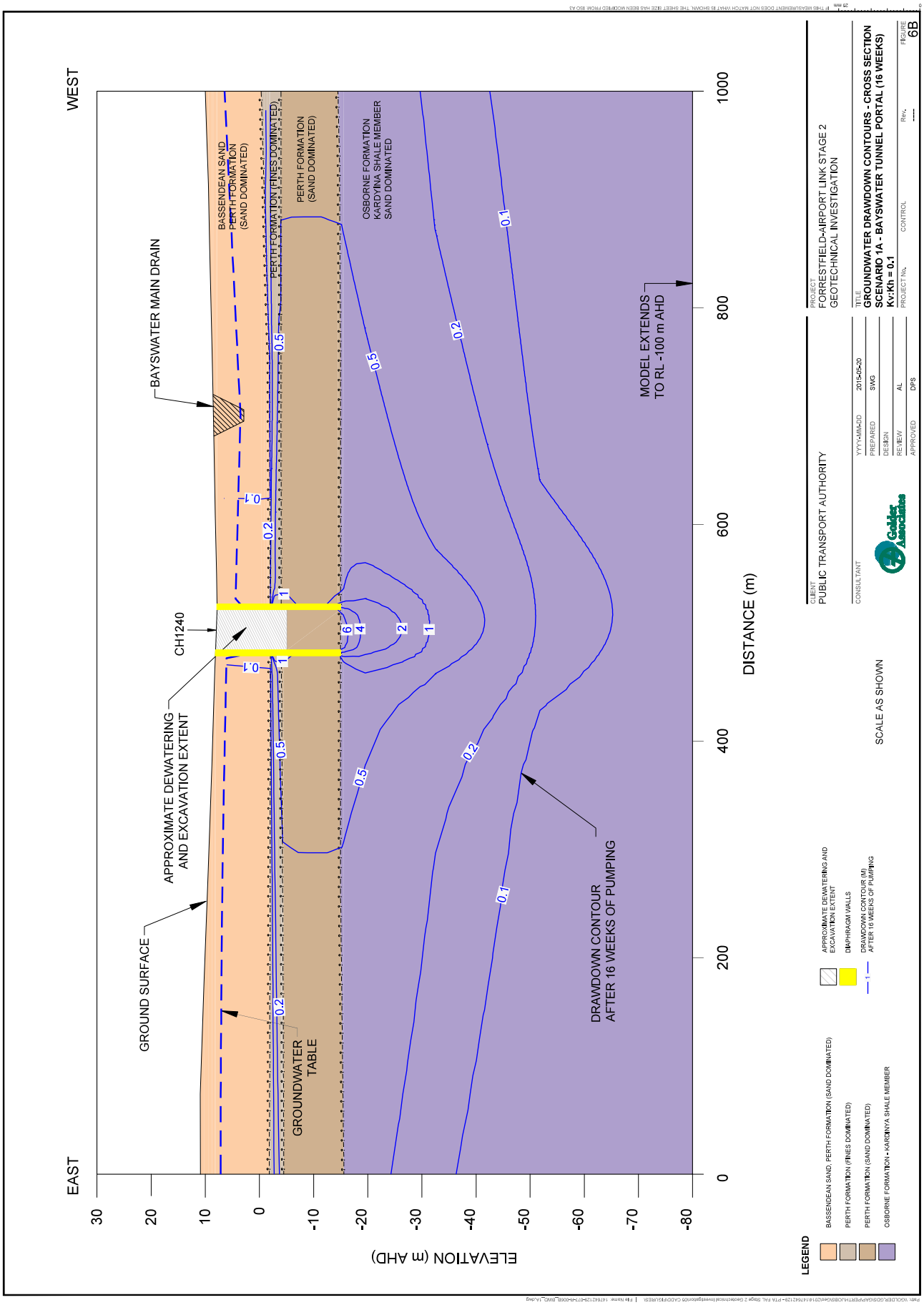
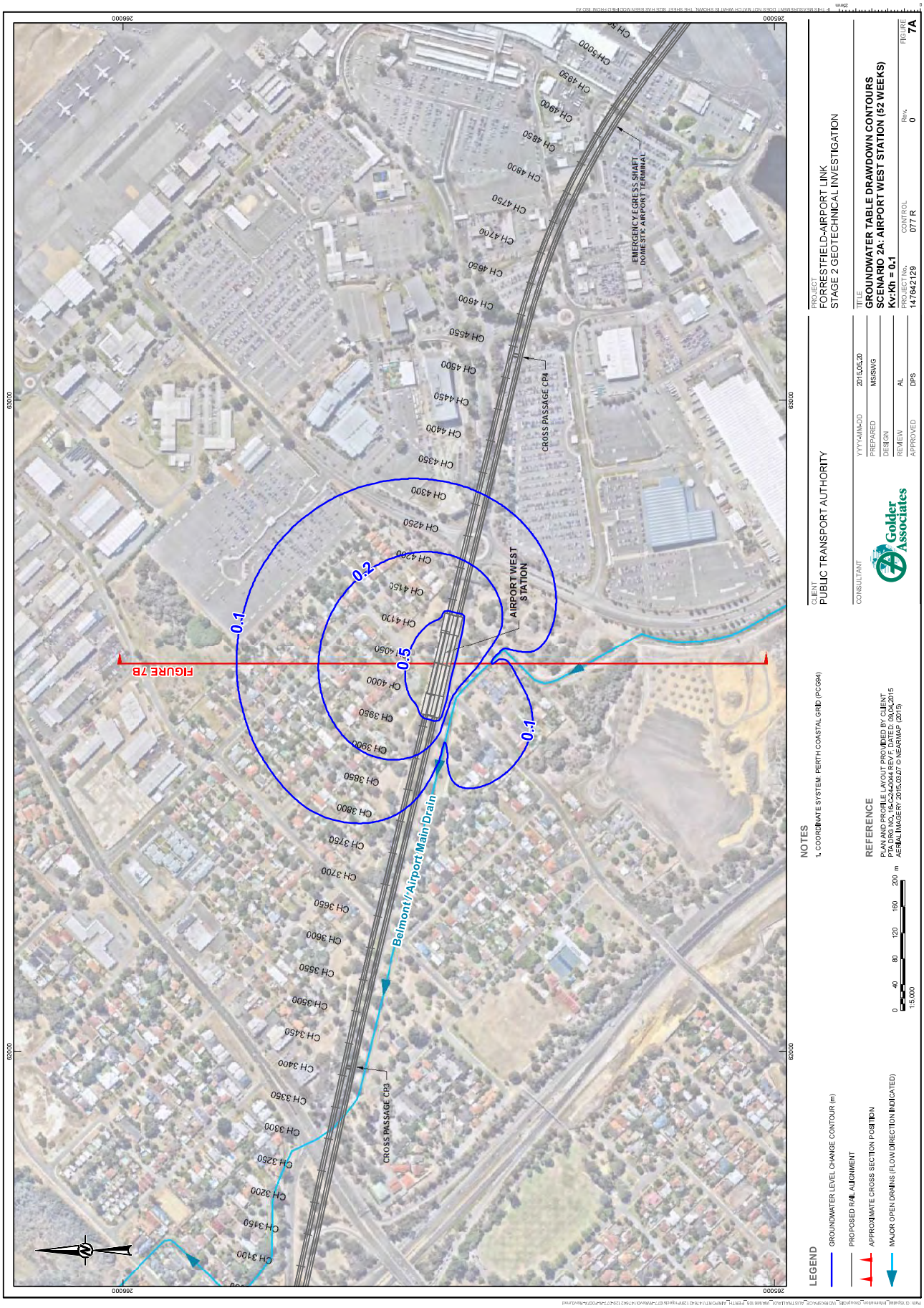
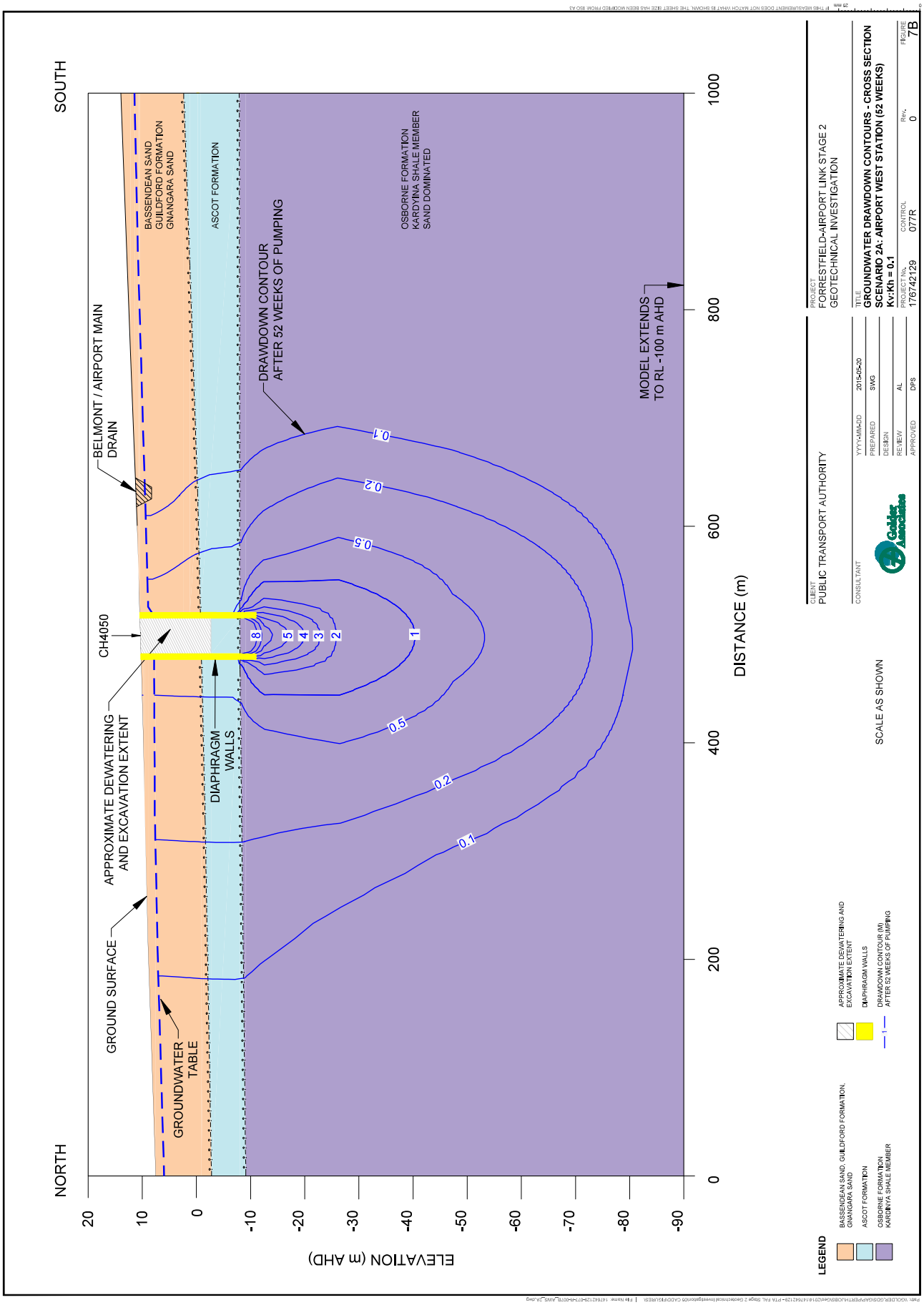
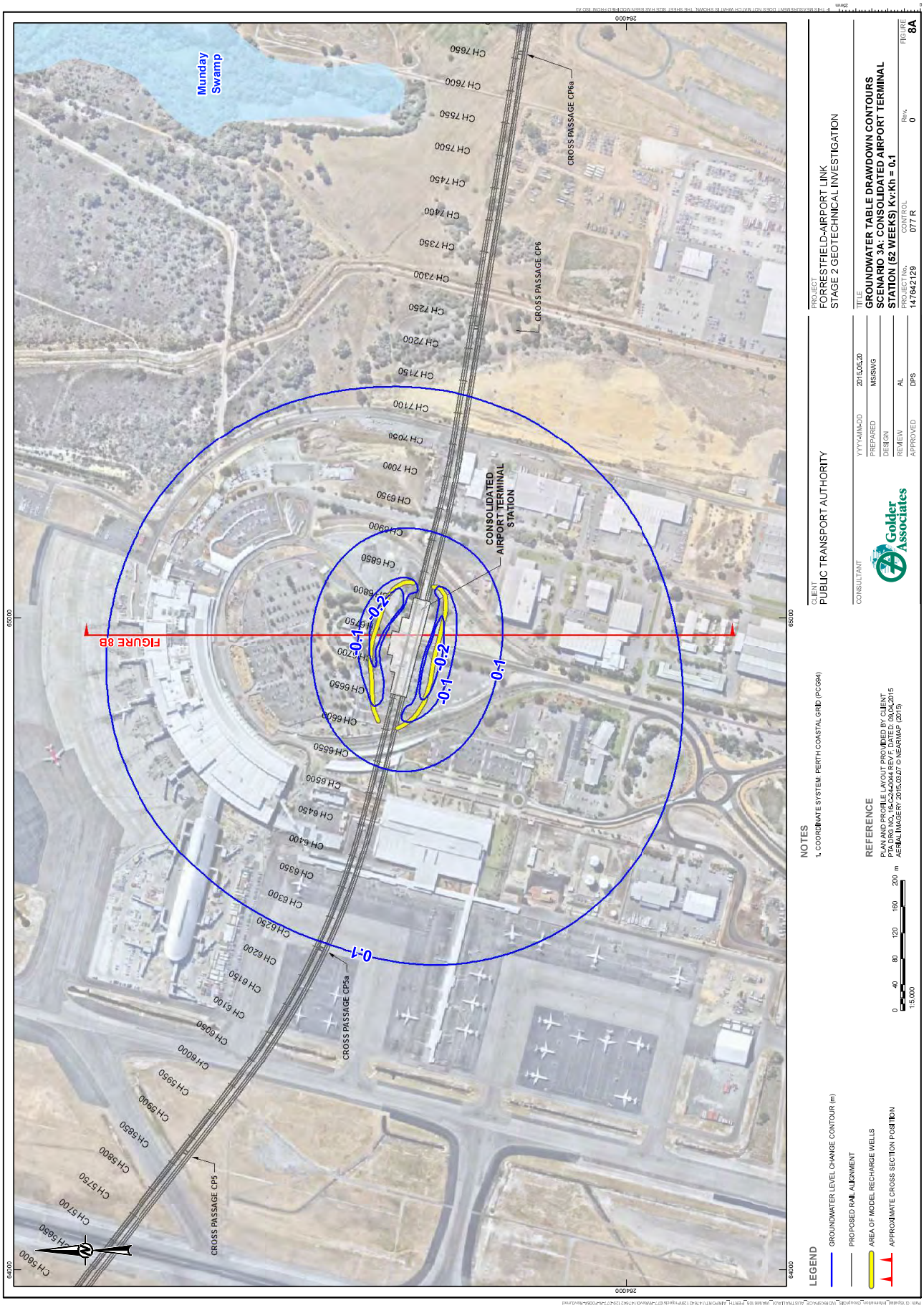


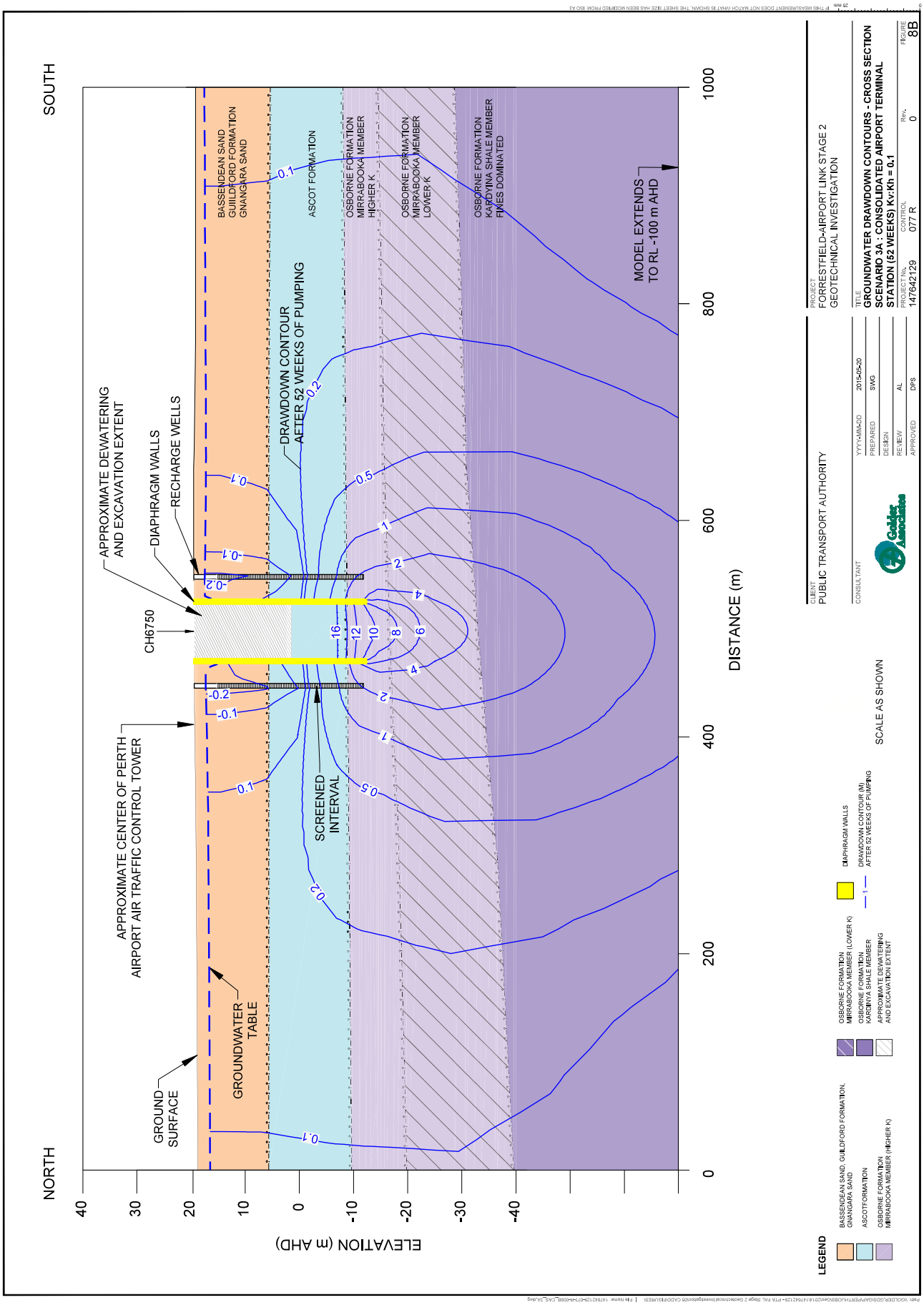
FIGURE 6B
0.1
0.1
BAYSWATER TUNNEL PORTAL
BAYSWATER DIVE STRUCTURE
SCISSORS CROSSOVER
BAYSWATER JUNCTION
CROSS PASSAGE CP1
Bayswater Main Drain
Bayswater Main Drain
CH 950
CH 900
CH 850
CH 800
CH 750
CH 700
CH 650
CH 600
CH 550
CH 500
CH 450
CH 428.121
CH 1000
CH 1050
CH 1100
CH 1150
CH 1200
CH 1250
CH 1300
CH 1350
CH 1400
CH 1450
CH 1500
CH 1550
CH 1600
CH 1650
CH 1700
CH 1750
CH 1800
CH 1850
CH 1900
CH 1950
61000
60000
267000
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY: CLIENT
PTA DRG NO.: 15-C-24-0044 REV: F, DATED: 09.04.2015
AERIAL IMAGERY: 2015.03.07 © NEARMAP (2015)
0 40 80 120 160 200 m
1:5,000
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
APPROXIMATE CROSS SECTION POSITION
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 1A: BAYSWATER TUNNEL PORTAL AND DIVE STRUCTURE (16 WEEKS) Kv:Kh = 0.1
YYYY-MM-DD 2015.05.21
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 6A

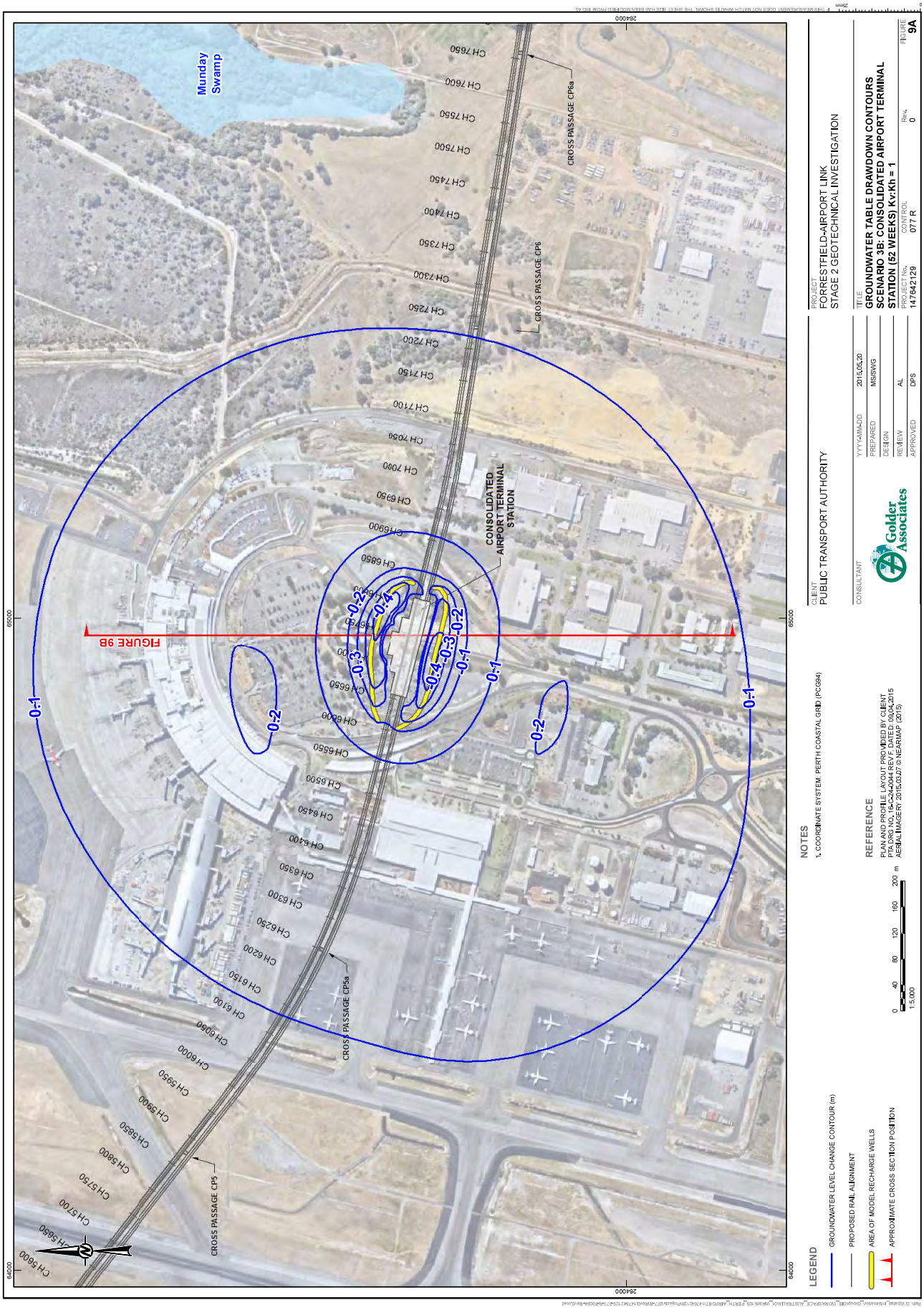

WEST
EAST
BAYSWATER MAIN DRAIN
BASSENDEAN SAND
PERTH FORMATION
(SAND DOMINATED)
PERTH FORMATION (FINES DOMINATED)
PERTH FORMATION
(SAND DOMINATED)
OSBORNE FORMATION
KARDINYA SHALE MEMBER
SAND DOMINATED
MODEL EXTENDS
TO RL -100 m AHD
CH1240
APPROXIMATE DEWATERING
AND EXCAVATION EXTENT
GROUND SURFACE
GROUNDWATER
TABLE
DRAWDOWN CONTOUR
AFTER 16 WEEKS OF PUMPING
DISTANCE (m)
ELEVATION (m AHD)
SCALE AS SHOWN
LEGEND
BASSENDEAN SAND, PERTH FORMATION (SAND DOMINATED)
PERTH FORMATION (FINES DOMINATED)
PERTH FORMATION (SAND DOMINATED)
OSBORNE FORMATION - KARDINYA SHALE MEMBER
APPROXIMATE DEWATERING AND
EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M)
AFTER 16 WEEKS OF PUMPING
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 1A - BAYSWATER TUNNEL PORTAL (16 WEEKS)
Kv:Kh = 0.1
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No.
CONTROL
REV.
FIGURE
6B

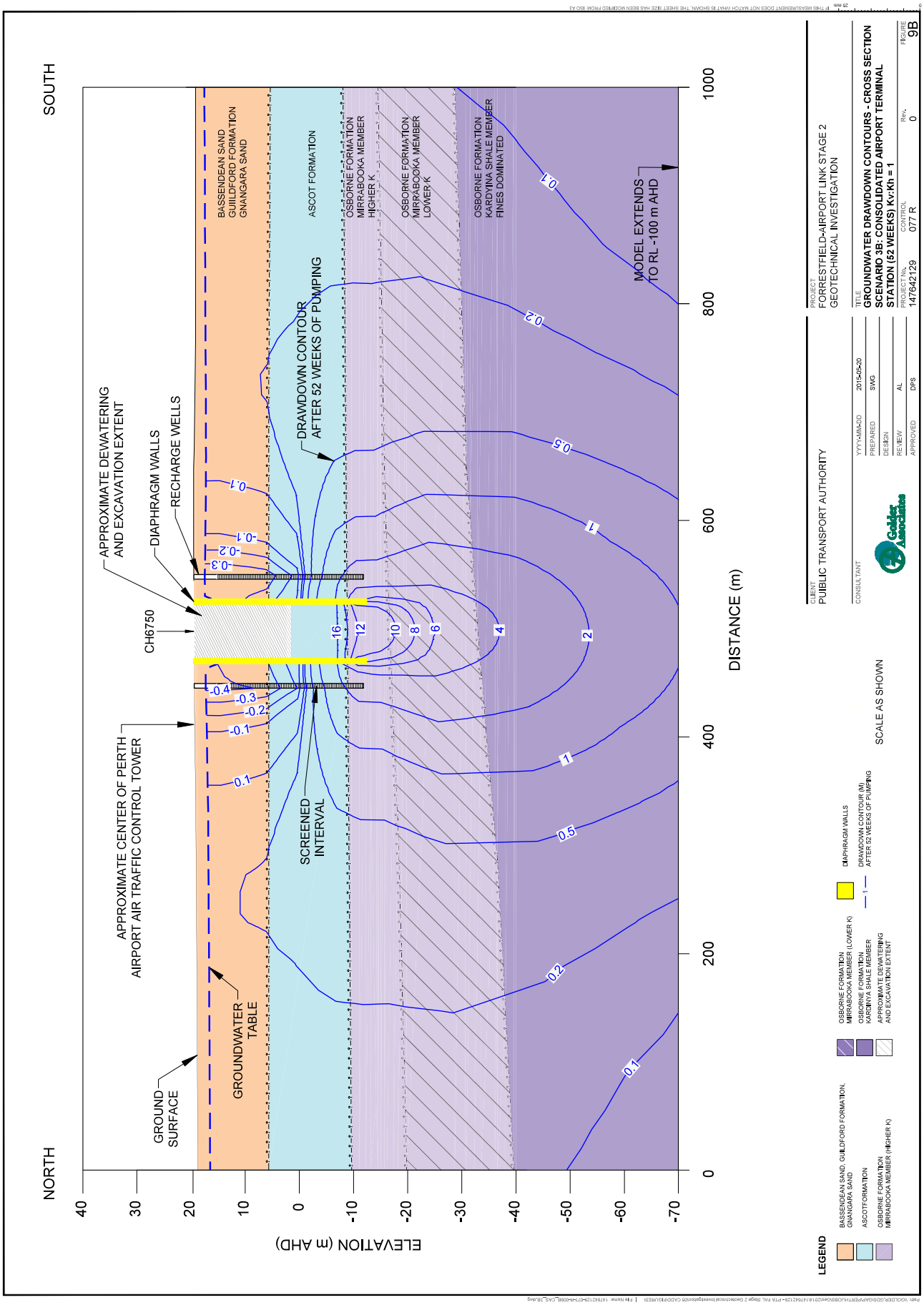

AIRPORT WEST STATION
EMERGENCY EGRESS SHAFT DOMESTIC AIRPORT TERMINAL
CROSS PASSAGE CP4
CROSS PASSAGE CP3
Belmont//Airport Main Drain
0.1
0.2
0.5
0.1
FIGURE 7B
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY: CLIENT
PTA DRG NO.: 15-C-24-0044 REV: F, DATED: 09.04.2015
AERIAL IMAGERY: 2015.03.07 © NEARMAP (2015)
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
APPROXIMATE CROSS SECTION POSITION
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
1:5,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 2A: AIRPORT WEST STATION (52 WEEKS)
Kv:Kh = 0.1
YYYY-MM-DD 2015.05.20
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 7A


SOUTH
NORTH
BELMONT / AIRPORT MAIN DRAIN
CH4050
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
GROUND SURFACE
GROUNDWATER TABLE
DIAPHRAGM WALLS
BASSENDEAN SAND
GUILDFORD FORMATION
GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION
KARDYNA SHALE MEMBER
SAND DOMINATED
DRAWDOWN CONTOUR
AFTER 52 WEEKS OF PUMPING
MODEL EXTENDS
TO RL -100 m AHD
0.1
0.2
0.5
1
2
3
4
5
8
ELEVATION (m AHD)
DISTANCE (m)
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
0
200
400
600
800
1000
LEGEND
BASSENDEAN SAND, GUILDFORD FORMATION, GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION KARDINYA SHALE MEMBER
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M) AFTER 52 WEEKS OF PUMPING
SCALE AS SHOWN
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 2A: AIRPORT WEST STATION (52 WEEKS)
Kv:Kh = 0.1
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 176742129
CONTROL 077R
REV. 0
FIGURE 7B


Munday Swamp
CONSOLIDATED AIRPORT TERMINAL STATION
CROSS PASSAGE CP6a
CROSS PASSAGE CP6
CROSS PASSAGE CP5a
CROSS PASSAGE CP5
FIGURE 8B
-0.1
-0.2
-0.1
-0.2
-0.1
-0.1
CH 7650
CH 7600
CH 7550
CH 7500
CH 7450
CH 7400
CH 7350
CH 7300
CH 7250
CH 7200
CH 7150
CH 7100
CH 7050
CH 7000
CH 6950
CH 6900
CH 6850
CH 6800
CH 6750
CH 6700
CH 6650
CH 6600
CH 6550
CH 6500
CH 6450
CH 6400
CH 6350
CH 6300
CH 6250
CH 6200
CH 6150
CH 6100
CH 6050
CH 6000
CH 5950
CH 5900
CH 5850
CH 5800
CH 5750
CH 5700
CH 5650
CH 5600
264000
65000
64000
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY: CLIENT
PTA DRG NO.: 15-C-24-0044 REV: F, DATED: 09.04.2015
AERIAL IMAGERY: 2015.03.07 © NEARMAP (2015)
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
AREA OF MODEL RECHARGE WELLS
APPROXIMATE CROSS SECTION POSITION
0 40 80 120 160 200 m
1:5,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 3A: CONSOLIDATED AIRPORT TERMINAL
STATION (52 WEEKS) Kv:Kh = 0.1
YYYY-MM-DD 2015.05.20
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 8A


SOUTH
NORTH
BASSENDEAN SAND
GUILDFORD FORMATION
GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION
MIRRABOOKA MEMBER
HIGHER K
OSBORNE FORMATION
MIRRABOOKA MEMBER
LOWER K
OSBORNE FORMATION
KARDINYA SHALE MEMBER
FINES DOMINATED
MODEL EXTENDS
TO RL -100 m AHD
APPROXIMATE DEWATERING
AND EXCAVATION EXTENT
DIAPHRAGM WALLS
RECHARGE WELLS
CH6750
DRAWDOWN CONTOUR
AFTER 52 WEEKS OF PUMPING
APPROXIMATE CENTER OF PERTH
AIRPORT AIR TRAFFIC CONTROL TOWER
GROUNDWATER
TABLE
GROUND
SURFACE
SCREENED
INTERVAL
ELEVATION (m AHD)
DISTANCE (m)
LEGEND
BASSENDEAN SAND, GUILDFORD FORMATION, GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION MIRRABOOKA MEMBER (HIGHER K)
OSBORNE FORMATION MIRRABOOKA MEMBER (LOWER K)
OSBORNE FORMATION KARDINYA SHALE MEMBER
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M) AFTER 52 WEEKS OF PUMPING
SCALE AS SHOWN
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 3A : CONSOLIDATED AIRPORT TERMINAL
STATION (52 WEEKS) Kv:Kh = 0.1
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
REV. 0
FIGURE 8B


Munday Swamp
CONSOLIDATED AIRPORT TERMINAL STATION
CROSS PASSAGE CP6a
CROSS PASSAGE CP6
CROSS PASSAGE CP5a
CROSS PASSAGE CP5
FIGURE 9B
-0.1
-0.2
-0.3
-0.4
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY: CLIENT
PTA DRG NO.: 15-C-24-0044 REV: F, DATED: 09.04.2015
AERIAL IMAGERY: 2015.03.07 © NEARMAP (2015)
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
AREA OF MODEL RECHARGE WELLS
APPROXIMATE CROSS SECTION POSITION
1:5,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 3B: CONSOLIDATED AIRPORT TERMINAL
STATION (52 WEEKS) Kv:Kh = 1
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 9A
YYYY-MM-DD 2015.05.20
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS


SOUTH
NORTH
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
RECHARGE WELLS
CH6750
APPROXIMATE CENTER OF PERTH AIRPORT AIR TRAFFIC CONTROL TOWER
GROUND SURFACE
GROUNDWATER TABLE
SCREENED INTERVAL
DRAWDOWN CONTOUR AFTER 52 WEEKS OF PUMPING
BASSENDEAN SAND
GUILDFORD FORMATION
GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION MIRRABOOKA MEMBER HIGHER K
OSBORNE FORMATION MIRRABOOKA MEMBER LOWER K
OSBORNE FORMATION KARDINYA SHALE MEMBER FINES DOMINATED
MODEL EXTENDS TO RL -100 m AHD
ELEVATION (m AHD)
DISTANCE (m)
LEGEND
BASSENDEAN SAND, GUILDFORD FORMATION, GNANGARA SAND
ASCOT FORMATION
OSBORNE FORMATION MIRRABOOKA MEMBER (HIGHER K)
OSBORNE FORMATION MIRRABOOKA MEMBER (LOWER K)
OSBORNE FORMATION KARDINYA SHALE MEMBER
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M) AFTER 52 WEEKS OF PUMPING
SCALE AS SHOWN
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 3B: CONSOLIDATED AIRPORT TERMINAL
STATION (52 WEEKS) Kv:Kh = 1
PROJECT No. 147642129
CONTROL 077 R
REV. 0
FIGURE 9B
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS

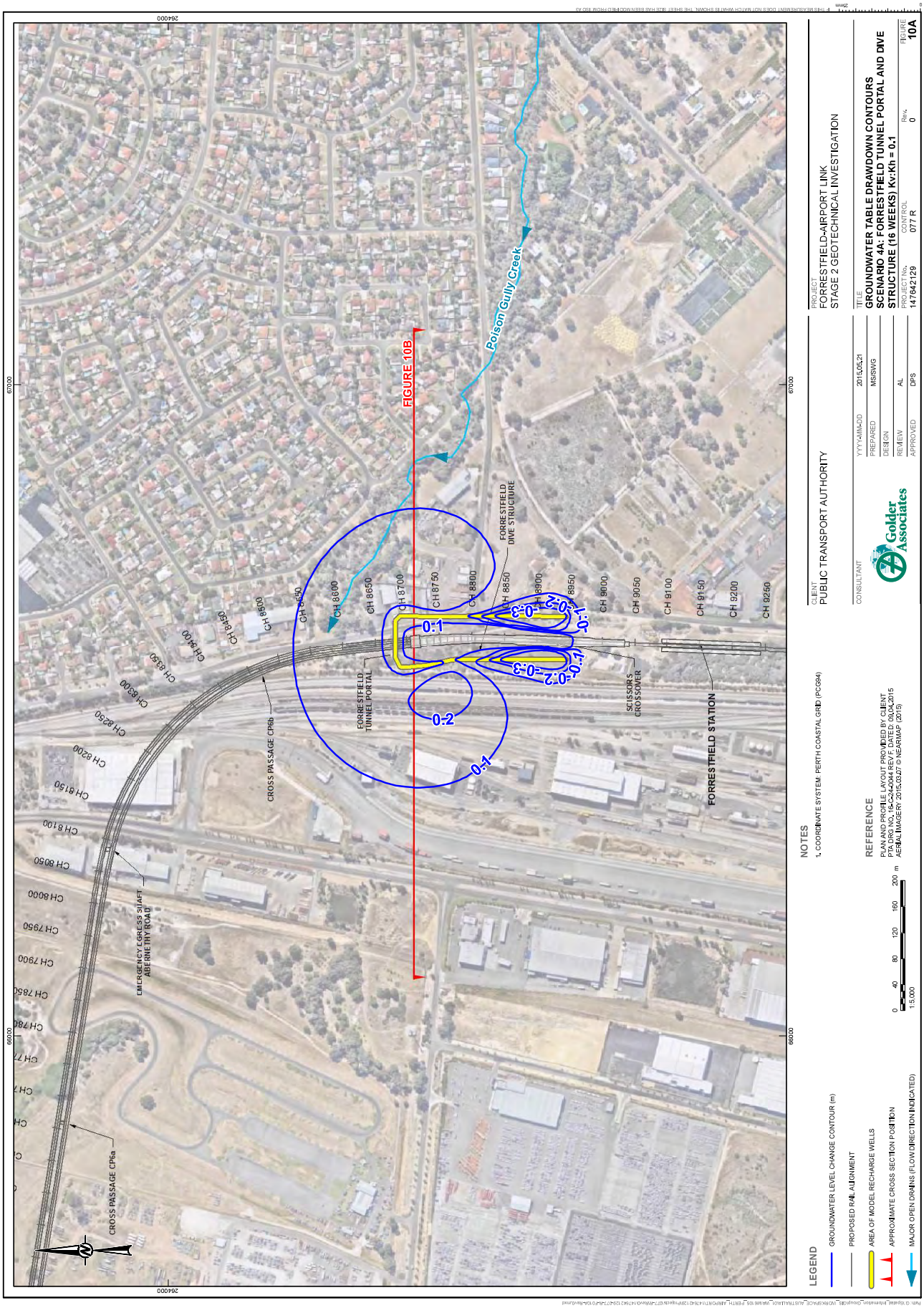
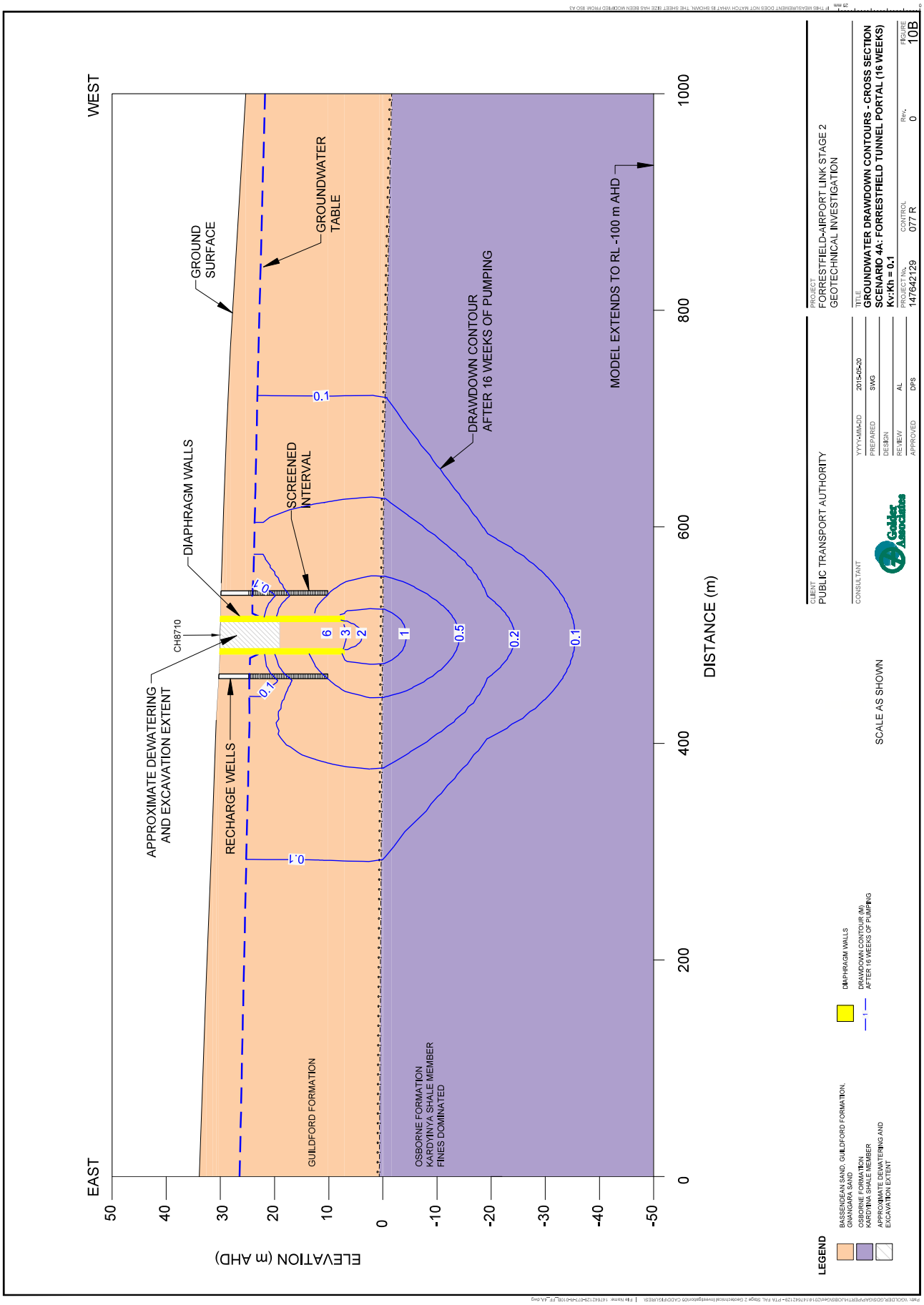


FIGURE 10B
Poison Gully Creek
FORRESTFIELD DIVE STRUCTURE
FORRESTFIELD TUNNEL PORTAL
SCISSORS CROSSOVER
FORRESTFIELD STATION
CROSS PASSAGE CP6b
CROSS PASSAGE CP6a
EMERGENCY EGRESS SHAFT ABERNETHY ROAD
CH 8600
CH 8650
CH 8700
CH 8750
CH 8800
CH 8850
CH 8900
CH 8950
CH 9000
CH 9050
CH 9100
CH 9150
CH 9200
CH 9250
CH 8500
CH 8450
CH 8400
CH 8350
CH 8300
CH 8250
CH 8200
CH 8150
CH 8100
CH 8050
CH 8000
CH 7950
CH 7900
0.1
0.2
-0.1
-0.2
-0.3
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
AREA OF MODEL RECHARGE WELLS
APPROXIMATE CROSS SECTION POSITION
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-24-0044 REV F, DATED: 09.04.2015
AERIAL IMAGERY 2015.03.07 © NEARMAP (2015)
0 40 80 120 160 200 m
1:5,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.21
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 4A: FORRESTFIELD TUNNEL PORTAL AND DIVE STRUCTURE (16 WEEKS) Kv:Kh = 0.1
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 10A


WEST
EAST
GROUND SURFACE
GROUNDWATER TABLE
DIAPHRAGM WALLS
SCREENED INTERVAL
CH8710
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
RECHARGE WELLS
DRAWDOWN CONTOUR AFTER 16 WEEKS OF PUMPING
MODEL EXTENDS TO RL -100 m AHD
GUILDFORD FORMATION
OSBORNE FORMATION KARDYINYA SHALE MEMBER FINES DOMINATED
0.1
0.2
0.5
1
2
3
6
DISTANCE (m)
ELEVATION (m AHD)
LEGEND
BASSENDEAN SAND, GUILDFORD FORMATION, GNANGARA SAND
OSBORNE FORMATION KARDYINYA SHALE MEMBER
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M) AFTER 16 WEEKS OF PUMPING
SCALE AS SHOWN
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 4A: FORRESTFIELD TUNNEL PORTAL (16 WEEKS)
Kv:Kh = 0.1
PROJECT No.
147642129
CONTROL
077 R
REV.
0
FIGURE
10B
YYYY-MM-DD
2015-05-20
PREPARED
SWG
DESIGN
REVIEW
AL
APPROVED
DPS

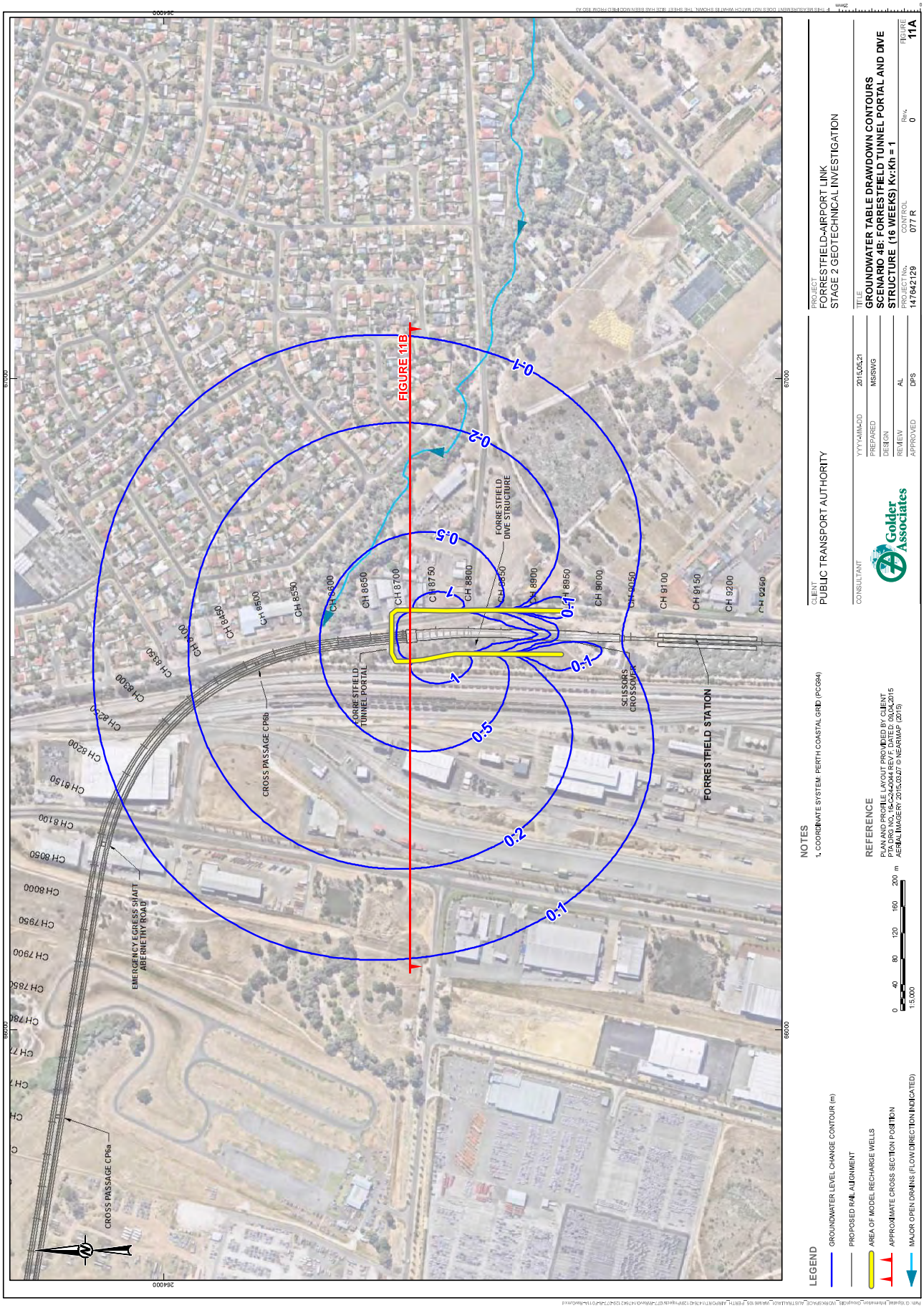
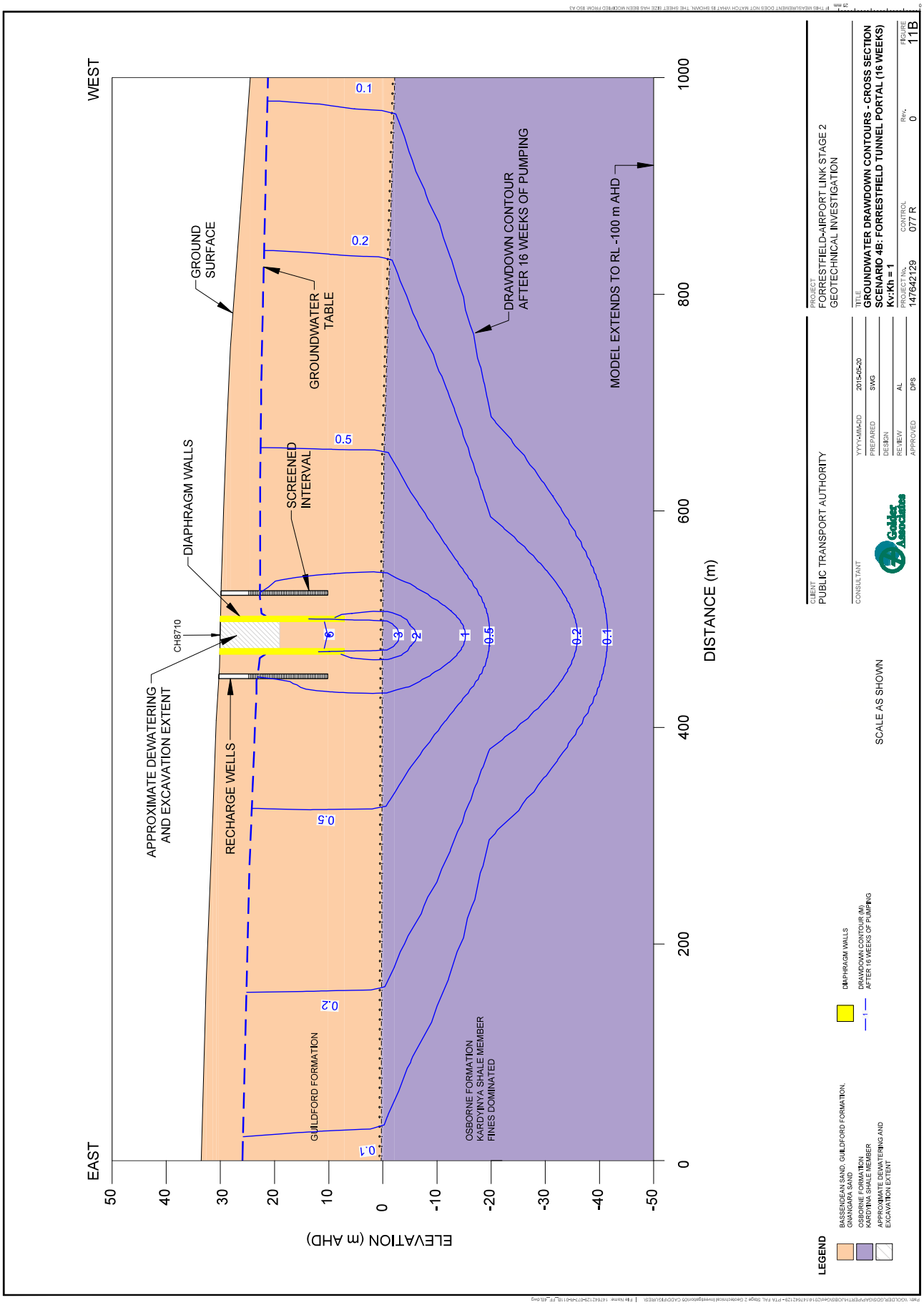
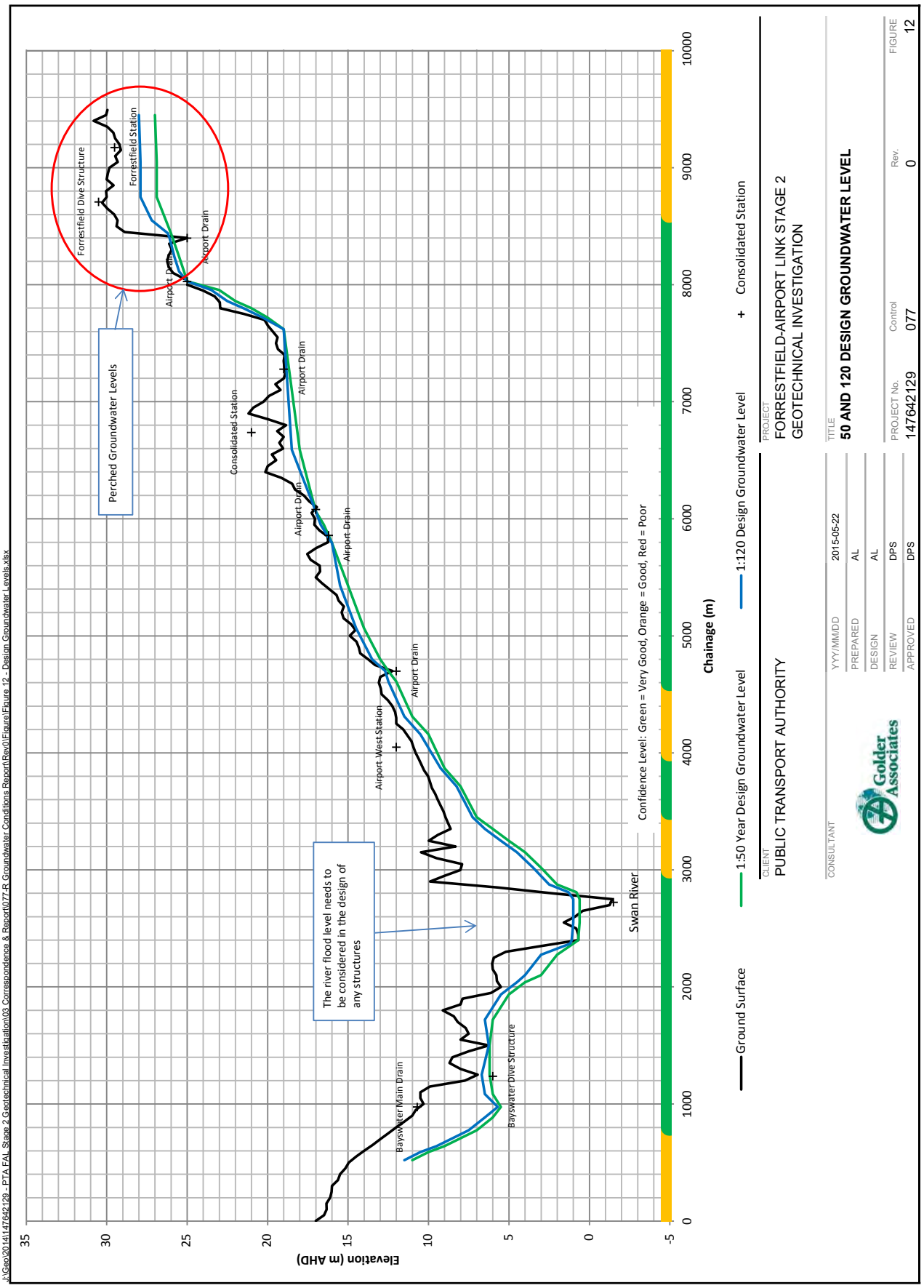


FIGURE 11B
0.1
0.2
0.5
1
FORRESTFIELD DIVE STRUCTURE
FORRESTFIELD TUNNEL PORTAL
CROSS PASSAGE CP6b
CROSS PASSAGE CP6a
EMERGENCY EGRESS SHAFT ABERNETHY ROAD
SCISSORS CROSSOVER
FORRESTFIELD STATION
CH 8450
CH 8500
CH 8550
CH 8600
CH 8650
CH 8700
CH 8750
CH 8800
CH 8850
CH 8900
CH 8950
CH 9000
CH 9050
CH 9100
CH 9150
CH 9200
CH 9250
CH 8100
CH 8150
CH 8200
CH 8250
CH 8300
CH 8350
CH 8400
CH 8450
CH 8500
CH 8550
CH 8600
CH 8650
CH 8700
CH 8750
CH 8800
CH 8850
CH 8900
CH 8950
CH 9000
CH 9050
CH 9100
CH 9150
CH 9200
CH 9250
LEGEND
GROUNDWATER LEVEL CHANGE CONTOUR (m)
PROPOSED RAIL ALIGNMENT
AREA OF MODEL RECHARGE WELLS
APPROXIMATE CROSS SECTION POSITION
MAJOR OPEN DRAINS (FLOW DIRECTION INDICATED)
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 16-C-24-0044 REV F, DATED: 09.04.2015
AERIAL IMAGERY 2015.03.07 © NEARMAP (2015)
0 40 80 120 160 200 m
1:5,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
YYYY-MM-DD 2015.05.21
PREPARED MS/SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER TABLE DRAWDOWN CONTOURS
SCENARIO 4B: FORRESTFIELD TUNNEL PORTAL AND DIVE STRUCTURE (16 WEEKS) Kv:Kh = 1
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE 11A

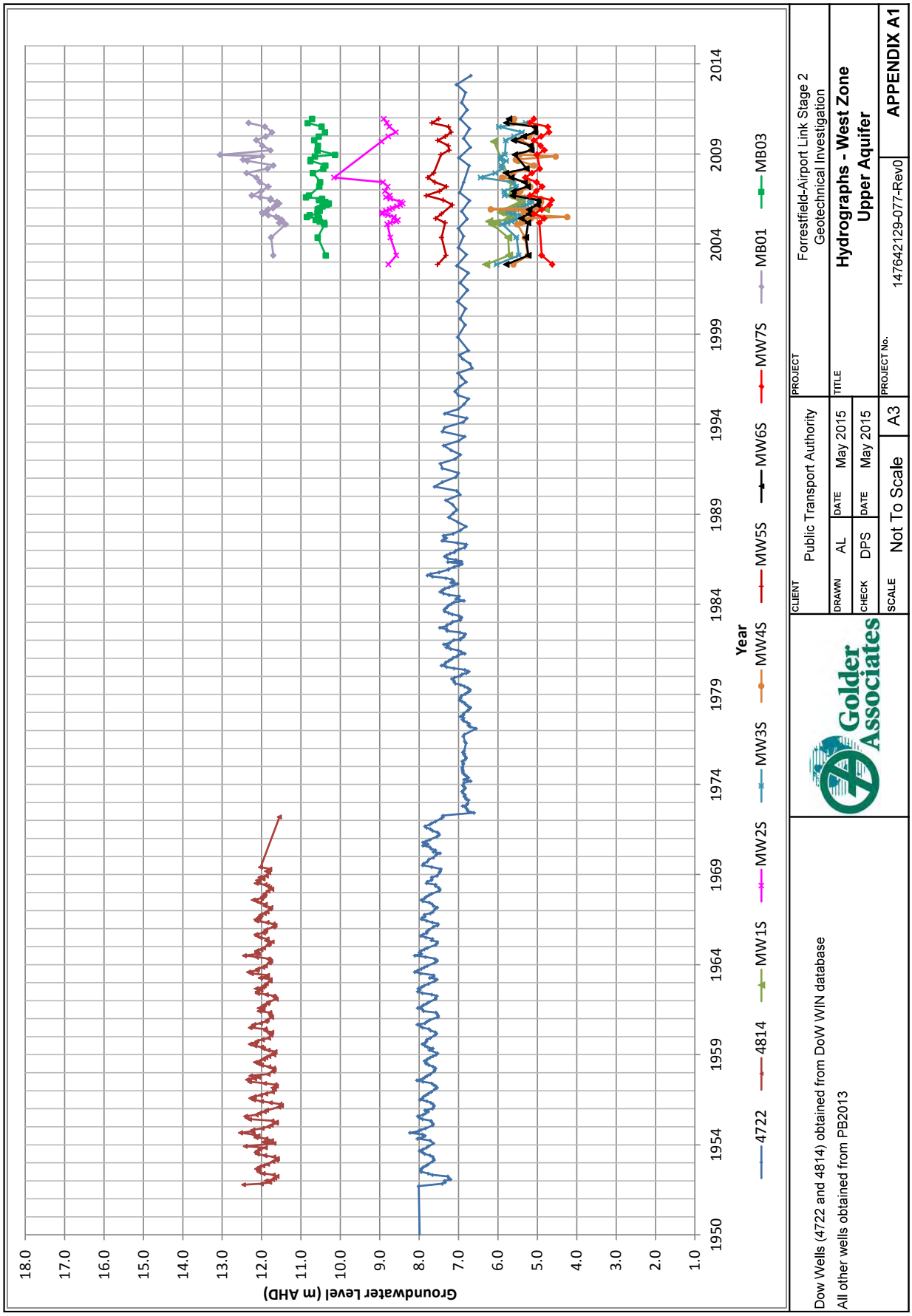

WEST
EAST
GROUND SURFACE
GROUNDWATER TABLE
DIAPHRAGM WALLS
SCREENED INTERVAL
CH8710
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
RECHARGE WELLS
DRAWDOWN CONTOUR AFTER 16 WEEKS OF PUMPING
MODEL EXTENDS TO RL -100 m AHD
GUILDFORD FORMATION
OSBORNE FORMATION KARDINYA SHALE MEMBER FINES DOMINATED
0.1
0.2
0.5
1
2
3
6
0.6
0.2
0.1
0.5
0.2
0.1
ELEVATION (m AHD)
DISTANCE (m)
50
40
30
20
10
0
-10
-20
-30
-40
-50
0
200
400
600
800
1000
LEGEND
BASSENDEAN SAND, GUILDFORD FORMATION, GNANGARA SAND
OSBORNE FORMATION
KARDINYA SHALE MEMBER
APPROXIMATE DEWATERING AND EXCAVATION EXTENT
DIAPHRAGM WALLS
DRAWDOWN CONTOUR (M) AFTER 16 WEEKS OF PUMPING
SCALE AS SHOWN
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER DRAWDOWN CONTOURS - CROSS SECTION
SCENARIO 4B: FORRESTFIELD TUNNEL PORTAL (16 WEEKS)
Kv:Kh = 1
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
REV. 0
FIGURE 11B

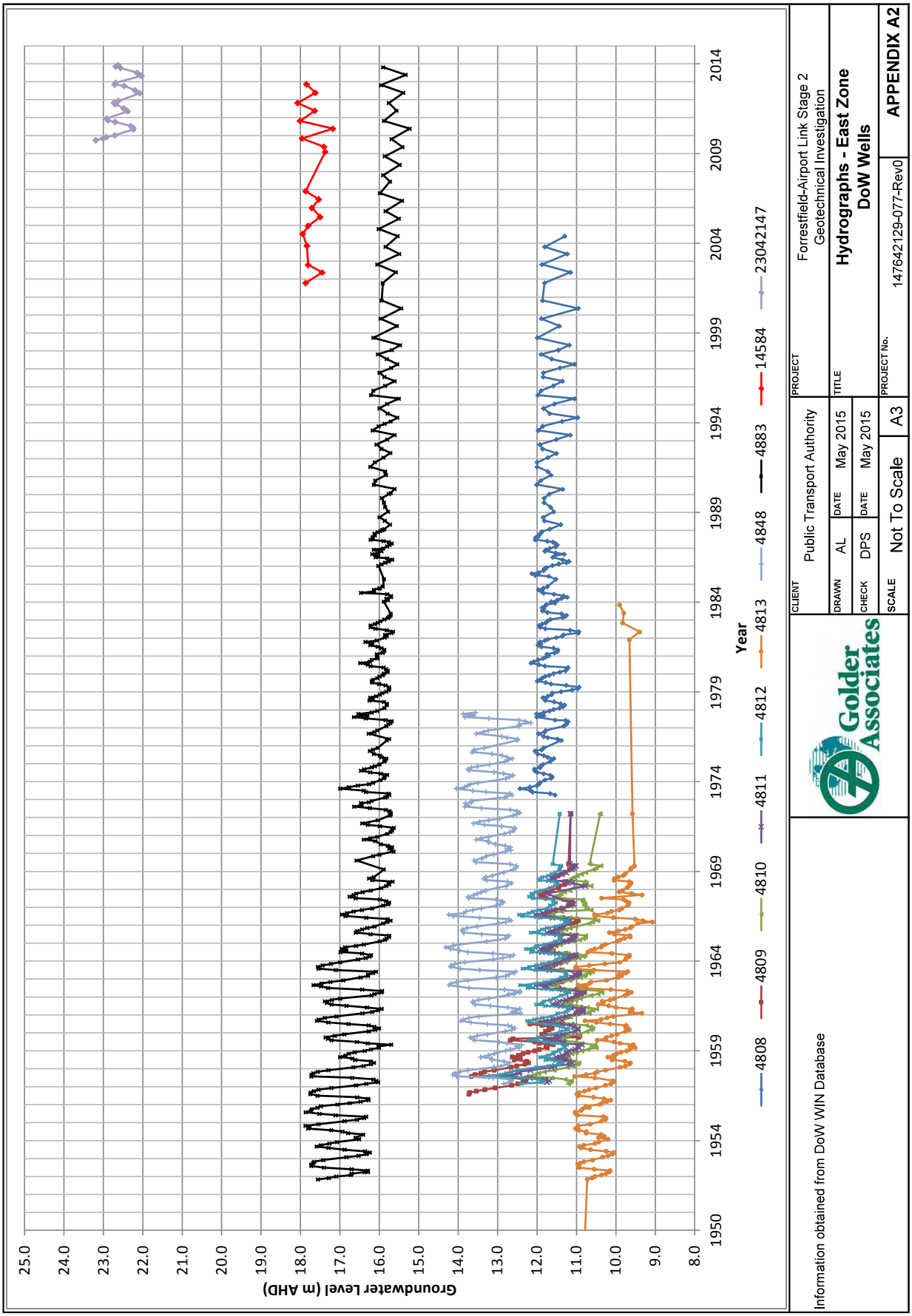

J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Figure\Figure 12 - Design Groundwater Levels.xlsx
Elevation (m AHD)
Chainage (m)
Ground Surface
1:50 Year Design Groundwater Level
1:120 Design Groundwater Level
Consolidated Station
Confidence Level: Green = Very Good, Orange = Good, Red = Poor
Swan River
Perched Groundwater Levels
The river flood level needs to be considered in the design of any structures
Forrestfield Dive Structure
Forrestfield Station
Airport Drain
Consolidated Station
Airport West Station
Bayswater Main Drain
Bayswater Dive Structure
CLIENT
PUBLIC TRANSPORT AUTHORITY
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
50 AND 120 DESIGN GROUNDWATER LEVEL
CONSULTANT
Golder Associates
YYYY/MM/DD 2015-05-22
PREPARED AL
DESIGN AL
REVIEW DPS
APPROVED DPS
PROJECT No. 147642129
Control 077
Rev. 0
FIGURE 12

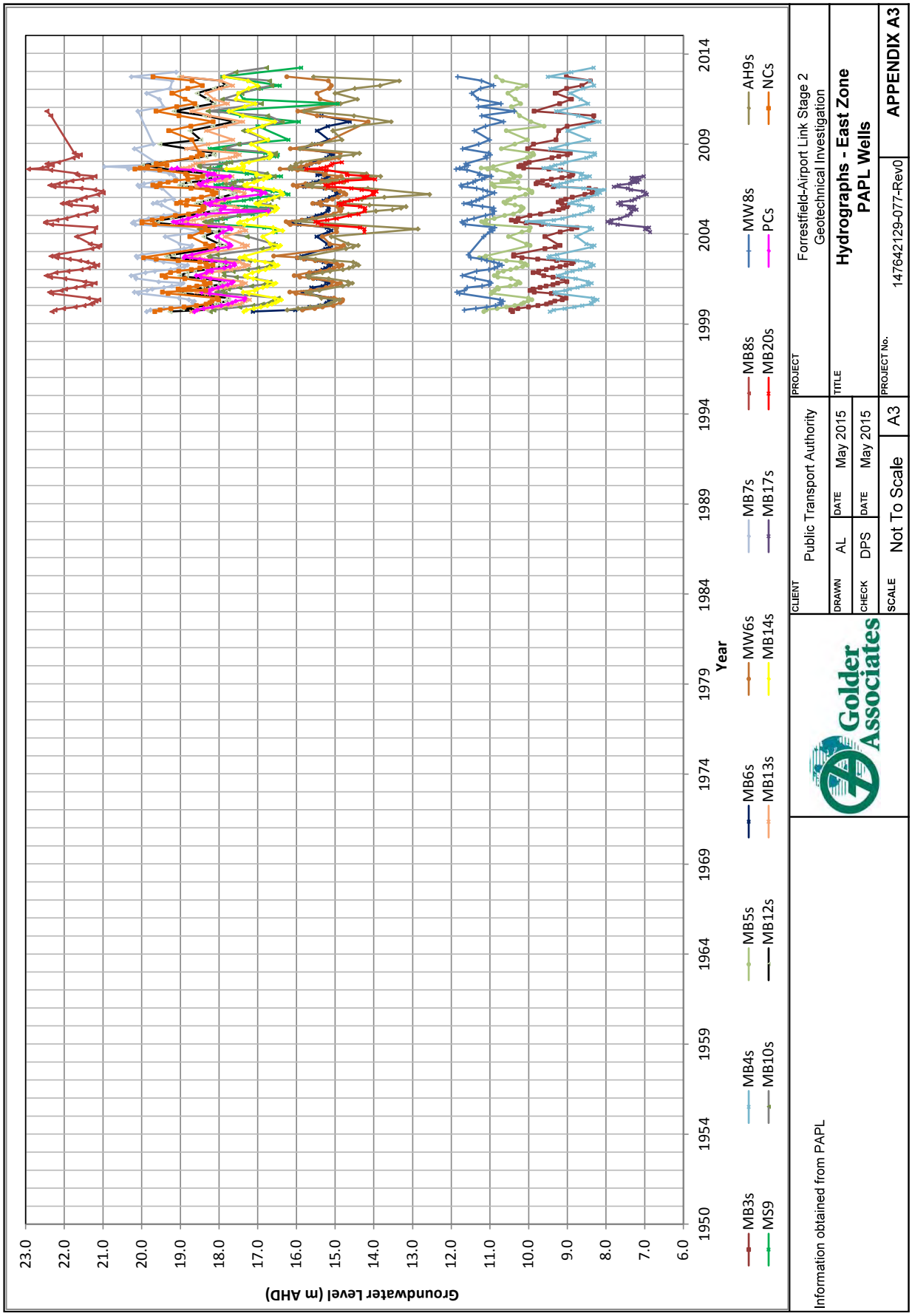
APPENDIX A

Historical Hydrographs

May 2015
Report No. 147642129-077-R-Rev0


Groundwater Level (m AHD)
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
1950
1954
1959
1964
1969
1974
1979
1984
1989
1994
1999
2004
2009
2014
Year
4722
4814
MW1S
MW2S
MW3S
MW4S
MW5S
MW6S
MW7S
MB01
MB03
Dow Wells (4722 and 4814) obtained from DoW WIN database
All other wells obtained from PB2013
Golder Associates
CLIENT
Public Transport Authority
PROJECT
Forrestfield-Airport Link Stage 2
Geotechnical Investigation
DRAWN
AL
DATE
May 2015
TITLE
Hydrographs - West Zone
Upper Aquifer
CHECK
DPS
DATE
May 2015
SCALE
Not To Scale
A3
PROJECT No.
147642129-077-Rev0
APPENDIX A1
J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Appendix A - Hydrographs\147642129-077-R-Rev0 - Appendix A - Hydrographs.xlsx


Groundwater Level (m AHD)
25.0
24.0
23.0
22.0
21.0
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
1950
1954
1959
1964
1969
1974
1979
1984
1989
1994
1999
2004
2009
2014
Year
4808
4809
4810
4811
4812
4813
4848
4883
14584
23042147
Information obtained from DoW WIN Database
Golder Associates
CLIENT
Public Transport Authority
PROJECT
Forrestfield-Airport Link Stage 2
Geotechnical Investigation
DRAWN
AL
DATE
May 2015
TITLE
Hydrographs - East Zone
DoW Wells
CHECK
DPS
DATE
May 2015
SCALE
Not To Scale
A3
PROJECT No.
147642129-077-Rev0
APPENDIX A2
J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Appendix A - Hydrographs\147642129-077-R-Rev0 - Appendix A - Hydrographs.xlsx


Groundwater Level (m AHD)
23.0
22.0
21.0
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
Year
1950
1954
1959
1964
1969
1974
1979
1984
1989
1994
1999
2004
2009
2014
MB3s
MS9
MB4s
MB10s
MB5s
MB12s
MB6s
MB13s
MW6s
MB14s
MB7s
MB17s
MB8s
MB20s
MW8s
PCs
AH9s
NCs
Information obtained from PAPL
Golder Associates
CLIENT
Public Transport Authority
PROJECT
Forrestfield-Airport Link Stage 2
Geotechnical Investigation
DRAWN
AL
DATE
May 2015
TITLE
Hydrographs - East Zone
PAPL Wells
CHECK
DPS
DATE
May 2015
SCALE
Not To Scale
A3
PROJECT No.
147642129-077-Rev0
APPENDIX A3
J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Appendix A - Hydrographs\147642129-077-R-Rev0 - Appendix A - Hydrographs.xlsx

APPENDIX B

March 2015 Groundwater Level Measurements

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX B

10 MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVEL MEASUREMENTS															
Well ID	Chainage *	Offset (m) **	Type	Easting (PCG94)	Northing (PCG94)	Surface Elevation (m AHD)	Top of Casing Elevation (m AHD)	Monument Type	Borehole Depth (m)	Well or VWP Sensor Depth (m)	Screen (m bgl)	Material Screened	Aquifer	Groundwater or Piezometric Level	
											from - to			m b TOC	m AHD
LG 3-153	1435	-30	LG	60,395	266,740	9.17	9.80	Riser	20.0	13.0	6.0 - 13.0	Bassendean Sand	Upper	4.59	5.21
LG 3-149	1575	-20	LG	60,500	266,647	8.83	9.49	Riser	18.0	18.0	11.0 - 18.0	Perth Formation Clay Dominated	Aquitard	6.86	2.63
LG 2-18	2425	-18	LG	61,082	266,029	0.62	0.55	Riser	39.0	21.5	15.5 - 21.5	Swan River Alluvium	Aquitard	0.30	0.25
LG 3-131	2600	-19	LG	61,200	265,899	0.55	0.43	Gatic	19.5	19.5	15.5 - 19.5	Swan River Formation	Aquitard	0.26	0.17
LG 2-14	2825	35	LG	61,415	265,822	2.58	2.44	Gatic	35.0	31.9	16.9 - 31.9	Osborne Formation	Osborne	1.85	0.59
MW 3-180	70	0	MW	59,182	267,262	16.38	16.29	Gatic	6.5	6.5	3.5 - 6.5	Bassendean Sand	Upper	4.88	11.41
MW 3-175	500	-25	MW	59,650	267,164	15.06	14.98	Gatic	7.5	7.3	4.3 - 7.3	Bassendean Sand	Upper	5.17	9.81
MW 3-172	970	-20	MW	60,040	267,025	7.08	6.98	Gatic	4.5	2.7	1.2 - 2.7	Bassendean Sand	Upper	2.10	4.88
BH 2-26	1130	0	MW	60,187	266,966	10.14	10.73	Riser	35.0	35.0	26.0 - 35.0	Osborne Formation	Osborne	7.98	2.75
MW 3-165	1175	10	MW	60,242	266,946	8.54	9.12	Riser	17.0	5.0	2.0 - 5.0	Bassendean Sand	Upper	3.85	5.27
MW 3-166	1185	15	MW	60,232	266,948	9.23	9.85	Riser	20.0	5.5	2.5 - 5.5	Bassendean Sand	Upper	4.62	5.23
MW 3-159	1195	67	MW	60,287	266,976	9.86	10.56	Riser	6.2	5.2	2.2 - 5.2	Bassendean Sand	Upper	4.96	5.60
MW 2-02C	1230	8	MW	60,274	266,911	6.92	7.62	Riser	20.0	19.0	11.5 - 19.0	Perth Formation	Lower	6.05	1.57
MW 2-02A	1233	8	MW	60,277	266,911	6.90	7.67	Riser	26.0	26.0	22.0 - 26.0	Perth Formation	Lower	6.10	1.57
MW 2-02B	1235	48	MW	60,303	266,939	7.90	8.67	Riser	26.0	26.0	10.5 - 26.0	Perth Formation	Lower	6.98	1.69
MW 2-02D	1236	8	MW	60,279	266,909	6.87	7.63	Riser	6.9	6.7	0.7 - 6.7	Bassendean Sand	Upper	2.36	5.27
MW 3-156	1325	17	MW	60,338	266,863	7.89	8.56	Riser	17.0	17.0	15.0 - 17.0	Perth Formation Sand Dominated	Lower	6.96	1.60
MW 3-145	1805	-18	MW	60,660	266,483	9.77	9.69	Gatic	6.5	5.2	3.2 - 5.2	Bassendean Sand	Upper	4.47	5.22
MW 3-140	2195	43	MW	60,975	266,240	6.37	6.27	Gatic	6.0	6.0	3.5 - 6.0	Bassendean Sand	Upper	4.90	1.37
MW 3-137	2285	18	MW	61,015	266,157	5.28	5.22	Gatic	6.0	6.0	3.0 - 6.0	Perth Formation Sand Dominated	Upper	4.36	0.86
MW 3-136 (d)	2295	-18	MW	60,992	266,128	5.25	5.21	Gatic	28.5	15.5	12.5 - 15.5	Perth Formation Clay Dominated	Upper	4.38	0.83
MW 3-136 (s)	2295	-18	MW	60,993	266,129	5.25	5.22	Gatic	5.0	5.0	1.0 - 5.0	Perth Formation Sand Dominated	Upper	4.38	0.84
MW 3-132	2620	19	MW	61,237	265,914	0.83	0.79	Gatic	27.0	4.0	1.0 - 4.0	Swan River Formation	Superficial	0.46	0.33
MW 3-128	2815	30	MW	61,402	265,821	2.26	2.24	Gatic	37.0	4.0	1.0 - 4.0	Perth Formation Sand Dominated	Superficial	2.02	0.22
MW 3-125	3270	-64	MW	61,818	265,624	8.60	8.50	Gatic	5.3	5.3	2.3 - 5.3	Perth Formation Sand Dominated	Superficial	2.86	5.64
MW 3-121 (d)	3320	-28	MW	61,880	265,647	8.75	8.69	Gatic	27.1	22.0	19.5 - 23.7	Ascot Formation	Superficial	2.75	5.94
MW 3-121 (s)	3320	-28	MW	61,881	265,647	8.72	8.65	Gatic	6.0	5.5	1.0 - 5.5	Bassendean Sand	Superficial	2.59	6.06
MW 2-03	3330	-19	MW	61,888	265,654	8.75	8.68	Gatic	21.0	21.0	18.0 - 21.0	Osborne Formation	Osborne	2.69	5.99
MW 3-118	3695	-71	MW	62,232	265,524	9.66	9.57	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.89	7.68
MW 3-115	3705	39	MW	62,269	265,627	8.20	8.09	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.06	7.03
MW 3-109 (d)	3845	22	MW	62,403	265,579	9.84	9.75	Gatic	14.5	14.5	12.0 - 14.5	Ascot Formation	Superficial	2.09	7.66
MW 3-109 (i)	3850	22	MW	62,406	265,578	9.82	9.74	Gatic	10.0	10.0	8.0 - 10.0	Gnangara Sand	Superficial	2.06	7.68
MW 3-109 (s)	3855	22	MW	62,409	265,577	9.92	9.82	Gatic	4.5	4.5	1.5 - 4.5	Bassendean Sand	Superficial	2.14	7.68
MW 2-01D	3870	-3	MW	62,419	265,548	10.08	10.05	Gatic	18.0	17.0	11.0 - 17.0	Ascot Formation	Superficial	2.17	7.88
MW 3-108	3870	-44	MW	62,410	265,508	10.00	9.91	Gatic	20.0	17.5	14.5 - 17.5	Ascot Formation	Superficial	1.90	8.01
MW 2-01B	3910	-48	MW	62,447	265,496	10.12	10.05	Gatic	8.0	8.0	2.0 - 8.0	Bassendean Sand	Superficial	1.88	8.17
MW 2-01A	3925	-6	MW	62,473	265,533	10.32	10.28	Gatic	8.9	8.0	2.0 - 8.0	Bassendean Sand	Superficial	2.16	8.12
MW 2-01C	3925	-2	MW	62,473	265,537	10.33	10.29	Gatic	19.5	17.0	10.5 - 17.0	Ascot Formation	Superficial	2.19	8.10
MW 2-01E	3925	-4	MW	62,473	265,535	10.29	10.26	Gatic	25.0	23.0	20.0 - 23.0	Osborne Formation	Osborne	2.19	8.07
MW 3-105	3935	68	MW	62,498	265,603	10.12	10.02	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	2.08	7.94
MW 3-104	3970	23	MW	62,524	265,550	10.02	9.92	Gatic	18.1	17.1	14.1 - 17.1	Ascot Formation	Superficial	1.75	8.17
MW 3-103	3975	-49	MW	62,510	265,480	10.26	10.17	Gatic	14.0	4.5	1.5 - 4.5	Bassendean Sand	Superficial	1.83	8.34
BH 2-01	4035	-30	MW	62,575	265,483	10.44	11.01	Riser	28.0	18.5	11.0 - 18.5	Ascot Formation	Superficial	2.51	8.50
MW 3-100	4070	-48	MW	62,600	265,459	10.41	11.11	Riser	4.5	4.5	1.5 - 4.5	Bassendean Sand	Superficial	2.65	8.46
MW 3-099	4090	50	MW	62,645	265,547	10.29	10.19	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.83	8.36
MW 3-096	4335	-34	MW	62,867	265,412	11.96	11.86	Gatic	18.5	3.8	0.8 - 3.8	Bassendean Sand	Superficial	1.84	10.02
MW 3-094	4590	18	MW	63,125	265,402	12.96	12.98	Gatic	23.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	2.19	10.79
MW 3-093	4665	-46	MW	63,178	265,320	13.44	13.36	Gatic	4.5	4.5	1.5 - 4.5	Bassendean Sand	Superficial	2.15	11.21
MW 3-092	4675	42	MW	63,214	265,400	12.88	12.79	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.72	11.07
MW 3-089	4775	40	MW	63,309	265,362	13.50	13.42	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.77	11.65
MW 3-090	4825	-21	MW	63,330	265,284	14.00	13.88	Gatic	4.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	1.63	12.25
MW 3-087	4970	-27	MW	63,445	265,208	14.81	14.71	Gatic	3.5	3.5	0.9 - 3.5	Bassendean Sand	Superficial	1.00	13.71
MW 1-03	5030	17	MW	63,522	265,208	14.68	14.59	Gatic	14.5	14.5	11.5 - 14.5	Ascot Formation	Superficial	1.89	12.70
MW 3-086 (d)	5035	15	MW	63,522	265,205	14.72	14.66	Gatic	28.0	22.0	20.0 - 22.0	Osborne Formation	Osborne	1.96	12.70
BH 1-25	6660	-5	MW	64,884	264,345	19.23	19.16	Gatic	48.0	42.0	29.0 - 42.0	Osborne Formation (Mirrabooka)	Osborne	2.28	16.88
MW 3-080 (d)	6660	30	MW	64,893	264,379	19.14	19.04	Gatic	20.0	20.0	17.0 - 20.0	Ascot Formation	Superficial	2.26	16.78
MW 3-080 (s)	6660	30	MW	64,891	264,378	19.17	19.08	Gatic	20.0	4.5	1.0 - 4.5	Bassendean Sand	Superficial	2.32	16.76
BH 1-21	6725	37	MW	64,955	264,371	19.03	19.54	Riser	36.0	24.9	15.9 - 24.9	Ascot Formation	Superficial	2.64	16.90
MW 3-077	6730	-27	MW	64,944	264,308	19.40	19.29	Gatic	14.0	5.0	1.5 - 5.0	Bassendean Sand	Superficial	2.38	16.91
MW 3-079	6730	-140	MW	64,915	264,197	20.38	20.32	Gatic	5.0	5.0	1.5 - 5.0	Bassendean Sand	Superficial	3.16	17.16
MW 1-02B	6780	-40	MW	64,989	264,283	19.45	20.08	Riser	28.5	26.0	14.0 - 26.0	Ascot Formation	Superficial	2.97	17.11
MW 1-02C	6785	-9	MW	65,005	264,311	18.63	19.32	Riser	11.0	11.0	5.0 - 11.0	Bassendean Sand	Superficial	2.30	17.02
MW 1-02D	6785	-6	MW	65,006	264,312	18.55	19.26	Riser	26.0	25.8	13.8 - 25.8	Ascot Formation	Superficial	2.17	17.09
MW 1-02A	6805	25	MW	65,032	264,340	18.56	19.05	Riser	36.0	26.0	14.0 - 26.0	Ascot Formation	Superficial	2.00	17.05
BH 1-13	7330	22	MW	65,544	264,238	18.90	19.55	Riser	36.0	16.5	15.5 - 16.5	Ascot Formation	Superficial	2.32	17.23
MW 3-065	7375	18	MW	65,588	264,226	18.72	19.22	Gatic	26.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	2.92	16.30
BH 1-12	7445	22	MW	65,658	264,219	18.94	19.60	Riser	35.0	23.5	15.5 - 23.5	Ascot Formation	Superficial	2.37	17.23
MW 3-064	7535	16	MW	65,746	264,200	19.09	19.56	Gatic	26.0	4.0	1.0 - 4.0	Bassendean Sand	Superficial	2.30	17.26

APPENDIX B

10 MARCH 2015 GROUNDWATER AND PIEZOMETRIC LEVEL MEASUREMENTS															
Well ID	Chainage *	Offset (m) **	Type	Easting (PCG94)	Northing (PCG94)	Surface Elevation (m AHD)	Top of Casing Elevation (m AHD)	Monument Type	Borehole Depth (m)	Well or VWP Sensor Depth (m)	Screen (m bgl)	Material Screened	Aquifer	Groundwater or Piezometric Level	
											from - to			m b TOC	m AHD
MW 1-04	8110	15	MW	66,311	264,103	26.30	26.27	Gatic	31.5	31.5	28.5 - 31.5	Osborne Formation	Osborne	3.87	22.40
MW 3-060 (d)	8110	0	MW	66,308	264,087	26.33	26.29	Gatic	27.0	17.5	15.5 - 17.5	Guildford Formation	Superficial	3.86	22.43
MW 3-060 (s)	8110	0	MW	66,308	264,088	26.33	26.29	Gatic	4.5	4.0	1.0 - 4.0	Bassendean Sand	Perched	2.40	23.89
MW 3-059	8130	-33	MW	66,303	264,054	26.39	26.28	Gatic	4.0	3.5	1.0 - 3.5	Bassendean Sand	Perched	2.36	23.92
MW 3-055	8360	112	MW	66,599	264,019	28.81	28.71	Gatic	7.0	7.0	4.0 - 7.0	Guildford Formation	Superficial	5.95	22.76
MW 3-052	8545	-21	MW	66,564	263,781	28.35	28.97	Riser	17.0	10.0	7.0 - 10.0	Guildford Formation	Superficial	6.11	22.86
MW 3-051 (d)	8665	-40	MW	66,560	263,664	29.41	30.05	Riser	17.5	17.5	15.5 - 17.5	Guildford Formation	Superficial	7.18	22.87
MW 3-051 (s)	8665	-40	MW	66,560	263,662	29.39	30.01	Riser	17.5	7.0	4.0 - 7.0	Guildford Formation	Superficial	7.13	22.88
MW 1-01A	8710	13	MW	66,617	263,625	30.47	30.37	Gatic	46.2	32.0	8.0 - 32.0	Guildford Formation	Superficial	7.45	22.92
MW 1-01C	8720	12	MW	66,619	263,615	30.47	30.40	Gatic	45.0	32.3	7.0 - 32.3	Guildford Formation	Superficial	7.49	22.91
MW 1-01B	8725	-48	MW	66,558	263,604	29.33	29.26	Gatic	32.5	32.5	8.5 - 32.5	Guildford Formation	Superficial	6.37	22.89
MW 1-01D	8755	15	MW	66,624	263,578	30.19	30.07	Gatic	33.0	32.0	7.0 - 32.0	Guildford Formation	Superficial	7.15	22.92
MW 3-047	8780	18	MW	66,628	263,555	30.36	30.27	Riser	17.0	12.0	9.0 - 12.0	Guildford Formation	Superficial	7.32	22.95
MW 3-044 (d)	8825	-3	MW	66,606	263,509	29.54	30.27	Riser	14.0	14.0	11.0 - 14.0	Guildford Formation	Superficial	7.35	22.92
MW 3-044 (s)	8825	-3	MW	66,606	263,510	29.50	30.17	Riser	7.0	6.3	3.3 - 6.3	Guildford Formation	Perched	6.95	23.22
MW 3-038	8940	-57	MW	66,549	263,393	29.33	29.95	Riser	7.5	7.5	4.5 - 7.5	Guildford Formation	Superficial	7.07	22.88
MW 3-034	9010	200	MW	66,805	263,318	31.23	31.89	Riser	7.7	7.0	4.0 - 7.0	Guildford Formation	Perched	DRY	
MW 3-026	9170	-55	MW	66,542	263,164	28.39	29.02	Gatic	8.0	8.0	5.0 - 8.0	Guildford Formation	Superficial	6.22	22.80
MW 3-019	9250	75	MW	66,670	263,080	30.61	30.52	Gatic	6.5	6.5	3.5 - 6.5	Guildford Formation	Perched	4.32	26.20
MW 3-013	9315	506	MW	67,095	263,011	34.68	34.57	Gatic	17.5	17.3	16.3 - 17.3	Guildford Formation	Superficial	11.50	23.07
MW 3-012	9360	-58	MW	66,531	262,979	29.24	29.86	Riser	10.5	10.5	7.5 - 10.5	Guildford Formation	Superficial	6.97	22.89
MW 1-05	9595	-34	MW	66,548	262,742	28.42	28.42	Gatic	10.0	10.0	7.0 - 10.0	Guildford Formation	Superficial	5.01	23.41
MW 3-009	9630	6	MW	66,587	262,706	30.20	30.10	Gatic	8.0	6.9	3.9 - 5.9	Guildford Formation	Perched	5.56	24.54
MW 3-004	9755	-14	MW	66,561	262,579	30.00	29.89	Gatic	8.0	8.0	5.0 - 8.0	Guildford Formation	Perched	5.55	24.34
MW 3-001	9985	-32	MW	66,533	262,353	27.78	27.70	Gatic	6.0	6.0	3.0 - 6.0	Guildford Formation	Superficial	4.45	23.25
PW 2-02	1235	5	PW	60,275	266,907	6.91	7.40	Riser	27.0	26.0	10.5 - 26.0	Perth Formation	Lower	5.81	1.59
PW 2-01	3920	-4	PW	62,468	265,536	10.24	10.69	Riser	19.0	17.8	10.5 - 17.8	Ascot Formation	Superficial	2.57	8.12
PW 1-02	6790	-10	PW	65,009	264,309	18.60	18.99	Riser	28.5	26.0	12.0 - 26.0	Ascot Formation	Superficial	1.85	17.14

* Transposed perpendicular to alignment (approximate to nearest 5 m)

** Distance is measured to project centreline (between tunnels). Negative value is when monitoring point is located south or west of alignment. Positive value is when the monitoring point is located north or east of alignment

MW = Monitoring Well

VWP = Vibrating Wire Piezometer

LG = Gas Monitoring Well

PW = Pumping Well

NA = Not Applicable

NM = Not Measured

APPENDIX C

Summary of Analysis of PSD Samples

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX C
Particle Size Distribution Analysis

The hydraulic conductivity was calculated using Hazen's formula, shown below, which is an empirical method that uses data from the particle size distribution of a sample.

$$k = C_1 \times f(n) \times D_e^2 \times 975124$$

Where:

C_1 = 0.0047

f(n) = 1

$D_e = D_{10}$ in m

Hazen's method is usually used for samples with low fines content. Therefore samples with fines content greater than 10% were generally not analysed except for some samples from the Guildford and Perth Formation where samples with a fines content of up to 20% were analysed. It is therefore important to note that the derived hydraulic conductivities will be skewed toward giving higher hydraulic conductivity (i.e. the samples with large fines content which would result in low hydraulic conductivity have been excluded from analysis due to the limitation of the method).

The hydraulic conductivity was estimated for a total of 207 PSD test results. Table 1 presents a summary of the derived hydraulic conductivities.

Table 1: Hydraulic Conductivities Derived from Particle Size Distribution Analysis (m/d)

Geological Unit	CH0 to CH2800			CH2800 to CH4850			CH4850 to CH7600			CH7600 to CH9492			All			Number of Tests
	Range		Median	Range		Median	Range		Median	Range		Median	Range		Median	
Swan River Formation	3	9	6	-	-	-	-	-	-	-	-	-	3	9	6	4
Bassendean Sand	9	22	16	4	23	9	1	25	5	3	5	3	1	37	9	43
Perth Formation	1	23	2	5	14	9	-	-	-	-	-	-	1	23	5	25
Guildford Formation	-	-	-	1	16	3	1	36	3	1	25	2	1	36	2	79
Gnangara Sand	-	-	-	1	8	3	0.4	3	1	-	-	-	0.4	8	2	12
Ascot Formation	-	-	-	3	16	3	3	20	4	-	-	-	3	20	4	27
Osborne Formation Mirrabooka Member	-	-	-	-	-	-	0.3	20	9	-	-	-	0.3	20	9	17

APPENDIX D

Summary of Slug Test Analysis

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX D
Summary of Slug Test Analysis

Single well hydraulic testing (slug testing) was carried out in 31 selected groundwater monitoring wells in March 2015.

The testing involved installing an automatic water level logger into the test well and introducing or removing a slug of PVC or water of known volume while measuring the resulting displacement in groundwater level at 1 second intervals. The groundwater level displacement data was then analysed using AQTESOLV Pro to provide an estimate of hydraulic conductivity (k) adjacent to the well screen.

Several tests were carried out at each well dependent on the time required for complete recovery of the initiated change in groundwater level. In most cases this comprised of three falling and three rising head tests.

Table 1 provides a summary of the estimated hydraulic conductivity for each well and for each hydrogeological unit. Table 1 only provides results for 29 of the 31 tests as the data collected at two of the wells was not suitable to estimate hydraulic conductivity.

Attachment D1 provides selected output files from AQTESOLV Pro of analysis of different groundwater displacement responses as examples:

- BH1-12 (Slug In, High k, Fast Response)
- MW1-03 (Slug Out, High k, Fast Response)
- MW1-02D (High k, Falling Head, Oscillatory Response)
- MW1-04 (Low k, Falling Head, Slow response)
- MW1-05 (Low k, Slug In, Moderate Response).

APPENDIX D
Summary of Slug Test Analysis

Table 1: Results of Slug Testing

Well ID	Location	Material Description at Well Screen	Well Screen Interval (m bgl)	Geological Unit (Formation)	Aquifer	Hydraulic Conductivity (m/d)			
						Individual Well		Hydrogeological Unit	
						Range	Median	Range	Median
MW2-02D	Bayswater	Sand (*f-c*)	0.7 – 6.7	Perth	Upper	5 – 6	6	0.3 – 35	11
MW2-02C	Bayswater	Sand (*c*)	11.5 – 19.0		Lower	10 – 32	27		
MW2-02A	Bayswater	Sandy clay/gravelly sand (*c*)	22.0 – 26.0			8 – 35	16		
MW2-02B	Bayswater	Sand (*f-m*),clayey sand(*f*), gravelly sand (*f-c*)	10.5 – 26.0			0.3 – 0.6	0.5		
LG2-18	Swan River	Clayey silt	15.5 – 21.5	Swan River	Aquitard	0.2 – 0.2	0.2	0.2	0.2
MW2-01A	Airport West	Sand (*m-c*), trace clay	2.0 – 8.0	Guildford	Superficial	1.1 – 2.2	2	0.1 – 12	2.5
MW2-01B	Airport West	Sand (*f-c*), trace clay	2.0 – 8.0			2.9 – 3.8	3		
MW1-02C	Consolidated	Sand (*f-c*)/clayey sand (*m-c*)	5.0 – 11.0			5 – 12	8		
MW1-01A	Forrestfield	Sand (*f-c*),silty sand (*f-m*)	8.0 – 32.0			0.1 – 0.3	0.2		
MW1-01B	Forrestfield	Sand (*f-c*), silty sand *(f-m)*, clayey sand (*f-m*)	8.5 – 32.5			0.1 – 1.5	0.4		
MW1-01C	Forrestfield	Clayey sand (*f-m*), sand (*f-c*), silty sand (*f-m*)	7.0 – 32.3			0.3 – 0.4	0.4		
MW1-01D	Forrestfield	Clayey sand (*f-m*), sand (*f-c*), clayey gravelly sand (*f*)	7.0 – 32.0			0.5 – 5.1	4		
MW1-05	Forrestfield	Sand (*f-m*), clayey sand (*f-m*)	7.0 – 10.0			0.1 – 0.2	0.2		

APPENDIX D
Summary of Slug Test Analysis

Well ID	Location	Material Description at Well Screen	Well Screen Interval (m bgl)	Geological Unit (Formation)	Aquifer	Hydraulic Conductivity (m/d)			
						Individual Well		Hydrogeological Unit	
						Range	Median	Range	Median
MW2-01D	Airport West	Silty gravelly sand (*f-c*)	11.0 – 17.0	Ascot	Superficial	2 – 26	6	1 – 75	7.5
MW2-01C	Airport West	Calcarenite (*m-c*)	10.5 – 17.0			3 – 23	18		
BH2-01	Airport West	Silty sand (*f*), sand (*f-m*), gravel (*f-c*)	11.0 – 18.5			2 – 3	3		
MW1-03	Airport	Silty sand Gravel (*f*)	11.5 – 14.5			5 – 8	7		
BH1-21	Consolidated	Silty sandy gravel (*f-c*), sand (*f-m*)	15.9 – 24.9			1 – 4	3		
MW1-02A	Consolidated	Gravelly silty sand (*f*), calcarenite (*f*)	14.0 – 26.0			2 – 8	5		
MW1-02D	Consolidated	Gravelly sand (*m-c*), calcarenite	13.8 – 25.8			15 – 21	17		
MW1-02B	Consolidated	Sandy gravel (*f*), gravelly sand (*f-c*)	14.0 – 26.0			3 – 12	12		
BH1-12	Consolidated	Sand (*f-c*), gravel (*f-c*), calcarenite (*f-c*)	15.5 – 23.5			5 – 12	8		
BH1-13	Consolidated	Sandy gravel (*f-c*)	15.5 – 16.5			20 – 75	50		
LG2-14	Swan River	Silty sandstone (*f*)	16.9 – 31.9	Osborne Formation (Kardinya Shale)	Osborne Aquitard	0.0002	0.0002	0.0002 – 0.008	0.0005
MW3-121 (d)	Airport West	Sandstone (*f*)	19.5 – 23.7			0.0005	0.0005		
MW2-01E	Airport West	Siltstone, claystone	20.0 – 23.0			0.0006	0.0006		
MW1-04	EE4	Sandy clay	28.5 – 31.5			0.008	0.008		
BH1-25	Consolidated	Sand (*f-c*), gravelly clayey sand (*f-c*)	29.0 – 42.0	Osborne Formation	Mirrabooka	1 – 4	3	1 – 7	4.5
MW1-02E	Consolidated	Sandy Gravel (*f-c*), calcarenite	27.0 – 34.0			4 – 7	6		

f,- fine grained, *m* – medium grained, *c* – coarse grained, m bgl – meters below ground level

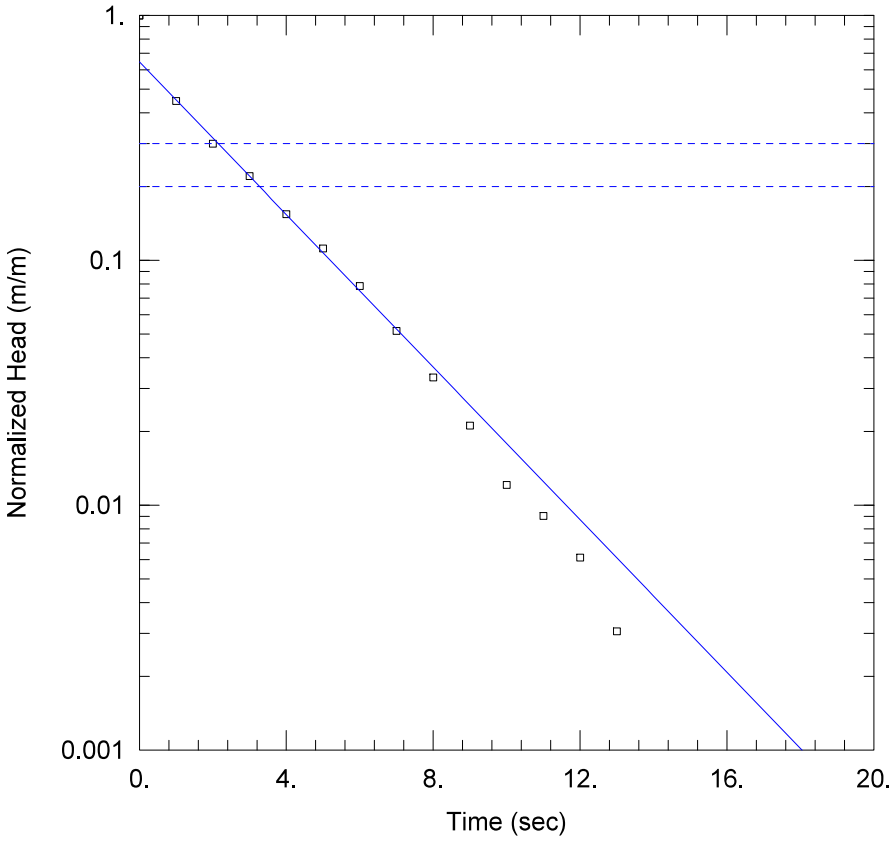


APPENDIX D
Summary of Slug Test Analysis

ATTACHMENT D1
Selected AQTESOLV Solutions from Slug Testing

May 2015
Reference No. 147642129-077-R-Rev0

Golder Associates



WELL TEST ANALYSIS

Data Set: J:\...\BH1-12 slug in 2_AL.aqt
Date: 04/08/15 Time: 09:15:44

PROJECT INFORMATION

Test Well: BH1-12

AQUIFER DATA

Saturated Thickness: 21.81 m Anisotropy Ratio (Kz/Kr): 0.1

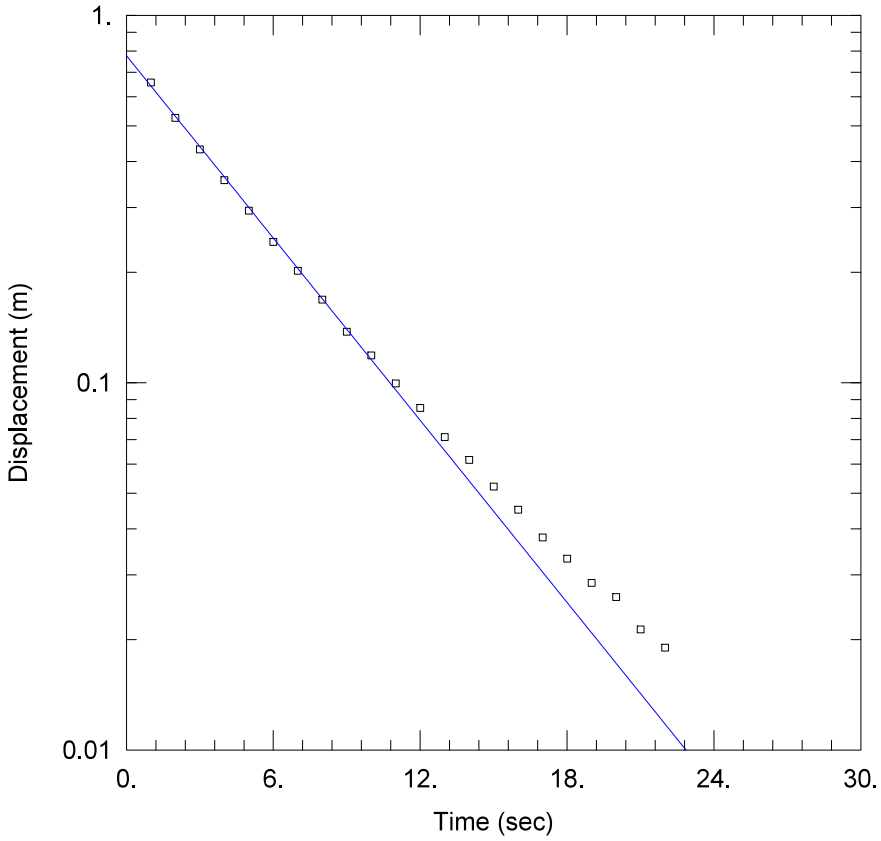
WELL DATA (BH1-12)

Initial Displacement: 0.7852 m
Total Well Penetration Depth: 21.81 m
Casing Radius: 0.0253 m
Static Water Column Height: 21.81 m
Screen Length: 9. m
Well Radius: 0.06 m
Gravel Pack Porosity: 0.

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 6.427 m/day y0 = 0.5072 m



WELL TEST ANALYSIS

Data Set: J:\...\MW1-03 slug out 1.aqt
Date: 04/08/15 Time: 09:27:58

PROJECT INFORMATION

Test Well: MW1-03

AQUIFER DATA

Saturated Thickness: 11.74 m Anisotropy Ratio (Kz/Kr): 0.1

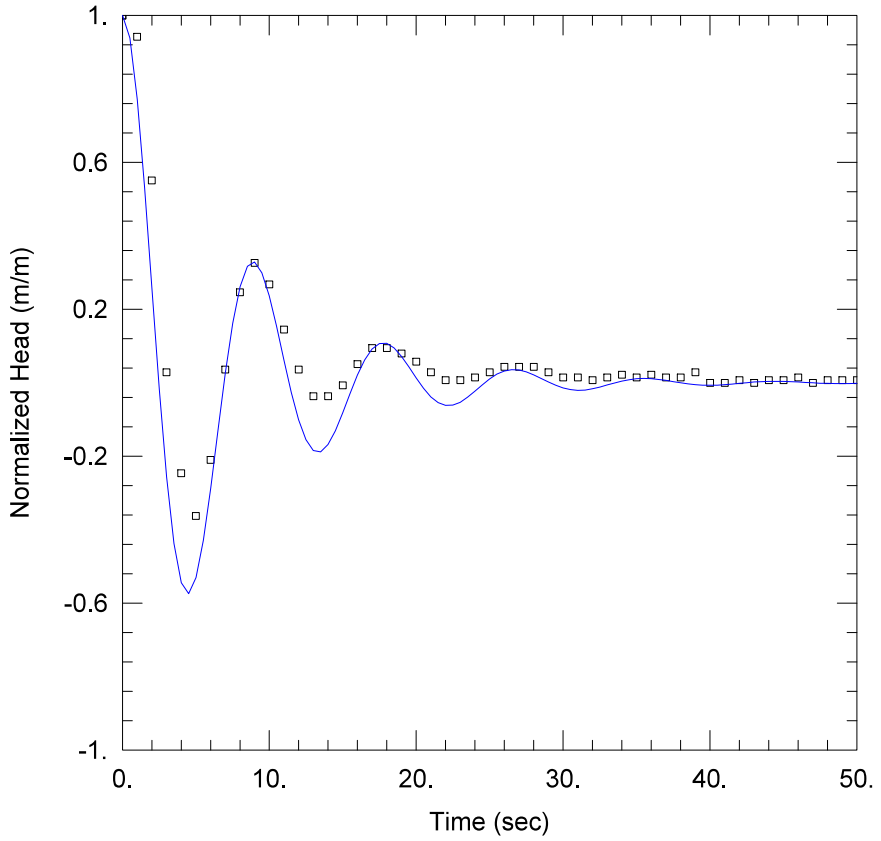
WELL DATA (MW1-03)

Initial Displacement: 1.233 m
Total Well Penetration Depth: 11.74 m
Casing Radius: 0.0253 m
Static Water Column Height: 11.74 m
Screen Length: 3.5 m
Well Radius: 0.0253 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 6.883 m/day y0 = 0.7768 m



WELL TEST ANALYSIS

Data Set: J:\...\MW1-02D - Falling Head 1_AL.aqt
Date: 04/08/15 Time: 09:29:14

PROJECT INFORMATION

Test Well: MW1-02D

AQUIFER DATA

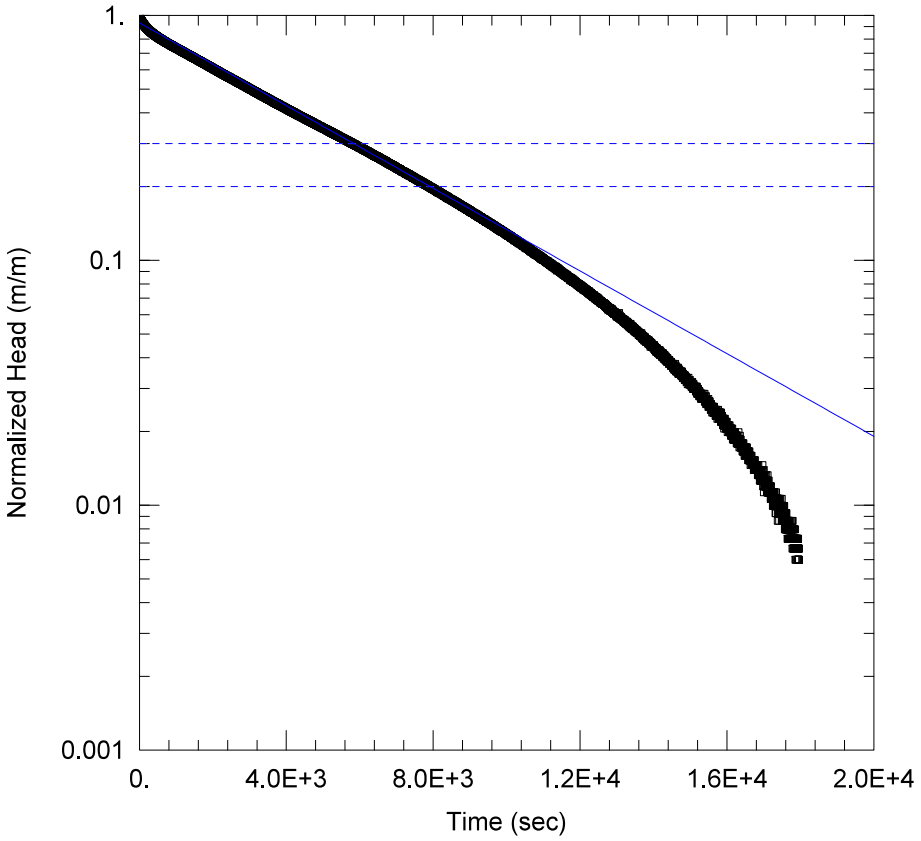
Saturated Thickness: 29.78 m Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (MW1-02D)

Initial Displacement: 0.3273 m
Total Well Penetration Depth: 23.78 m
Casing Radius: 0.0253 m
Static Water Column Height: 23.78 m
Screen Length: 12. m
Well Radius: 0.06 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Springer-Gelhar
K = 20.53 m/day
Le = 19. m



WELL TEST ANALYSIS

Data Set: J:\...\MW1-04-Confined.aqt
Date: 04/08/15 Time: 09:20:55

PROJECT INFORMATION

Test Well: MW1-04

AQUIFER DATA

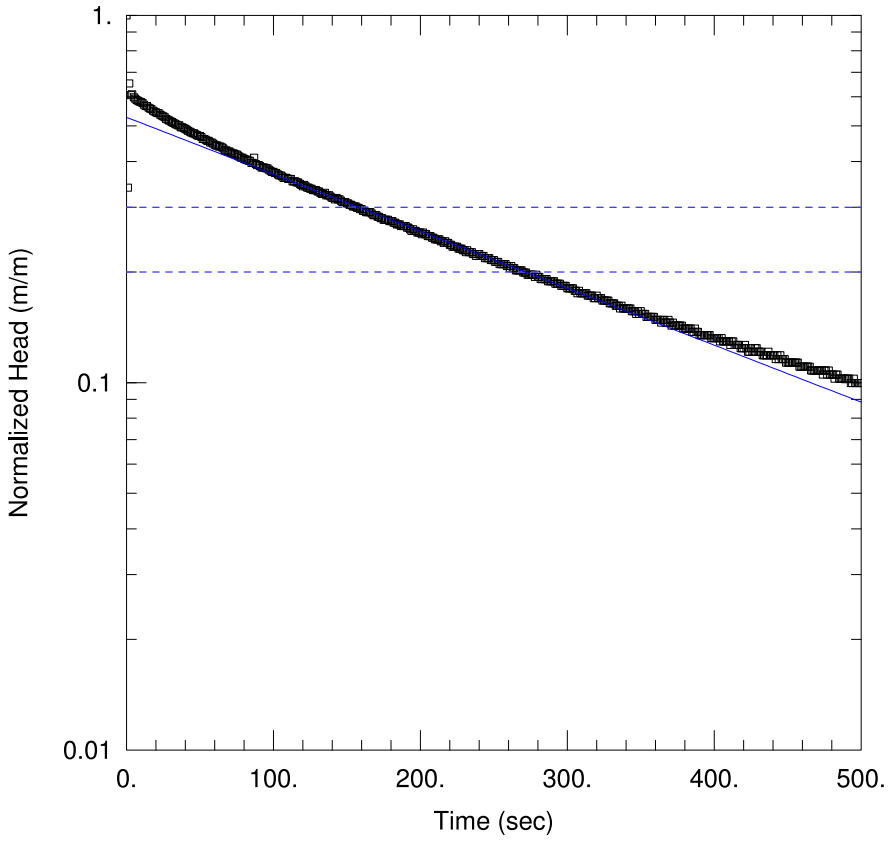
Saturated Thickness: 4.5 m Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (MW1-04)

Initial Displacement: 3.578 m
Total Well Penetration Depth: 27.89 m
Casing Radius: 0.0253 m
Static Water Column Height: 27.89 m
Screen Length: 3.5 m
Well Radius: 0.06 m

SOLUTION

Aquifer Model: Confined
Solution Method: Bouwer-Rice
K = 0.006694 m/day
y0 = 3.333 m



WELL TEST ANALYSIS

Data Set: J:\...\MW1-05- Slug In_AL.aqt
Date: 05/16/15 Time: 16:56:23

PROJECT INFORMATION

Test Well: MW1-05

AQUIFER DATA

Saturated Thickness: 4.87 m
Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (MW1-05)

Initial Displacement: 0.8801 m
Total Well Penetration Depth: 4.87 m
Casing Radius: 0.0253 m
Static Water Column Height: 4.87 m
Screen Length: 3. m
Well Radius: 0.06 m
Gravel Pack Porosity: 0.

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice
K = 0.1633 m/day
y0 = 0.4643 m

APPENDIX E

Summary of Test Pumping Analysis

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX E
Summary of Test Pumping Analysis

1.0 INTRODUCTION

This Appendix describes and presents the results from the four pumping tests carried out as part of the Stage 2 Geotechnical Investigation.

The following pumping tests were completed at the following sites:

- PW1-01 at Forrestfield Dive Structure – 8 to 12 December 2014
- PW1-02 at Consolidated Airport Station – 5 to 9 January 2015
- PW2-01 at Airport West Station – 19 to 22 January 2015
- PW2-02 at Bayswater Dive Structure – 2 to 6 February 2015.

A 26D licence under the Rights in Water and Irrigation Act 1914 was obtained to install all the pumping test wells and to carry out all the pumping tests. This licence was issued by the Department of Water (DoW) and allowed for the installation, development and testing of the pumping test wells. Additional approvals were obtained for disposal of the groundwater abstracted during the pumping tests. The required disposal approvals are site specific and are discussed specifically for each site in the sections below.

2.0 GENERAL METHODOLOGY

2.1 Pumping Tests

The pumping tests were completed by McArthur Drilling and Pumping (MDP) under the supervision of Golder in general accordance with AS 2368-1990 – "Test pumping of water wells". The following tests were carried out at each of the four sites:

- Step-Rate – The test involved pumping the test well at four different pumping rates (with the rate constant during each step, but increasing between steps) with each step lasting around one hour. The purpose of this test is to assess the pumping rate for the constant rate test.
- Constant Rate – This test involved pumping the test well at a constant rate for up to 72 hours depending on the response observed in the pumping and monitoring wells during the test.
- Recovery – The recovery occurred immediately following the cessation of the constant rate test (i.e. the start of the test is when the pump is turned off for the constant rate test) and was continued until approximately 95% recovery had occurred.

Some changes to the methodology did occur at each site due to site specific conditions and logistics. The design of each pumping test focused on maximising the return of information from each of the four pumping test locations.

2.2 Monitoring

Monitoring of both groundwater level and quality was carried out as part of the pumping tests in accordance with AS 2368-1990. In summary, the groundwater monitoring consisted of:

- Background barometric pressure and groundwater levels in monitoring and pumping wells were monitored prior to the start of each pumping test using automated barometric and groundwater level loggers.
- Groundwater level monitoring in the pumping well and at least four monitoring wells was completed during each pumping test using time-synchronised, automated groundwater level loggers logging at 1 minute intervals.

- Manual measurement of groundwater level in the pumping wells and the monitoring wells closest to the pumping well was undertaken at the intervals outlined in AS 2368-1990 as a quality assurance control and to verify automated water level logger information during each pumping test. In addition, groundwater levels in other monitoring wells with automated loggers was manually measured during the pumping tests as time permitted.
- Barometric pressure was measured for the duration of the pumping tests using an automated pressure logger that was synchronised with the groundwater level loggers to allow for barometric correction of the groundwater level logger data.
- Flow volume and discharge rate from the pumping well was measured using a digital flow meter at the wellhead.
- Water quality field monitoring of pH, electrical conductivity (EC), and temperature was carried out on the abstracted groundwater.
- Sampling of discharge water for laboratory water quality analysis for major ions, metals, nutrients, TPH, BTEX, PAH, MAH and pesticides was undertaken as a minimum at the beginning of each pumping test and at the end of each pumping test. A sample was also taken halfway through the test at Airport West (PW2-01) at the request of Swan River Trust (SRT).

Some variations in the monitoring at each site did occur due to site specific conditions and logistics.

2.3 Groundwater Disposal

The groundwater abstracted during pumping well development was either reinfiltrated on site (Bayswater), pumped to a turkeys nest for construction use by others (Consolidated Airport Terminal Station) or pumped to the stormwater drain (Forrestfield and Airport West). The method of disposal was evaluated on a case by case basis. Approval from the relevant asset owner and regulators was obtained before proceeding with each pumping test.

2.4 Analysis

The pumping test data was analysed analytically [1, 2] using multiple analytical solutions and type curves and numerically using Visual MODFLOW. A basic numerical 3D groundwater model developed using Visual MODFLOW for the area around each pumping well was used to match the pumping test data by manually changing hydraulic conductivity and storage.

3.0 PW 1-01 (FORRESTFIELD DIVE STRUCTURE)

The results from the Stage 2 Geotechnical Investigation indicate that the regional Superficial Aquifer at the Forrestfield Dive Structure is comprised of Guildford Formation with a depth of approximately 32 m from current ground surface. The pumping well and associated monitoring wells were installed to the bottom of the Guildford Formation and screened across the whole aquifer (screen lengths of approximately 24 m to 27 m).

3.1 Site and Pumping Test Layout

Pumping tests were carried out at PW 1-01 between 8 and 12 December 2014 and included:

- A step test with pumping rates set at 6, 8, 10 and 12 L/s. The first three steps lasted for 60 minutes each, while the fourth step was stopped after 8 minutes due to the groundwater level drawdown reaching the pump.

[1] Kruseman, G.P. and N.A. de Ridder, 1994. Analysis and Evaluation of Pumping Test Data. Second Edition. Publication 47. International Institute for Land Reclamation and Improvement, Wageningen, The Netherlands, 1994.

[2] Cooper, H.H. and C.E. Jacob, 1946. A generalized graphical method for evaluating formation constants and summarizing well field history. Am. Geophys. Union Trans. 27: 526-534.

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- A 72-hour constant rate test with a discharge rate of approximately 10 L/s; followed by
- A 9-hour recovery test.

Sketch 1 shows the site layout of the pumping test well (PW1-01), four dedicated monitoring wells (MW1-01A to MW1-01D) and Vibrating Wire Piezometers (VWPs) (BH0-07) while Table 1 presents the distances and directions of the different monitoring locations from the pumping test well.



Sketch 1: Site Layout for pumping test at (PW 1-01)

Table 1: Distance and Direction from the Pumping Test Well to the Monitoring Locations

Well ID	Type	Distance to Pumping Well (m)	General Direction to Pumping Well	Screened Geological Unit
PW1-01	Pumping Well	-	-	GF
MW1-01-A	Dedicated Monitoring Well	18	North	GF
MW1-01-B	Dedicated Monitoring Well	61	West	GF
MW1-01-C	Dedicated Monitoring Well	7	North	GF
MW1-01-D	Dedicated Monitoring Well	30	South	GF
BH0-07	Vibrating Wire Piezometer	58	North	GF
BH1-02	Vibrating Wire Piezometer	74	South	GF
BH1-04	Vibrating Wire Piezometer	197	North	GF

* Location of BH1-02 and BH1-04 are not shown on Sketch 1 due to the distance from PW1-01. Refer to Figure 5 in the main report for location of these VWPs.

3.2 Discharge Location

The abstracted groundwater during the pumping test was discharged to Poison Gully Creek at the confluence just before it becomes a man-made drain and enters part of the Perth Airport Drain system (Sketch 1). Discharge approval was obtained from the Shire of Kalamunda.

3.3 Analysis and Results

Figures E1–1 to E1–2 summarise the pumping test results while Table 2 presents the estimated hydraulic properties from the different analysis methods.

Table 2: Results from Pumping Test Analysis – Forrestfield Dive Structure

Method	Transmissivity (m^2/d)	Storage Coefficient	Figures
Distance Drawdown	180	6.0×10^{-3}	E1-3
Groundwater Model	104	2.5×10^{-3}	E1–4 to E1–5
Analytical Type Curves	94–316	-	E1–6 to E1–15 *

* Selected type curves only

The results indicate:

- The groundwater level drawdown in the pumping well reached almost 16 m toward the end of the test and the groundwater level almost reached steady-state conditions at the end of the pumping test
- The groundwater level drawdown in the monitoring wells ranged between 0.9 m and 3.2 m, depending on their distance from the pumping well.
- Four hours after starting the Constant Rate test the flow rate was adjusted as it was decreasing. The increase in flow rate resulted in a step change in the groundwater level within the test well. However, the groundwater level decline was similar before and after the step change, suggesting that the step change in level within the pumping well was due to removal of bore storage when the pumping rate was increased.
- The distance drawdown curve (Figure E1–3) indicates that
  - There is a relatively good correlation between distance (from the pumping well) and drawdown, which suggests generally homogenous conditions.
  - The expected groundwater level drawdown in the aquifer just outside the pumping well is around 6.5 m, indicating that 9.5 m of the 16 m of drawdown in the well was caused by well loss.
- There is a reasonably good match between observed and modelled groundwater level drawdown (E1–4 and E1–5). There was found to be only a small difference when varying the vertical hydraulic conductivity in the model, which is likely due to the wells being screened over the full aquifer thickness and the presence of relatively homogenous conditions.
- The estimated range in the derived storage coefficient suggests semi-confined to confined conditions.

3.4 Groundwater Quality

Figure E1–16 shows the field monitoring results of pH and Electrical Conductivity (EC) from the Constant Rate Test while Table E1 presents the groundwater quality laboratory results from samples taken at the beginning and end of the pumping test. The results indicate:

- pH is acidic and ranged between 5.5 and 6.0 during the pumping test with no particular trend, excluding one outlier.

- EC was generally steady throughout the pumping test at around 1000 uS/cm, indicating slightly brackish water.
- Total Nitrogen was 44 mg/L and 33 mg/L at the beginning and end of test respectively, indicating high nutrient concentrations.
- Total iron increased from below detection limit at the beginning of the test to 0.018 mg/L at the end of the test, which is well below SRT criterion of 1 mg/L.
- BTEX, MAH, PAH, hydrocarbons and pesticides were not detected in either of the two samples.

4.0 PW1-02 (CONSOLIDATED AIRPORT TERMINAL STATION)

The results from the Stage 2 Geotechnical Investigation indicated that the Superficial Aquifer at this location consists of the Bassendean Sand, Guildford Formation and Ascot Formation, overlying the Mirrabooka Member of the Osborne Formation. The Superficial Aquifer is approximately 24 m thick at the Consolidated Airport Terminal Station, of which approximately 12 m consists of the Ascot Formation.

The pumping test well was installed and screened across the full thickness of the Ascot Formation with two of the dedicated monitoring wells (MW1-02B and MW1-02D) installed in the Ascot Formation, one monitoring well in the Bassendean Sand and Guildford Formation (MW1-02C) and one well (MW1-02E) in the Mirrabooka Aquifer.

4.1 Site and Pumping Test Layout

Pumping tests were carried out at PW 1-02 between 5 and 9 January 2015 and included:

- A step test with pumping rates set at 6, 8, 10, 12 and 13.5 L/s. The first four steps lasted for 60 minutes each, while the fifth step was stopped after 3 minutes due to excessive drawdown and the pump sucking in air.
- CRT 1 - A 52-hour constant rate test with a discharge rate of approximately 12.0 L/s followed by an 18.5-hour recovery test.
- CRT 2 - A 6-hour constant rate test with a discharge rate of approximately 9.5 L/s followed by a 1.5 hour recovery test.

CRT 1 was found to pump some air during the pumping test which resulted in some fluctuations in the groundwater level inside the pumping well. Following the completion of the recovery after CRT 1 it was therefore decided to run a short second test (CRT2) at a lower pumping rate to see if less groundwater level fluctuation would be observed in the pumping well.

Sketch 2 shows the site layout of the pumping test well (PW1-02), monitoring wells and VWPs while Table 3 presents the distances and directions of the different monitoring locations from the pumping test well.

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Summary of Test Pumping Analysis



Sketch 2: Site Layout for pumping test at PW 1-02

Table 3: Distance and Direction from the Pumping Test Well to the Monitoring Locations

Well ID	Type	Distance to Pumping Well (m)	General Direction to Pumping Well	Screened Geological Unit
PW1-02	Pumping Well	-	-	AF
MW1-02-A	Dedicated Monitoring Well	39	Northeast	AF
MW1-02-B	Dedicated Monitoring Well	33	Southwest	AF
MW1-02-C	Dedicated Monitoring Well	5	West	BS, GF, GS
MW1-02-D	Dedicated Monitoring Well	5	West	AF
MW1-02-E	Dedicated Monitoring Well	4	Northwest	OFm
BH1-21	Monitoring Well	82	Northwest	AF
BH1-25	Monitoring Well	130	West	OFm
BH0-06	Vibrating Wire Piezometer (two sensors)	27	Northwest	GF and AF
BH1-22	Vibrating Wire Piezometer (two sensors	67	Northwest	AF and OFm
BH1-24	Vibrating Wire Piezometer (three sensors)	132	West	GF, AF and OFm

4.2 Discharge Location

The abstracted groundwater during the pumping test was discharged into a turkeys nest located approximately 30 m west PW1-02 (Sketch 2). The water from the turkeys nest was used during the day by Densford Civil for dust suppression. Normally Densford Civil obtained their construction water from a reticulation bore located approximately 50 m north of the site. However, the pumping rate from PW1-02 was sufficient to supply Densford Civil with their water demand and therefore the reticulation bore could be turned off during the test. While the pump test was being carried out, the turkeys nest overflowed into the Perth Airport Drain (Sketch 2) during the night when the water was not being used by Densford Civil. Discharge approval into the Perth Airport Drain was obtained from PAPL.

4.3 Analysis and Results

Figures E2–1 to E2–3 summarise the pumping test results while Table 4 presents the estimated hydraulic properties from the different analysis methods.

Table 4: Results from Pumping Test Analysis – Consolidated Airport Terminal Station

Method	Transmissivity (m^2/d)	Storage Coefficient	Figures
Distance Drawdown	271–281	1×10^{-2} - 5×10^{-2}	E2–4 to E2–5
Groundwater Model	274	6×10^{-4}	E2–6
Analytical Type Curves	136–690 *	-	E2–7 to E2–20 *

* Selected type curves only

The results indicate:

- The drawdown in piezometric level in the pumping well reached approximately 17 m and 9 m during Test 1 and Test 2, respectively, and the groundwater level reached steady-state within only a few hours of pumping.
- The groundwater and piezometric level drawdown in the monitoring wells ranged between 0.1 m and 2.3 m, depending on the distance of the monitoring well from the pumping well. This suggests that there is a large well loss in the pumping well (according to the distance drawdown curves the expected groundwater level drawdown in the aquifer just outside the pumping well is estimated to be around 4.5 m and 3.5 m instead of the observed 17 m and 9 m, for the two tests respectively).
- MW1-02E and BH1-25, both screened in the underlying Osborne Formation, show significant hydraulic connection with the Ascot Formation. MW1-02E shows a very similar response to pumping when compared with MW1-02D which is screened in the Ascot Formation and located next to MW1-02E.
- BH1-25 shows a small response to pumping at a distance of 130 m from the test well. The hydraulic connection between the Osborne Formation and the Ascot Formation is supported by the distance drawdown plot where the measurements for both formations plot on a straight line.
- The drawdown in piezometric level at BH1-21 (located closest to the Perth Airport control tower) was approximately 0.5 m at the end of the test.
- MW1-02C, screened in the overlying Bassendean Sand, Guildford Formation and Gnangara Sand, shows a similar but reduced response to pumping when compared with MW1-02D and MW2-01E. This response indicates some connection with the Ascot Formation, semi-confined conditions and a lower vertical hydraulic conductivity.
- A good match between observed and modelled groundwater level drawdown (Figure E2–6) was obtained. The Ascot Formation was found to have the greatest hydraulic conductivity. The best match was obtained when using a vertical to horizontal hydraulic conductivity ratio of 0.1 (= k_v/k_h).

- The estimated range in storage coefficient suggests semi-confined to confined conditions.

4.3.1 Groundwater Quality

Figure E2–21 shows the field monitoring results of pH and Electrical Conductivity (EC) from the Constant Rate tests while Table E1 presents the groundwater quality laboratory results from samples taken at the beginning and end of the pumping test. The results indicate:

- pH was stable throughout the pumping tests ranging between 7.5 and 8.0, indicating slightly alkaline conditions, excluding one outlier.
- EC was stable throughout the pumping tests ranging between ranging between 600 and 800 uS/cm, indicating fresh water.
- Total Nitrogen was between 0.16 mg/L and 0.1 mg/L at the beginning and end of each test respectively, which is below the SRT criterion for disposal to Swan River (1 mg/L). However, total phosphorous (0.2 mg/L and 0.13 mg/L) was above the SRT criterion (0.1 mg/L).
- Total iron was low and ranged between 0.57 mg/L and 0.87 mg/L, which is below the SRT criterion of 1 mg/L.
- BTEX, MAH, PAH, hydrocarbons and pesticides were not detected in either of the two samples.

5.0 PW 2-01 (AIRPORT WEST STATION)

The results from the Stage 2 Geotechnical Investigation indicated that the Superficial Aquifer at Airport West Station consists of Bassendean Sand, Guildford Formation, Gnangara Sand and Ascot Formation, overlying the Kardinya Shale of the Osborne Formation. The Superficial Aquifer is approximately 16 m thick at the Airport West Station, of which approximately 8 m consists of the Ascot Formation. The Osborne Formation below the Airport West Station consists of Kardinya Shale which was tested to have a very low hydraulic conductivity. A pumping test in the Osborne Formation at this location was therefore considered to be not necessary. The most permeable unit at the Airport West Station is the Ascot Formation and it was therefore decided to undertake the pumping test in a pumping well screened across the Ascot Formation. The pumping test well was installed and screened across the full thickness of the Ascot Formation with two of the dedicated monitoring wells (MW2-01C and MW2-01D) installed in the Ascot Formation, one monitoring well in the Kardinya Shale (MW2-01E) and two wells in the Bassendean Sand (MW2-01A and MW2-01B). MW2-01B was located on the other side of Belmont Main Drain to assess the effect the drain had on the pumping test results.

5.1 Site and Pumping Test Layout

Pumping tests were carried out at PW 2-01 between 19 and 22 January 2015 and included:

- A step-test with pumping rates set at 2, 3, 4, 5 and 5.5 L/s. The steps lasted for 60 minutes each.
- Constant Rate Test 1 (CRT 1) - A 26-hour constant rate test with a discharge rate of approximately 5.0 L/s followed by a 5-hour recovery test.
- Constant Rate Test 1 (CRT 2) - A 22-hour constant rate test with a discharge rate of approximately 6.1 L/s followed by a 3.5-hour recovery test.

During CRT 1 reticulation systems from garden bores in adjacent properties and parks were found to affect the groundwater levels in the monitoring wells and VWPs. In addition the groundwater levels reached steady state within a short period of time indicating that the aquifer system could be pumped at a higher pumping rate. The short period of time to reach steady state may also be due to a recharge boundary associated with the nearby drain.

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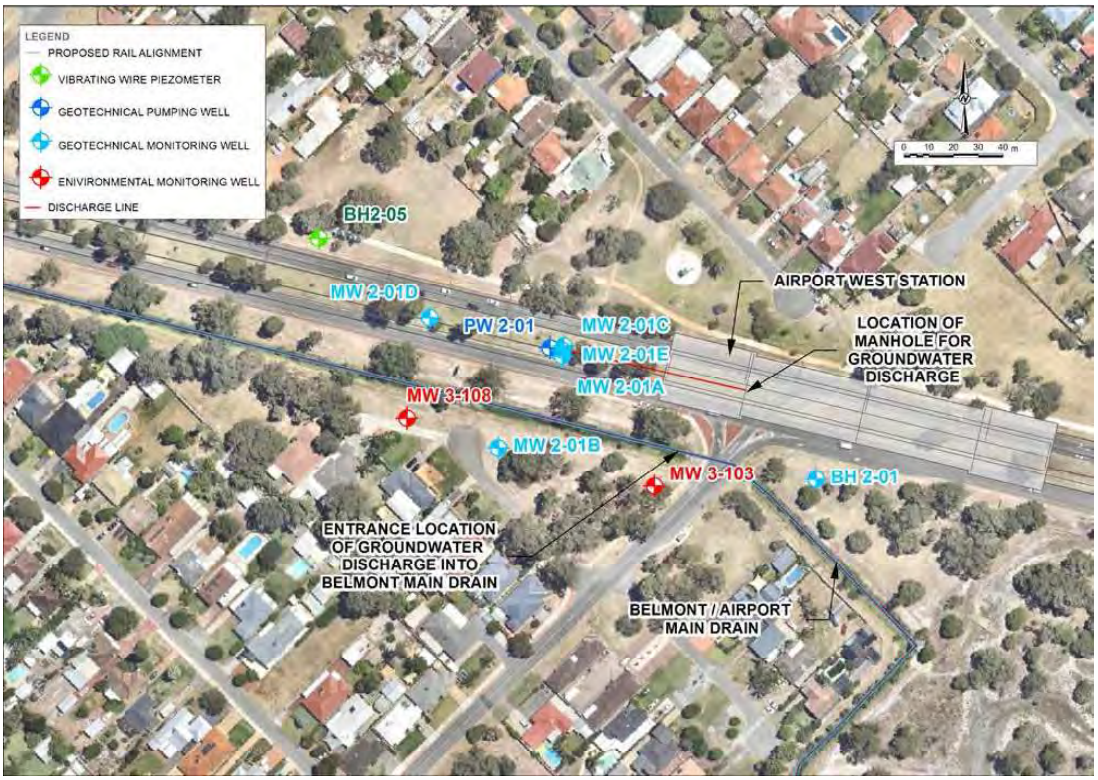
CRT 2 was carried out after it was arranged with the City of Belmont to turn off their reticulation systems and the test was started in the middle of the morning when the household reticulation bores were no longer pumping. A higher pumping rate was chosen based on the results of the first test.

Sketch 3 shows the site layout of the pumping test well (PW1-02), monitoring wells and VWPs while Table 5 presents the distances and directions of the different monitoring locations from the pumping test well.



Sketch 3: Site Layout for pumping test at PW 2-01.

Table 5: Distance and Direction from the Pumping Test Well to the Monitoring Locations

Well ID	Type	Distance to Pumping Well (m)	General Direction to Pumping Well	Screened Geological Unit
PW2-01	Pumping Well	-	-	AF
MW2-01A	Dedicated Monitoring Well	6	East	BS, GF, GS
MW2-01B	Dedicated Monitoring Well	45	South	BS, GF, GS
MW2-01C	Dedicated Monitoring Well	6	East	AF
MW2-01D	Dedicated Monitoring Well	50	West	AF
MW2-01E	Dedicated Monitoring Well	6	East	OFs
BH2-01	Monitoring Well	120	Southeast	AF
MW3-108	Monitoring Well	64	Southwest	AF
MW3-103	Monitoring Well	70	Southeast	BS
BH2-05	Vibrating Wire Piezometer	103	Northwest	GS and AF



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5.2 Discharge Location

The abstracted groundwater during the pumping test was discharged into the Belmont Main Drain via a man hole located on the road island where the pumping well was installed (Sketch 3). Approval for the discharge of the groundwater was given by the Water Corporation, the City of Belmont and Swan River Trust on the condition that water quality monitoring and sampling was also undertaken in the Belmont Main Drain. pH and EC was therefore measured in the morning and evening in the Belmont Drain during the CRT at two locations:

- Corner of Brearley Ave and Second Street, approximately 5 m downstream of the discharge point into the drain.
- Corner of Brearley Ave and First Street, approximately 365 m downstream of the discharge point into the drain.

Three water samples were collected at the corner of Brearley Ave and Second Street, at the beginning, halfway through and at the end of the pumping tests.

5.3 Analysis and Results

Figures E3–1 to E3–3 summarise the pumping test results while Table 6 presents the estimated hydraulic properties from the different analysis methods.

Table 6: Results from Pumping Test Analysis – Airport West Station

Method	Transmissivity (m^2/d)	Storage Coefficient	Figures
Distance Drawdown	74–76	6×10^{-3}	E3–4 to E3–5
Groundwater Model	76	2×10^{-4}	E3–6
Analytical Type Curves	39–180 *	-	E3–7 to E3–15 *

* Selected type curves only

The results indicate:

- The drawdown in piezometric level in the pumping well reached approximately 8.5 m and 11 m during Test 1 and Test 2, respectively, and the groundwater level reached steady-state within only one hour of pumping.
- The groundwater and piezometric level drawdown in the monitoring wells ranged between 0.1 m and 4.0 m, depending on the distance of the monitoring well from the pumping well.
- According to the distance drawdown curves the expected groundwater level drawdown in the aquifer just outside the pumping well is estimated to be 7 m and 8 m instead of the observed 8.5 m and 11 m, in the two CRT respectively.
- The monitoring wells, particularly MW3-108 and MW2-01B showed a clear effect from pumping of nearby household reticulation bores, resulting in temporary drawdown of up to 0.5 m in these wells.
- MW2-01E (screened in the underlying Osborne Formation) showed a different response to any of the other monitoring locations with a steady decrease in piezometric pressure throughout the tests. At the end of the tests the piezometric head in the Osborne Formation had been reduced by approximately 2 m. After the pumping ceased only a very small recovery (<0.1 m) occurred suggesting a very low hydraulic conductivity and slow or limited recharge of the Osborne Formation.

- MW3-103 which is shallow (4.5 m deep) and located south of Belmont Main Drain showed no noticeable response to the pumping, suggesting that the Belmont Main Drain acts a positive head boundary to the groundwater table. MW3-108 and MW2-01B, which are also located south of the Belmont Main Drain but installed deeper in the Ascot Formation, showed a reduction in piezometric level of up to approximately 1.2 m, indicating that a piezometric pressure reduction occured below the Belmont Main drain and that the radius of influence extended beyond the drain.
- MW2-01A, screened in the overlying Bassendean Sand, Guildford Formation and Gnangara Sand, shows a similar but reduced response to pumping when compared with MW2-01C. This response indicates some connection with the Ascot Formation, semi-confined conditions and a lower vertical hydraulic conductivity.
- A reasonable match between observed and modelled groundwater level drawdown (Figure E3-6) was obtained in the groundwater model. The Ascot Formation was found to have the greatest hydraulic conductivity. The best match was obtained when using a vertical to horizontal hydraulic conductivity ratio of 0.1 (= k_v/k_h).
- The groundwater model indicates that water inflow from the Belmont Main Drain into the aquifer during the pumping tests could have been up to 0.4 L/s.
- The estimated range in storage coefficient suggests confined conditions.

5.4 Groundwater Quality

Figure E3–16 and E3–17 show the field monitoring results of pH and Electrical Conductivity (EC) from the Constant Rate Test while Table E1 presents the groundwater quality laboratory results from samples taken at the beginning and end of the pumping test. The results indicate:

- pH was generally steady through the pumping test with the following ranges, excluding outliers:
  - 6.9 to 7.4 in PW 2-01 (average of 7.25)
  - 6.7 to 7.4 in Belmont Drain at Second Street (average of 7.19)
  - 7.6 to 7.3 in Belmont Drain at First Street (average of 7.43)
  - pH was within the upper and lower bounds of the Swan River Trust criteria.
  - pH of the discharge water remained within one pH unit of the receiving environment.
- EC was brackish in the both the discharge water and the drain with the following ranges:
  - 1191 to 1339 uS/cm in PW2-01 (average of 1313 uS/cm)
  - 628 to 969 uS/cm in Belmont Drain at Second Street (average of 820 uS/cm)
  - 608 to 955 uS/cm in Belmont Drain at First Street (average of 820 uS/cm)
  - EC was higher in the discharge water than in the drain
  - The EC increased in the drain from around 600 uS/cm prior to discharge to approximately 900 to 950 uS/cm during discharge into the drain
  - The EC decreased at the discharge to pre-discharge concentrations within 0.5 hour of ceasing the discharge into the drain.
- Total Nitrogen ranged between 0.31 mg/L and 0.13 mg/L at the beginning and end of test respectively, which is below the SRT criteria for disposal to Swan River (1 mg/L).

- Total iron remained stable throughout the pumping and ranged between 2.5 mg/L and 2.8 mg/L.
- BTEX, MAH, PAH, hydrocarbons and pesticides were not detected in either of the two samples.

The results indicated that discharge to Belmont Main Drain did not have a detrimental effect on the receiving environment.

6.0 PW 2-02 (BAYSWATER DIVE STRUCTURE)

The results from the Stage 2 Geotechnical Investigation indicated that the Superficial Aquifer at the Bayswater Dive Structure consists of two aquifers (Upper and Lower Aquifer), separated by a 5 m thick impermeable aquitard. The Superficial Aquifer is approximately 25 m thick (including the aquitard) and is underlain by the Kardinya Shale of the Osborne Formation. The Upper Aquifer consists mainly of Bassendean Sand while the Lower Aquifer consists of Perth Formation. The Lower Aquifer is approximately 15 m thick at the Bayswater Dive Structure portal.

The pumping test well was installed and screened across the full thickness of the Lower Aquifer with three of the dedicated monitoring wells (MW2-02A, MW2-02B and MW2-02C) installed in the Lower Aquifer and one monitoring well (MW2-02D) in the Upper Aquifer. Two other environmental monitoring wells (MW3-165 and MW3-166) were installed in the Upper Aquifer while one geotechnical borehole drilled into the Kardinya Shale was converted to a monitoring well (BH2-26).

6.1 Site and Pumping Test Layout

Pumping tests were carried out at PW 2-02 between 2 and 6 February 2015 and included:

- A step test with pumping rates set at 10, 14, 16 and 18 L/s. The first three steps lasted 60 minutes, while the fourth step was stopped after 4 minutes due to excessive drawdown.
- A 32-hour constant rate test with a discharge rate of approximately 12 L/s followed by a 46-hour recovery test.

Sketch 4 shows the site layout of the pumping test well (PW2-02), monitoring wells and VWPs while Table 7 presents the distances and directions of the different monitoring locations from the pumping test well.

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Sketch 4: Site Layout for pumping test at PW 2-02

Table 7: Distance and Direction from the Pumping Test Well to the Monitoring Locations

Well ID	Type	Distance to Pumping Well (m)	General Direction to Pumping Well	Screened Geological Unit	Aquifer
PW2-02	Pumping Well	-	-	PF	Lower
MW2-02A	Dedicated Monitoring Well	4	North	PF	Lower
MW2-02B	Dedicated Monitoring Well	42	Northeast	PF	Lower
MW2-02C	Dedicated Monitoring Well	5	North	PF	Lower
MW2-02D	Dedicated Monitoring Well	4	Northeast	BS	Upper
BH2-26	Monitoring Well	106	Northwest	OFf	Osborne Aquitard
MW3-156	Monitoring Well	77	Southeast	PF	Lower
MW3-165	Monitoring Well	51	Northwest	BS	Upper
MW3-166	Monitoring Well	60	Northwest	BS	Upper
BH0-02	Vibrating Wire Piezometer#	16	Northwest	BS, PF	Upper and Lower
BH2-27	Vibrating Wire Piezometer#	165	Northwest	BS, PF	Upper and Aquitard
BH2-22 *	Vibrating Wire Piezometer#	501	Southeast	PF, OFf	Lower and Osborne Aquitard
BH2-19 *	Vibrating Wire Piezometer#	1054	Southeast	PF, PF	Lower

* Locations of BH2-19 and BH2-22 are not shown on Sketch 1 due to the distance from PW2-02. Refer to Figure 5 in the main report for location of these VWPs. # Each VWP has two sensors installed.

6.2 Discharge Location

Originally the plan was to discharge the pumped groundwater into the Bayswater Main Drain via a stormwater drain located to the south of the site. However, approval could not be obtained from the City of Bayswater and Swan River Trust due to the known groundwater contamination in the area from the CSBP site (located to the northeast of the Bayswater Dive structure). Therefore, arrangements were instead made with the Water Corporation to discharge the pumped water to the sewer (approval to discharge up to 15 L/s to the sewer was granted by the Water Corporation). However, on the day of the connection to the sewer discharge point (manhole), it was found that it had been covered and that it would take several months before access could be obtained.

Approval to infiltrate on site was then sought and obtained from the City of Bayswater and the site owner (Western Power) and the groundwater abstracted during the pumping tests was therefore re-infiltrated on site (Sketch 4). The water was infiltrated into two infiltration basins with discharge being alternated between the basins when they had filled. The capacity of the infiltration basins was found to be insufficient as the pumping test progressed and it was therefore decided to start alternating the discharge between the infiltration basins and a natural low depression south of the main infiltration basin. After 32 hours of pumping it was considered that the pumping test data was sufficient for analysis and the pumping test was therefore terminated due to issues with the available infiltration capacity.

6.3 Analysis and Results

Figures E4–1 to E4–2 summarise the pumping test results while Table 8 presents the estimated hydraulic properties from the different analysis methods.

Table 8: Results from Pumping Test Analysis – Bayswater Dive Structure

Method	Transmissivity (m^2/d)	Storage Coefficient	Figures
Distance Drawdown	120	7.0×10^{-5}	E4–3
Groundwater Model	60–113	1.5×10^{-5} - 7.5×10^{-5}	E4–4 and E4–5
Analytical Type Curves	21–300 *	-	E4–6 to E4–13 *

* Selected type curves only

The results indicate:

- The drawdown in piezometric level in the pumping well reached almost 14 m toward the end of the test and the piezometric level almost reached steady-state conditions at the end of the pumping test. The drawdown in the monitoring wells and VWPs ranged between 0.1 m and 9.2 m.
- There was no groundwater level drawdown response in the Upper Aquifer due to the pumping of the Lower Aquifer. Instead groundwater level rose in some of the wells due to the infiltration of the water back into the Upper Aquifer. This indicates that there is no noticeable connection between the two aquifers.
- BH2-26 (screened in the Osborne Formation), showed a small delayed response to the pumping, indicating limited connection of the Lower Aquifer with the Osborne Formation.
- The recovery of the piezometric level after pumping ceased took a much longer time than for the other pumping tests described in this Appendix, suggesting that there is limited direct recharge to the Lower Aquifer.
- The distance drawdown curve (Figure E4–3) indicates that there is a medium correlation between distance (from the pumping well) and drawdown, suggesting heterogeneous conditions. MW2-02B falls far from the curve, which is also the well that showed a significantly lower hydraulic conductivity during slug testing than the other wells.

- The groundwater model was not able to provide a good match for all observed piezometric levels by using a homogenous aquifer approach. The closer wells (MW2-01C and BH0-02) showed a better match with a higher transmissivity, while the wells and VWPs located further away (BH2-19, BH2-22 and MW2-02B) showed a better match with a lower transmissivity (E4–4 and E4–5). There was found to be only a small difference in the calculated values when varying the vertical hydraulic conductivity in the model, which is likely due to the wells being screened over the full aquifer thickness.
- The type curve analysis provided larger transmissivity for the pumping curves and lower transmissivity for the recovery curves, again indicating slow or limited recharge to the Lower Aquifer. MW2-02C reached steady-state more quickly than any other well, suggesting that the hydraulic conductivity could be higher in the upper part of the Lower Aquifer.

6.4 Groundwater Quality

Figure E4–14 shows the field monitoring results of pH and Electrical Conductivity (EC) from the Constant Rate Test while Table E1 presents the groundwater quality laboratory results from samples taken at the beginning and end of the pumping test. The results indicate:

- pH decreased from 7.5 to 7.0 within the first 400 minutes of the pumping test after which it remained stable at about 7.0, suggesting neutral conditions.
- EC ranged between 500 and 700 uS/cm during the test indicating fresh water. There was a slight increasing trend from the beginning of the test to 1600 minutes into the test, after which EC decreased again.
- Total Nitrogen was 0.33 mg/L and 0.49 mg/L at the beginning and end of test, respectively, which is below the SRT criterion for disposal to Swan River (1 mg/L).
- Total iron increased from 3 mg/L at the beginning of the test to 22 mg/L at the end of the test.
- BTEX, MAH, PAH, hydrocarbons and pesticides were not detected in either of the collected samples.

TABLE E1 - SUMMARY OF WATER QUALITY RESULTS

			Area	Forrestfield		Airport West									Consolidated		Bayswater Dive Structure		
			Well ID	PW1-01	PW1-01	PW2-01	SW08	PW2-01	Drain	PW2-01	Drain	PW2-01	PW2-01 DUP	Drain	PW1-02	PW1-02	MW2-02A	MW2-02C	PW2-02
			Test Time	Beginning	End	Before		Beginning		Mid		End			Beginning	End	Before		End
			Sample Date	8/12/2014	11/12/2014	5/12/2014		19/1/2015		21/1/2015		22/1/2015			6/1/2015	8/1/2015	5/12/2014		4/2/2015
			Matrix	Groundwater		Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater		Water	Groundwater		Groundwater		
pH	pH Units	6.0-8.5	0	6.3	6.5	9.1	8.0	7.9	7.7	7.7	7.7	7.8		7.6	8.0	8.1	7.1	7.4	7.0
Total Dissolved Solids Dried at 175-185°C	mg/L		10	540	520	770	360	720	390	740	540	740		460	410	420	250	380	420
Conductivity @ 25 C	µS/cm		2	930	930			1300	670	1300	920	1300		660	720	750			670
Salinity	mg/L		2	610	610			860	440	860	600	850		430	470	490			440
Total Suspended Solids Dried at 103-105°C	mg/L		5			7	<5										<5	9	
Turbidity	NTU		0.5			5.7	3.5										4.1	15	
Hydroxide Alkalinity as OH	mg/L		5	<5	<5	<5	<5	<5	<5	<5	<5	<5		<5	<5	<5	<5	<5	<5
Carbonate Alkalinity as CO3	mg/L		1	<1	<1	57	<1	<1	<1	<1	<1	<1		<1	<1	<1	<1	<1	<1
Bicarbonate Alkalinity as HCO3	mg/L		5	25	22	170	110	340	110	330	200	330		110	180	180	100	150	93
Total Alkalinity as CaCO3	mg/L		5	21	18	240	90	280	88	270	160	270		90	150	150	84	120	76
Acidity to pH 8.3	mg CaCO3/L	>40	5	39	33	<5	<5	16	10	18	10	17		9	7	<5	16	19	50
Chloride, Cl	mg/L		1	130	120	260	130	260	140	260	180	260		140	110	120	91	110	92
Sulphate, SO4	mg/L		1	69	65	24	30	<1	18	<1	20	<1		29	58	58	6	20	110
Sulphide	mg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		0	<0.1	<0.1			<0.1
Sodium, Na	mg/L		0.5	110	100	150	67	140	76	130	92	140		63	67	66	55	110	59
Calcium, Ca	mg/L		0.2	10.0	10.0	95	28	94.0	29	86.0	51	87.0		27	57.0	53.0	14	8.7	30.0
Magnesium, Mg	mg/L		0.1	28.0	28.0	11	12	18.0	14	16.0	14	16.0		12	11.0	12.0	7.5	5.5	14.0
Potassium, K	mg/L		0.1	4.7	4.7	9.7	5.0	6.1	4.8	5.5	5.0	6.3		4.9	4.0	4.0	4.3	3.5	5.5
Total Hardness by Calculation	mg CaCO3/L		1	140	140	280	120	310	130	280	180	290		120	190	180	66	45	130
Total Organic Carbon	mg/L		0.2			21	24										4.1	7.2	
Total Cyanide	mg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	0.1	<0.004			<0.004
Thiocyanate	mg/L		0.1	<0.1	<0.1			<0.1	0	0.4	1	0.3		1	<0.1	<0.1			<0.1
Chlorophyll a	mg/L		0.0005			<0.0005	0.0010										0.0031	<0.0005	
NUTRIENTS																			
Nitrite, NO_2 as NO_2	mg/L		0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Nitrate, NO_3 as NO_3	mg/L		0.2	190	150	<0.2	0.4	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2	<0.2
Nitrate/Nitrite Nitrogen, NOx as N	v		0.05	44	33			<0.05	0.1	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05			<0.05
Ammonia, NH_3	mg/L		0.05	0.07	0.25	0.17	<0.05	0.52	<0.05	0.54	0.19	0.55	0.54	<0.05	<0.05	0.05	0.15	0.13	0.32
Total Kjeldahl Nitrogen	mg/L		0.05	<0.05	<0.05	0.82	0.85	0.70	<0.05	0.76	0.76	0.77	0.82	0.89	0.13	0.10	0.30	0.33	0.47
Total Nitrogen (calc)	mg/L	1	0.05	44.00	33.00	0.83	0.94	0.72	0.06	0.77	0.76	0.78	0.82	0.89	0.16	0.10	0.30	0.33	0.49
Total Phosphorus (Kjeldahl Digestion)	mg/L	0.1	0.01	0.09	0.07	1.10	<0.01	0.31	<0.01	0.12	0.06	0.13	0.12	0.01	0.20	0.13	0.76	0.12	0.07
Filterable Reactive Phosphorus	mg/L		0.00	0.09	0.06			0.04	0.01	0.09	0.05	0.09	0.09	0.004	0.08	0.07			<0.002
DISSOLVED METALS																			
Aluminium, Al	mg/L	0.15	0.02	0.01	<0.005	<0.02	0.10	0.01	0.12	<0.005	0.07	<0.005		0.08	0.04	<0.005	<0.02	0.03	<0.005
Arsenic, As	mg/L		0.02	<0.001	<0.001	<0.02	<0.02	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	0.002	0.002	<0.02	<0.02	0.003
Boron, B	mg/L		0.05	0.05	0.06	0.09	0.07	0.03	0.03	0.02	0.02	0.03		0.04	0.02	0.02	<0.05	0.07	0.02
Cadmium, Cd	mg/L		0.001	<0.0001	<0.0001	<0.001	<0.001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001		<0.0001	<0.0001	<0.0001	<0.001	<0.001	<0.0001
Chromium, Cr	mg/L		0.005	<0.001	<0.001	<0.005	<0.005	0.002	0.001	0.002	0.002	0.002		<0.001	<0.001	<0.001	<0.005	<0.005	<0.001
Copper, Cu	mg/L		0.005	0.001	0.008	<0.005	<0.005	<0.001	0.001	<0.001	<0.001	<0.001		0.001	<0.001	<0.001	<0.005	<0.005	<0.001
Iron, Fe	mg/L		0.02	0.02	0.03	<0.02	0.23	2.10	0.33	2.70	1.10	1.10		0.18	0.13	0.51	2.90	0.25	20.00
Manganese, Mn	mg/L		0.005	0.031	0.002	0.026	0.007	0.093	0.007	0.028	0.014	0.027		<0.001	0.038	0.034	0.44	0.094	0.064
Molybdenum, Mo	mg/L		0.01	<0.001	<0.001	<0.01	<0.01	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.01	<0.01	<0.001
Lead, Pb	mg/L		0.02	0.001	0.002	<0.02	<0.02	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.02	<0.02	<0.001
Nickel, Ni	mg/L		0.005	0.001	<0.001	<0.005	<0.005	<0.001	0.001	<0.001	<0.001	<0.001		0.002	<0.001	<0.001	<0.005	<0.005	0.001
Selenium, Se	mg/L		0.05	<0.002	<0.002	<0.05	<0.05	<0.002	<0.002	<0.002	<0.002	<0.002		<0.002	<0.002	<0.002	<0.05	<0.05	<0.002
Tin, Sn	mg/L		0.05	<0.001	<0.001	<0.05	<0.05	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.05	<0.05	<0.001
Zinc, Zn	mg/L		0.01	0.009	0.065	<0.01	0.01	<0.005	<0.005	<0.005	0.01	<0.005		<0.005	<0.005	<0.005	<0.01	<0.01	0.008
Mercury	mg/L		0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005		<0.00005	<0.00005	<0.00005	<0.00005	<0.00005	<0.00005
TOTAL METALS																			
Total Aluminium	mg/L	1	0.02	0.08	0.01	0.18	0.27	0.26	0.13	0.02	0.09	0.01		0.11	0.08	0.02	0.03	1.50	0.01
Total Arsenic	mg/L		0.02	<0.001	<0.001	<0.02	<0.02	0.001	<0.001	<0.001	<0.001	<0.001		<0.001	0.002	0.002	<0.02	<0.02	0.003
Total Boron	mg/L		0.05	0.07	0.08	0.09	0.07	0.03	0.03	0.03	0.04	0.03		0.04	0.03	0.03	<0.05	0.07	0.03
Total Cadmium	mg/L		0.001	<0.0001	<0.0001	<0.001	<0.001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001		<0.0001	<0.0001	<0.0001	<0.001	<0.001	<0.0001
Total Chromium	mg/L		0.005	<0.001	<0.001	0.008	<0.005	0.007	0.001	0.003	0.002	0.002		0.001	<0.001	<0.001	<0.005	<0.005	<0.001
Total Copper	mg/L		0.005	0.002	0.003	<0.005	<0.005	<0.001	0.001	<0.001	<0.001	<0.001		0.001	<0.001	<0.001	<0.005	0.006	<0.001
Ferrous Iron, Fe2+	mg/L		0.05	<0.05	<0.05			2.50	0.37	2.70	0.99	2.80		0.33	0.51	0.49			22.00
Ferric Iron, Fe3+	mg/L		0.05	<0.05	0.018			<0.05	<0.05	<0.05	0.32	<0.05		<0.05	0.06	0.38			1.00
Total Iron	mg/L	1	0.02	0.03	0.02	0.10	0.43	2.50	0.41	2.70	1.30	2.80		0.33	0.57	0.87	3.00	1.20	22.00
Total Manganese	mg/L		0.005	0.033	0.002	0.032	0.008	0.096	0.008	0.028	0.016	0.028		0.008	0.042	0.037	0.46	0.099	0.071
Total Molybdenum	mg/L		0.01	<0.001	<0.001	<0.01	<0.01	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.01	<0.01	<0.001
Total Lead	mg/L		0.02	0.002	<0.001	<0.02	<0.02	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.02	<0.02	<0.001
Total Nickel	mg/L		0.005	0.001	<0.001	0.005	<0.005	0.001	0.001	<0.001	<0.001	<0.001		0.002	0.001	<0.001	<0.005	<0.005	0.001
Total Selenium	mg/L		0.05	<0.002	<0.002	<0.05	<0.05	<0.002	<0.002	<0.002	<0.002	<0.002		<0.002	<0.002	<0.002	<0.05	<0.05	<0.002
Total Tin	mg/L		0.05	<0.001	<0.001	<0.05	<0.05	<0.001	<0.001	<0.001	<0.001	<0.001		<0.001	<0.001	<0.001	<0.05	<0.05	<0.001
Total Zinc	mg/L		0.01	0.012	0.008	<0.01	<0.01	<0.005	0.007	<0.005	0.01	<0.005		0.009	<0.005	0.029	<0.01	<0.01	0.011
Total Mercury	mg/L		0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001		<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001

TABLE E1 - SUMMARY OF WATER QUALITY RESULTS

			Area	Forrestfield		Airport West									Consolidated		Bayswater Dive Structure		
			Well ID	PW1-01	PW1-01	PW2-01	SW08	PW2-01	Drain	PW2-01	Drain	PW2-01	PW2-01 DUP	Drain	PW1-02	PW1-02	MW2-02A	MW2-02C	PW2-02
			Test Time	Beginning	End	Before		Beginning		Mid		End			Beginning	End	Before		End
			Sample Date	8/12/2014	11/12/2014	5/12/2014		19/1/2015		21/1/2015		22/1/2015			6/1/2015	8/1/2015	5/12/2014		4/2/2015
			Matrix	Groundwater		Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater		Water	Groundwater		Groundwater		
HYDROCARBONS																			
Benzene (F0)	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
TRH C6-C9	µg/L		40	<40	<40	71	<40	<40	<40	<40	<40	<40		<40	<40	<40	<40	<40	<40
TRH C6-C10	µg/L		50	<50	<50			<50	<50	<50	<50	<50		<50	<50				<50
TRH C6-C10 minus BTEX (F1)	µg/L		50	<50	<50	79	<50	<50	<50	<50	<50	<50		<50	<50	<50	<50	<50	<50
TRH C10-C14	µg/L		50	55	<50	<50	<50	<50	<50	<50	<50	<50		<50	<50	<50	<50	<50	<50
TRH C15-C28	µg/L		200	<200	<200	<200	<200	<200	<200	<200	<200	<200		<200	<200	<200	<200	<200	<200
TRH C29-C36	µg/L		200	<200	<200	<200	<200	<200	<200	<200	<200	<200		<200	<200	<200	<200	<200	<200
TRH >C10-C16 (F2)	µg/L		60	<60	<60	<60	<60	<60	<60	<60	<60	<60		<60	<60	<60	<60	<60	<60
TRH >C16-C34 (F3)	µg/L		500	<500	<500	<500	<500	<500	<500	<500	<500	<500		<500	<500	<500	<500	<500	<500
TRH >C34-C40 (F4)	µg/L		500	<500	<500	<500	<500	<500	<500	<500	<500	<500		<500	<500	<500	<500	<500	<500
VOLATILES																			
Benzene	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Toluene	µg/L		0.5	<0.5	<0.5	1.3	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Ethylbenzene	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
m/p-xylene	µg/L		1	<1	<1	<1	<1	<1	<1	<1	<1	<1		<1	<1	<1	<1	<1	<1
o-xylene	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Total Xylenes			1.5	<1.5	<1.5			<1.5	<1.5	<1.5	<1.5	<1.5		<1.5	<1.5	<1.5			<1.5
Styrene (Vinyl benzene)			0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Naphthalene	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
MtBE (Methyl-tert-butyl ether)				<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
PESTICIDES																			
Hexachlorobenzene (HCB)	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Alpha BHC	µg/L			<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
Lindane (gamma BHC)	µg/L		0.002	<0.05	<0.05	<0.002	<0.002	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.002	<0.002	<0.05
Beta BHC	µg/L			<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
Heptachlor	µg/L		0.002	<0.02	<0.02	<0.002	<0.002	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.002	<0.002	<0.02
Delta BHC	µg/L			<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
Aldrin	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Isodrin	µg/L			<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			<0.02
Heptachlor epoxide	µg/L		0.002	<0.02	<0.02	<0.002	<0.002	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.002	<0.002	<0.02
Gamma Chlordane	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Alpha Chlordane	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Alpha Endosulfan	µg/L		0.005	<0.02	<0.02	<0.005	<0.005	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.005	<0.005	<0.02
p,p'-DDE	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Dieldrin	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Endrin	µg/L		0.004	<0.02	<0.02	<0.004	<0.004	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.004	<0.004	<0.02
p,p'-DDD	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Beta Endosulfan	µg/L		0.005	<0.02	<0.02	<0.005	<0.005	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.005	<0.005	<0.02
p,p'-DDT	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Endrin Aldehyde	µg/L			<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			<0.02
Endosulfan sulphate	µg/L		0.005	<0.02	<0.02	<0.005	<0.005	<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02	<0.005	<0.005	<0.02
Methoxychlor	µg/L		0.100	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
Mirex	µg/L		0.010	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Endrin Ketone	µg/L		0.010	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Oxychlordane	µg/L		0.002	<0.01	<0.01	<0.002	<0.002	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.002	<0.002	<0.01
Azinphos-methyl (Guthion)	µg/L		0.050	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Bromophos Ethyl	µg/L		0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Chlorpyrifos (Chlorpyrifos Ethyl)	µg/L		0.009	<0.01	<0.01	<0.009	<0.009	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.009	<0.009	<0.01
Diazinon (Dimpylate)	µg/L		0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Dichlorvos	µg/L		0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Dimethoate	µg/L		0.1	<0.15	<0.15	<0.1	<0.1	<0.15	<0.15	<0.15	<0.15	<0.15		<0.15	<0.15	<0.15	<0.1	<0.1	<0.15
Ethion	µg/L		0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Fenitrothion	µg/L		0.1	<0.2	<0.2	<0.1	<0.1	<0.2	<0.2	<0.2	<0.2	<0.2		<0.2	<0.2	<0.2	<0.1	<0.1	<0.2
Malathion	µg/L		0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Methidathion	µg/L		0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05	<0.05	<0.05	<0.05
Parathion-ethyl (Parathion)	µg/L		0.004	<0.01	<0.01	<0.004	<0.004	<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01	<0.004	<0.004	<0.01
PCBs																			
PCB Congener C28	µg/L		0.02	<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			<0.02
PCB Congener C52	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
PCB Congener C101	µg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	<0.004	<0.004			<0.004
PCB Congener C118	µg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	<0.004	<0.004			<0.004
PCB Congener C153	µg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	<0.004	<0.004			<0.004
PCB Congener C138	µg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	<0.004	<0.004			<0.004
PCB Congener C180	µg/L		0.004	<0.004	<0.004			<0.004	<0.004	<0.004	<0.004	<0.004		<0.004	<0.004	<0.004			<0.004

City of Kalamunda

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Ordinary Council Meeting 12 October 2021 Attachments

Attachment 10.5.2.3

TABLE E1 - SUMMARY OF WATER QUALITY RESULTS

			Area	Forrestfield		Airport West									Consolidated		Bayswater Dive Structure		
			Well ID	PW1-01	PW1-01	PW2-01	SW08	PW2-01	Drain	PW2-01	Drain	PW2-01	PW2-01 DUP	Drain	PW1-02	PW1-02	MW2-02A	MW2-02C	PW2-02
			Test Time	Beginning	End	Before		Beginning		Mid		End			Beginning	End	Before		End
			Sample Date	8/12/2014	11/12/2014	5/12/2014		19/1/2015		21/1/2015		22/1/2015			6/1/2015	8/1/2015	5/12/2014		4/2/2015
			Matrix	Groundwater		Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater	Surfacewater	Groundwater		Water	Groundwater		Groundwater		
PAH																			
Naphthalene	µg/L		0.02	<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			0.03
2-methylnaphthalene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
1-methylnaphthalene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Acenaphthylene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Acenaphthene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Fluorene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Phenanthrene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Anthracene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Fluoranthene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Pyrene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(a)anthracene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Chrysene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(k)fluoranthene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(b&j)fluoranthene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(a)pyrene	µg/L		0.005	<0.005	<0.005			<0.005	<0.005	<0.005	<0.005	<0.005		<0.005	<0.005	<0.005			<0.005
Indeno(1,2,3-cd)pyrene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Dibenzo(a&h)anthracene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(ghi)perylene	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Benzo(b&j&k)fluoranthene	µg/L		0.02	<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			<0.02
Total PAH (18)	µg/L		0.1					<0.1	<0.1	<0.1	<0.1	<0.1		<0.1					<0.1
Phenols																			
4-chloro-3-methylphenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
2-chlorophenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
2,4-dichlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
2,6-dichlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Pentachlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
2,5-dichlorophenol	µg/L		0.05	<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
2,3,4,5-tetrachlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
2,3,4,6/2,3,5,6-tetrachlorophenol	µg/L		0.02	<0.02	<0.02			<0.02	<0.02	<0.02	<0.02	<0.02		<0.02	<0.02	<0.02			<0.02
2,4,5-trichlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
2,4,6-trichlorophenol	µg/L		0.01	<0.01	<0.01			<0.01	<0.01	<0.01	<0.01	<0.01		<0.01	<0.01	<0.01			<0.01
Phenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	0.1	<0.1	<0.1		<0.1	0.1	<0.1			0.3
2-methyl phenol (o-cresol)	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
3/4-methyl phenol (m/p-cresol)	µg/L		0.2	<0.2	<0.2			<0.2	<0.2	<0.2	<0.2	<0.2		<0.2	<0.2	<0.2			<0.2
2,4-dimethylphenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
2,4-dinitrophenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
2-methyl-4,6-dinitrophenol	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
2-nitrophenol	µg/L		0.05	<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
4-nitrophenol	µg/L		0.05	<0.05	<0.05			<0.05	<0.05	<0.05	<0.05	<0.05		<0.05	<0.05	<0.05			<0.05
Dinex (2-cyclohexyl-4,6-dinitrophenol)	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
Dinoseb	µg/L		0.1	<0.1	<0.1			<0.1	<0.1	<0.1	<0.1	<0.1		<0.1	<0.1	<0.1			<0.1
Carbamates																			
Carbofuran	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Carbaryl	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Triazines																			
Simazine	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Atrazine	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Propazine	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Terbuthylazine	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Metribuzin	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Prometryn	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Terbutryn	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Cyanazine	µg/L		0.5	<0.5	<0.5			<0.5	<0.5	<0.5	<0.5	<0.5		<0.5	<0.5	<0.5			<0.5
Hexazinone	µg/L		1	<1	<1			<1	<1	<1	<1	<1		<1	<1	<1			<1
E.coli & Coliforms																			
E. coli	CFU/100mL		1	<1	<1			<1	490	<1	170	<1		500	<1	<1			<1
Thermotolerant Coliforms	CFU/100mL		1	<1	<1			<1	720	<1	250	<1		500	<1	<1			<1

City of Kalamunda

Ordinary Council Meeting 12 October 2021 Attachments

Attachment 10.5.2.3

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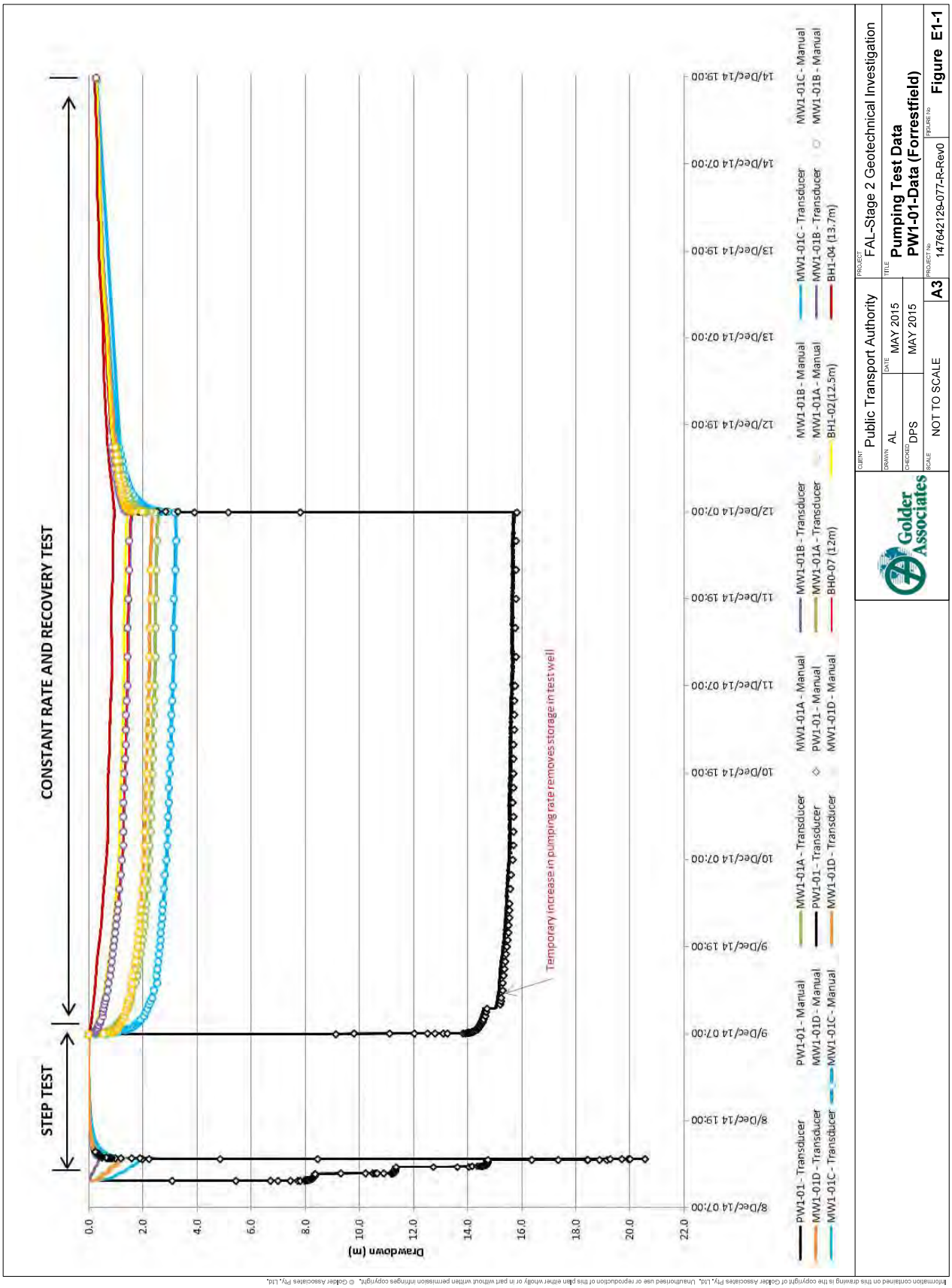
APPENDIX E
Summary of Test Pumping Analysis

ATTACHMENT E1
Constant Rate Test – Forrestfield Dive Structure (PW1-01)



Golder Associates

CLIENT: Public Transport Authority
DRAWN: AL — DATE: MAY 2015
CHECKED: DPS — DATE: MAY 2015
SCALE: NOT TO SCALE — A3
PROJECT: FAL-Stage 2 Geotechnical Investigation
TITLE: Pumping Test Data PW1-01-Data (Forrestfield)
PROJECT No: 147642129-077-R-Rev0
FIGURE No: Figure E1-1

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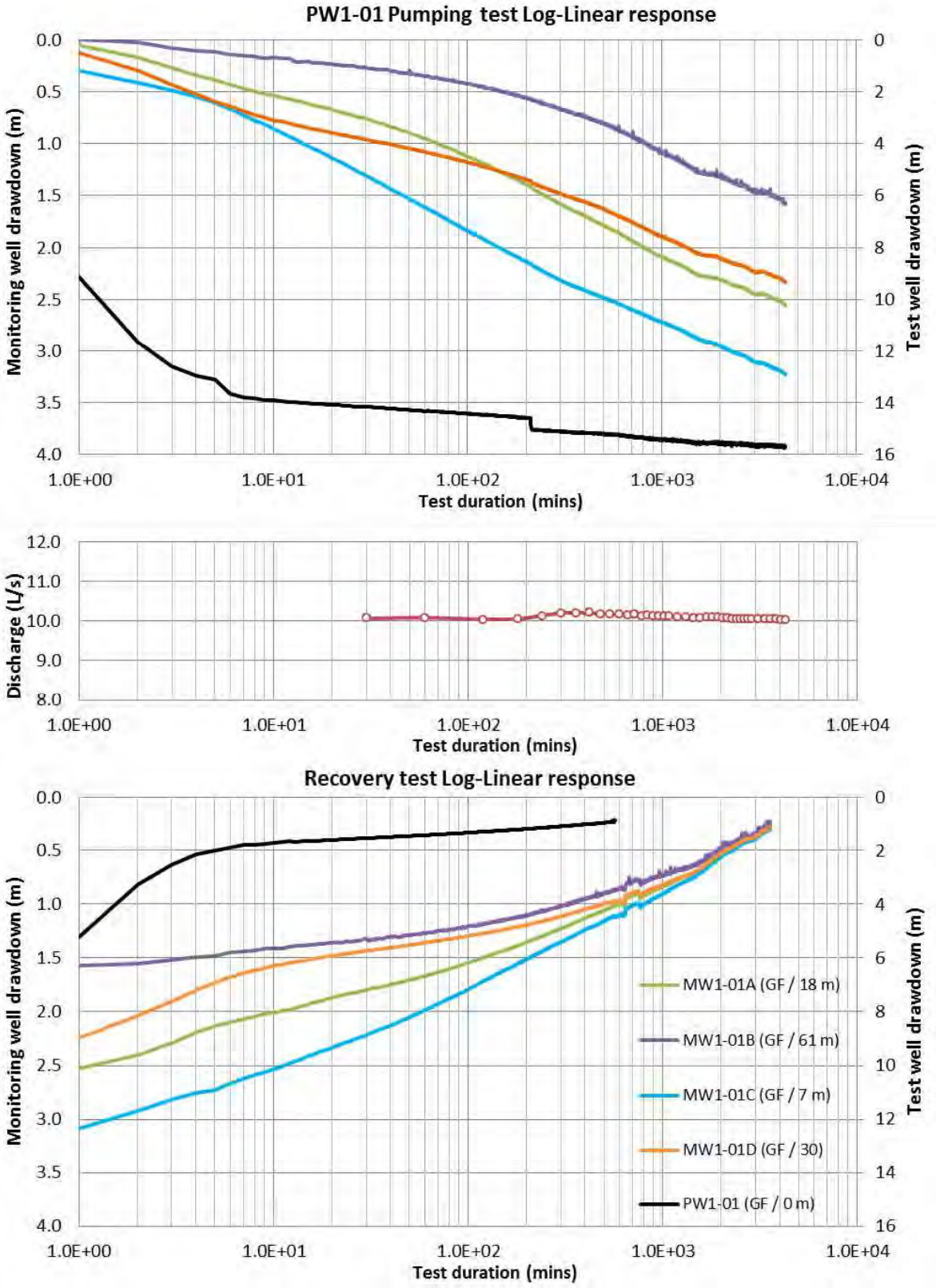

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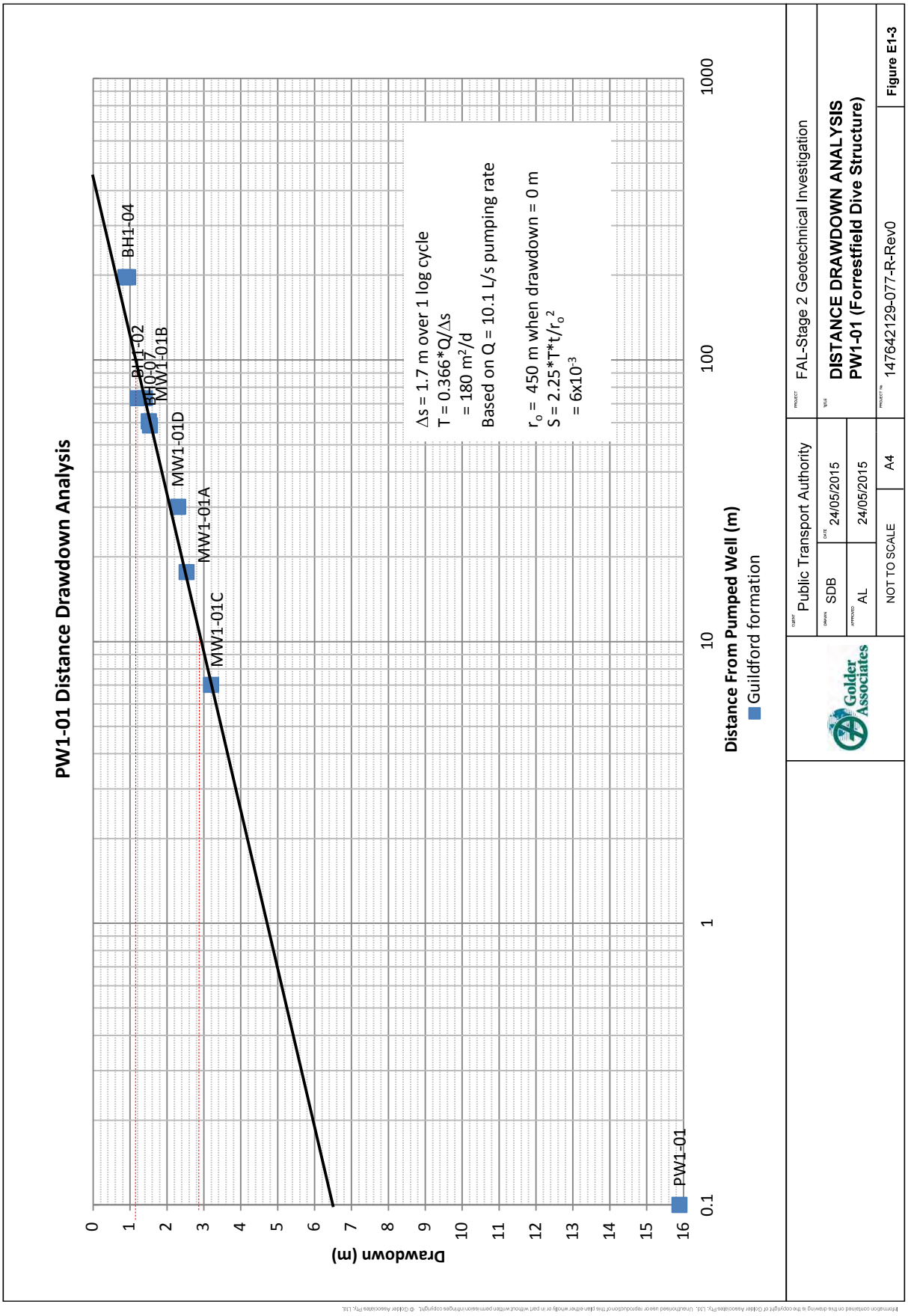
Golder Associates
Level 3, 1 Havelock St West Perth WA 6005
Ph: +618 9213 7600 Fx: +618 9213 7611

CLIENT	Public Transport Authority	PROJECT	FAL-Stage 2 Geotechnical Investigation
DRAWN	SDB	DATE 23/05/2015	TITLE **Pumping Test Data PW1-01 (Forrestfield Dive Structure)**
CHECKED	AL	DATE 23/05/2015	
SCALE	NOT TO SCALE	A4	PROJECT No 147642129-077-R-Rev0
FIGURE No	**Figure E1-2**		



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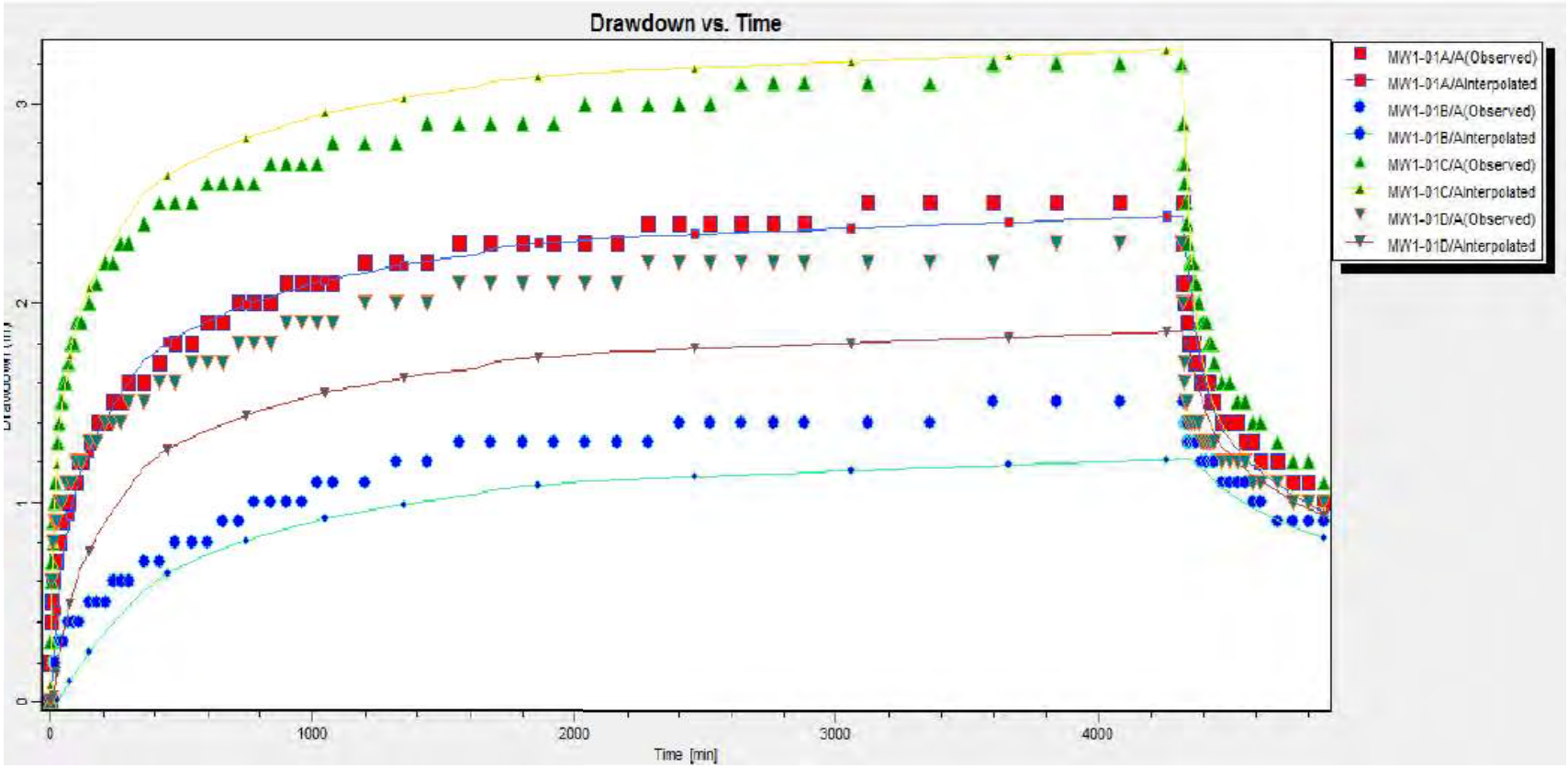
Specific Storage =	1.00E-04	1/m
Transmissivity =	104	m2/d
Hydraulic Conductivity		
K(GF)h	4	m/d
K(GF)v	4	m/d

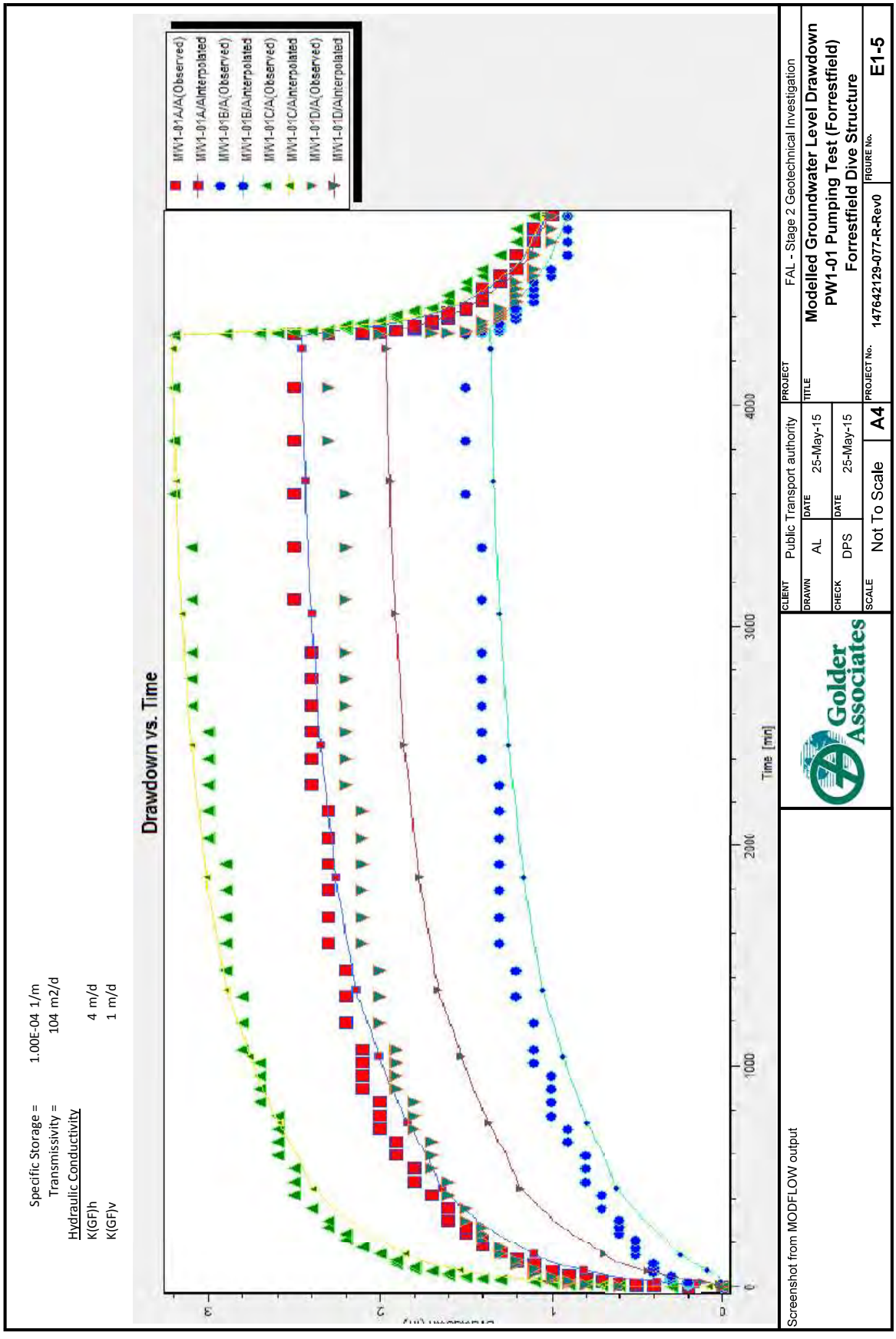


Screenshot from MODFLOW output

Golder Associates

CLIENT	Public Transport authority		PROJECT	FAL - Stage 2 Geotechnical Investigation	
DRAWN	AL	DATE 25-May-15	TITLE	Modelled Groundwater Level Drawdown PW1-01 Pumping Test (Forrestfield) Forrestfield Dive Structure	
CHECK	DPS	DATE 25-May-15			
SCALE	Not To Scale	A4	PROJECT No.	147642129-077-R-Rev0	FIGURE No. E1-4

J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Appendix E - Test Pumping Results\Attachment E1\Figure E1-4&5.xlsx


Drawdown vs. Time
Time [min]
MW1-01A/A(Observed)
MW1-01A/AInterpolated
MW1-01B/A(Observed)
MW1-01B/AInterpolated
MW1-01C/A(Observed)
MW1-01C/AInterpolated
MW1-01D/A(Observed)
MW1-01D/AInterpolated
Specific Storage = 1.00E-04 1/m
Transmissivity = 104 m2/d
Hydraulic Conductivity
K(GF)h 4 m/d
K(GF)v 1 m/d
Screenshot from MODFLOW output
Golder Associates
CLIENT Public Transport authority
DRAWN AL DATE 25-May-15
CHECK DPS DATE 25-May-15
SCALE Not To Scale A4
PROJECT FAL - Stage 2 Geotechnical Investigation
TITLE Modelled Groundwater Level Drawdown PW1-01 Pumping Test (Forrestfield) Forrestfield Dive Structure
PROJECT No. 147642129-077-R-Rev0
FIGURE No. E1-5
J:\Geo\2014\147642129 - PTA FAL Stage 2 Geotechnical Investigation\03 Correspondence & Report\077-R Groundwater Conditions Report\Rev0\Appendix E - Test Pumping Results\Attachment E1\Figure E1-4&5.xlsx

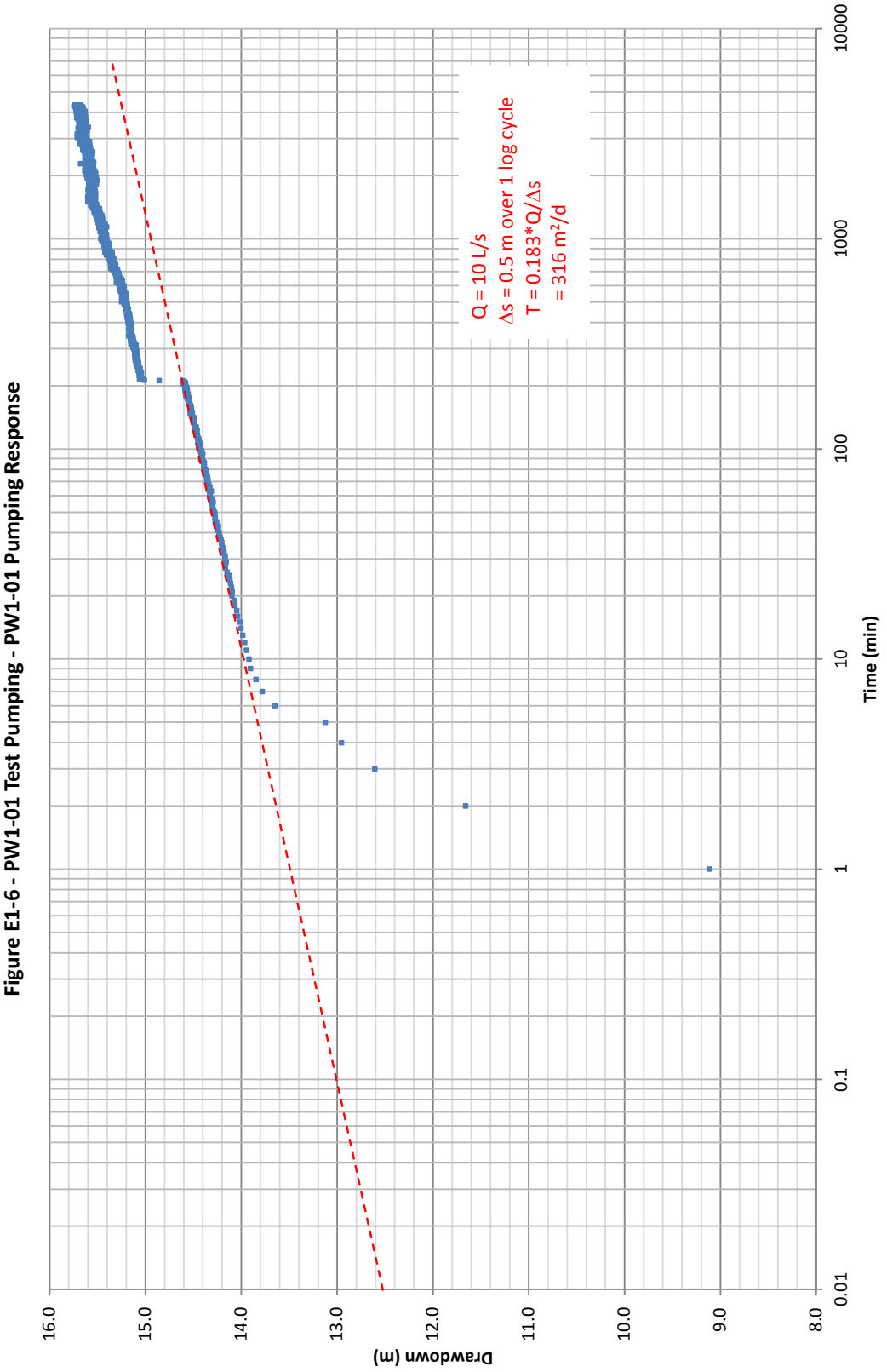


Figure E1-6 - PW1-01 Test Pumping - PW1-01 Pumping Response
Drawdown (m)
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 10 L/s
Δs = 0.5 m over 1 log cycle
T = 0.183*Q/Δs
= 316 m²/d

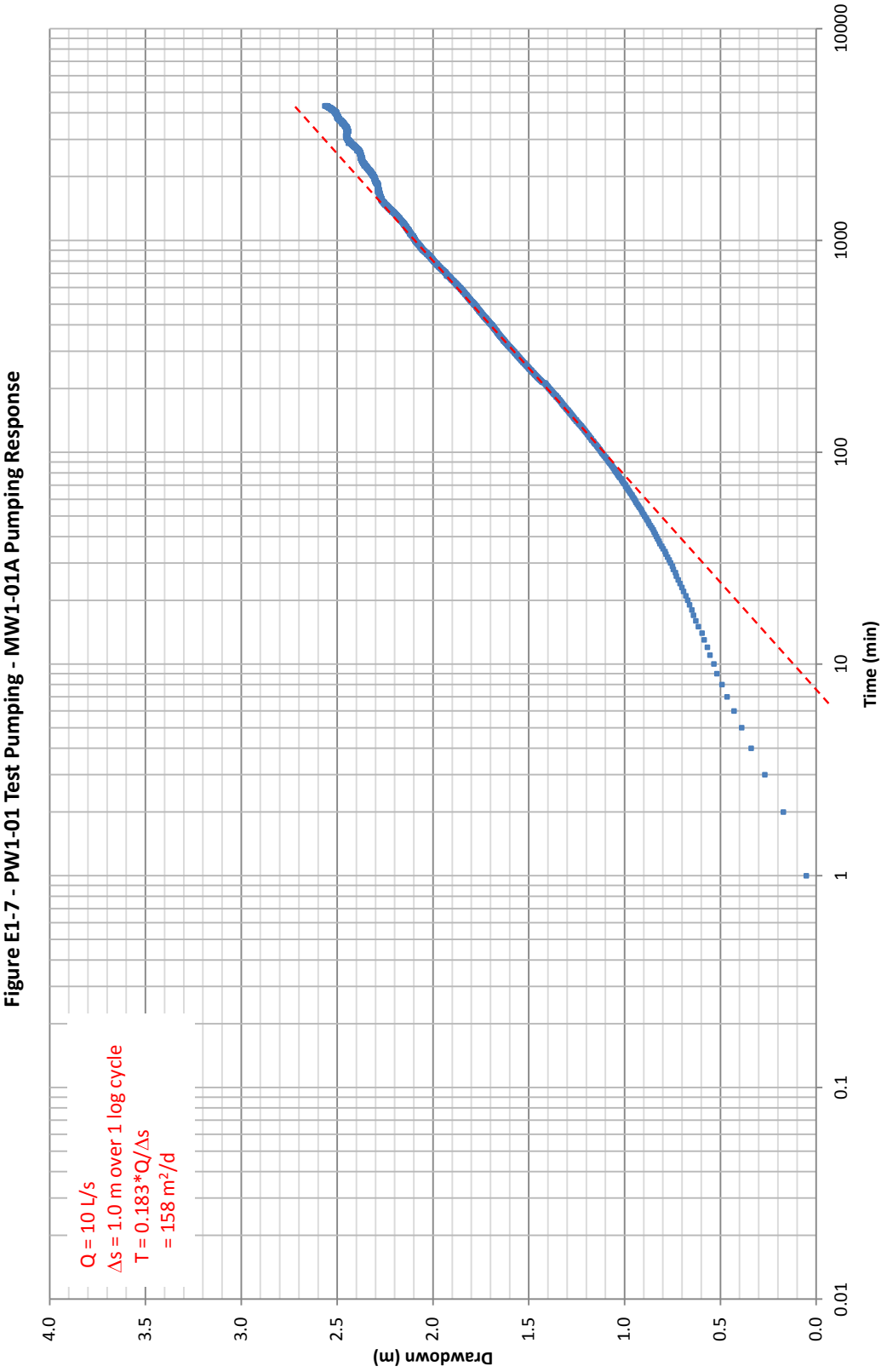


Figure E1-7 - PW1-01 Test Pumping - MW1-01A Pumping Response
Drawdown (m)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 10 L/s
Δs = 1.0 m over 1 log cycle
T = 0.183*Q/Δs
= 158 m²/d

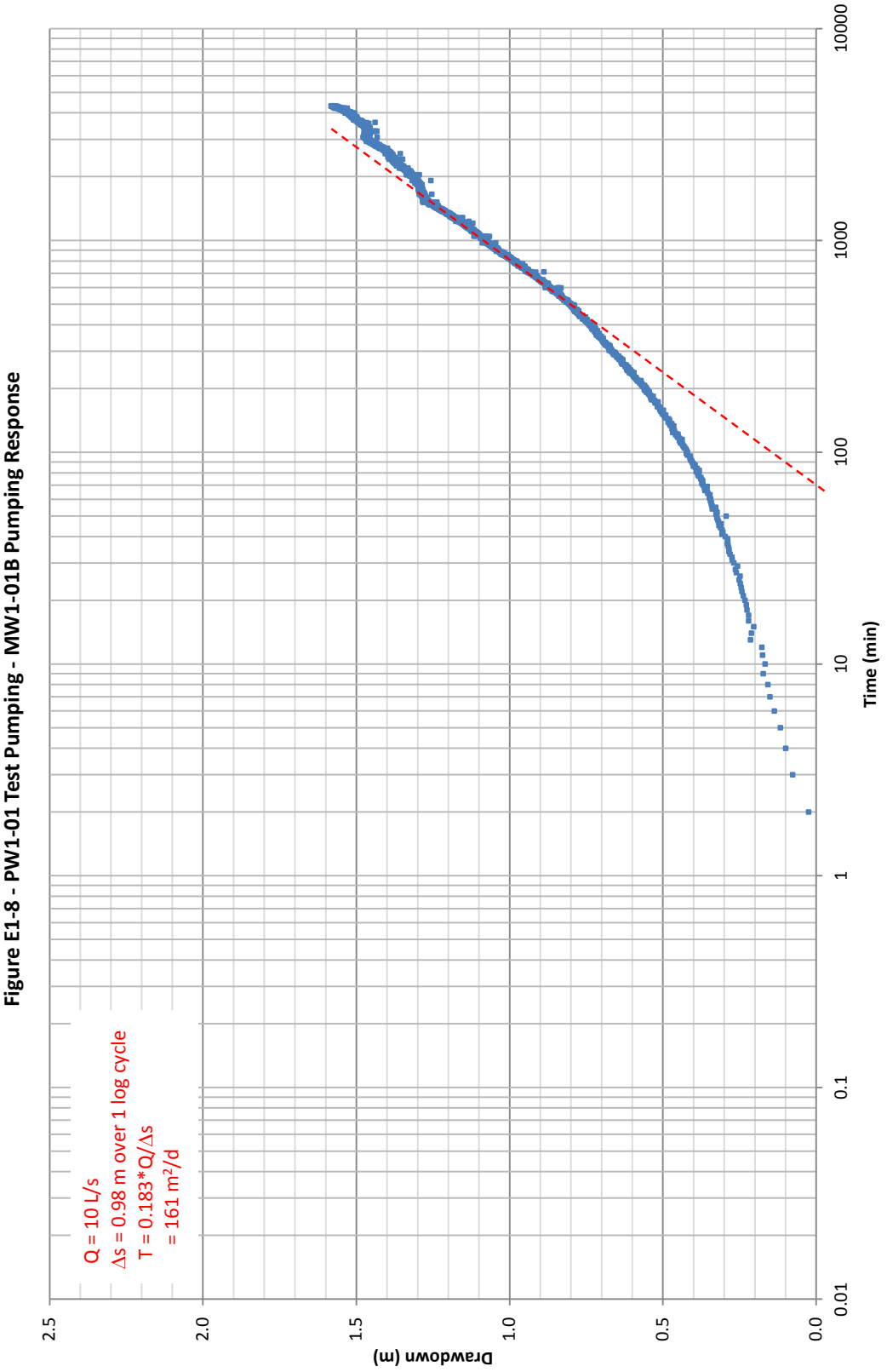


Figure E1-8 - PW1-01 Test Pumping - MW1-01B Pumping Response
Drawdown (m)
2.5
2.0
1.5
1.0
0.5
0.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 10 L/s
Δs = 0.98 m over 1 log cycle
T = 0.183*Q/Δs
= 161 m²/d

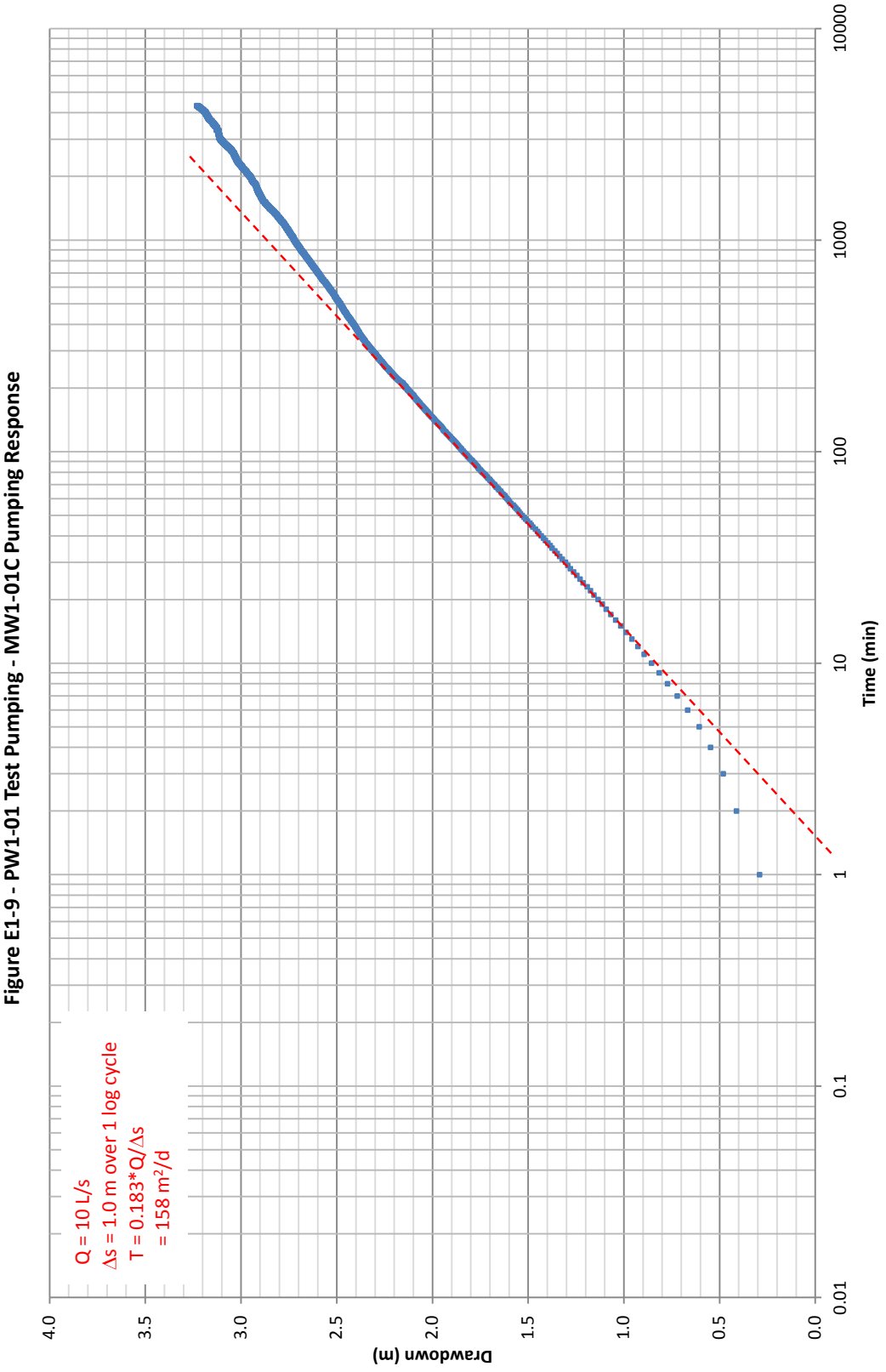


Figure E1-9 - PW1-01 Test Pumping - MW1-01C Pumping Response

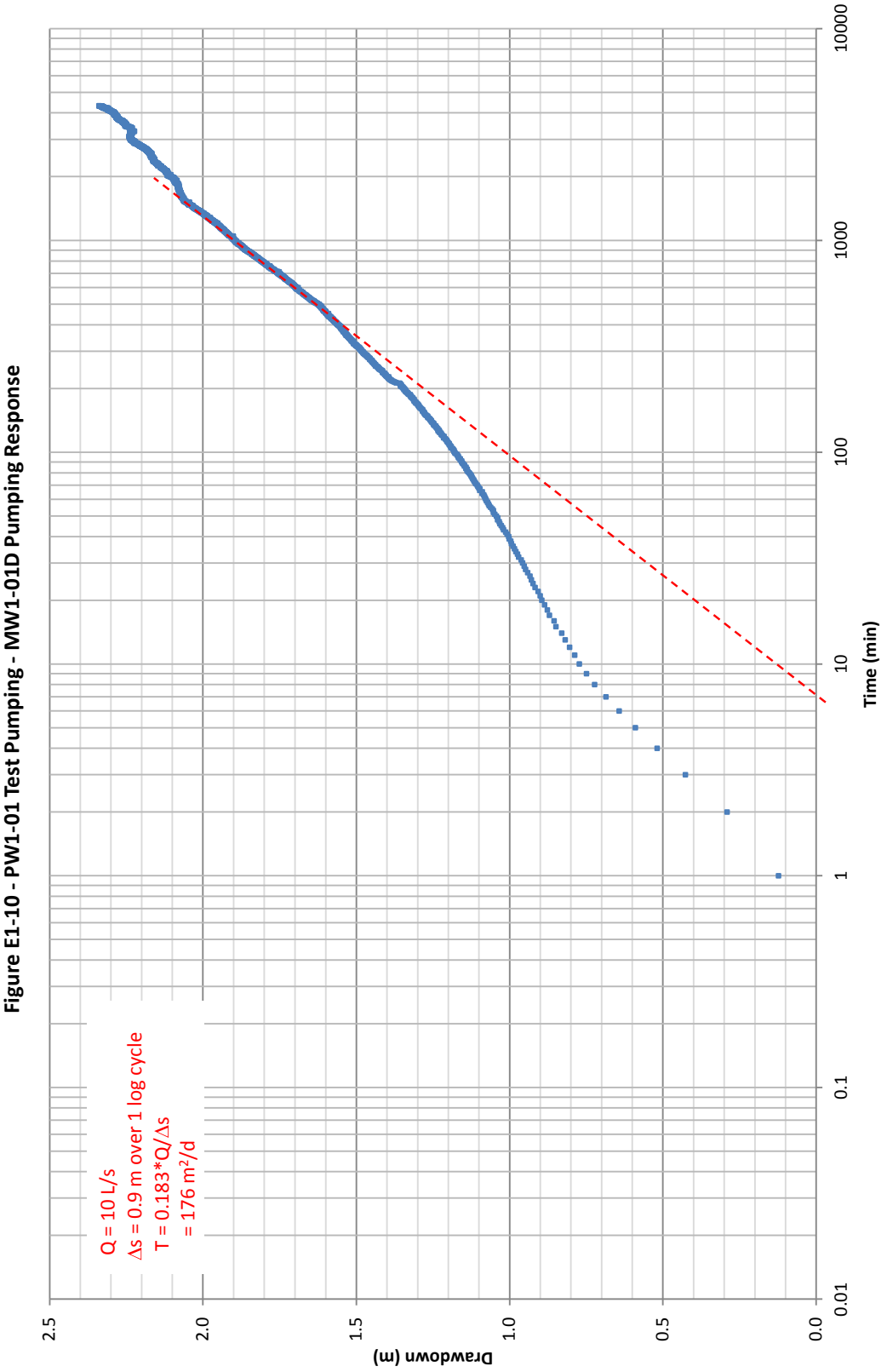


Figure E1-10 - PW1-01 Test Pumping - MW1-01D Pumping Response
Drawdown (m)
2.5
2.0
1.5
1.0
0.5
0.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 10 L/s
Δs = 0.9 m over 1 log cycle
T = 0.183*Q/Δs
= 176 m²/d

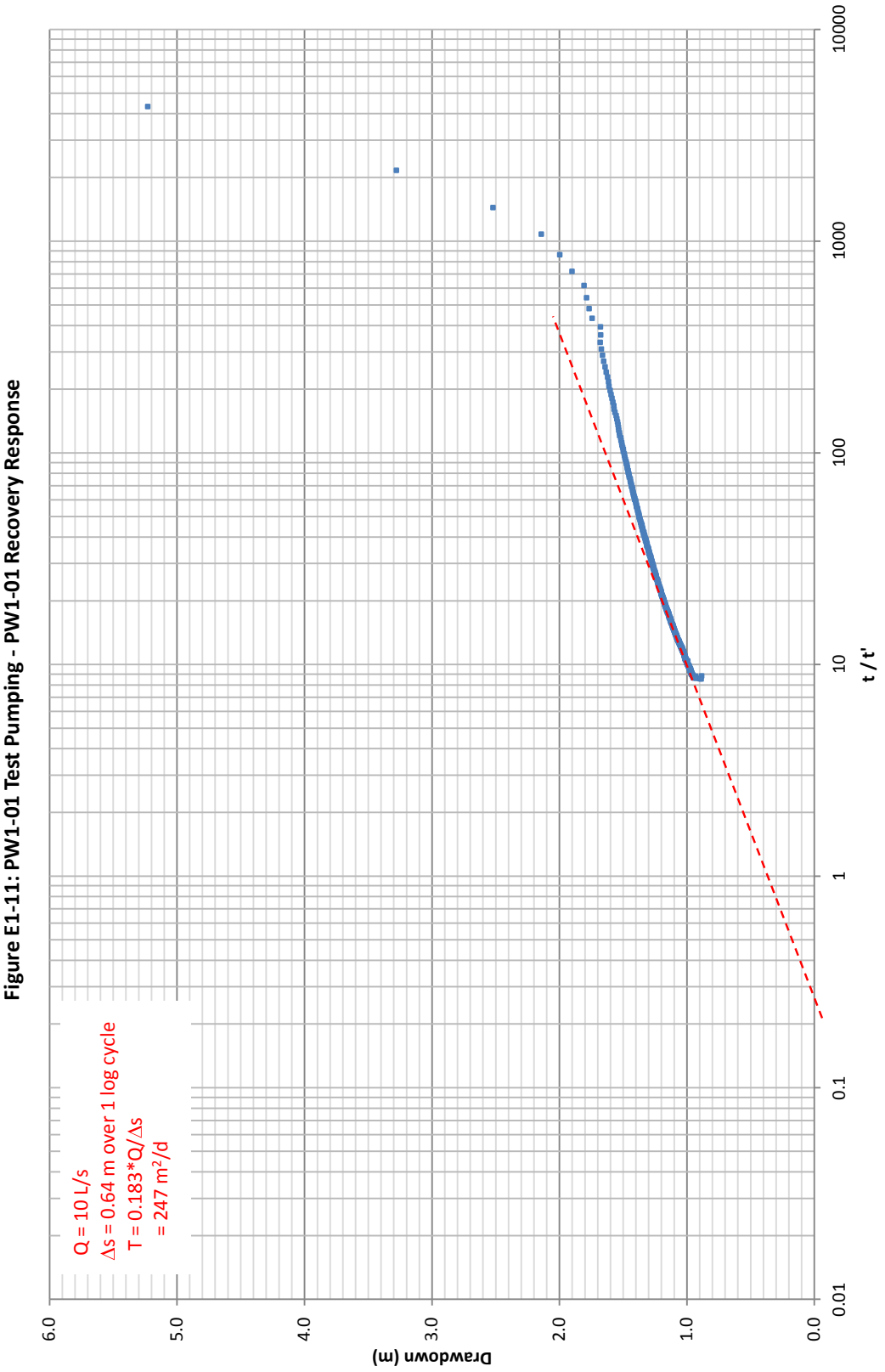


Figure E1-11: PW1-01 Test Pumping - PW1-01 Recovery Response

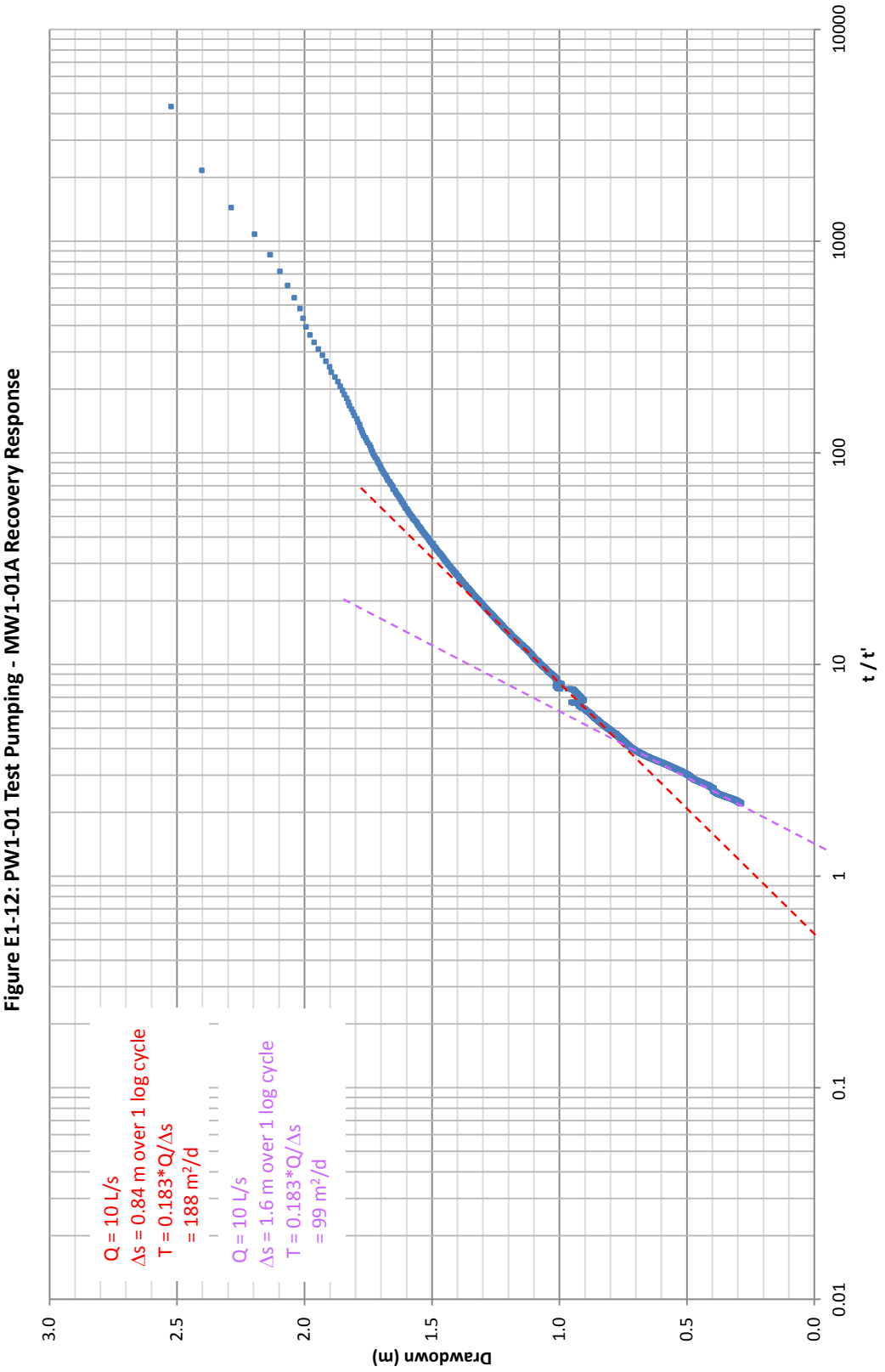


Figure E1-12: PW1-01 Test Pumping - MW1-01A Recovery Response

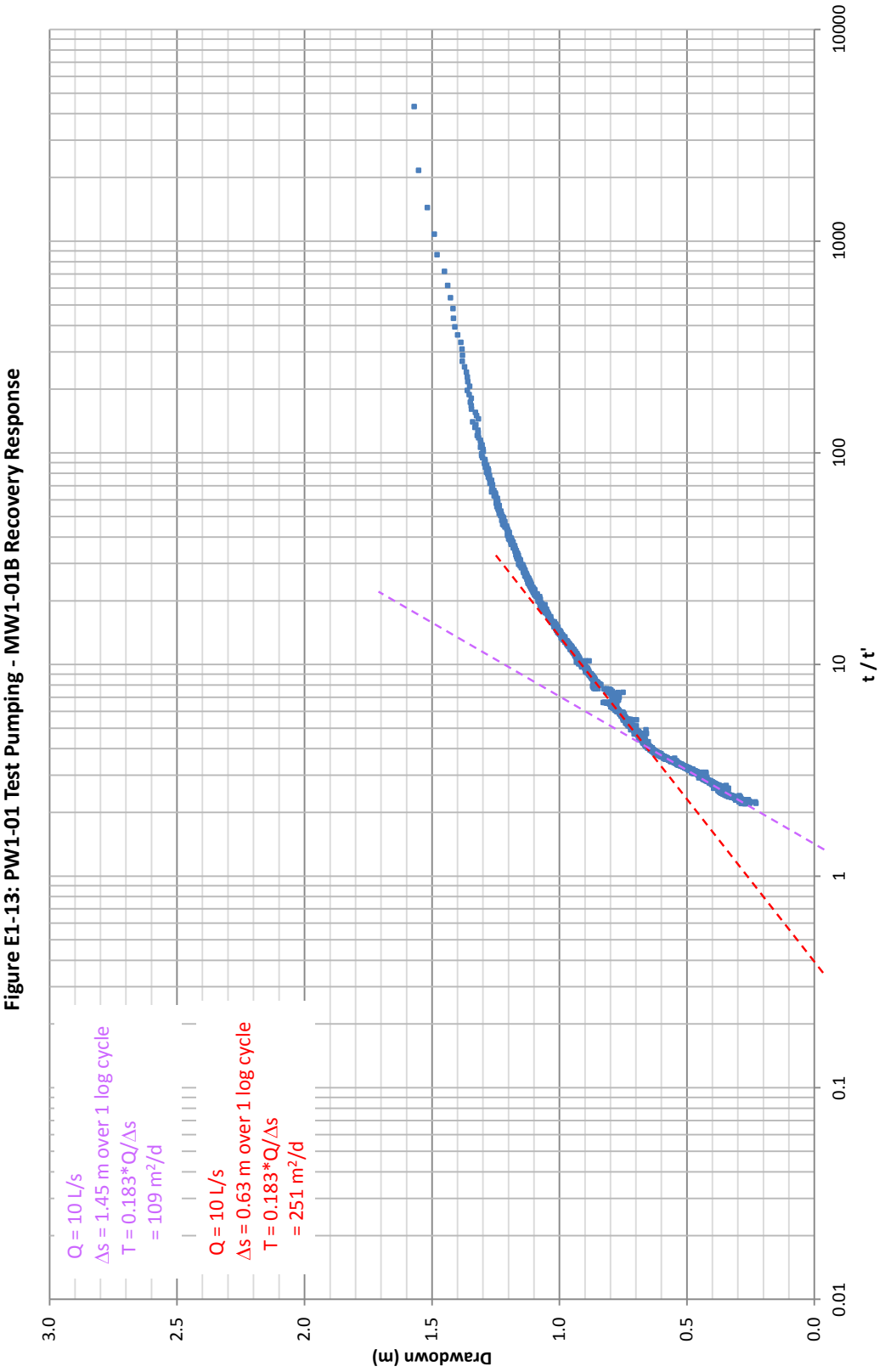


Figure E1-13: PW1-01 Test Pumping - MW1-01B Recovery Response

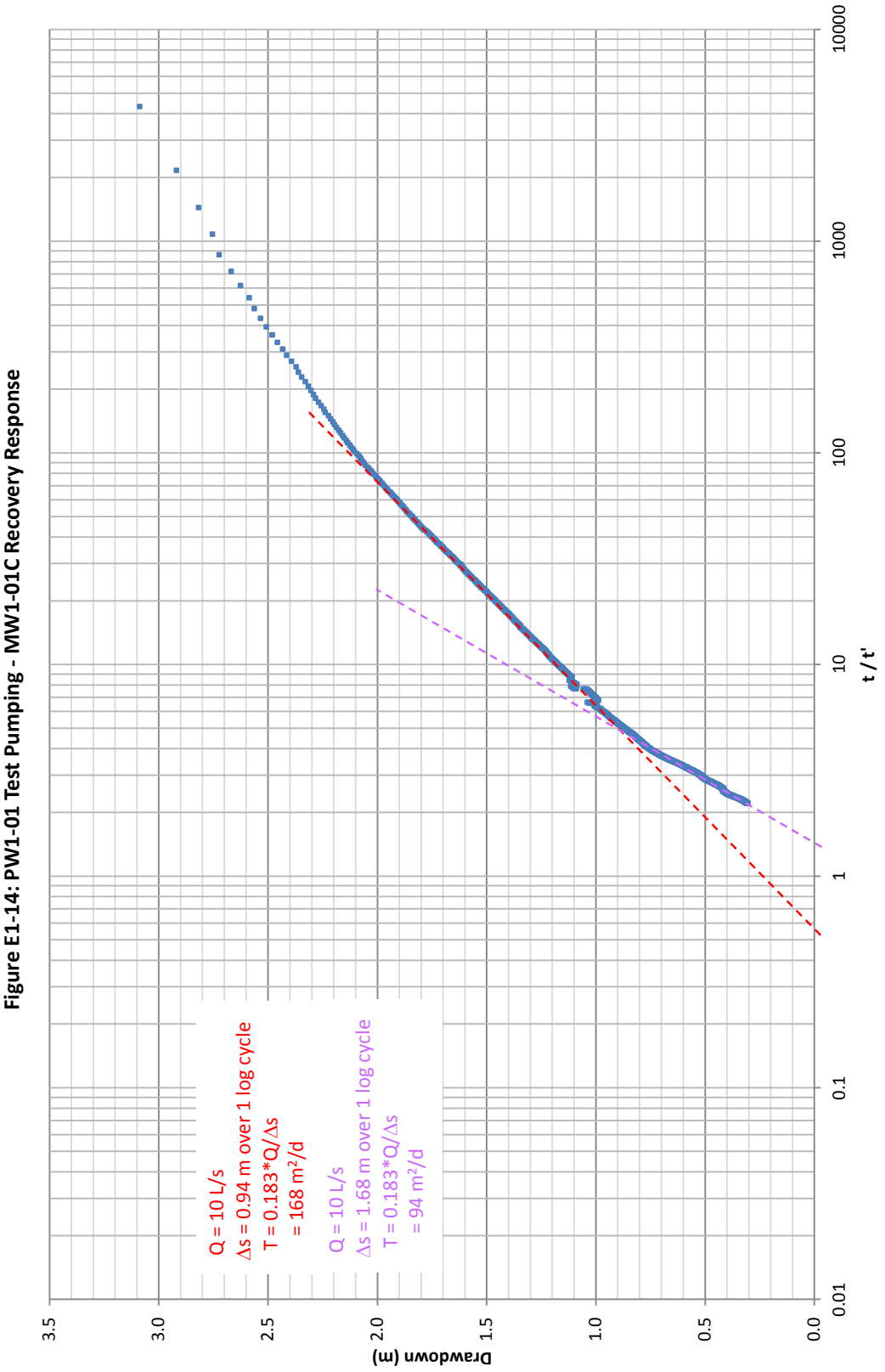


Figure E1-14: PW1-01 Test Pumping - MW1-01C Recovery Response

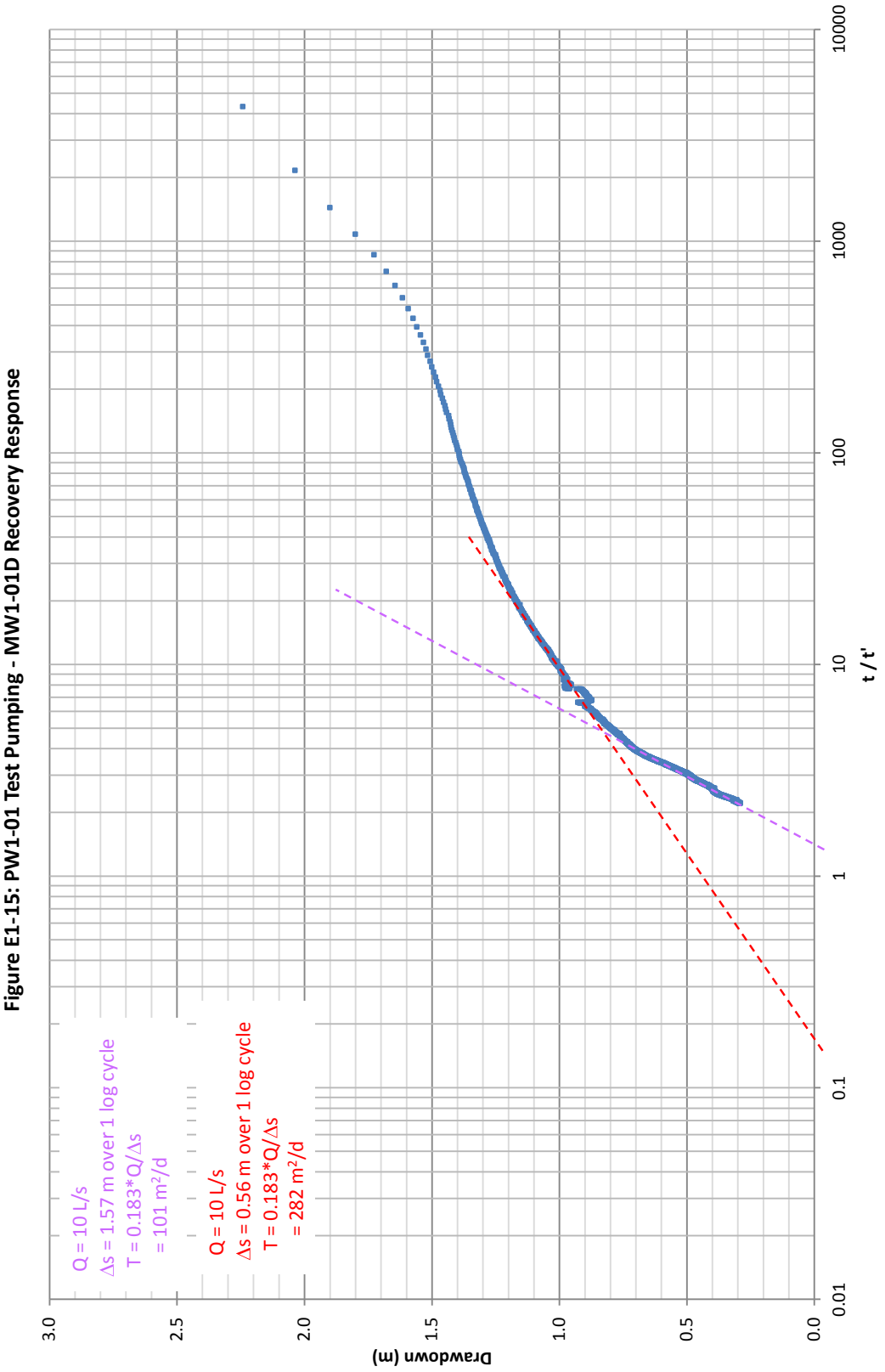


Figure E1-15: PW1-01 Test Pumping - MW1-01D Recovery Response

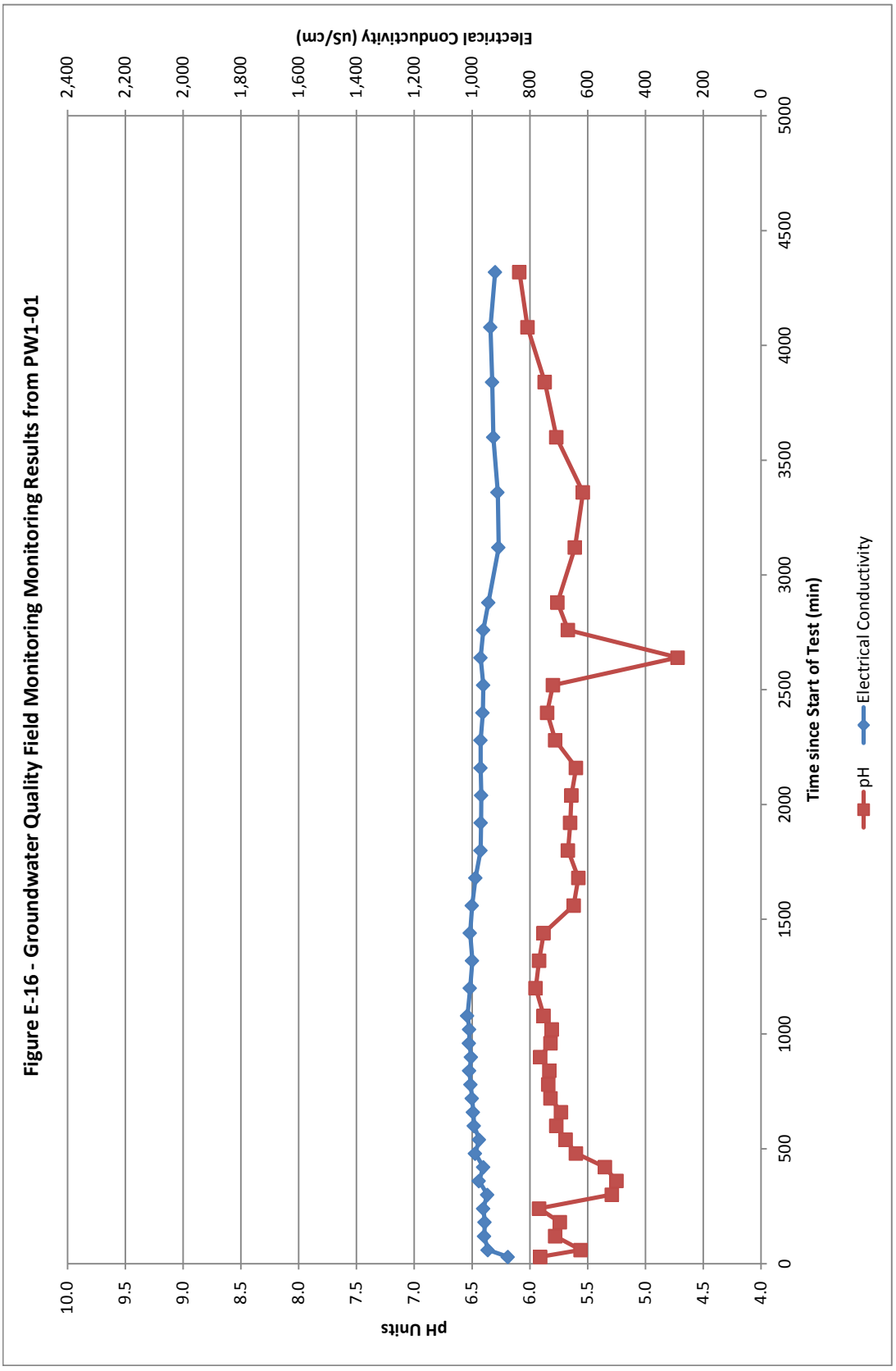
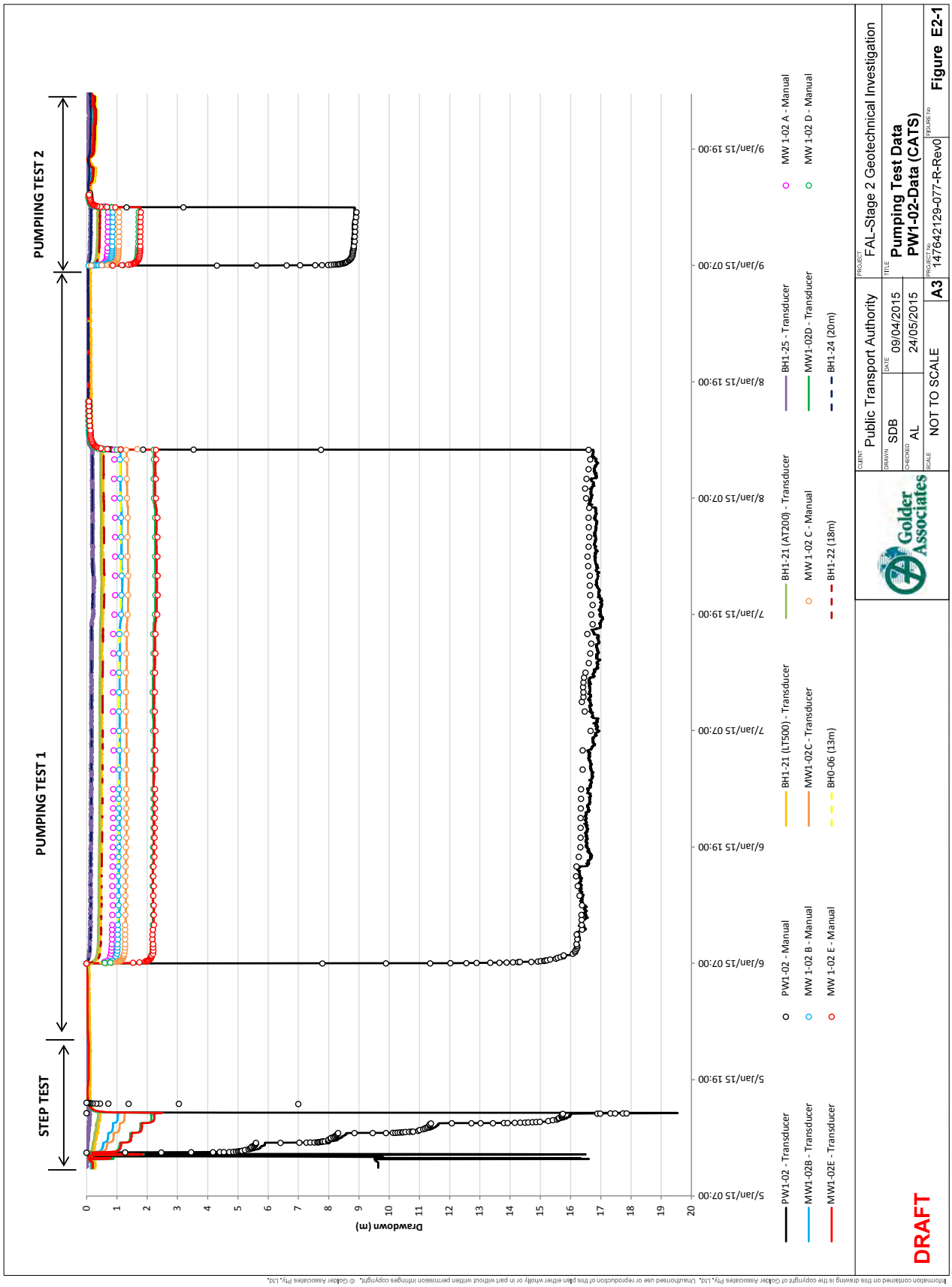


Figure E-16 - Groundwater Quality Field Monitoring Results from PW1-01
pH Units
Electrical Conductivity (uS/cm)
Time since Start of Test (min)
pH
Electrical Conductivity
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
2,400
2,200
2,000
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000



APPENDIX E
Summary of Test Pumping Analysis

ATTACHMENT E2
Constant Rate Test – Consolidated Terminal Station (PW1-02)


STEP TEST
PUMPING TEST 1
PUMPIING TEST 2
Drawdown (m)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
5/Jan/15 07:00
5/Jan/15 19:00
6/Jan/15 07:00
6/Jan/15 19:00
7/Jan/15 07:00
7/Jan/15 19:00
8/Jan/15 07:00
8/Jan/15 19:00
9/Jan/15 07:00
9/Jan/15 19:00
PW1-02 - Transducer
MW1-02B - Transducer
MW1-02E - Transducer
PW1-02 - Manual
MW 1-02 B - Manual
MW 1-02 E - Manual
BH1-21 (LT500) - Transducer
MW1-02C - Transducer
BH0-06 (13m)
BH1-21 (AT200) - Transducer
MW 1-02 C - Manual
BH1-22 (18m)
BH1-25 - Transducer
MW1-02D - Transducer
BH1-24 (20m)
MW 1-02 A - Manual
MW 1-02 D - Manual
DRAFT
Golder Associates
CLIENT
Public Transport Authority
DRAWN
SDB
DATE
09/04/2015
CHECKED
AL
24/05/2015
SCALE
NOT TO SCALE
A3
PROJECT
FAL-Stage 2 Geotechnical Investigation
TITLE
Pumping Test Data
PW1-02-Data (CATS)
PROJECT No
147642129-077-R-Rev0
FIGURE No
Figure E2-1
File Location: t:\powerpoint\golder_a3_landscape.pptm
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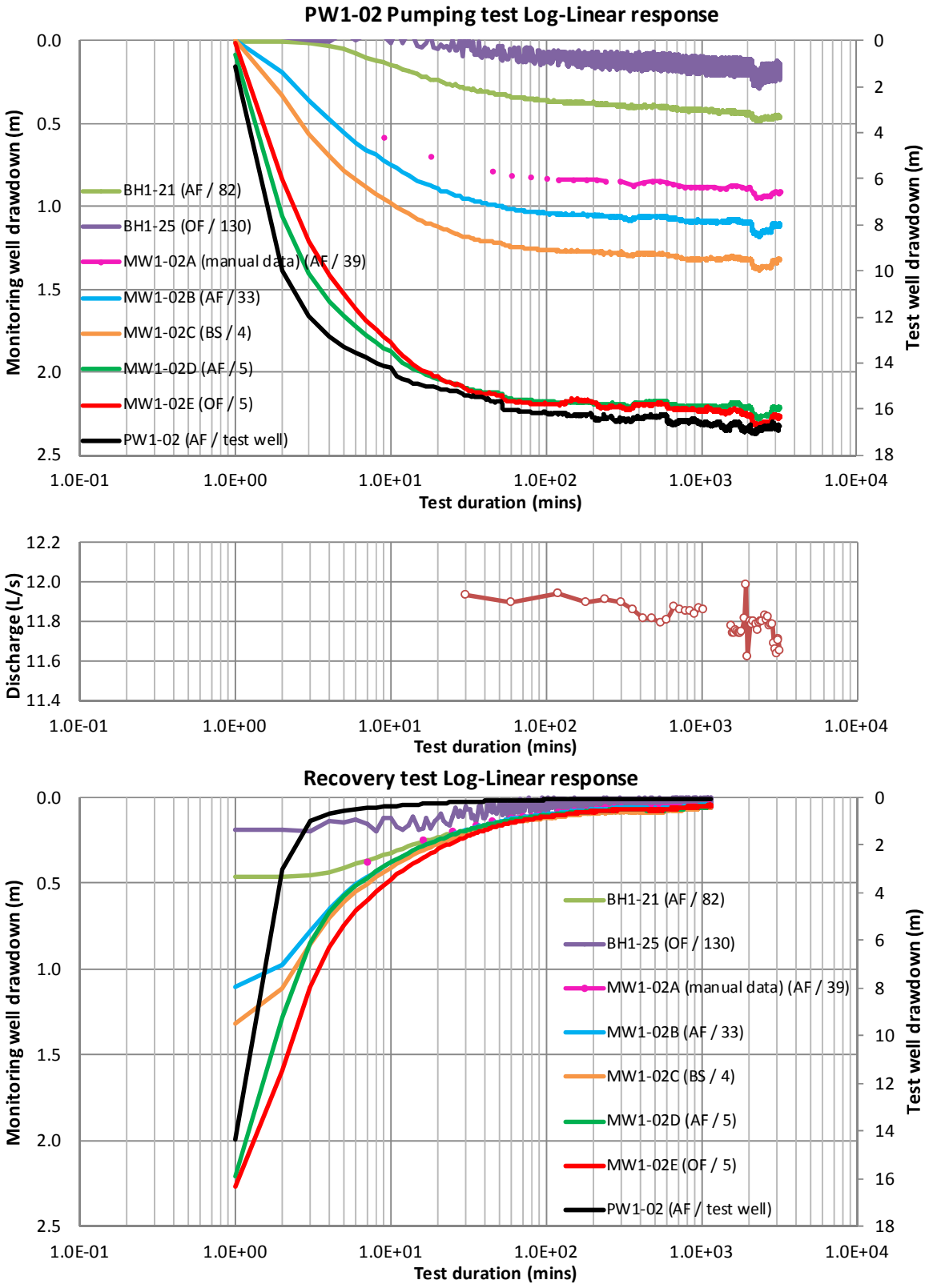

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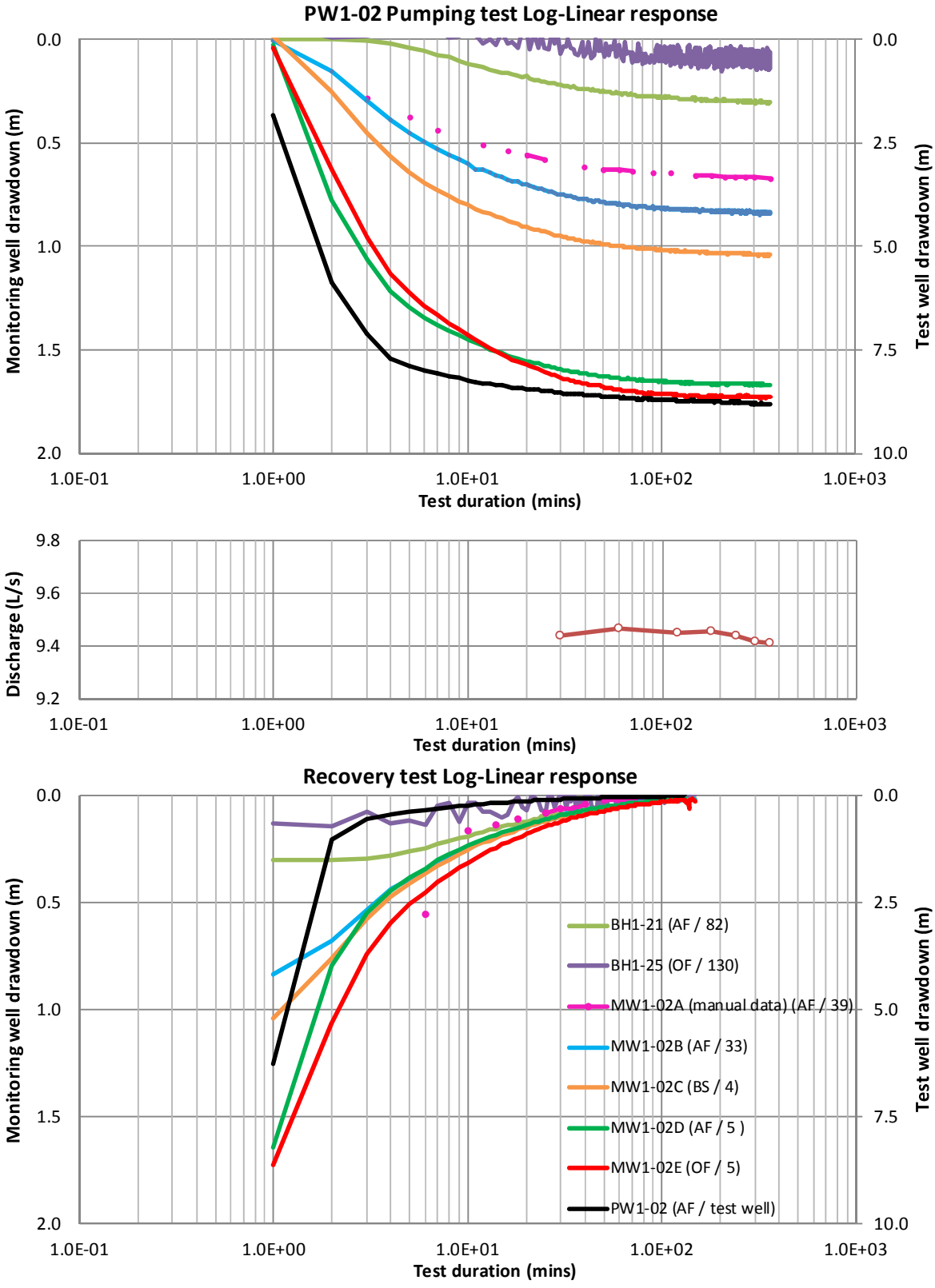
Golder Associates
Level 3, 1 Havelock St West Perth WA 6005
Ph: +618 9213 7600 Fx: +618 9213 7611

CLIENT	Public Transport Authority	PROJECT	FAL-Stage 2 Geotechnical Investigation
DRAWN	SDB	DATE 24/05/2015	TITLE Pumping Test Data PW1-02-Test 1 (Consolidated Station)
CHECKED	AL	DATE 24/05/2015	
SCALE	NOT TO SCALE	A4	PROJECT No 147642129-077-R-Rev0
FIGURE No	Figure E2-2		

File Location:



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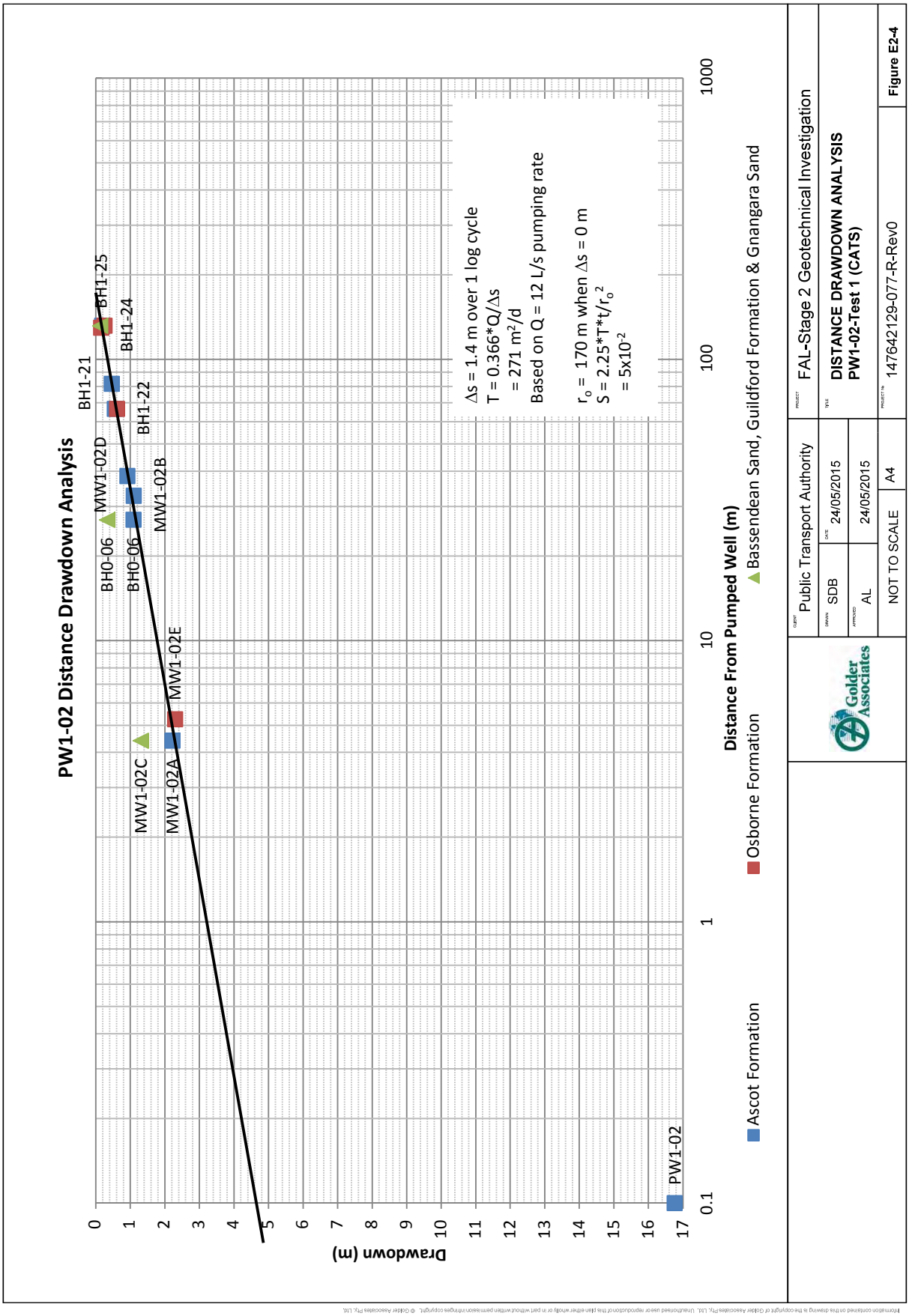
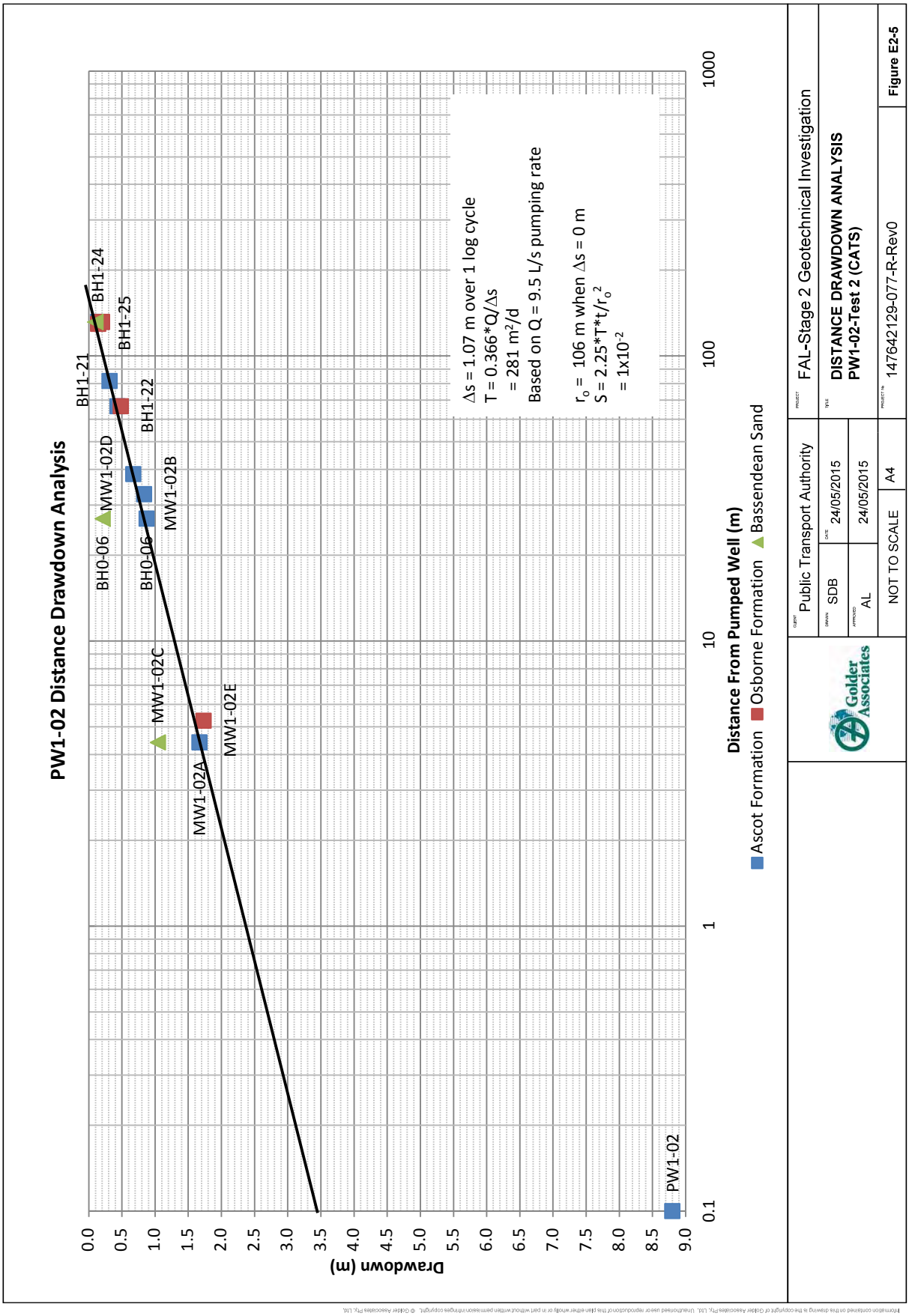
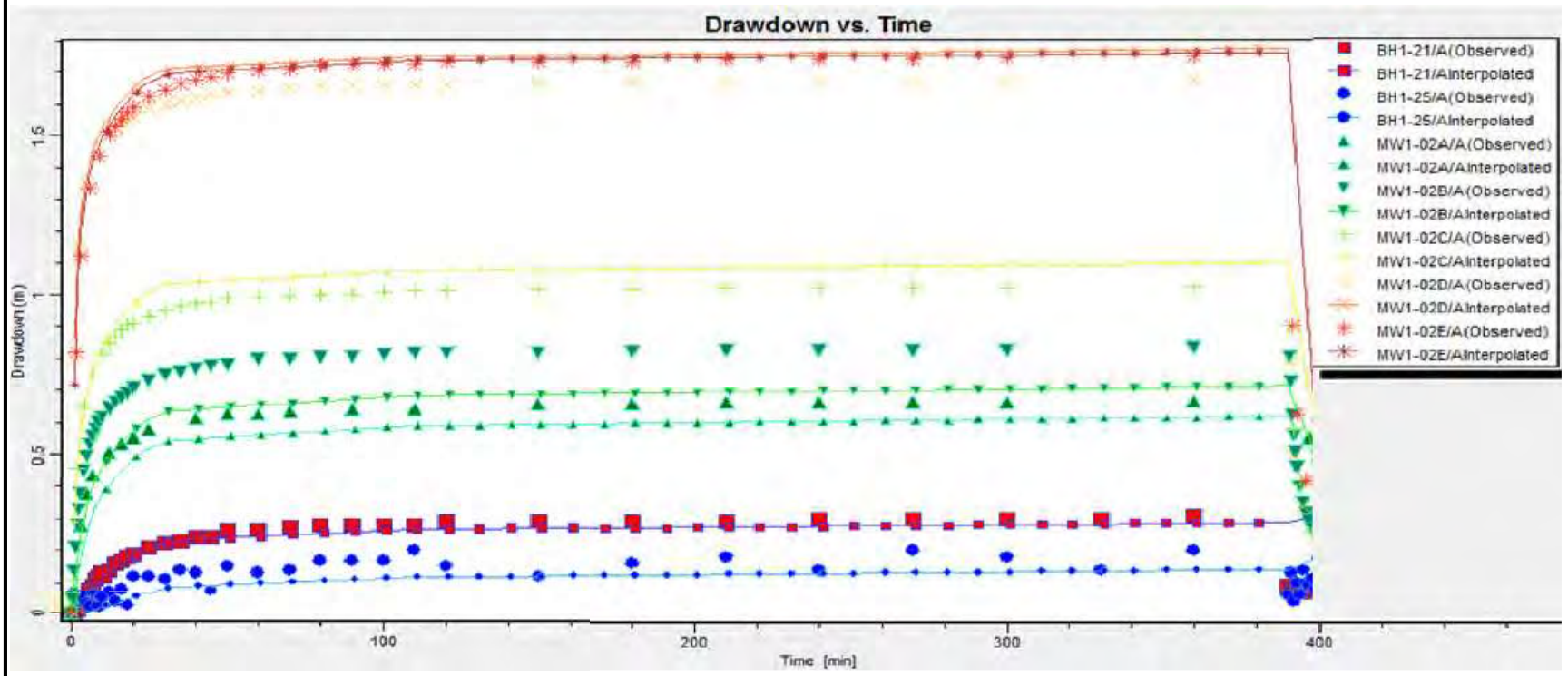
Golder Associates Level 3, 1 Havelock St West Perth WA 6005 Ph: +618 9213 7600 Fx: +618 9213 7611	CLIENT: Public Transport Authority		PROJECT: FAL-Stage 2 Geotechnical Investigation
	DRAWN: SDB	DATE: 24/05/2015	TITLE: **Pumping Test Data**
	CHECKED: AL	DATE: 24/05/2015	**PW1-02-Test 2 (Consolidated Station)**
	SCALE: NOT TO SCALE	A4	PROJECT No: 147642129-077-R-Rev0
			FIGURE No: **Figure E2-3**

File Location:


PW1-02 Distance Drawdown Analysis
Drawdown (m)
Distance From Pumped Well (m)
Ascot Formation
Osborne Formation
Bassendean Sand, Guildford Formation & Gnangara Sand
PW1-02
MW1-02A
MW1-02C
MW1-02E
BH0-06
BH0-06
MW1-02B
BH1-21
MW1-02D
BH1-22
BH1-24
BH1-25
Δs = 1.4 m over 1 log cycle
T = 0.366*Q/Δs
= 271 m²/d
Based on Q = 12 L/s pumping rate
r_o = 170 m when Δs = 0 m
S = 2.25*T*t/r_o²
= 5x10⁻²
Public Transport Authority
FAL-Stage 2 Geotechnical Investigation
SDB
24/05/2015
AL
24/05/2015
DISTANCE DRAWDOWN ANALYSIS
PW1-02-Test 1 (CATS)
NOT TO SCALE
A4
147642129-077-R-Rev0
Figure E2-4


PW1-02 Distance Drawdown Analysis
Drawdown (m)
Distance From Pumped Well (m)
Ascot Formation
Osborne Formation
Bassendean Sand
PW1-02
MW1-02A
MW1-02C
MW1-02E
BH0-06
MW1-02D
BH1-21
BH1-22
MW1-02B
BH0-06
BH1-24
BH1-25
Δs = 1.07 m over 1 log cycle
T = 0.366*Q/Δs
= 281 m²/d
Based on Q = 9.5 L/s pumping rate
r_o = 106 m when Δs = 0 m
S = 2.25*T*t/r_o²
= 1x10⁻²
Golder Associates
Public Transport Authority
SDB
24/05/2015
AL
24/05/2015
NOT TO SCALE
A4
FAL-Stage 2 Geotechnical Investigation
DISTANCE DRAWDOWN ANALYSIS
PW1-02-Test 2 (CATS)
147642129-077-R-Rev0
Figure E2-5

Specific Storage =	2.50E-05	1/m
Transmissivity =	274	m2/d
Hydraulic Conductivity		
K(BS,GF,GS)h	5	m/d
K(BS,GF,GS)v	0.5	m/d
K(AF)h	14	m/d
K(AF)v	1.4	m/d
K(OF)h	3	m/d
K(OF)v	0.3	m/d



Screenshot from MODFLOW output

Note: Other combinations in hydraulic conductivity and storage may also be possible

Golder Associates

CLIENT	Public Transport authority			PROJECT	FAL - Stage 2 Geotechnical Investigation
DRAWN	AL	DATE	24-May-15	TITLE	**Analysis of Pumping Test 2 using Numerical Groundwater Model Airport West Station**
CHECK	DPS	DATE	24-May-15		
SCALE	Not To Scale	**A4**		PROJECT No.	147642129-077-R-Rev0
				FIGURE No.	**E2-6**

C:\Users\alundorf\Desktop\FAL\APPENDIX E\Appendix E - Test Pumping Results\Attachment E2\Figure E2-6.xlsx

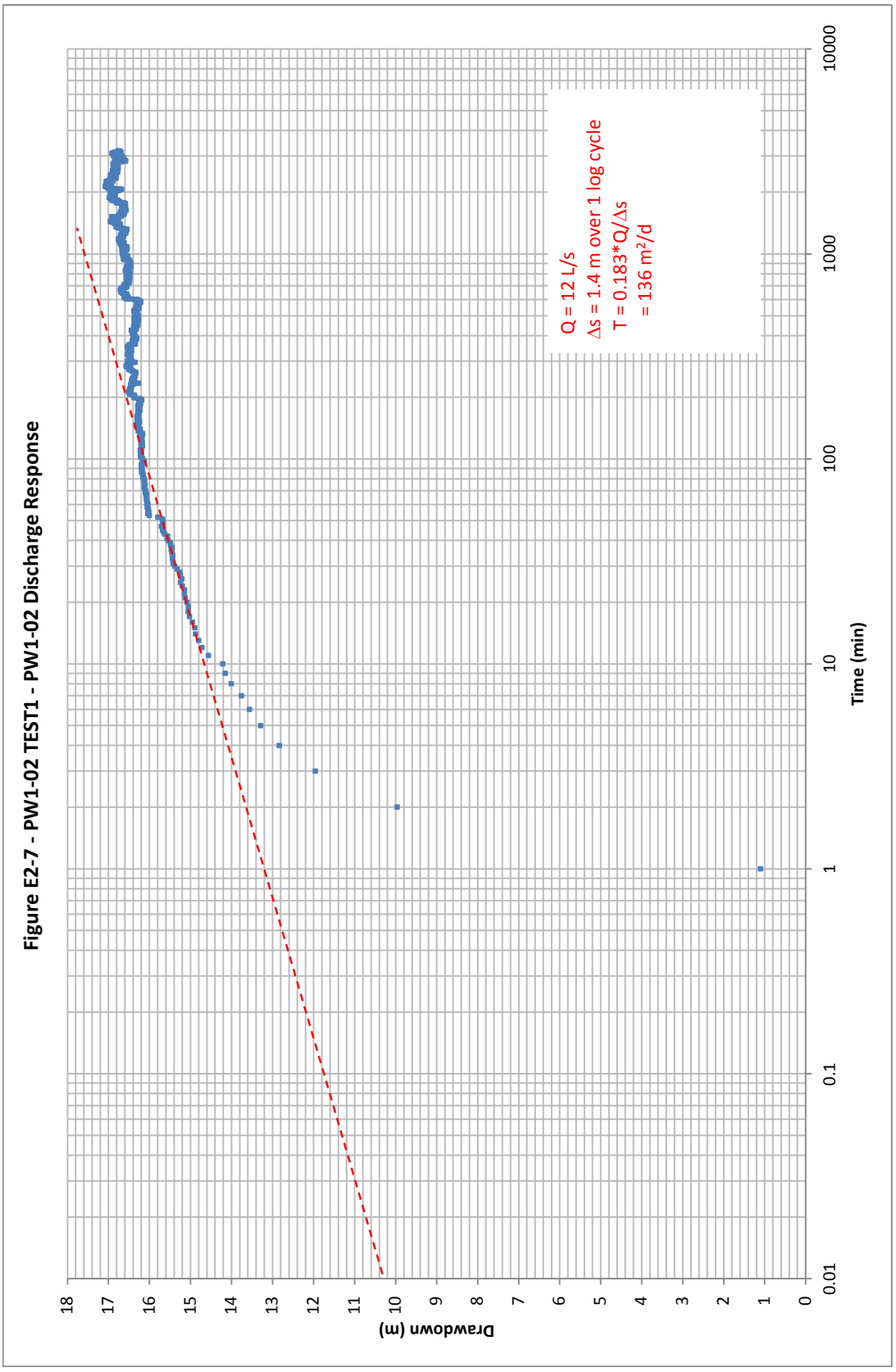


Figure E2-7 - PW1-02 TEST1 - PW1-02 Discharge Response
Drawdown (m)
Time (min)
Q = 12 L/s
Δs = 1.4 m over 1 log cycle
T = 0.183*Q/Δs
= 136 m²/d

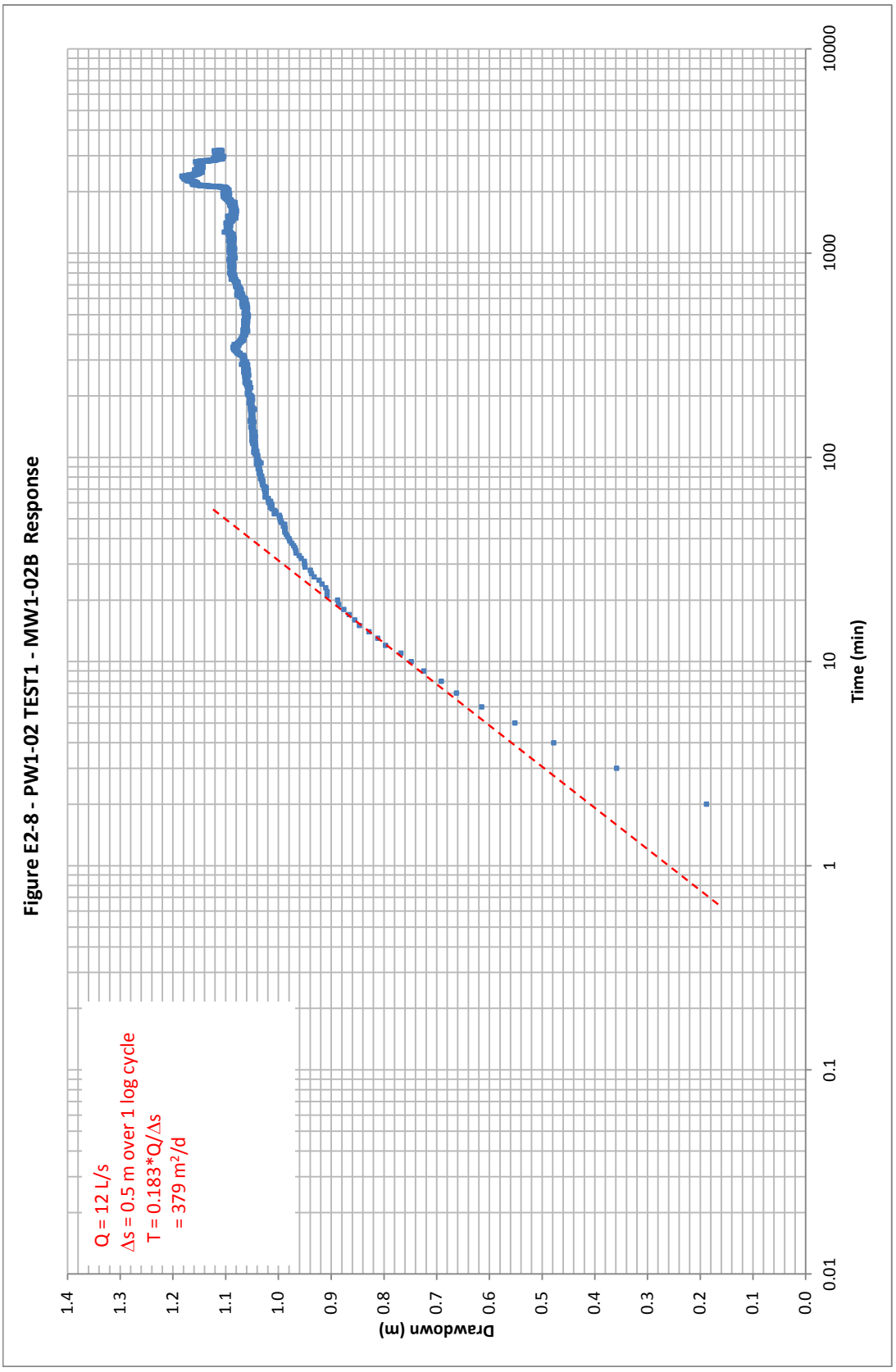


Figure E2-8 - PW1-02 TEST1 - MW1-02B Response
Drawdown (m)
Time (min)
Q = 12 L/s
Δs = 0.5 m over 1 log cycle
T = 0.183*Q/Δs
= 379 m²/d

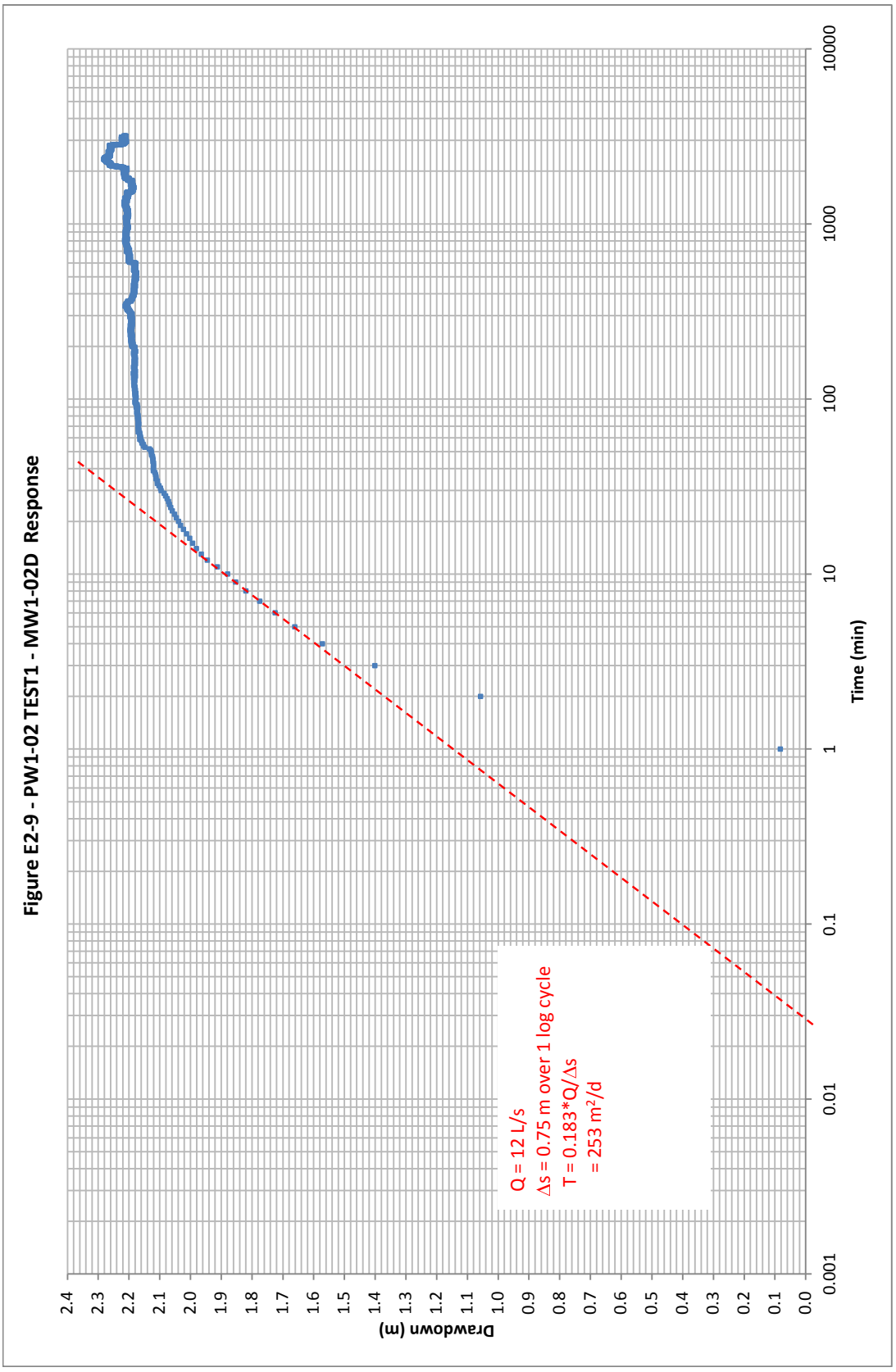


Figure E2-9 - PW1-02 TEST1 - MW1-02D Response
Drawdown (m)
Time (min)
Q = 12 L/s
Δs = 0.75 m over 1 log cycle
T = 0.183*Q/Δs
= 253 m²/d

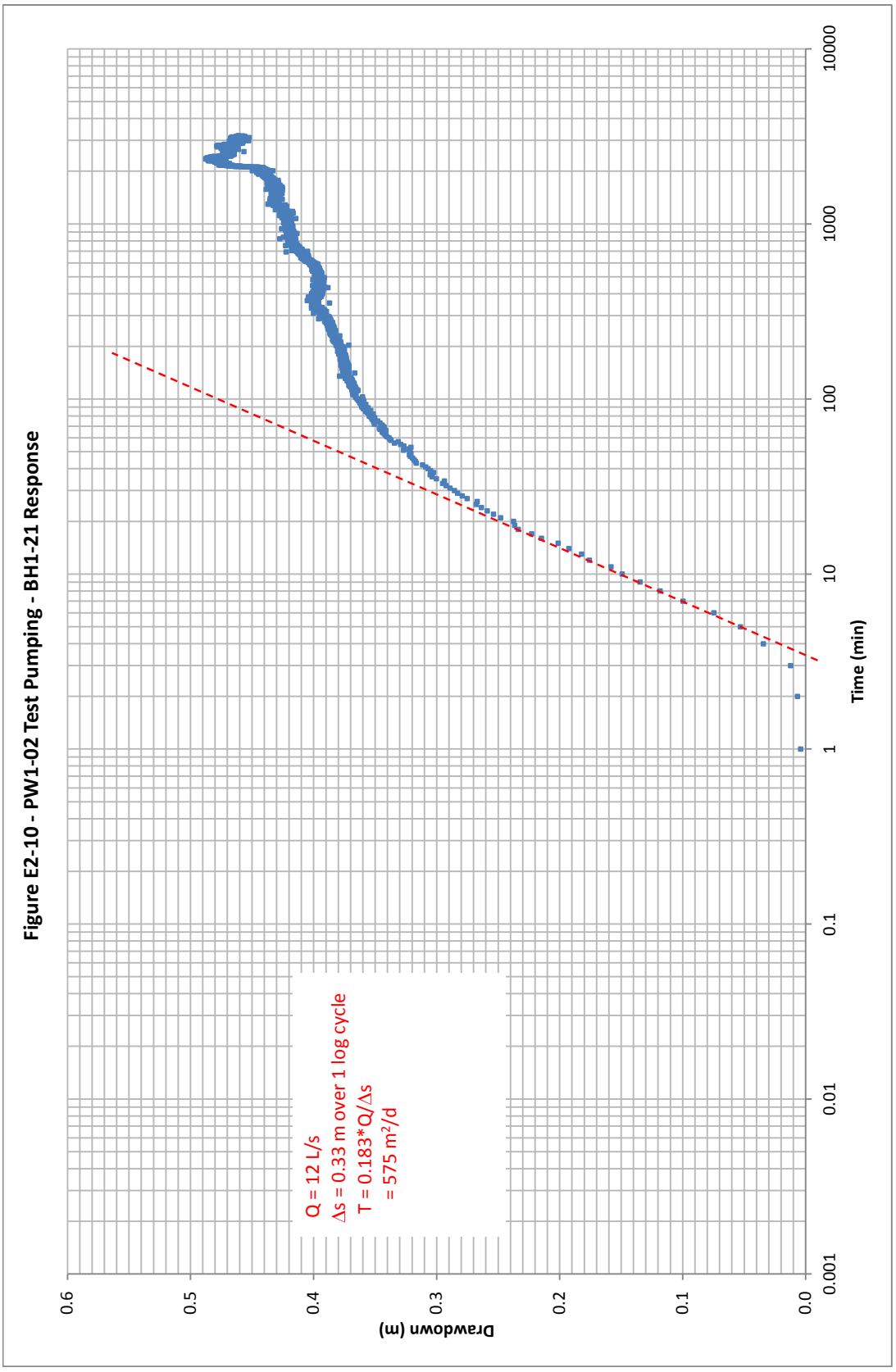


Figure E2-10 - PW1-02 Test Pumping - BH1-21 Response
Drawdown (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Time (min)
0.001
0.01
0.1
1
10
100
1000
10000
Q = 12 L/s
Δs = 0.33 m over 1 log cycle
T = 0.183*Q/Δs
= 575 m²/d

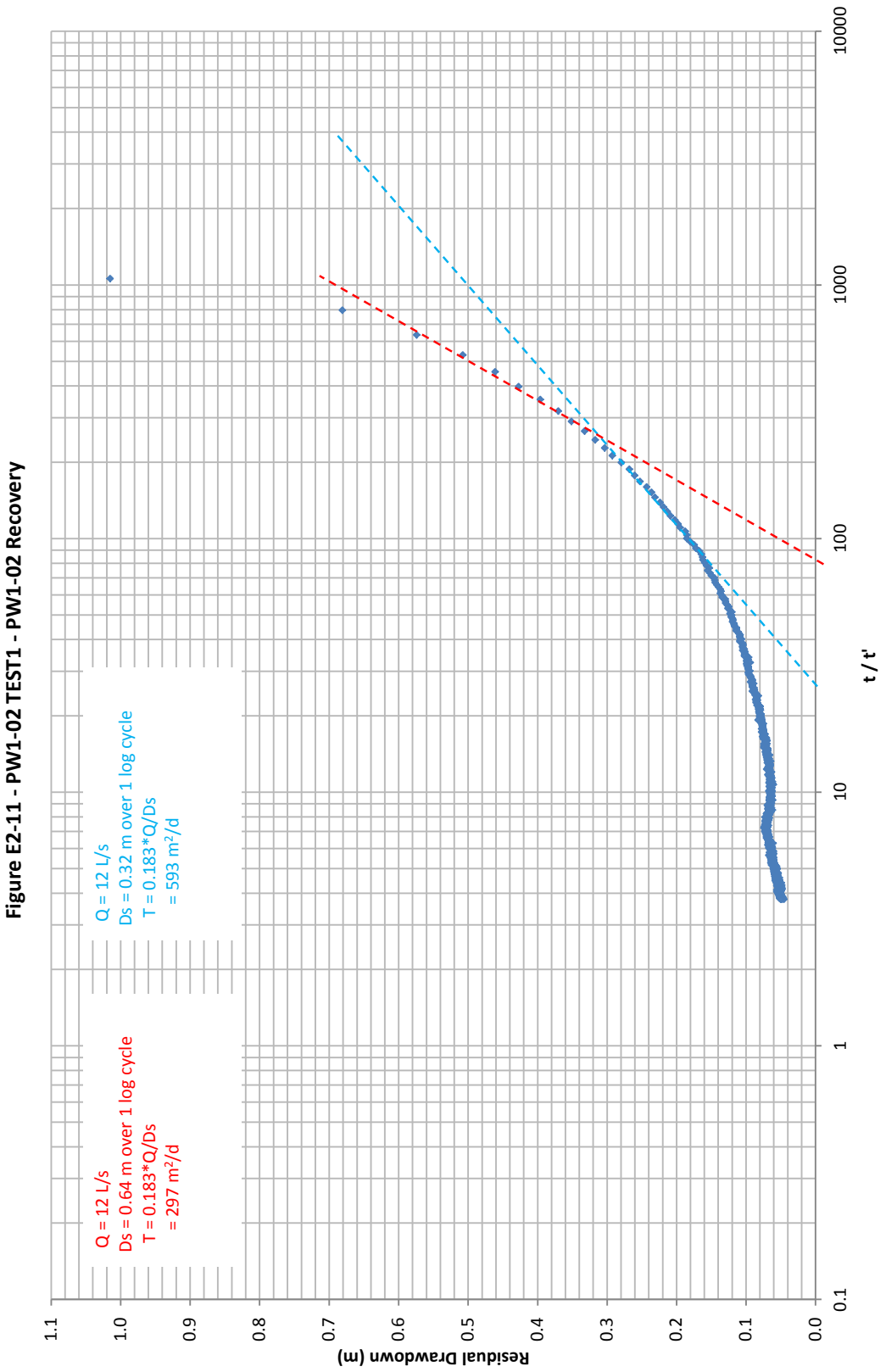


Figure E2-11 - PW1-02 TEST1 - PW1-02 Recovery

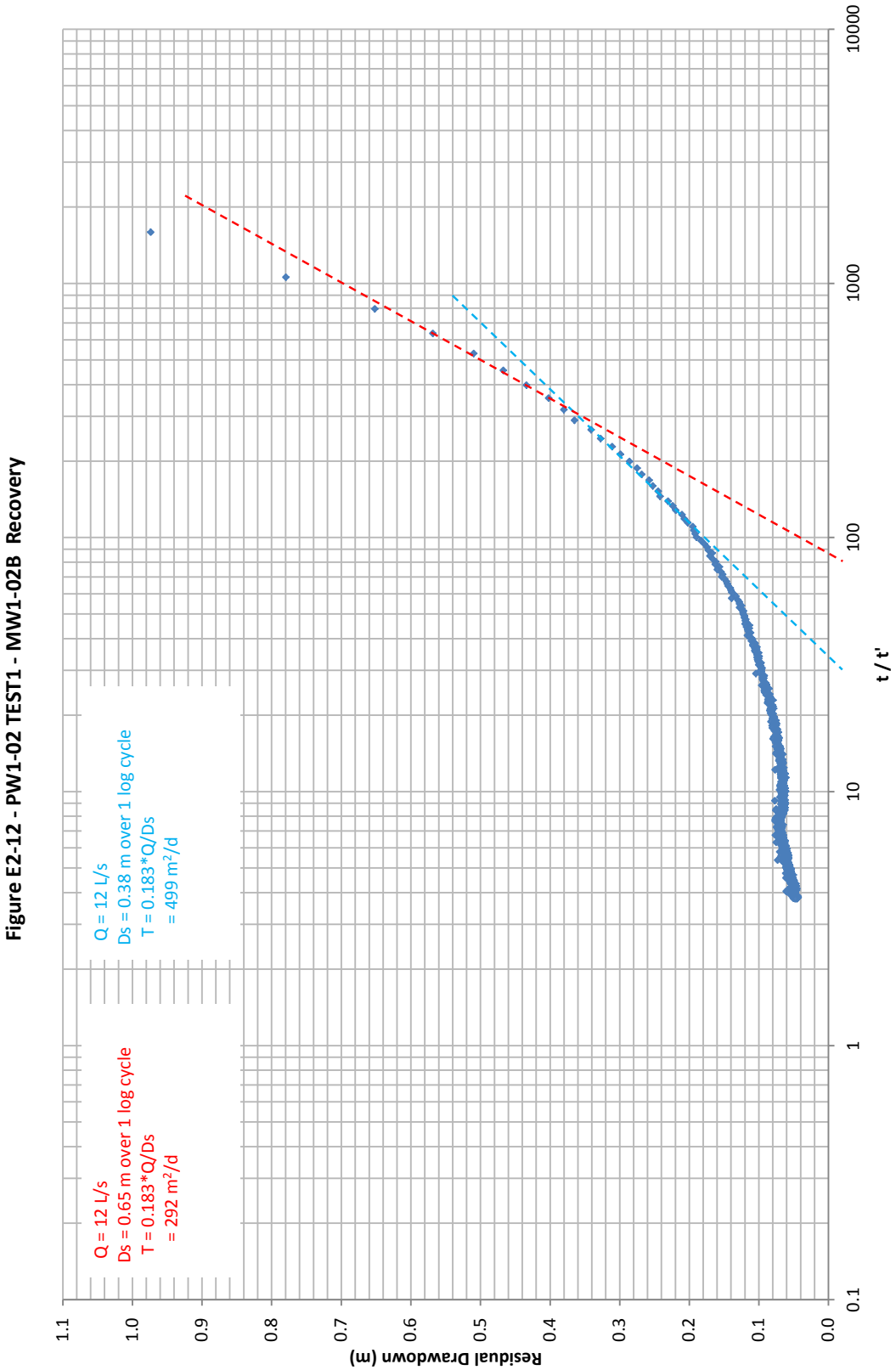


Figure E2-12 - PW1-02 TEST1 - MW1-02B Recovery
Residual Drawdown (m)
t / t'
Q = 12 L/s
Ds = 0.65 m over 1 log cycle
T = 0.183*Q/Ds
= 292 m²/d
Q = 12 L/s
Ds = 0.38 m over 1 log cycle
T = 0.183*Q/Ds
= 499 m²/d

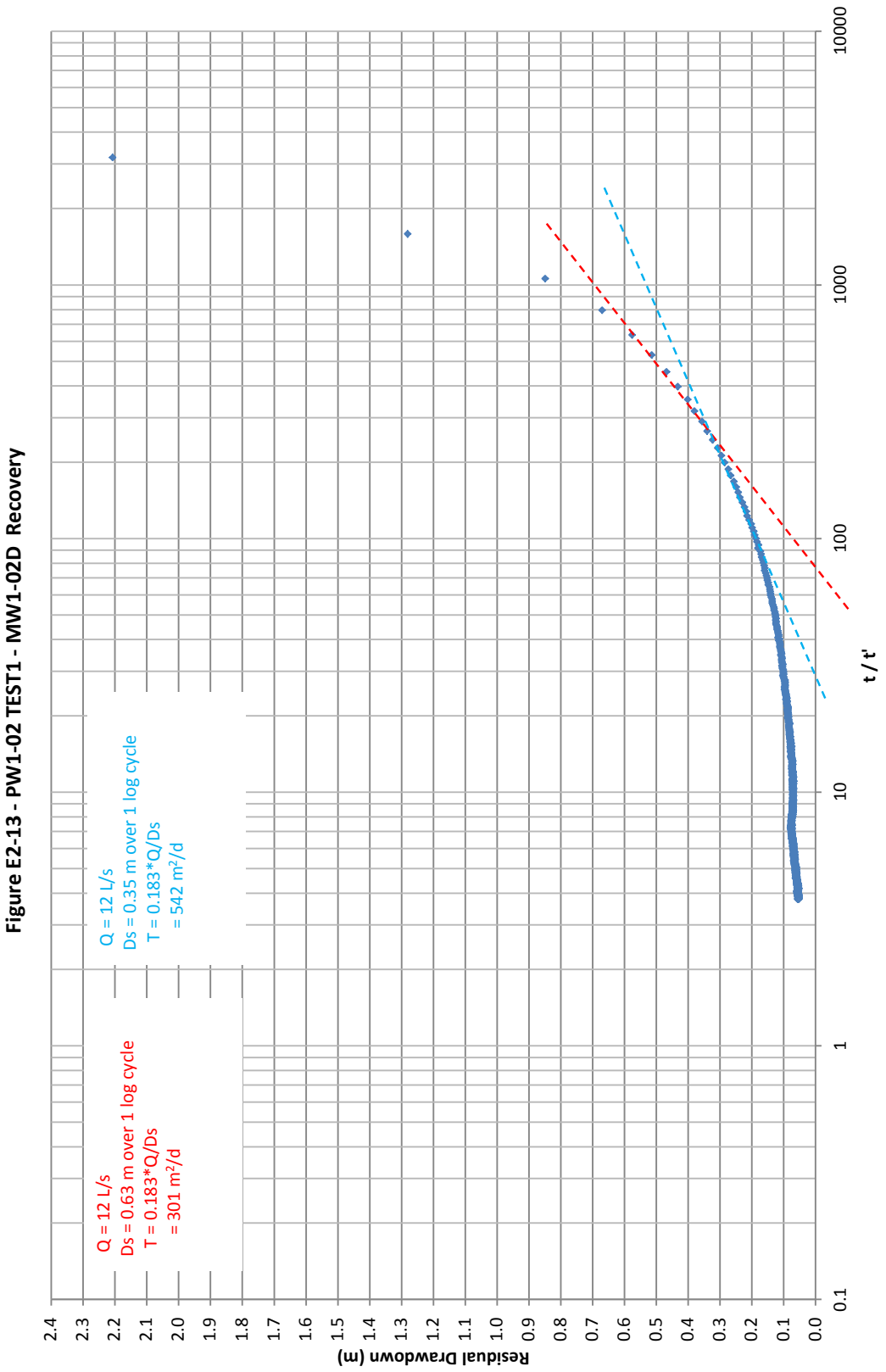


Figure E2-13 - PW1-02 TEST1 - MW1-02D Recovery
Residual Drawdown (m)
t / t'
Q = 12 L/s
Ds = 0.63 m over 1 log cycle
T = 0.183*Q/Ds
= 301 m²/d
Q = 12 L/s
Ds = 0.35 m over 1 log cycle
T = 0.183*Q/Ds
= 542 m²/d

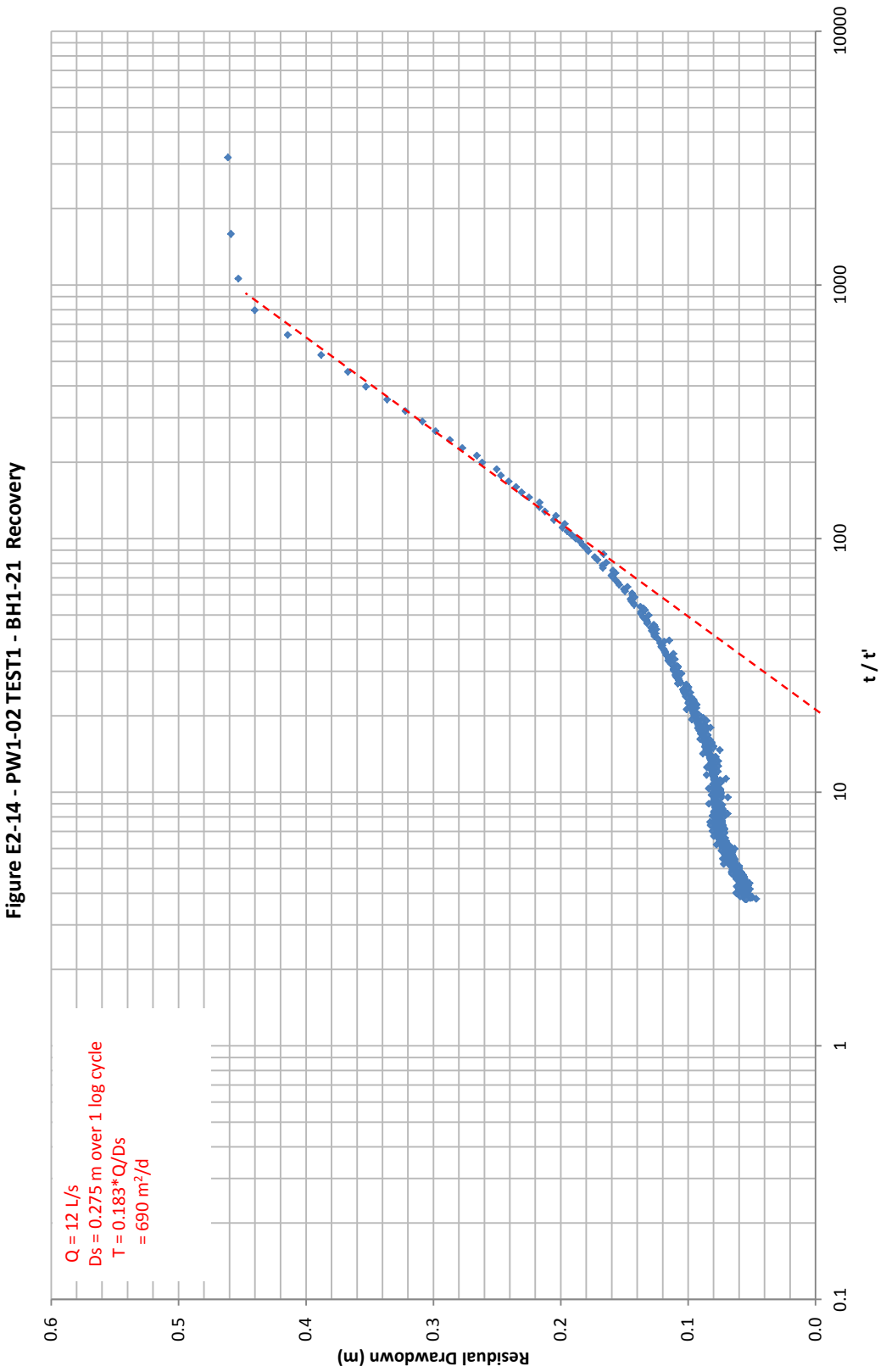


Figure E2-14 - PW1-02 TEST1 - BH1-21 Recovery
Residual Drawdown (m)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
t / t'
0.1
1
10
100
1000
10000
Q = 12 L/s
Ds = 0.275 m over 1 log cycle
T = 0.183*Q/Ds
= 690 m²/d

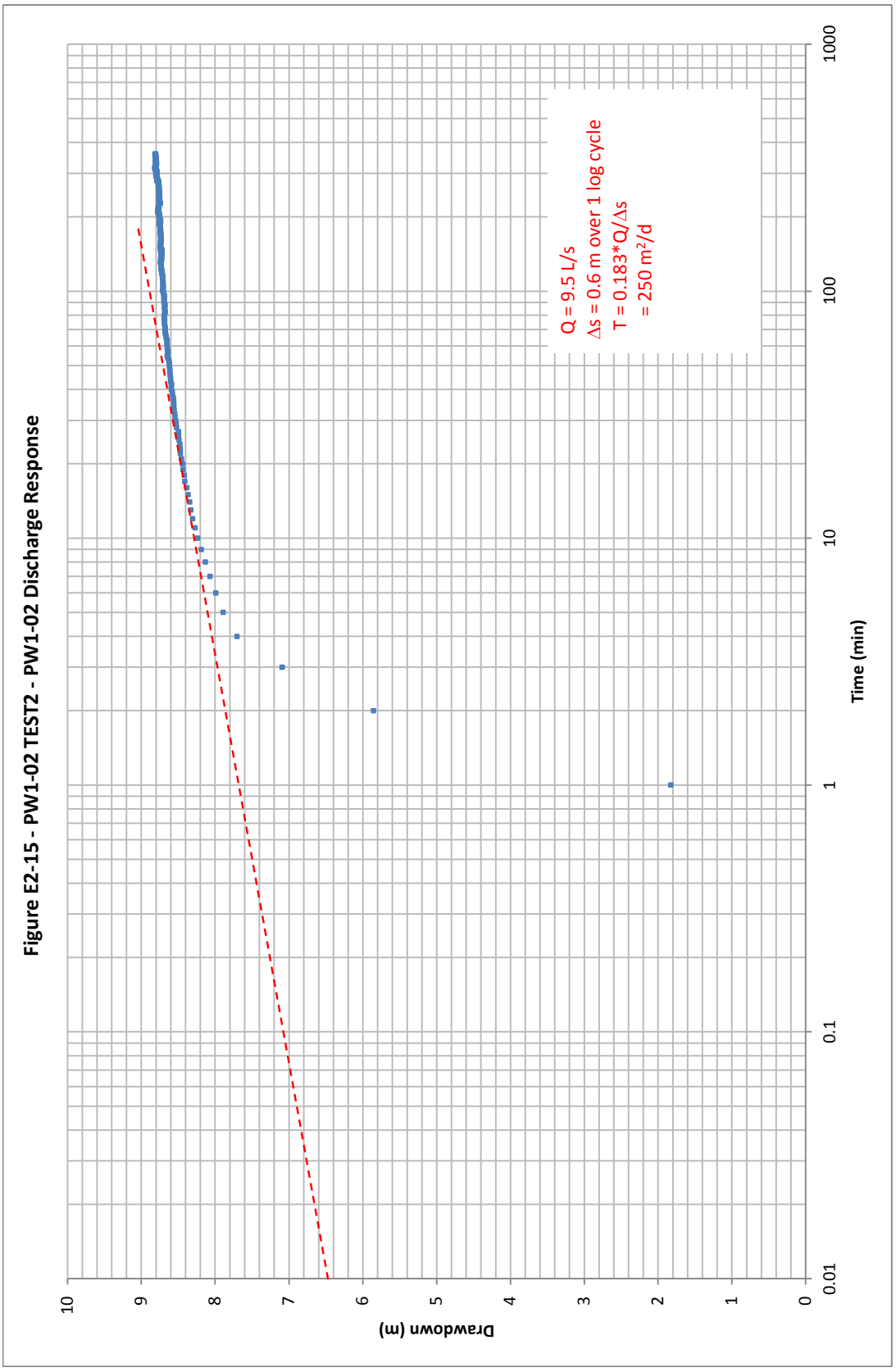


Figure E2-15 - PW1-02 TEST2 - PW1-02 Discharge Response

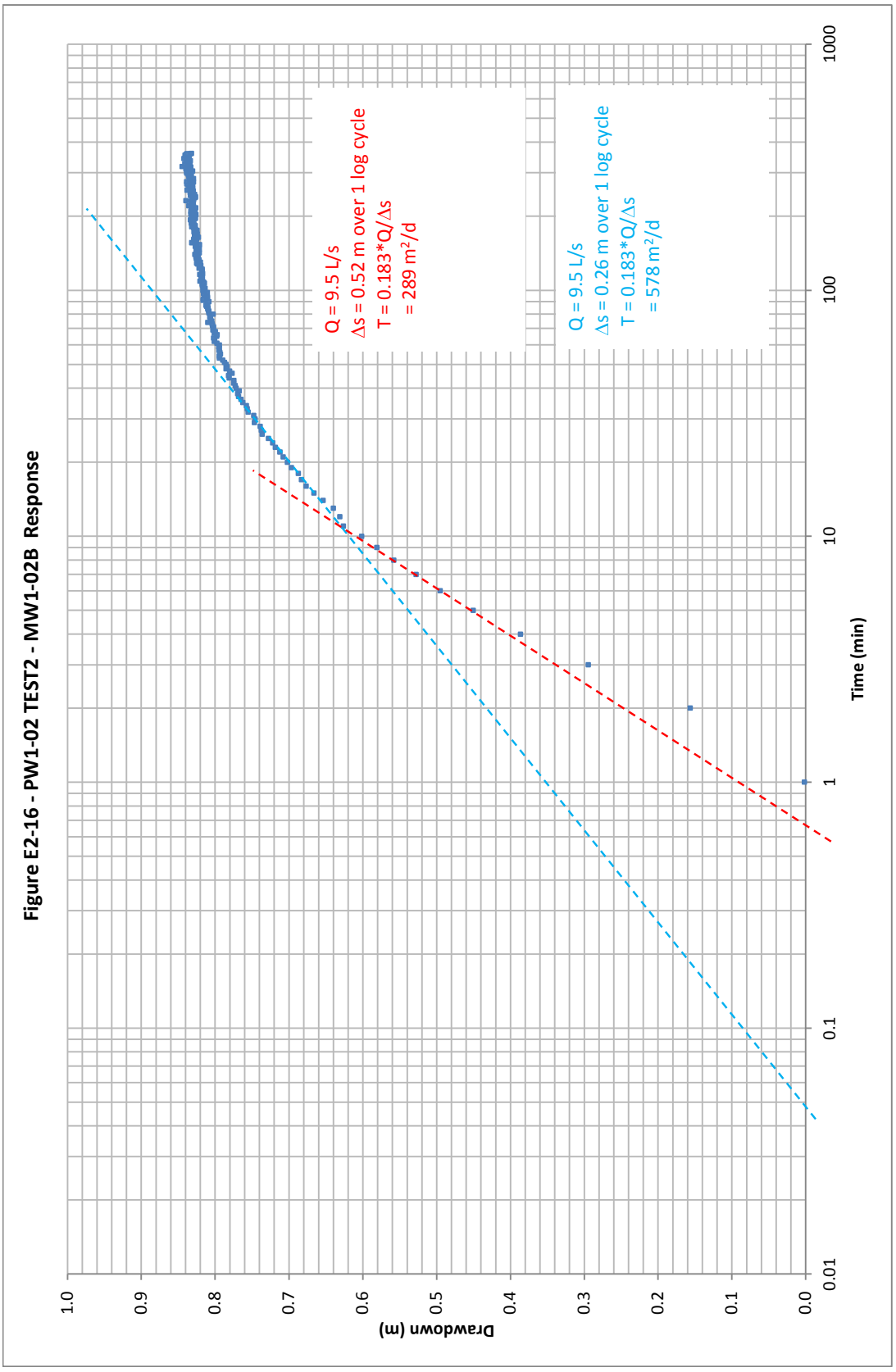


Figure E2-16 - PW1-02 TEST2 - MW1-02B Response
Drawdown (m)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0.01
0.1
1
10
100
1000
Time (min)
Q = 9.5 L/s
Δs = 0.52 m over 1 log cycle
T = 0.183*Q/Δs
= 289 m²/d
Q = 9.5 L/s
Δs = 0.26 m over 1 log cycle
T = 0.183*Q/Δs
= 578 m²/d

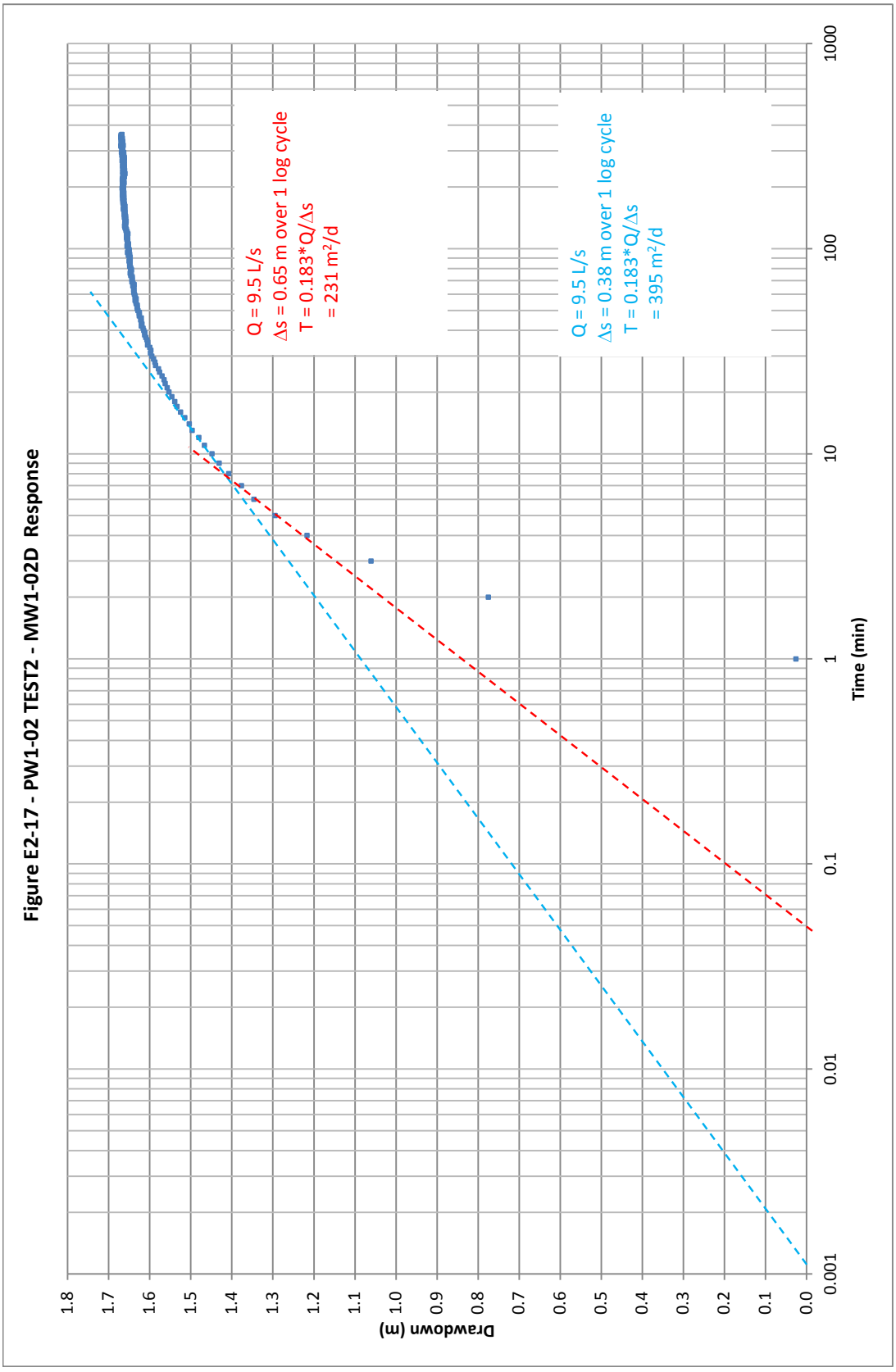


Figure E2-17 - PW1-02 TEST2 - MW1-02D Response
Drawdown (m)
Time (min)
Q = 9.5 L/s
Δs = 0.65 m over 1 log cycle
T = 0.183*Q/Δs
= 231 m²/d
Q = 9.5 L/s
Δs = 0.38 m over 1 log cycle
T = 0.183*Q/Δs
= 395 m²/d

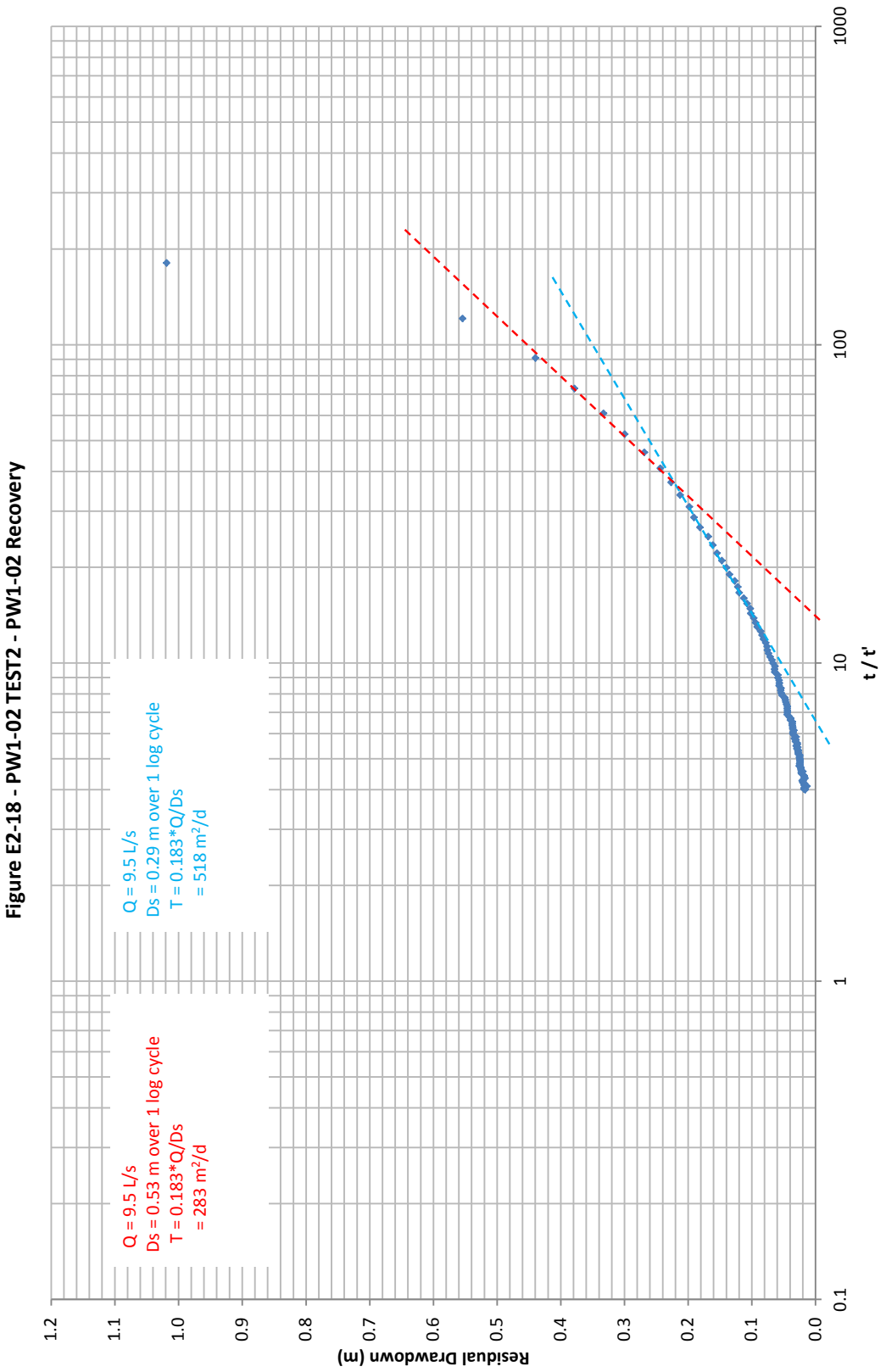


Figure E2-18 - PW1-02 TEST2 - PW1-02 Recovery

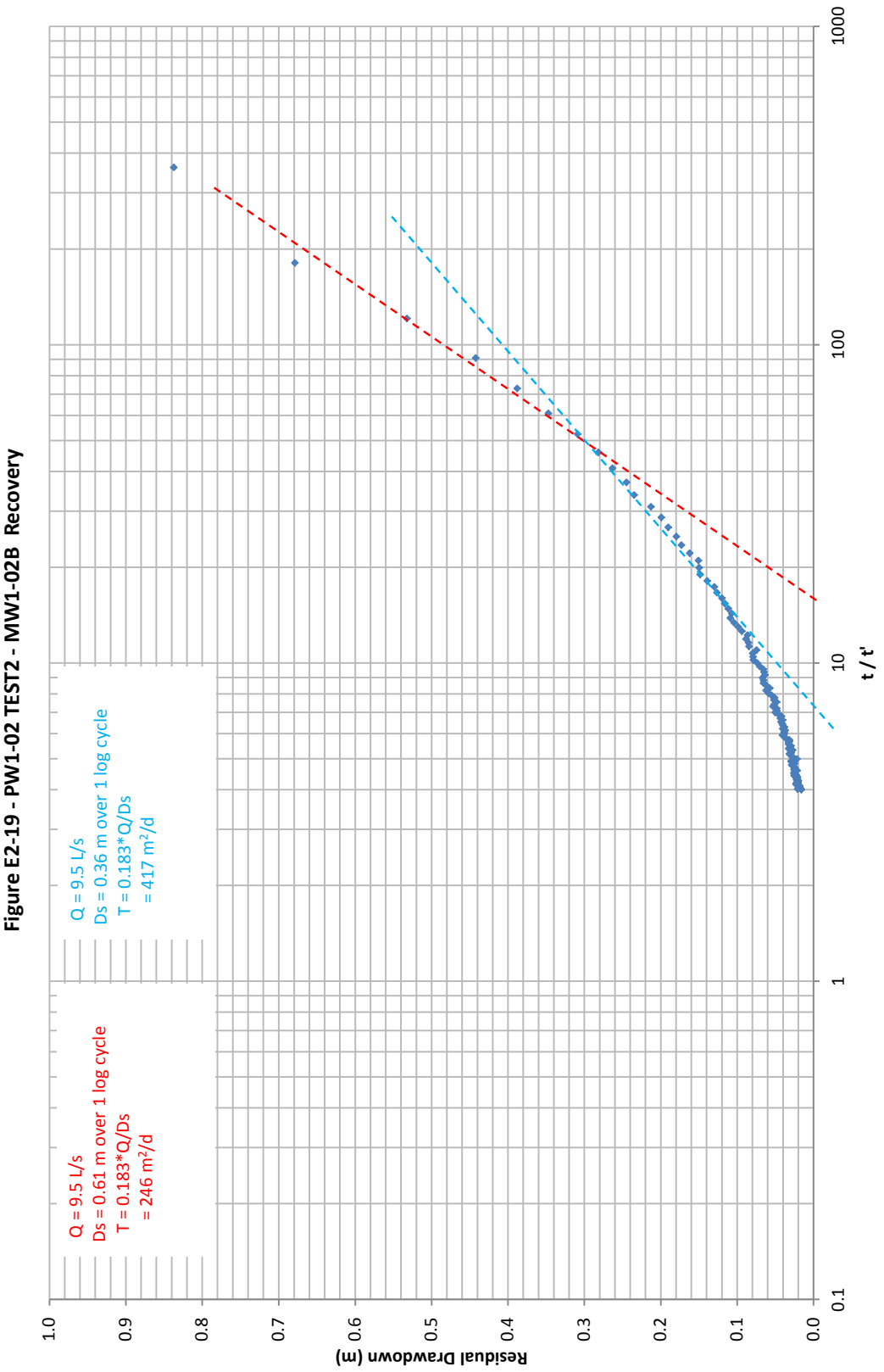


Figure E2-19 - PW1-02 TEST2 - MW1-02B Recovery
Residual Drawdown (m)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0.1
1
10
100
1000
t / t'
Q = 9.5 L/s
Ds = 0.61 m over 1 log cycle
T = 0.183*Q/Ds
= 246 m²/d
Q = 9.5 L/s
Ds = 0.36 m over 1 log cycle
T = 0.183*Q/Ds
= 417 m²/d

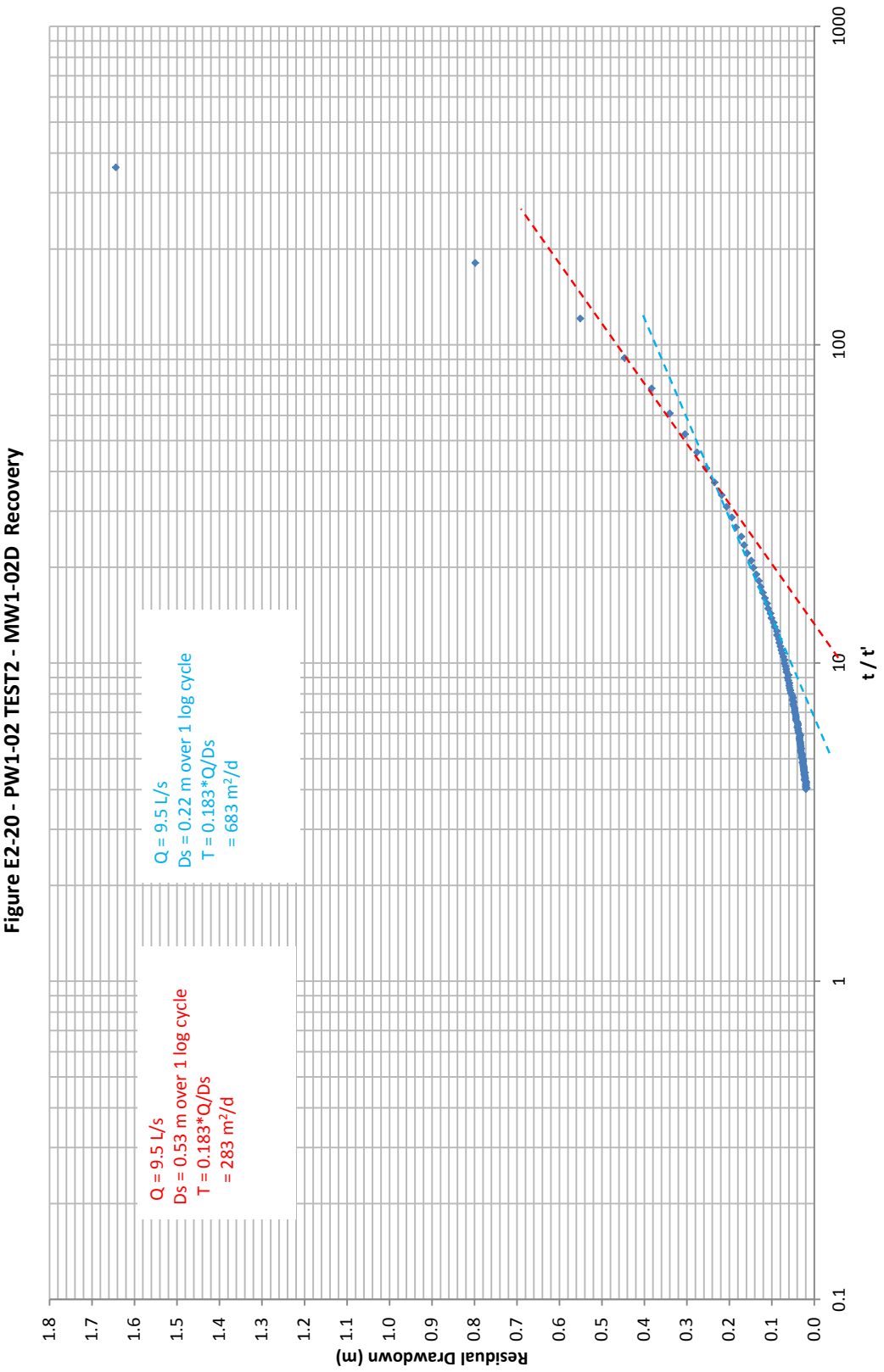


Figure E2-20 - PW1-02 TEST2 - MW1-02D Recovery
Residual Drawdown (m)
t / t'
Q = 9.5 L/s
Ds = 0.53 m over 1 log cycle
T = 0.183*Q/Ds
= 283 m²/d
Q = 9.5 L/s
Ds = 0.22 m over 1 log cycle
T = 0.183*Q/Ds
= 683 m²/d

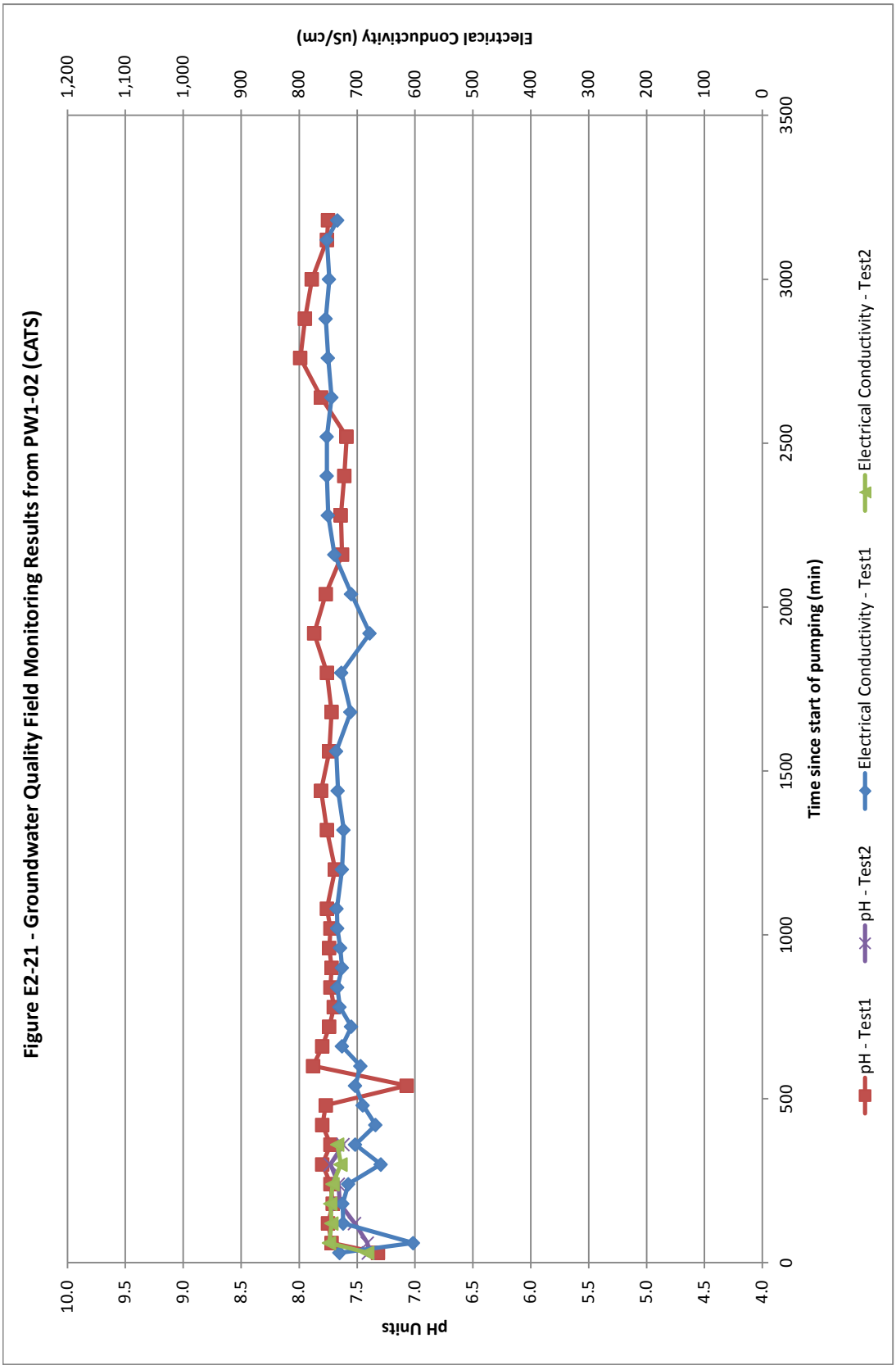
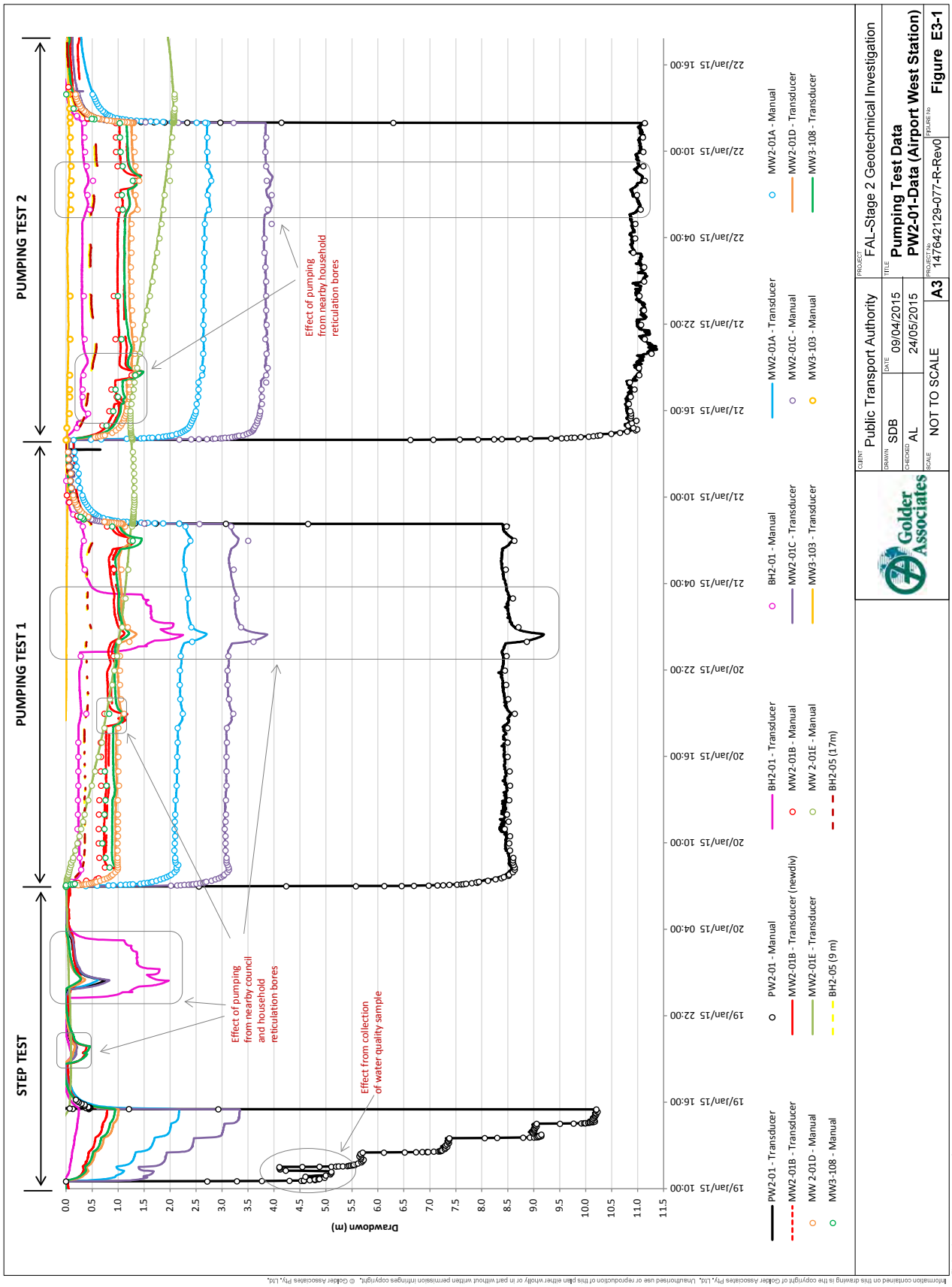


Figure E2-21 - Groundwater Quality Field Monitoring Results from PW1-02 (CATS)
pH Units
Electrical Conductivity (uS/cm)
Time since start of pumping (min)
pH - Test1
pH - Test2
Electrical Conductivity - Test1
Electrical Conductivity - Test2



APPENDIX E
Summary of Test Pumping Analysis

ATTACHMENT E3
Constant Rate Test – Airport West Station (PW2-01)



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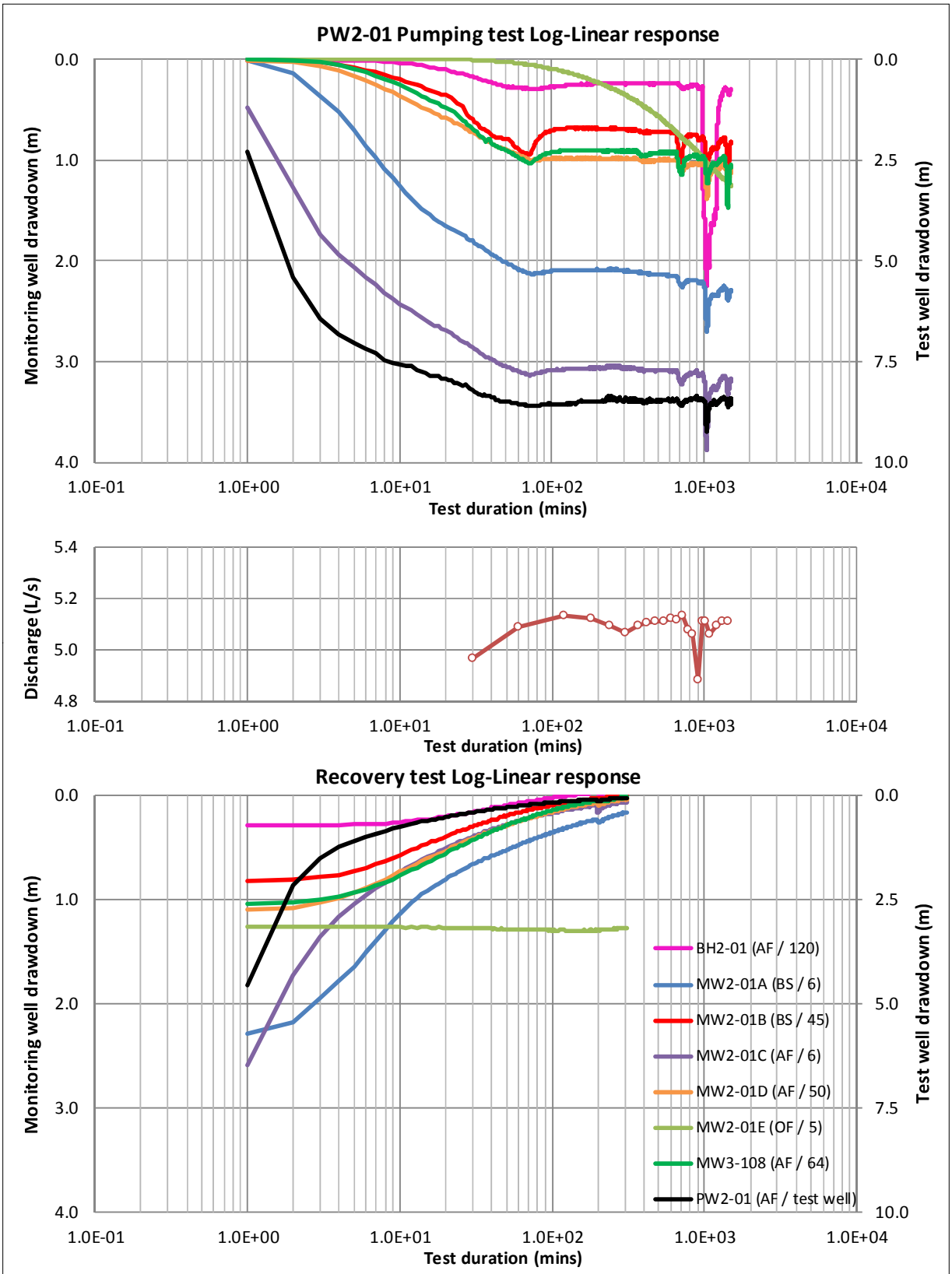

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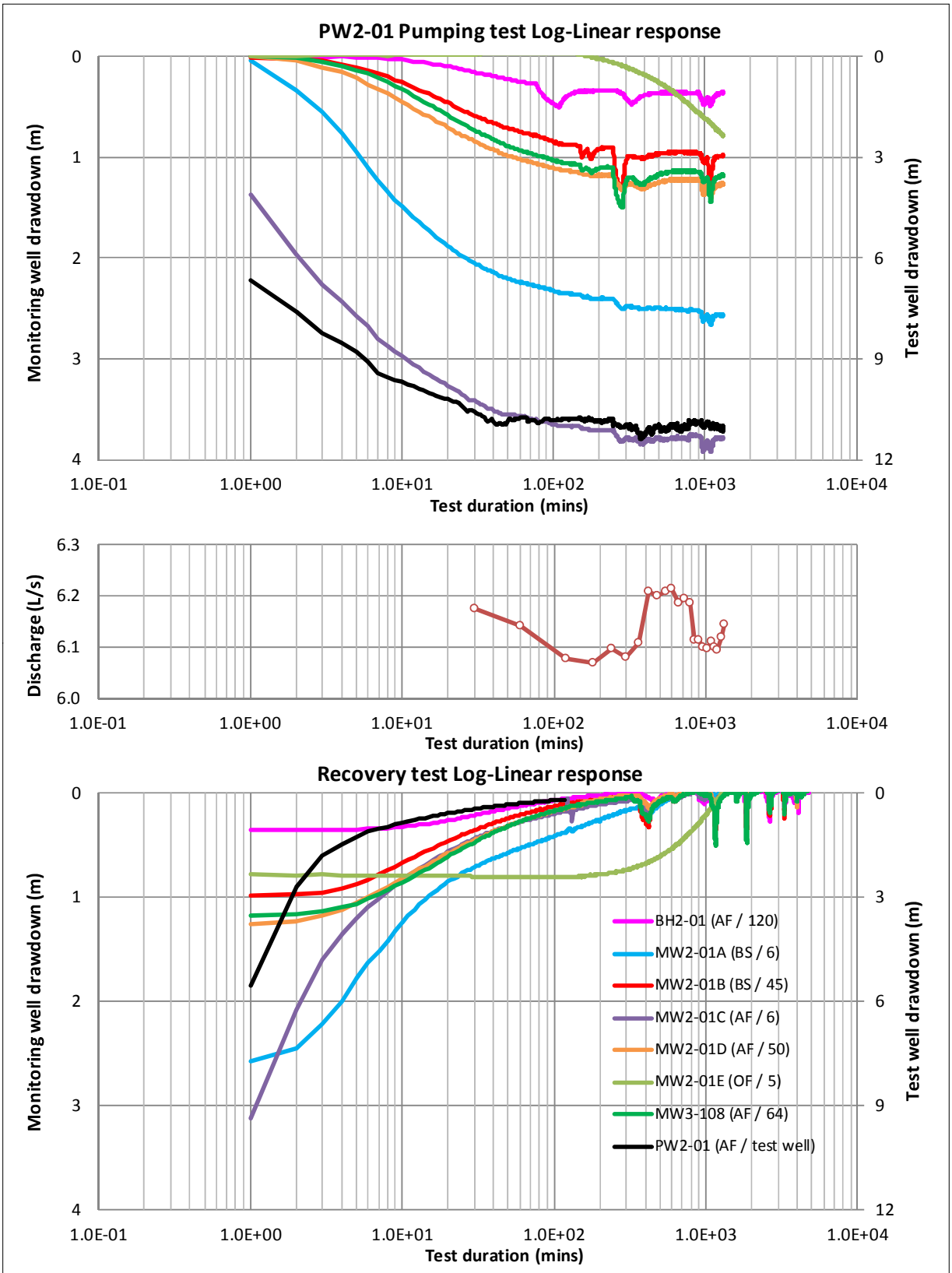
Ph: +618 9213 7600 Fx: +618 9213 7611

CLIENT	Public Transport Authority	PROJECT	FAL-Stage 2 Geotechnical Investigation
DRAWN	SDB	DATE 24/05/2015	TITLE **Pumping Test Data**
CHECKED	AL	DATE 24/05/2015	**PW2-01-Test 1 (Airport West Station)**
SCALE	NOT TO SCALE	A4	PROJECT No 147642129-077-R-Rev0
FIGURE No	**Figure E3-2**		

File Location:



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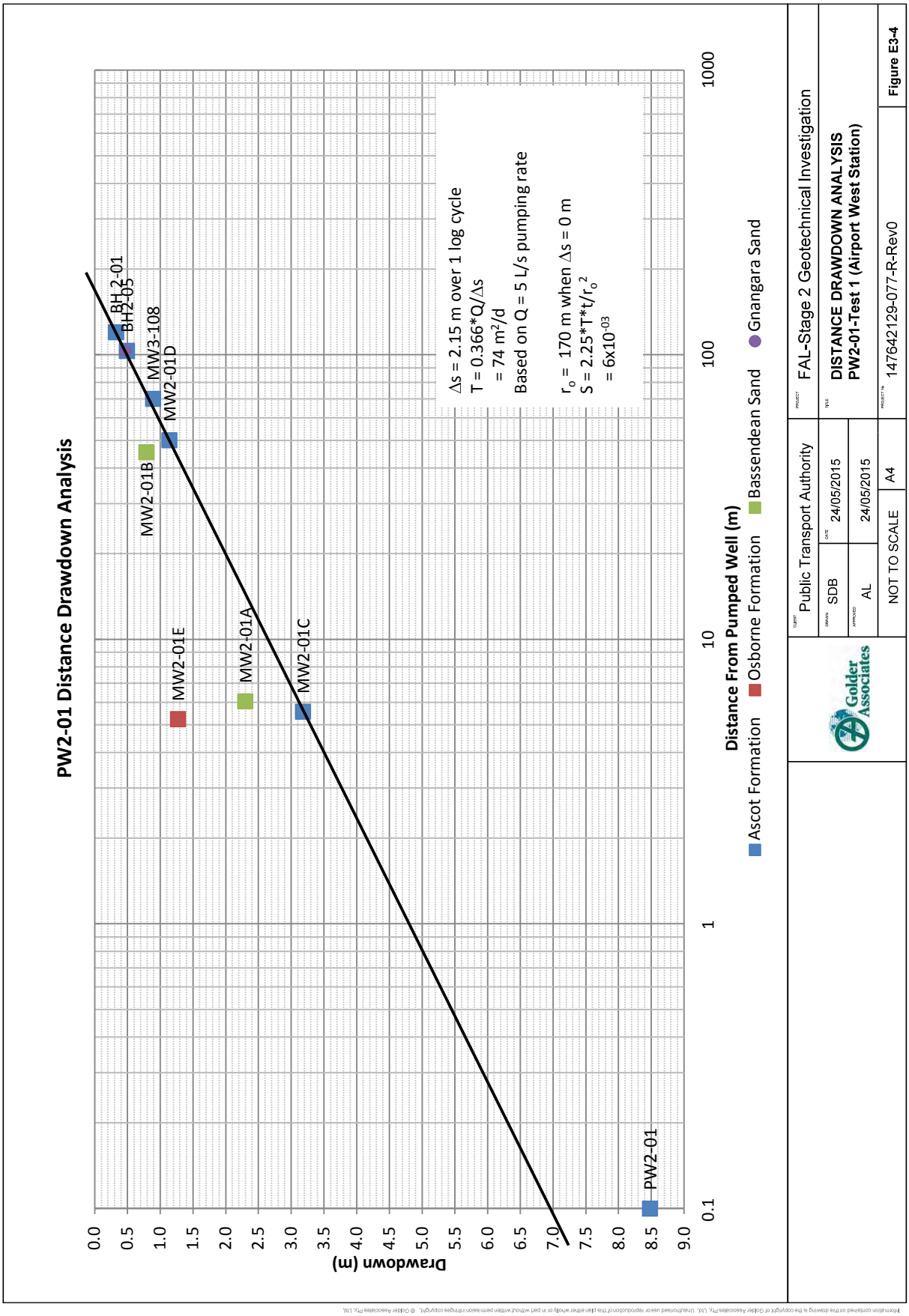
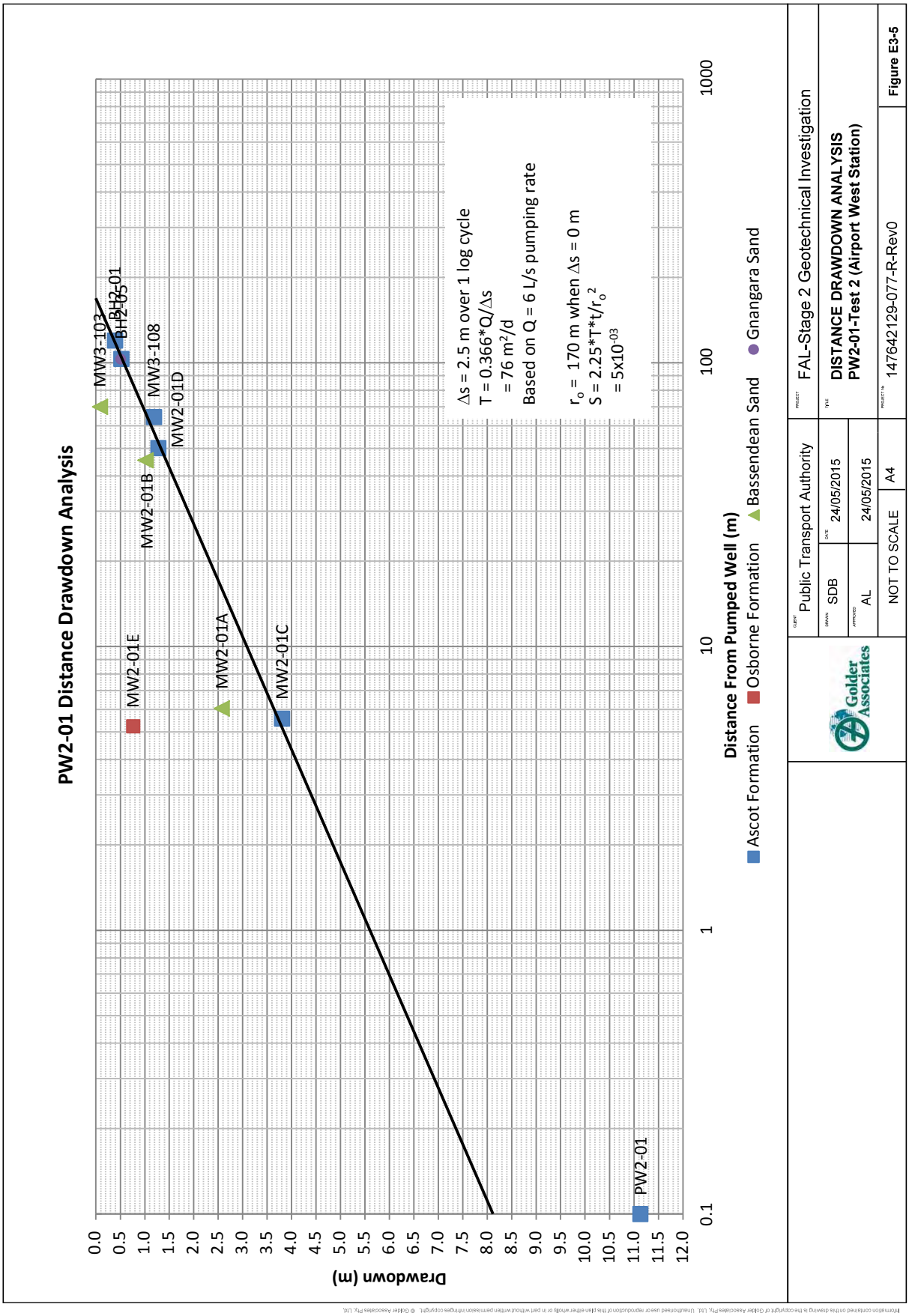
Ph: +618 9213 7600 Fx: +618 9213 7611

CLIENT	Public Transport Authority	PROJECT	FAL-Stage 2 Geotechnical Investigation
DRAWN	SDB	DATE 24/05/2015	TITLE Pumping Test Data
CHECKED	AL	DATE 24/05/2015	PW2-01-Test 2 (Airport West Station)
SCALE	NOT TO SCALE	A4	PROJECT No 147642129-077-R-Rev0
			FIGURE No Figure E3-3

File Location:

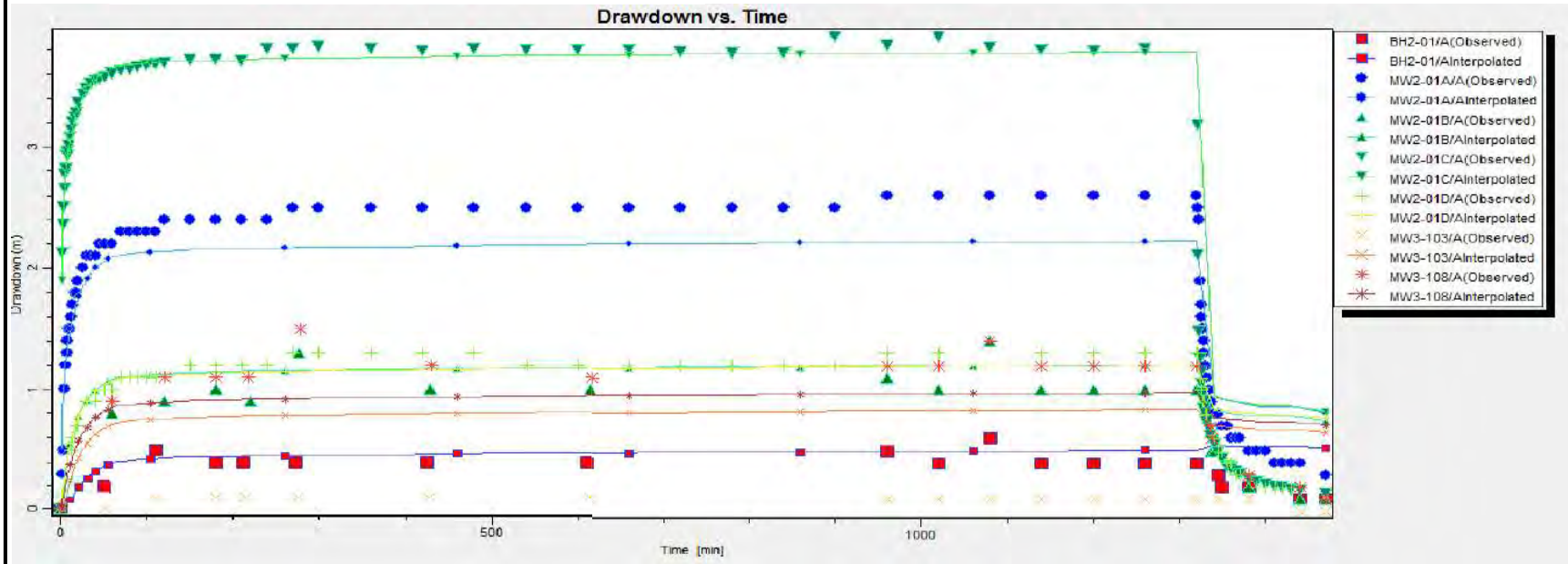

PW2-01 Distance Drawdown Analysis
Drawdown (m)
Distance From Pumped Well (m)
Ascot Formation
Osborne Formation
Bassendean Sand
Gnangara Sand
BH2-01
BH2-05
MW3-108
MW2-01D
MW2-01B
MW2-01E
MW2-01A
MW2-01C
PW2-01
Δs = 2.15 m over 1 log cycle
T = 0.366*Q/Δs
= 74 m²/d
Based on Q = 5 L/s pumping rate
r_o = 170 m when Δs = 0 m
S = 2.25*T*t/r_o²
= 6x10⁻⁰³
Public Transport Authority
FAL-Stage 2 Geotechnical Investigation
DISTANCE DRAWDOWN ANALYSIS
PW2-01-Test 1 (Airport West Station)
SDB
AL
24/05/2015
24/05/2015
NOT TO SCALE
A4
147642129-077-R-Rev0
Figure E3-4


PW2-01 Distance Drawdown Analysis
Drawdown (m)
Distance From Pumped Well (m)
Ascot Formation
Osborne Formation
Bassendean Sand
Gnangara Sand
MW3-103
BH2-01
BH2-05
MW3-108
MW2-01D
MW2-01B
MW2-01A
MW2-01C
MW2-01E
PW2-01
Δs = 2.5 m over 1 log cycle
T = 0.366*Q/Δs
= 76 m²/d
Based on Q = 6 L/s pumping rate
r_o = 170 m when Δs = 0 m
S = 2.25*T*t/r_o²
= 5x10⁻⁰³
Public Transport Authority
FAL-Stage 2 Geotechnical Investigation
DISTANCE DRAWDOWN ANALYSIS
PW2-01-Test 2 (Airport West Station)
SDB
24/05/2015
AL
24/05/2015
NOT TO SCALE
A4
147642129-077-R-Rev0
Figure E3-5

Specific Storage =	1.00E-05	1/m
Transmissivity =	76	m2/d
Hydraulic Conductivity		
K(BS,GF,GS)h	3	m/d
K(BS,GF,GS)v	0.3	m/d
K(AF)h	8	m/d
K(AF)v	0.8	m/d
K(OF)h	0.0005	m/d
K(OF)v	0.00005	m/d
River Conductance =	9	m2/d

River Leakage

Time (min)	Rate (L/S)
10	0.1
20	0.2
30	0.3
40	0.3
50	0.3
120	0.4
320	0.4
520	0.4
920	0.4
1120	0.4
1320	0.4
1470	0.4



Screenshot from MODFLOW output

Note: Other combinations in hydraulic conductivity and storage could may be possible

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CLIENT	Public Transport authority			PROJECT	FAL - Stage 2 Geotechnical Investigation
DRAWN	AL	DATE	24-May-15	TITLE	Analysis of Pumping Test 2 using Numerical Groundwater Model Airport West Station
CHECK	DPS	DATE	24-May-15		
SCALE	Not To Scale	A4		PROJECT No.	147642129-077-R-Rev0
				FIGURE No.	E3-6

C:\Users\alundorf\Desktop\FAL\APPENDIX E\Appendix E - Test Pumping Results\Attachment E3\Figure E3-6.xlsx

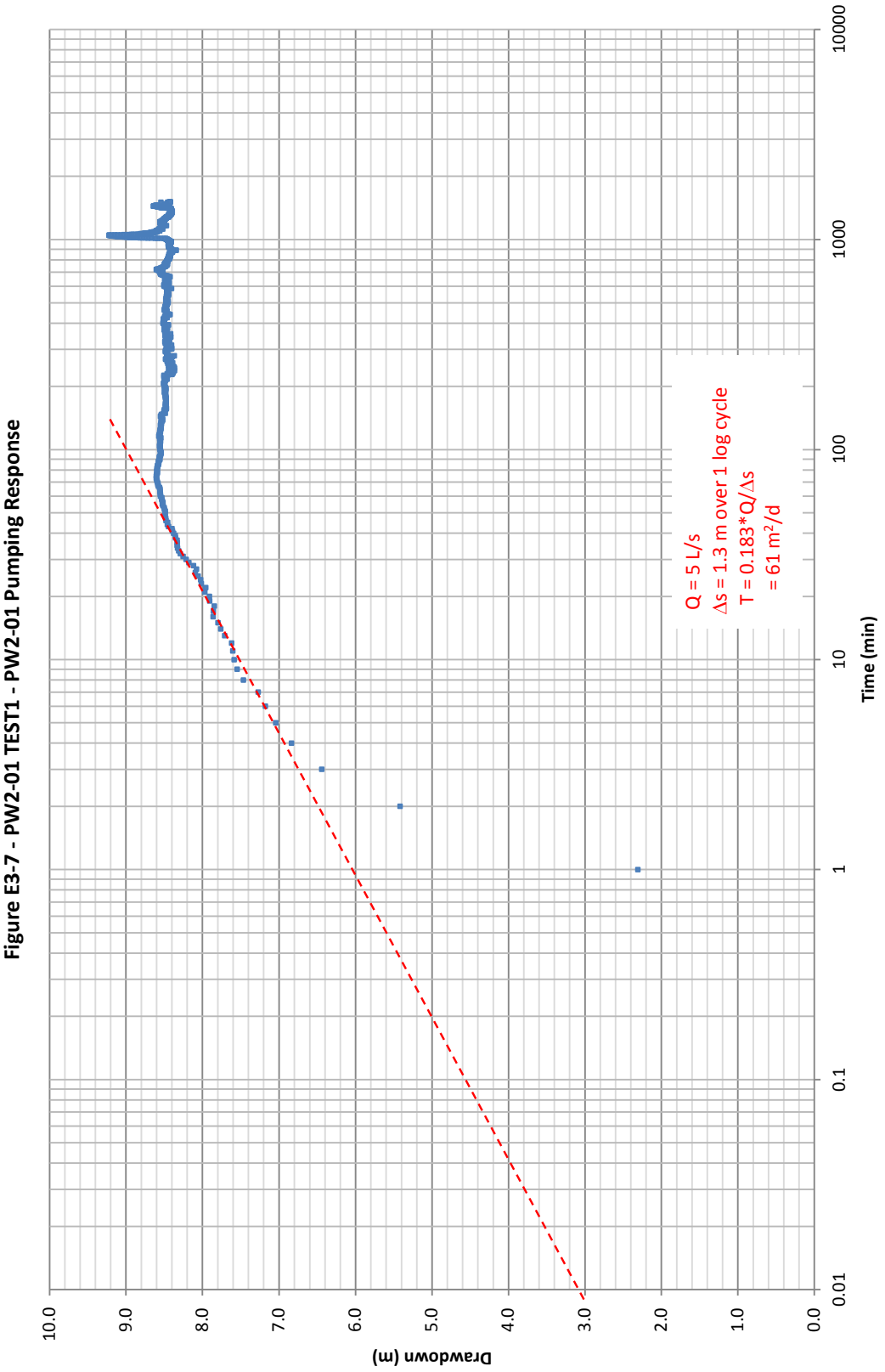


Figure E3-7 - PW2-01 TEST1 - PW2-01 Pumping Response
Drawdown (m)
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 5 L/s
Δs = 1.3 m over 1 log cycle
T = 0.183*Q/Δs
= 61 m²/d

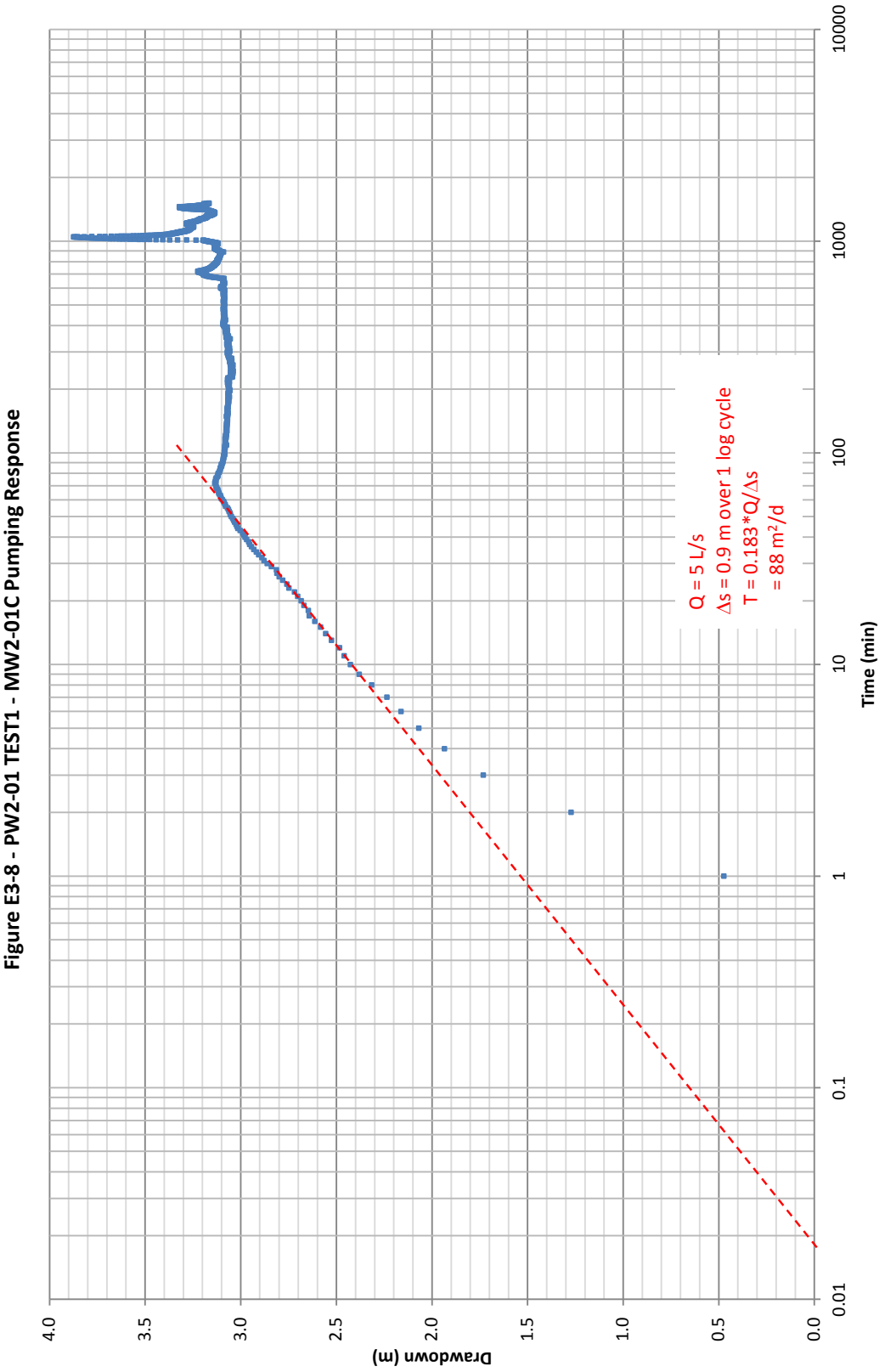


Figure E3-8 - PW2-01 TEST1 - MW2-01C Pumping Response
Drawdown (m)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Time (min)
0.01
0.1
1
10
100
1000
10000
Q = 5 L/s
Δs = 0.9 m over 1 log cycle
T = 0.183*Q/Δs
= 88 m²/d

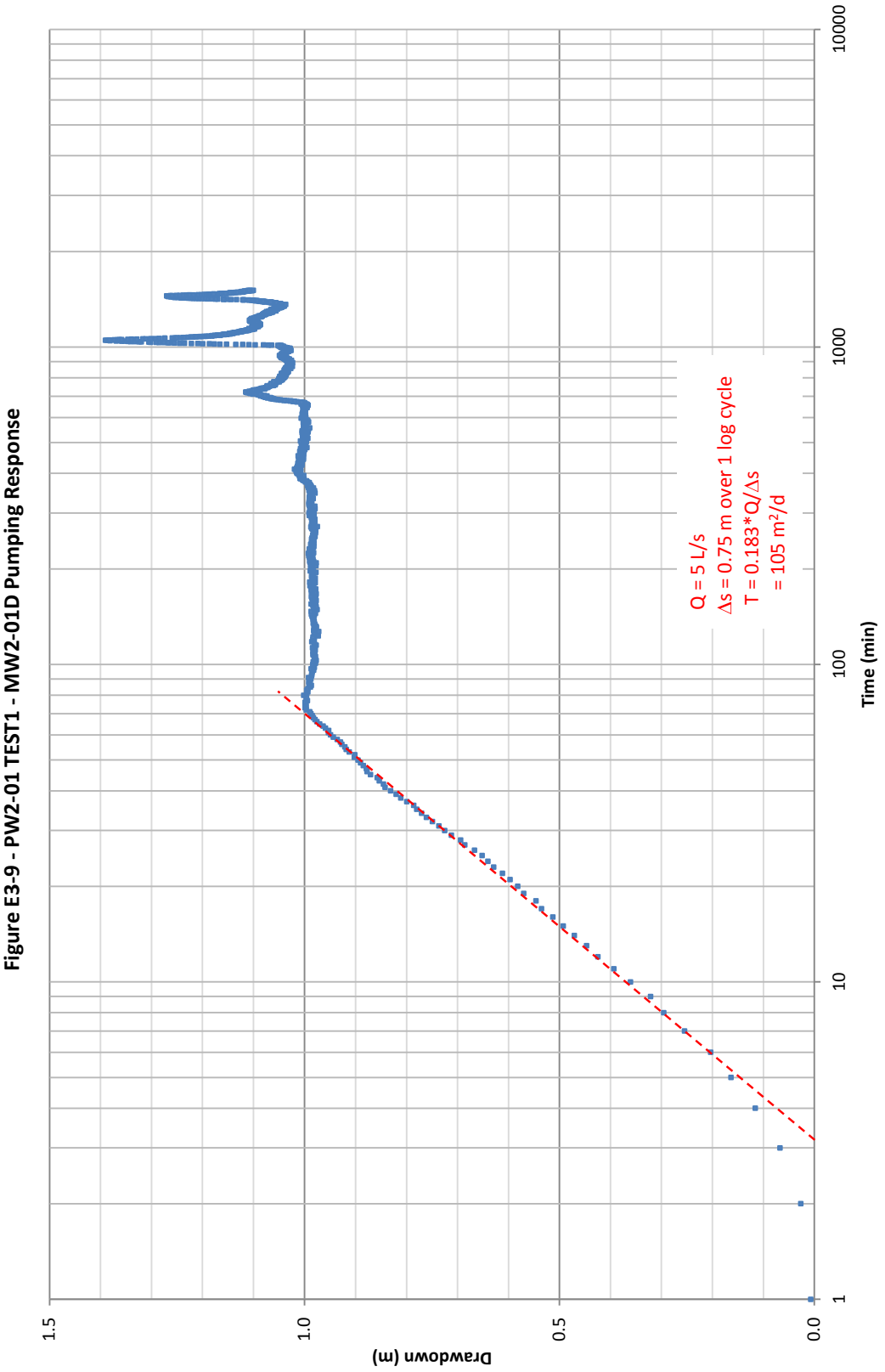


Figure E3-9 - PW2-01 TEST1 - MW2-01D Pumping Response
Drawdown (m)
1.5
1.0
0.5
0.0
Time (min)
1
10
100
1000
10000
Q = 5 L/s
Δs = 0.75 m over 1 log cycle
T = 0.183*Q/Δs
= 105 m²/d

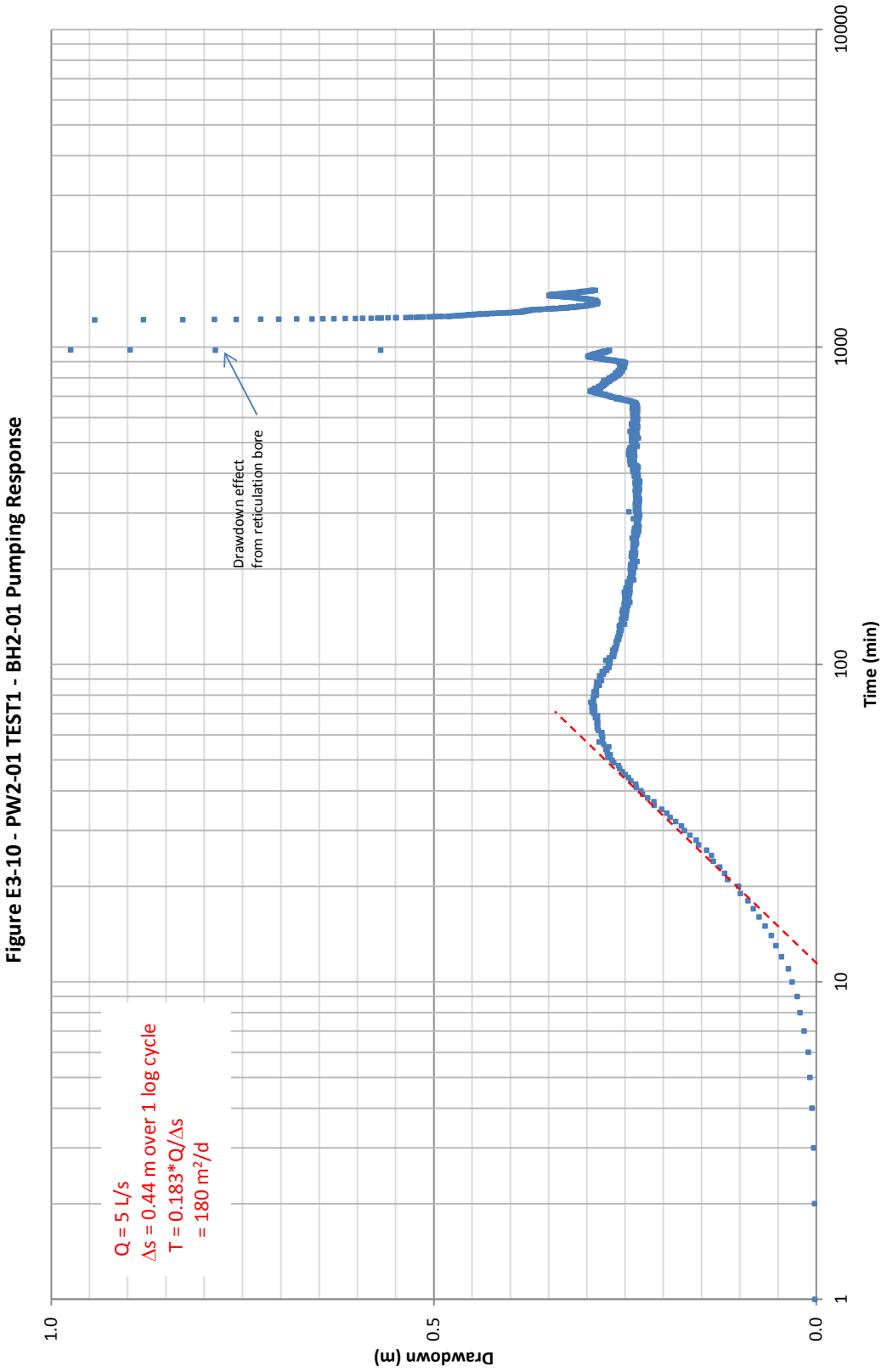


Figure E3-10 - PW2-01 TEST1 - BH2-01 Pumping Response
Drawdown (m)
1.0
0.5
0.0
Time (min)
1
10
100
1000
10000
Drawdown effect
from reticulation bore
Q = 5 L/s
Δs = 0.44 m over 1 log cycle
T = 0.183*Q/Δs
= 180 m²/d

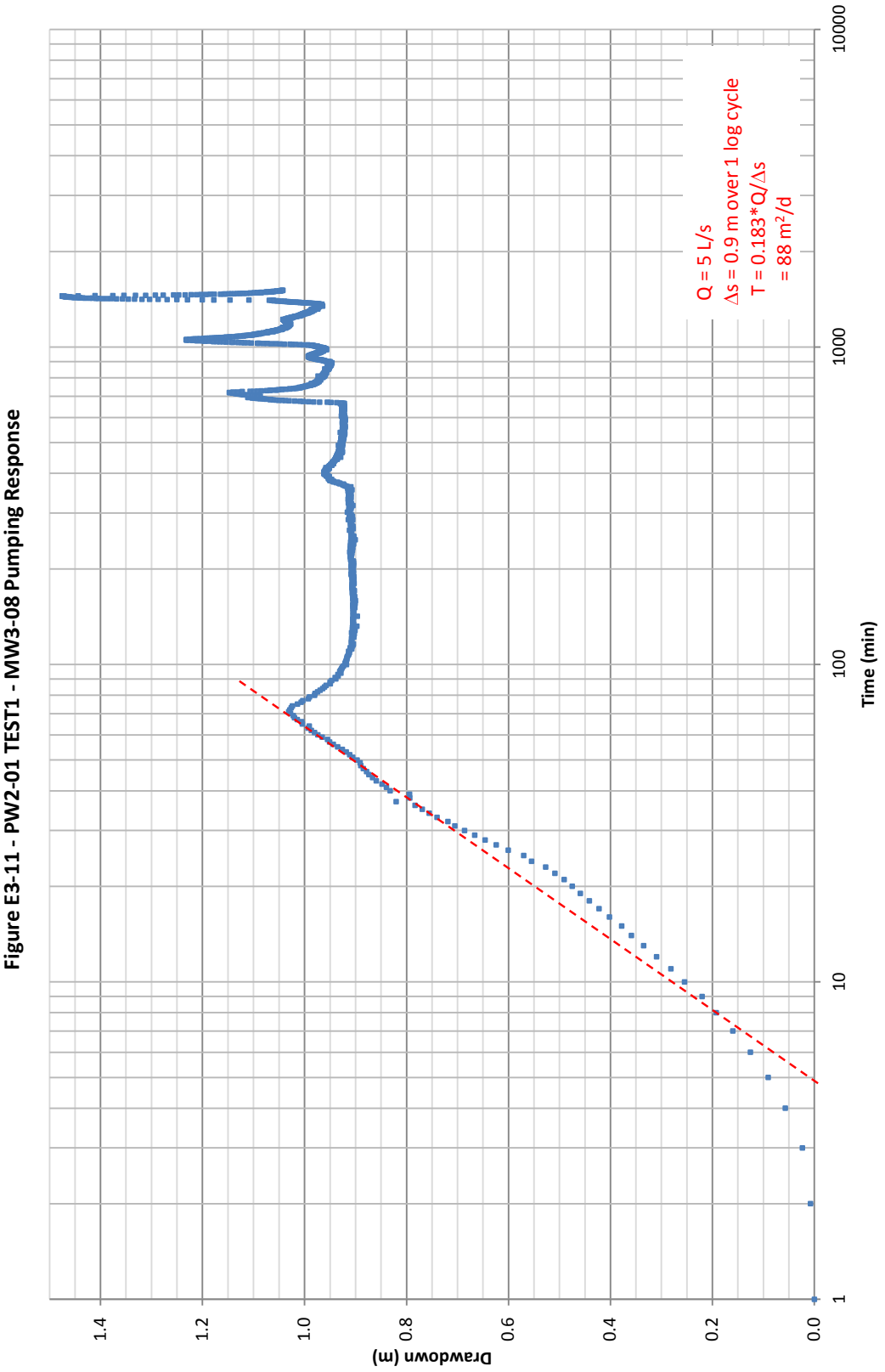


Figure E3-11 - PW2-01 TEST1 - MW3-08 Pumping Response
Drawdown (m)
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
Time (min)
1
10
100
1000
10000
Q = 5 L/s
Δs = 0.9 m over 1 log cycle
T = 0.183*Q/Δs
= 88 m²/d

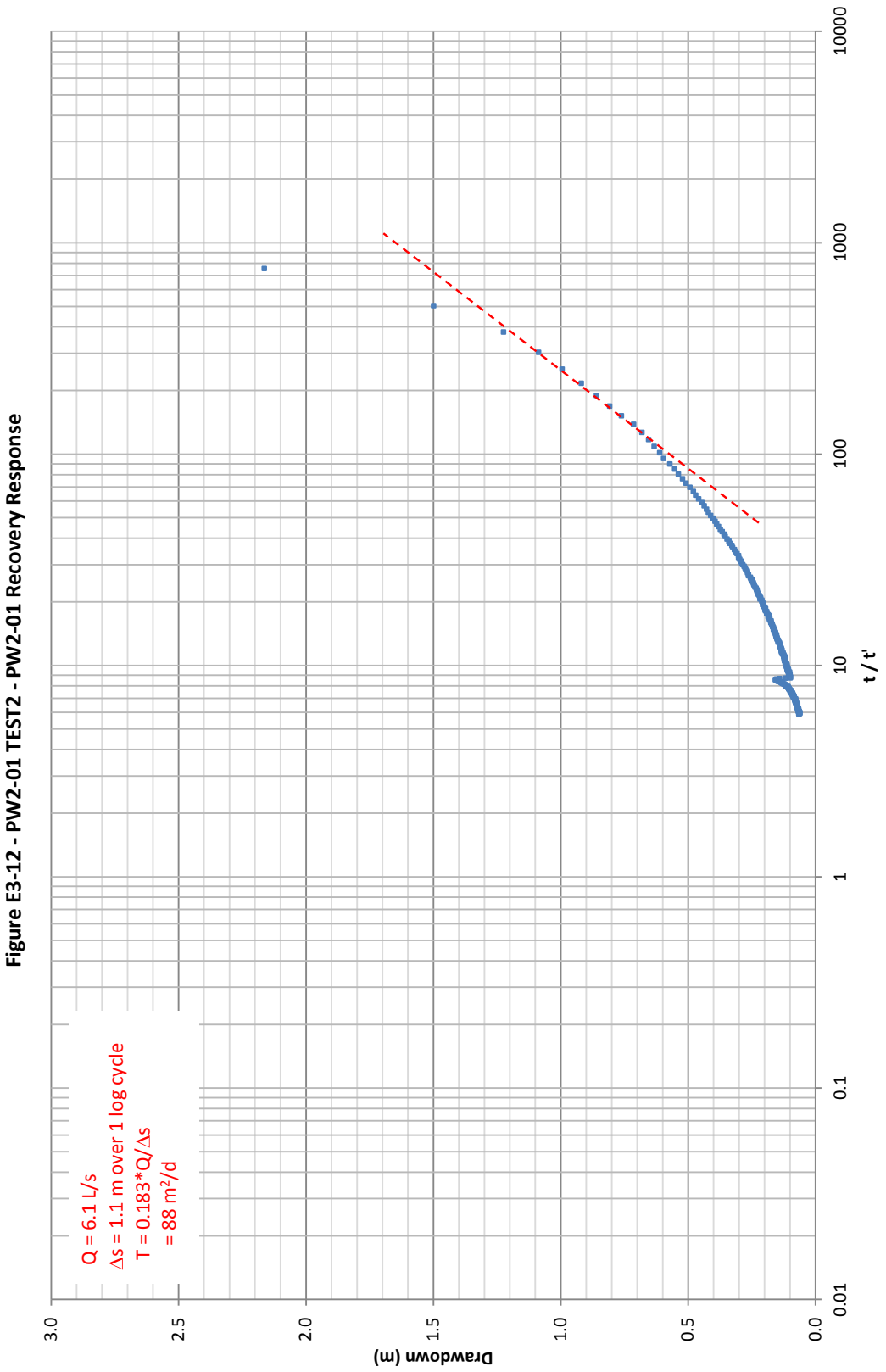


Figure E3-12 - PW2-01 TEST2 - PW2-01 Recovery Response

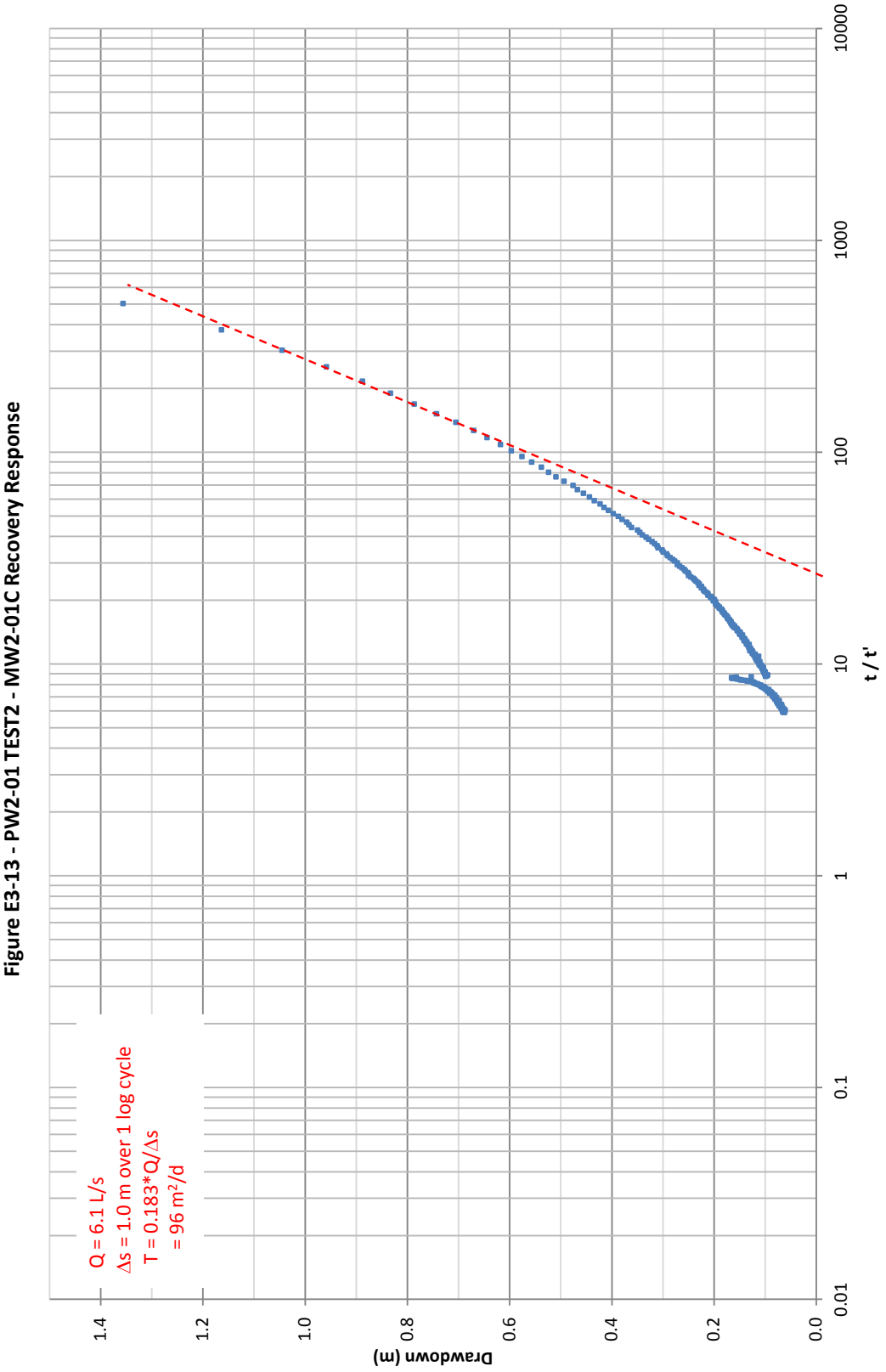


Figure E3-13 - PW2-01 TEST2 - MW2-01C Recovery Response
Drawdown (m)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.01
0.1
1
10
100
1000
10000
t / t'
Q = 6.1 L/s
Δs = 1.0 m over 1 log cycle
T = 0.183*Q/Δs
= 96 m²/d

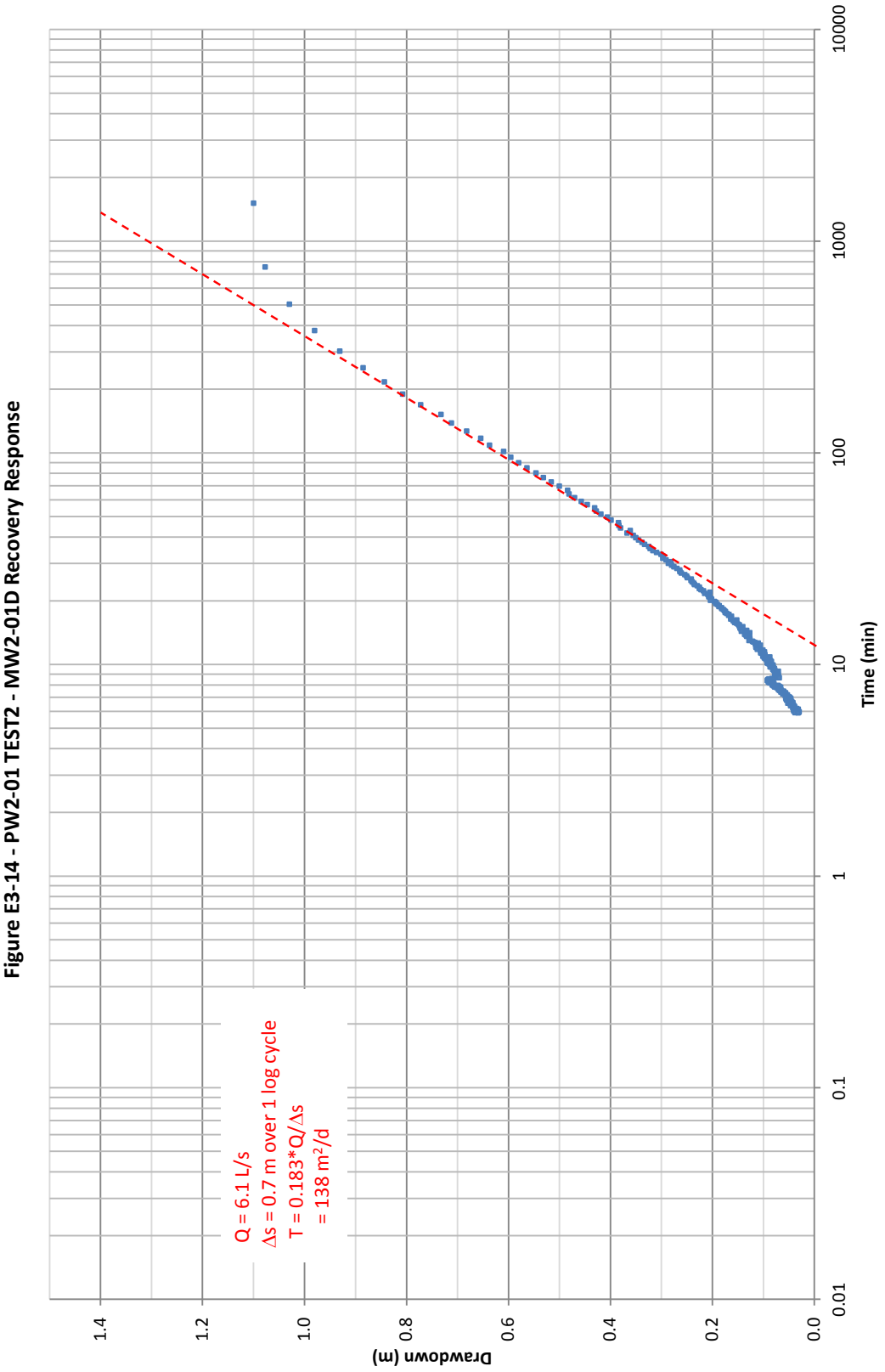


Figure E3-14 - PW2-01 TEST2 - MW2-01D Recovery Response
Drawdown (m)
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Time (min)
10000
1000
100
10
1
0.1
0.01
Q = 6.1 L/s
Δs = 0.7 m over 1 log cycle
T = 0.183*Q/Δs
= 138 m²/d

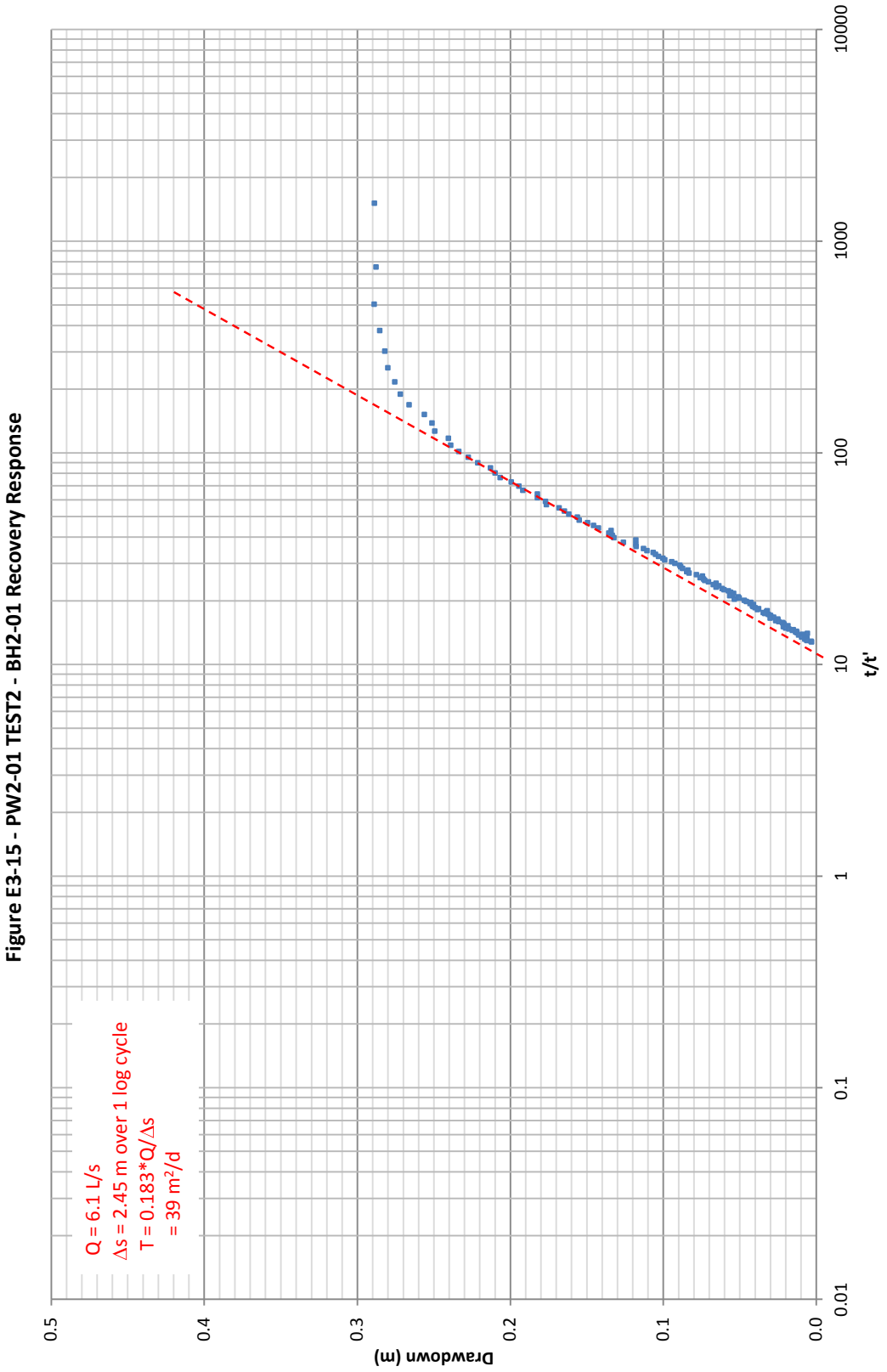


Figure E3-15 - PW2-01 TEST2 - BH2-01 Recovery Response
Drawdown (m)
0.5
0.4
0.3
0.2
0.1
0.0
t/t'
10000
1000
100
10
1
0.1
0.01
Q = 6.1 L/s
Δs = 2.45 m over 1 log cycle
T = 0.183*Q/Δs
= 39 m²/d

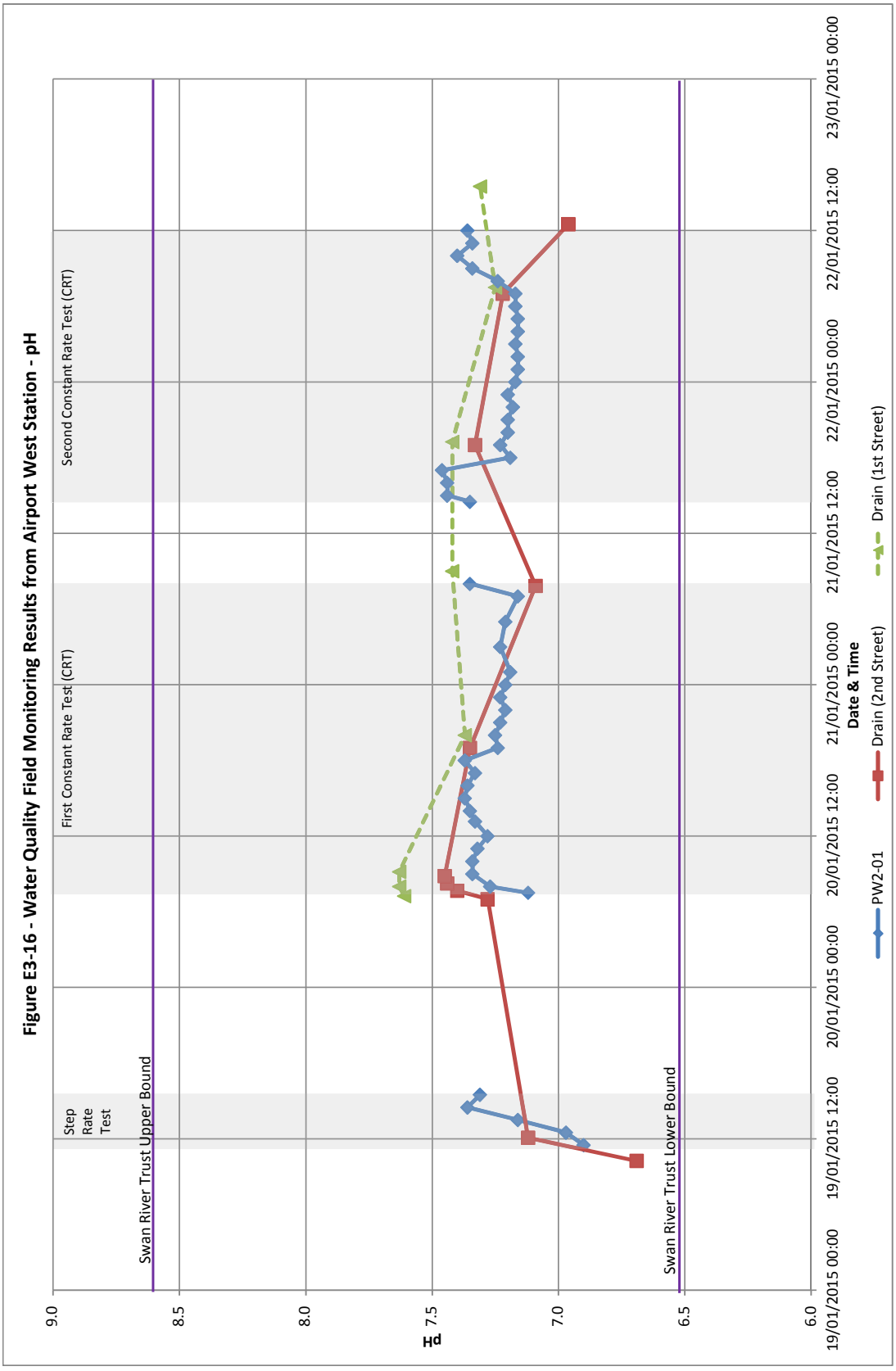


Figure E3-16 - Water Quality Field Monitoring Results from Airport West Station - pH
Step Rate Test
First Constant Rate Test (CRT)
Second Constant Rate Test (CRT)
Swan River Trust Upper Bound
Swan River Trust Lower Bound
pH
9.0
8.5
8.0
7.5
7.0
6.5
6.0
19/01/2015 00:00
19/01/2015 12:00
20/01/2015 00:00
20/01/2015 12:00
21/01/2015 00:00
21/01/2015 12:00
22/01/2015 00:00
22/01/2015 12:00
23/01/2015 00:00
Date & Time
PW2-01
Drain (2nd Street)
Drain (1st Street)

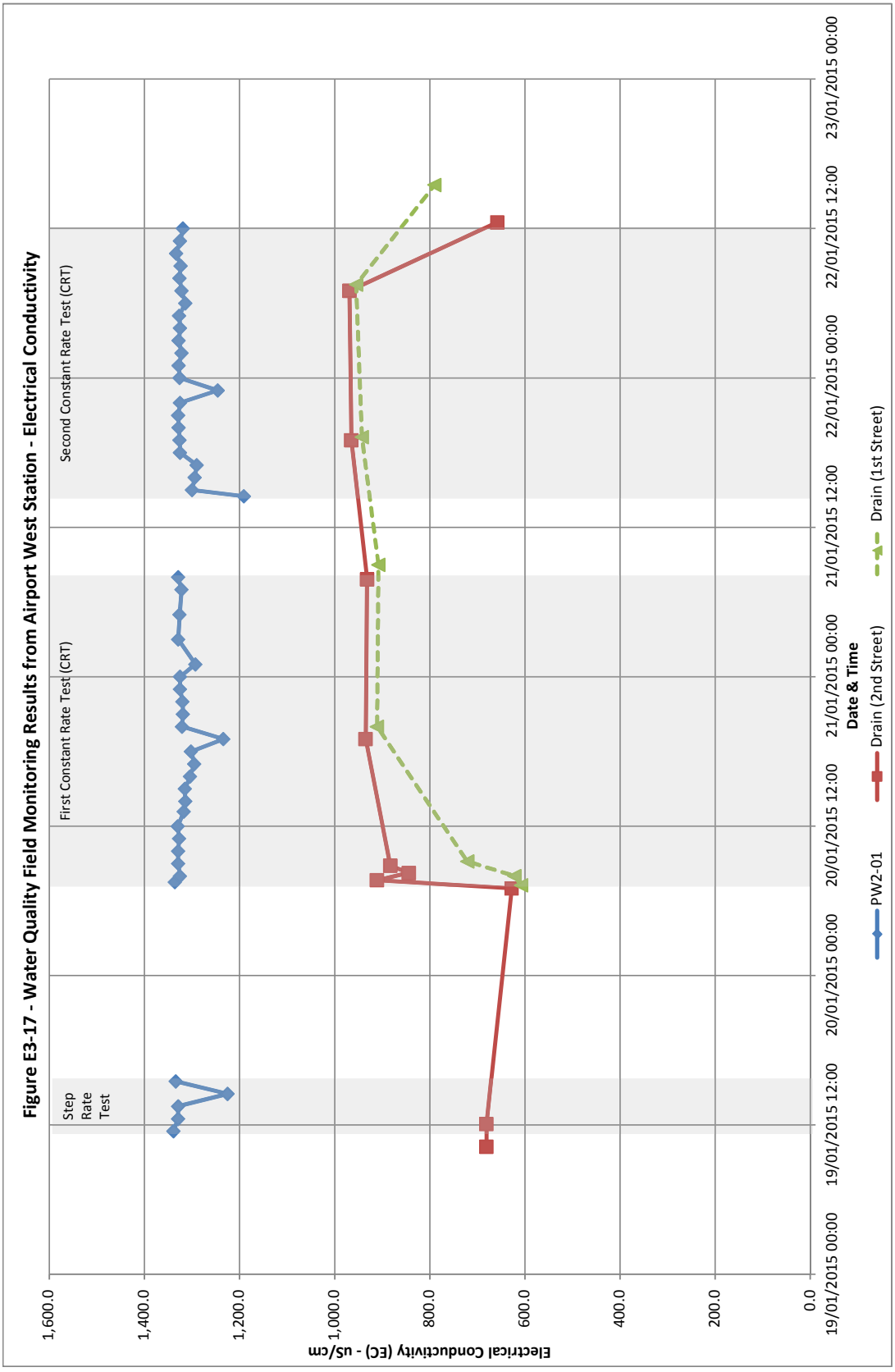
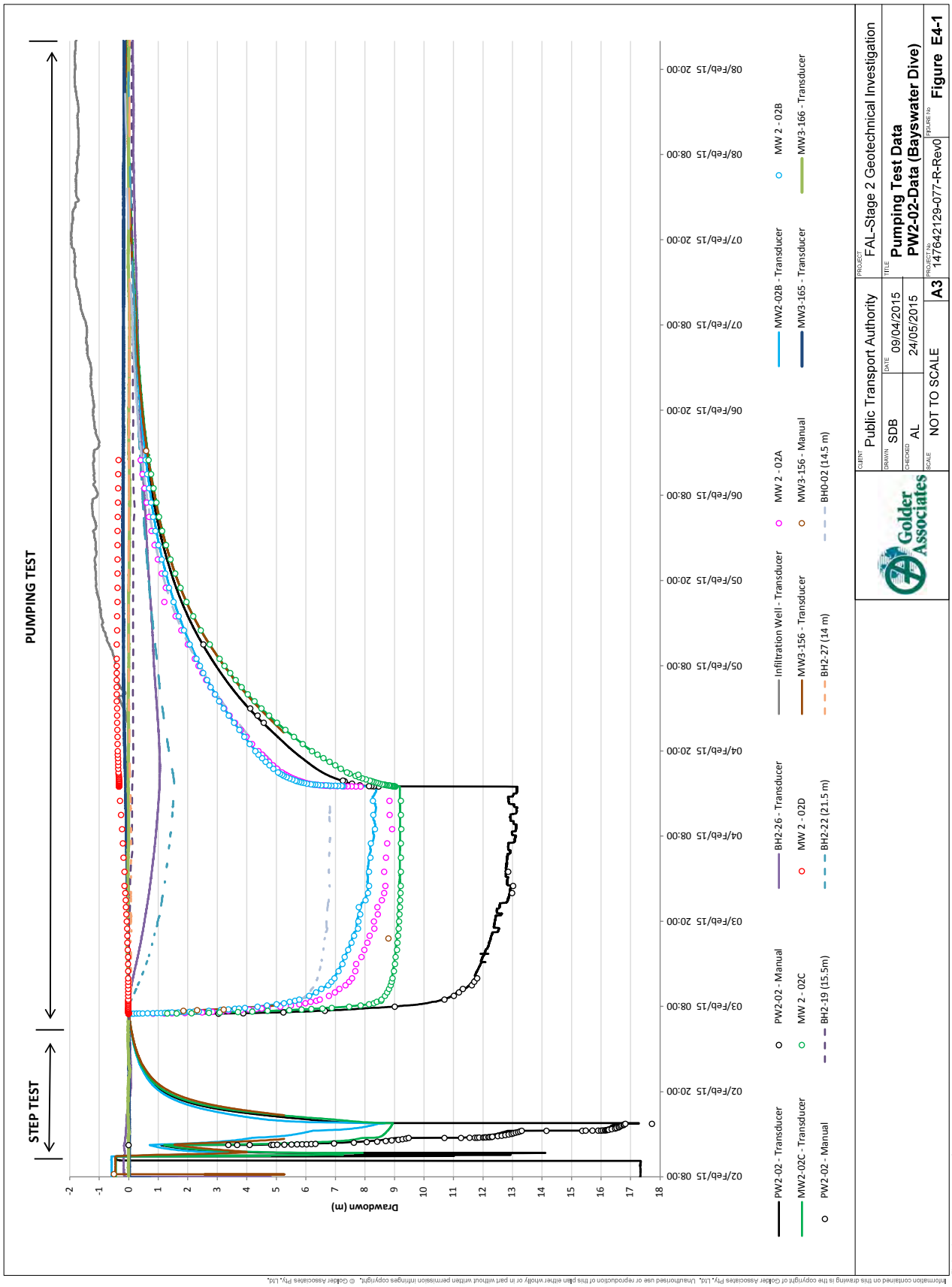


Figure E3-17 - Water Quality Field Monitoring Results from Airport West Station - Electrical Conductivity
Step Rate Test
First Constant Rate Test (CRT)
Second Constant Rate Test (CRT)
Electrical Conductivity (EC) - uS/cm
1,600.0
1,400.0
1,200.0
1,000.0
800.0
600.0
400.0
200.0
0.0
19/01/2015 00:00
19/01/2015 12:00
20/01/2015 00:00
20/01/2015 12:00
21/01/2015 00:00
21/01/2015 12:00
22/01/2015 00:00
22/01/2015 12:00
23/01/2015 00:00
Date & Time
PW2-01
Drain (2nd Street)
Drain (1st Street)



APPENDIX E
Summary of Test Pumping Analysis

ATTACHMENT E4
Constant Rate Test – Bayswater Dive Structure (PW2-02)


STEP TEST
PUMPING TEST
Drawdown (m)
-2
-1
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
02/Feb/15 08:00
02/Feb/15 20:00
03/Feb/15 08:00
03/Feb/15 20:00
04/Feb/15 08:00
04/Feb/15 20:00
05/Feb/15 08:00
05/Feb/15 20:00
06/Feb/15 08:00
06/Feb/15 20:00
07/Feb/15 08:00
07/Feb/15 20:00
08/Feb/15 08:00
08/Feb/15 20:00
PW2-02 - Transducer
MW2-02C - Transducer
PW2-02 - Manual
PW2-02 - Manual
MW 2 - 02C
BH2-19 (15.5m)
BH2-26 - Transducer
MW 2 - 02D
BH2-22 (21.5 m)
Infiltration Well - Transducer
MW3-156 - Transducer
BH2-27 (14 m)
MW 2 - 02A
MW3-156 - Manual
BH0-02 (14.5 m)
MW2-02B - Transducer
MW3-165 - Transducer
MW 2 - 02B
MW3-166 - Transducer
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CLIENT
Public Transport Authority
DRAWN
SDB
DATE
09/04/2015
CHECKED
AL
24/05/2015
SCALE
NOT TO SCALE
A3
PROJECT
FAL-Stage 2 Geotechnical Investigation
TITLE
Pumping Test Data
PW2-02-Data (Bayswater Dive)
PROJECT No
147642129-077-R-Rev0
FIGURE No
Figure E4-1
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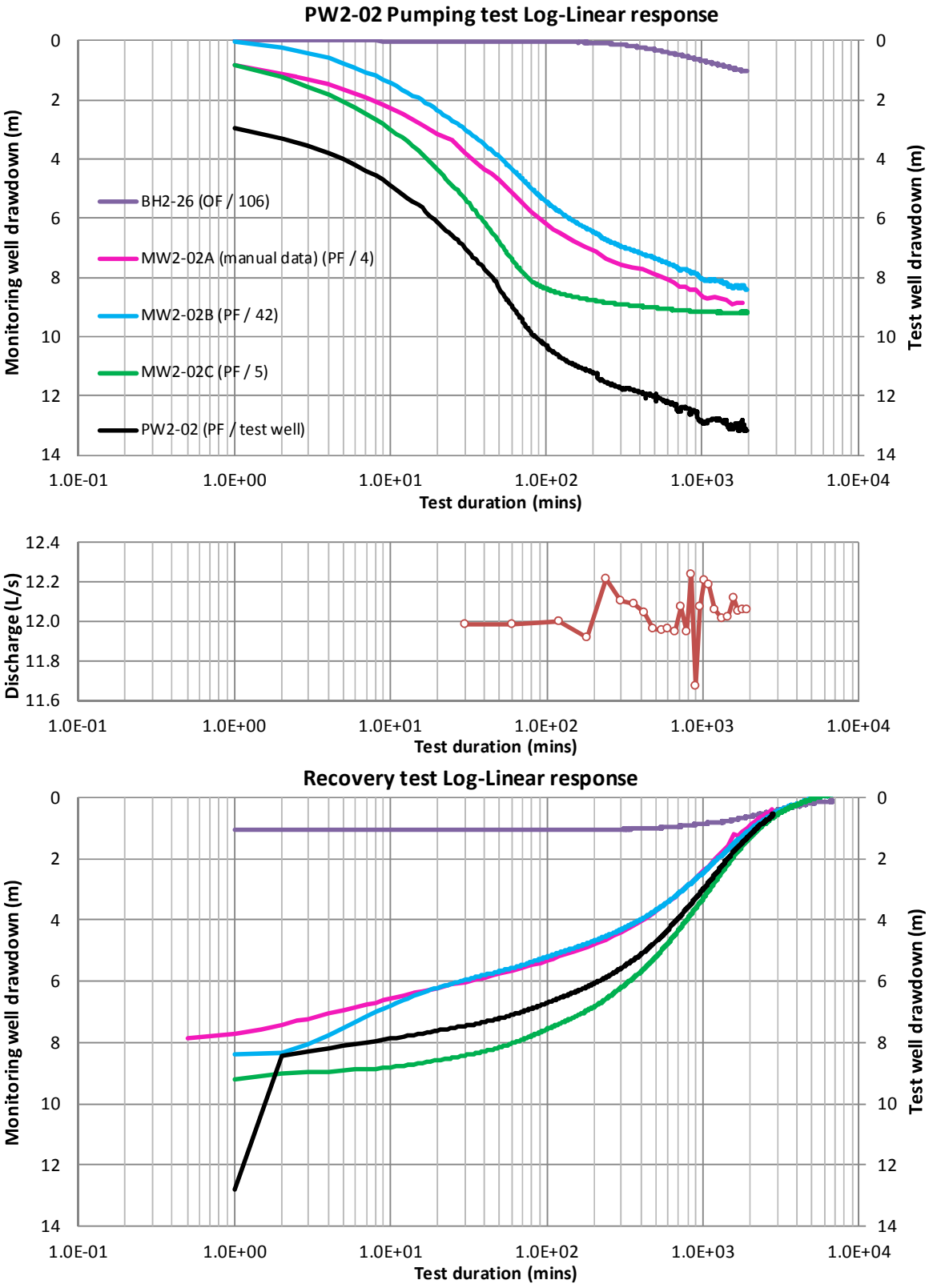

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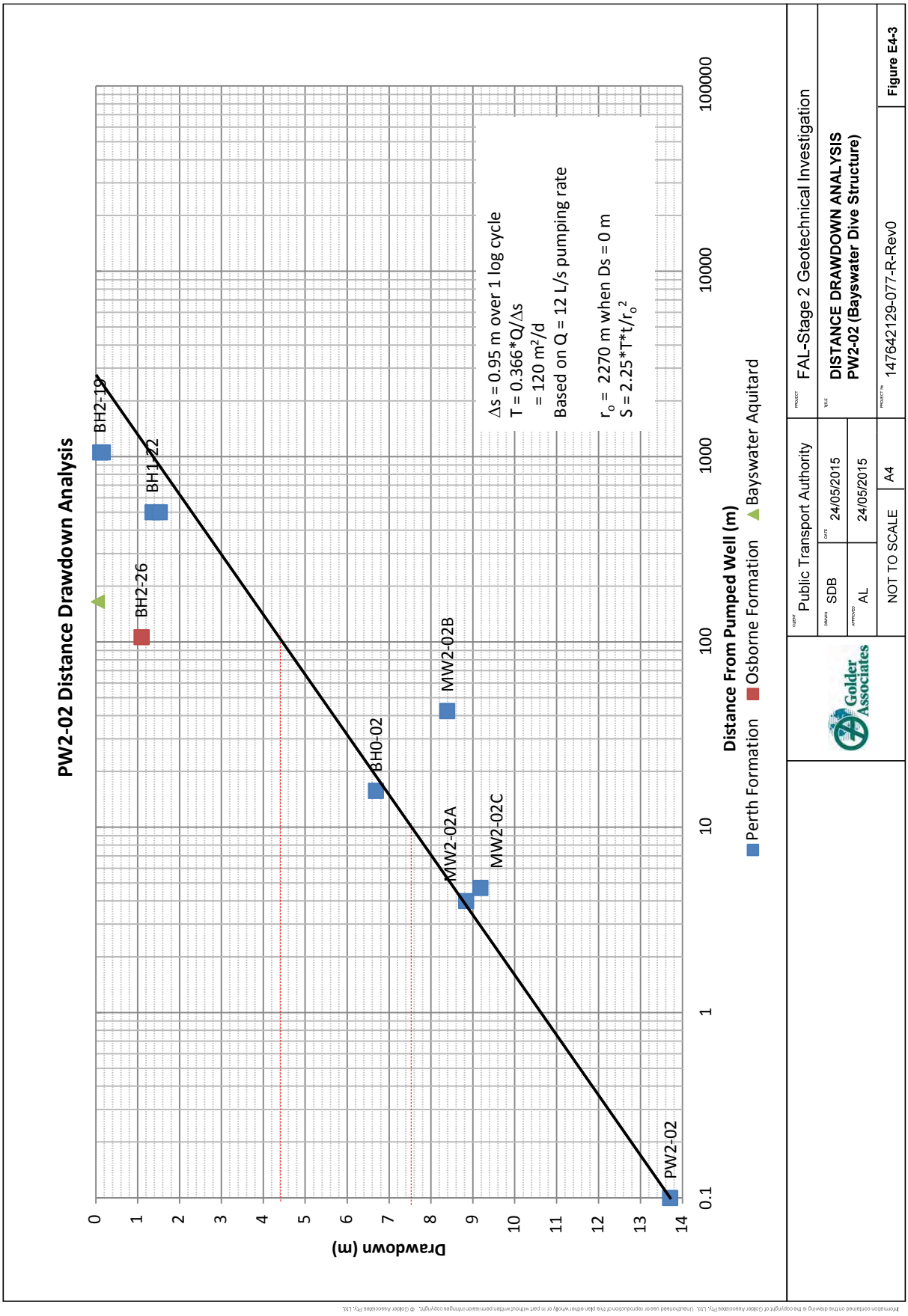
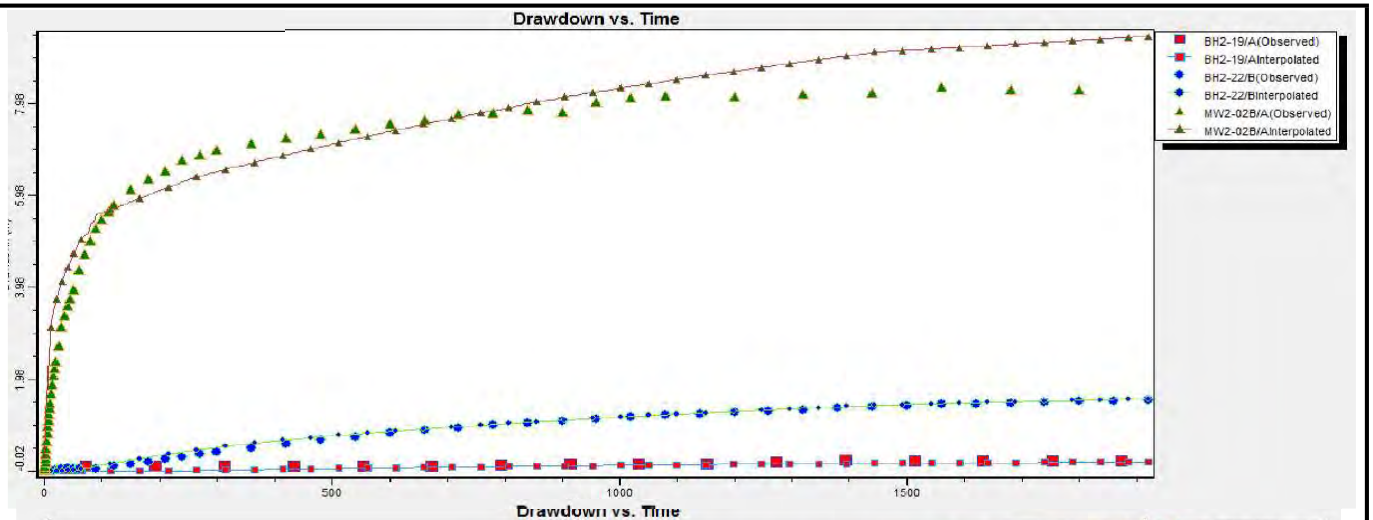
CLIENT	Public Transport Authority	PROJECT	FAL-Stage 2 Geotechnical Investigation
DRAWN	SDB	DATE 24/05/2015	TITLE Pumping Test Data PW2-02 (Bayswater Dive Structure)
CHECKED	AL	DATE 24/05/2015	
SCALE	NOT TO SCALE	A4	PROJECT No 147642129-077-R-Rev0
FIGURE No	Figure E4-2		

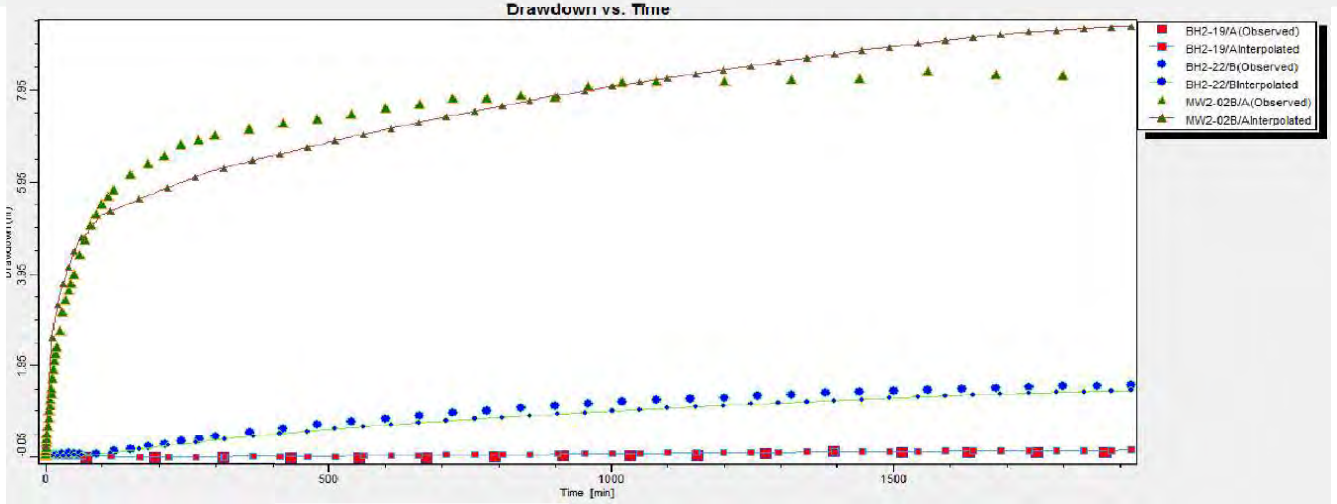
File Location:


PW2-02 Distance Drawdown Analysis
Drawdown (m)
Distance From Pumped Well (m)
Perth Formation
Osborne Formation
Bayswater Aquitard
BH2-19
BH1-22
BH2-26
MW2-02B
BH0-02
MW2-02A
MW2-02C
PW2-02
Δs = 0.95 m over 1 log cycle
T = 0.366*Q/Δs
= 120 m²/d
Based on Q = 12 L/s pumping rate
r_o = 2270 m when Ds = 0 m
S = 2.25*T*t/r_o²
Public Transport Authority
FAL-Stage 2 Geotechnical Investigation
DISTANCE DRAWDOWN ANALYSIS
PW2-02 (Bayswater Dive Structure)
SDB
24/05/2015
AL
24/05/2015
NOT TO SCALE
A4
147642129-077-R-Rev0
Figure E4-3

Specific Storage =	1.00E-06 1/m
Transmissivity =	60 m2/d
Hydraulic Conductivity	
K(GF)h	4 m/d
K(GF)v	0.4 m/d



Specific Storage =	5.00E-06 1/m
Transmissivity =	60 m2/d
Hydraulic Conductivity	
K(GF)h	4 m/d
K(GF)v	4 m/d



Screenshot from MODFLOW output

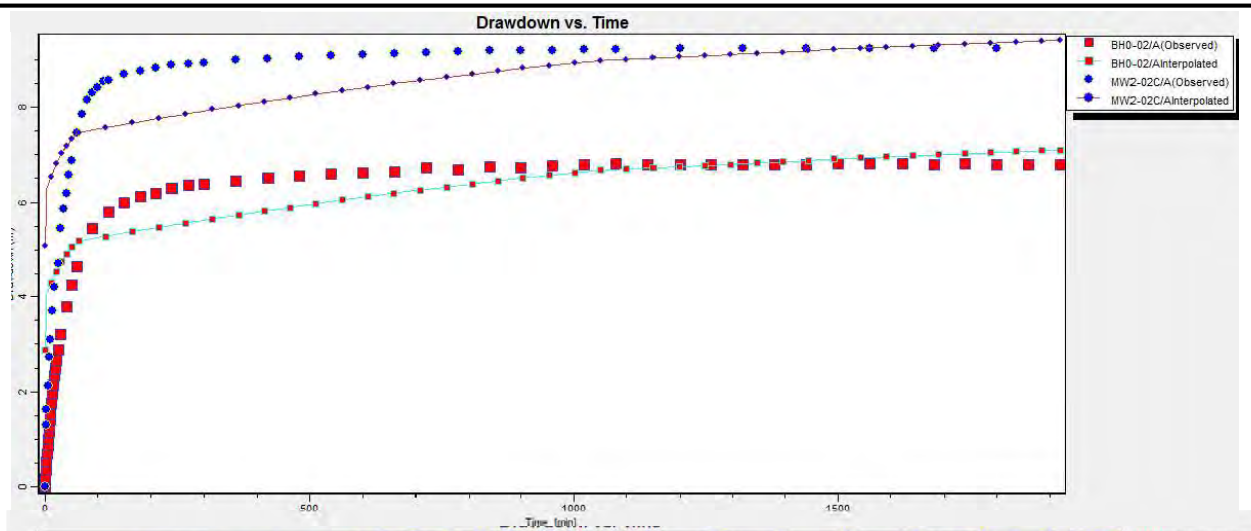
Graphs for BH2-19, BH2-22 and MW2-02B (further from PW2-02)

Golder Associates

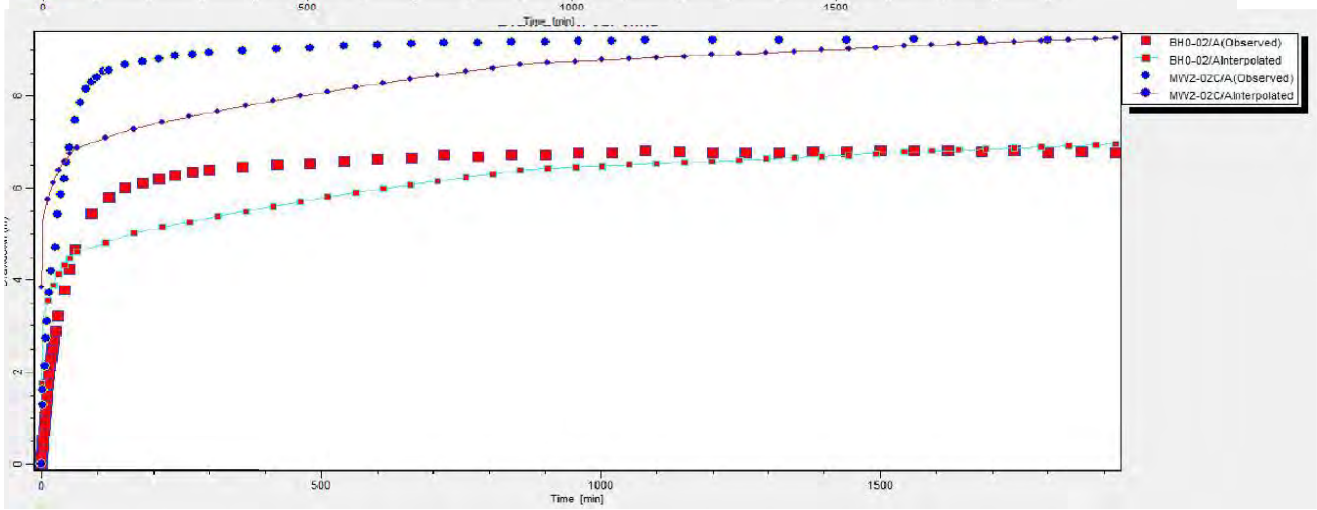
CLIENT	Public Transport authority	PROJECT	FAL - Stage 2 Geotechnical Investigation
DRAWN	AL	DATE	24-May-15
CHECK	DPS	DATE	24-May-15
SCALE	Not To Scale	A4	
TITLE	Modelled Groundwater Level Drawdown PW2-02 Pumping Test (Bayswater) Forrestfield Dive Structure		
PROJECT No.	147642129-077-R-Rev0	FIGURE No.	E4-4

C:\Users\alundorf\Desktop\FAL\APPENDIX E\Appendix E - Test Pumping Results\Attachment E4\Figure E4-4&E4-5.xlsx

Specific Storage = 1.00E-06 1/m
Transmissivity = 113 m2/d
Hydraulic Conductivity
K(GF)h 7.5 m/d
K(GF)v 0.75 m/d



Specific Storage = 5.00E-06 1/m
Transmissivity = 113 m2/d
Hydraulic Conductivity
K(GF)h 7.5 m/d
K(GF)v 7.5 m/d



Screenshot from MODFLOW output

Graphs for BH0-02 and MW2-02C (Closer to PW2-02)

Golder Associates

CLIENT	Public Transport authority	PROJECT	FAL - Stage 2 Geotechnical Investigation
DRAWN	AL / DATE 24-May-15	TITLE	Modelled Groundwater Level Drawdown PW2-02 Pumping Test (Bayswater) Forrestfield Dive Structure
CHECK	DPS / DATE 24-May-15		
SCALE	Not To Scale / A4	PROJECT No.	147642129-077-R-Rev0
		FIGURE No.	E4-5

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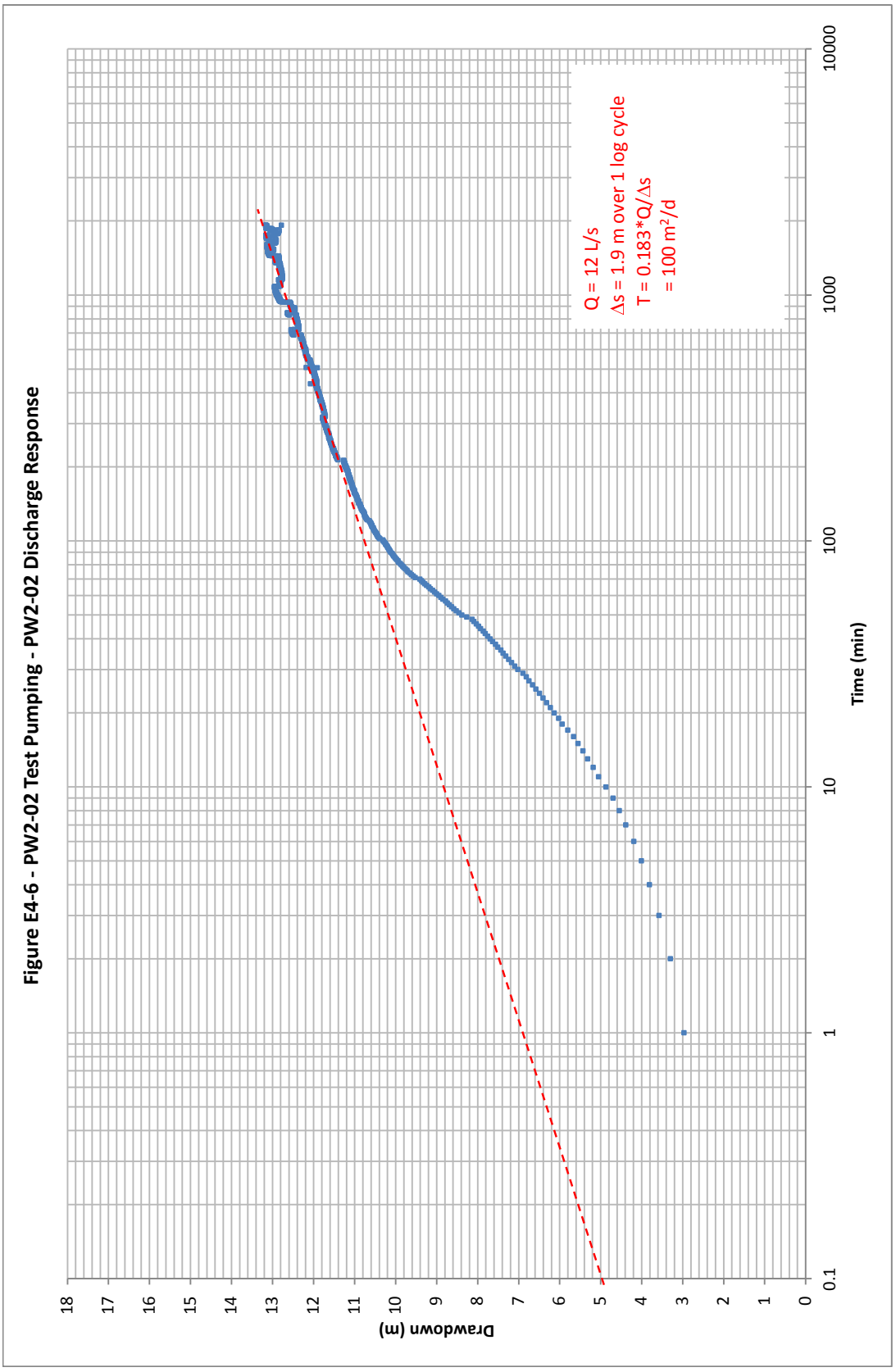


Figure E4-6 - PW2-02 Test Pumping - PW2-02 Discharge Response
Drawdown (m)
Time (min)
Q = 12 L/s
Δs = 1.9 m over 1 log cycle
T = 0.183*Q/Δs
= 100 m²/d
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
0.1
1
10
100
1000
10000

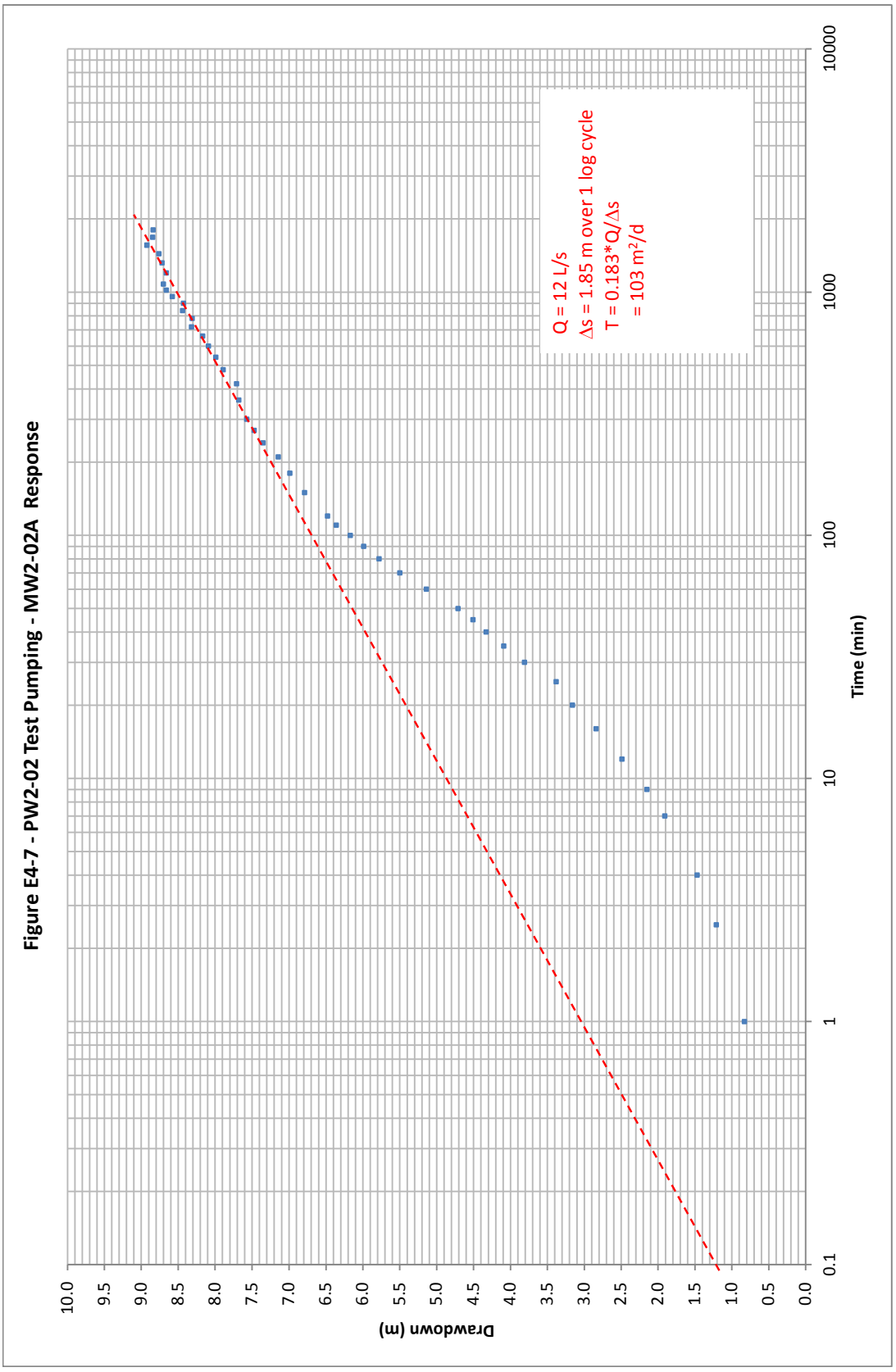


Figure E4-7 - PW2-02 Test Pumping - MW2-02A Response
Drawdown (m)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Time (min)
0.1
1
10
100
1000
10000
Q = 12 L/s
Δs = 1.85 m over 1 log cycle
T = 0.183*Q/Δs
= 103 m²/d

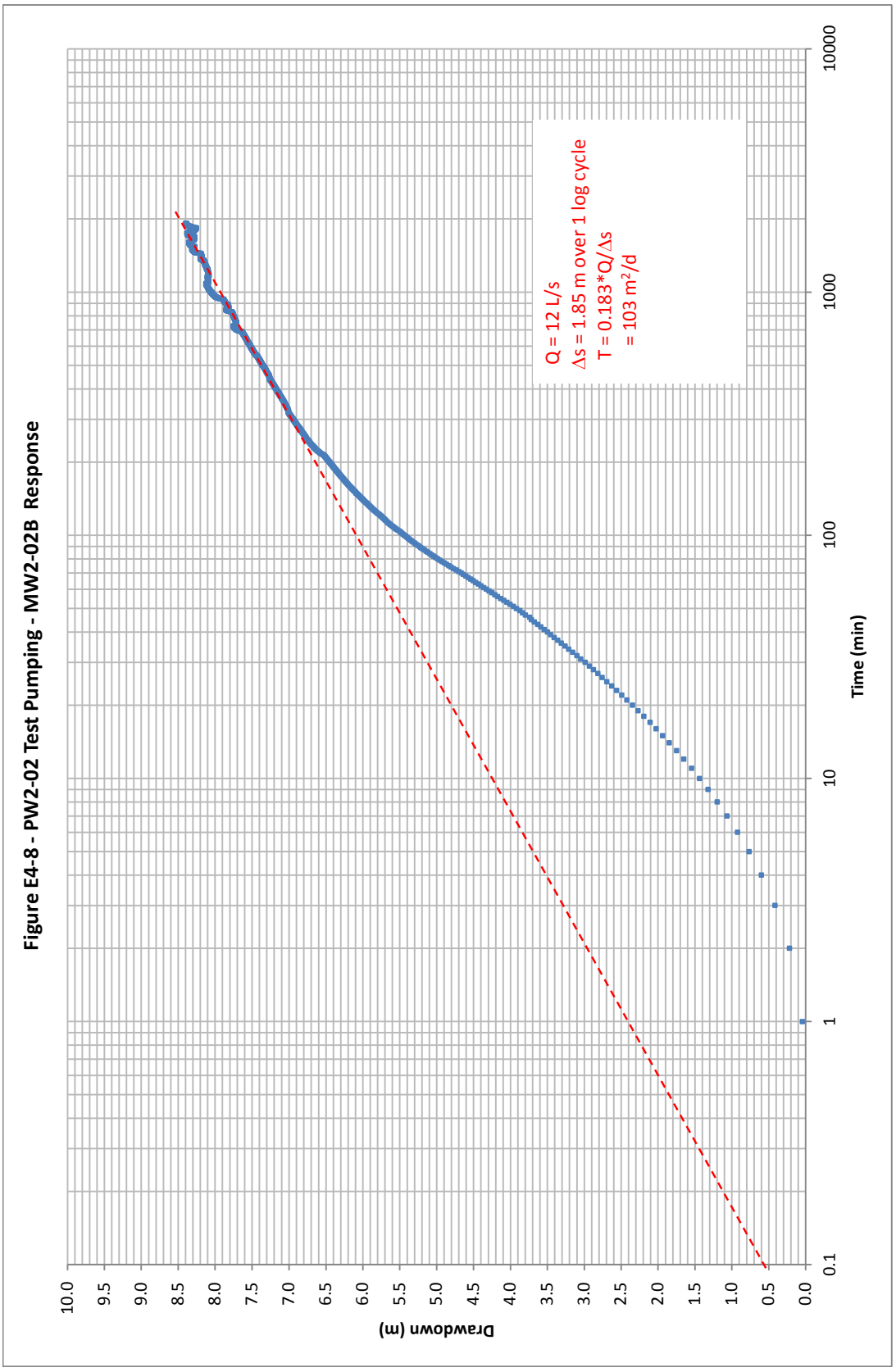


Figure E4-8 - PW2-02 Test Pumping - MW2-02B Response
Drawdown (m)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
1
10
100
1000
10000
Time (min)
Q = 12 L/s
Δs = 1.85 m over 1 log cycle
T = 0.183*Q/Δs
= 103 m²/d

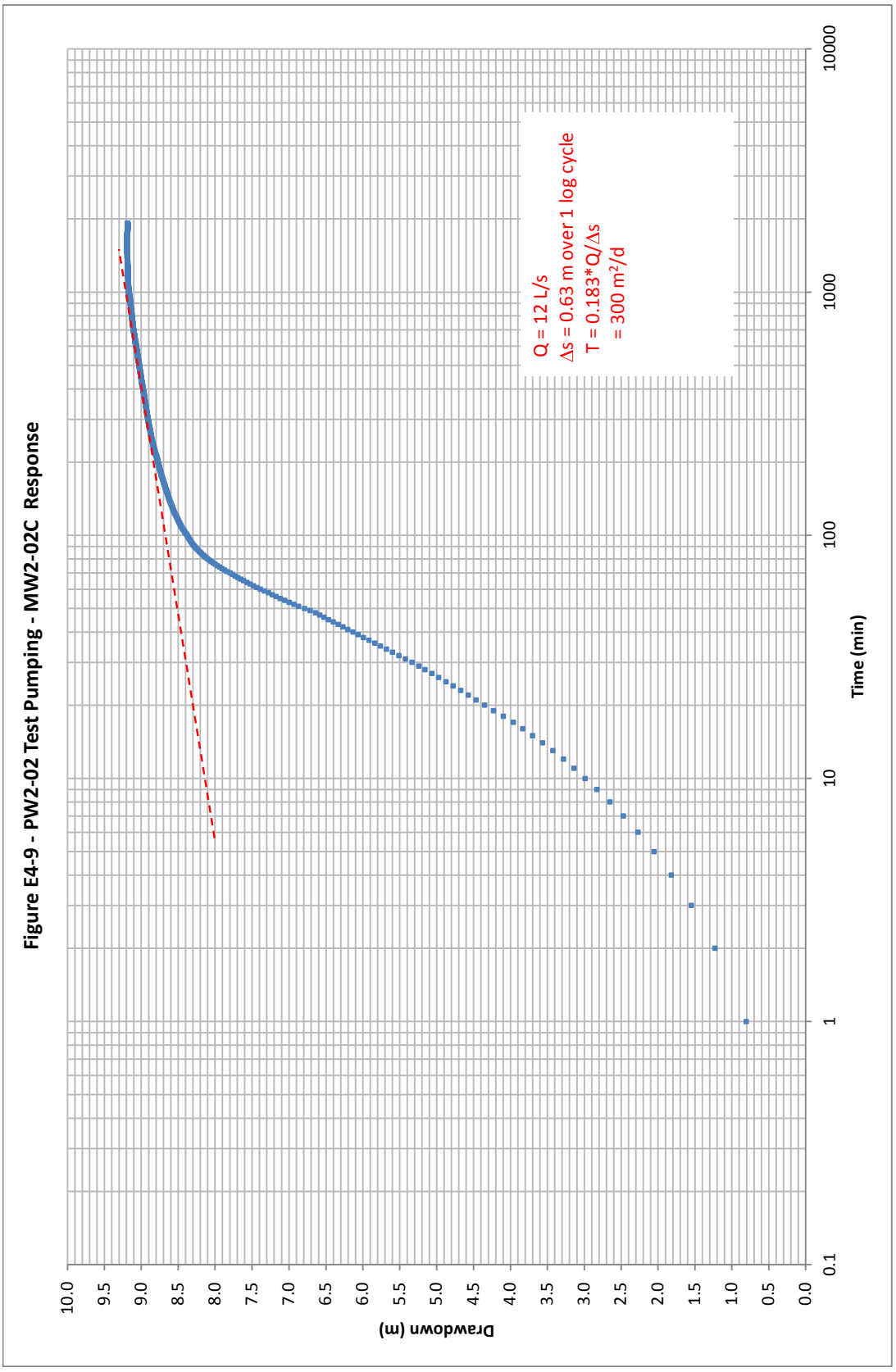


Figure E4-9 - PW2-02 Test Pumping - MW2-02C Response
Drawdown (m)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
1
10
100
1000
10000
Time (min)
Q = 12 L/s
Δs = 0.63 m over 1 log cycle
T = 0.183*Q/Δs
= 300 m²/d

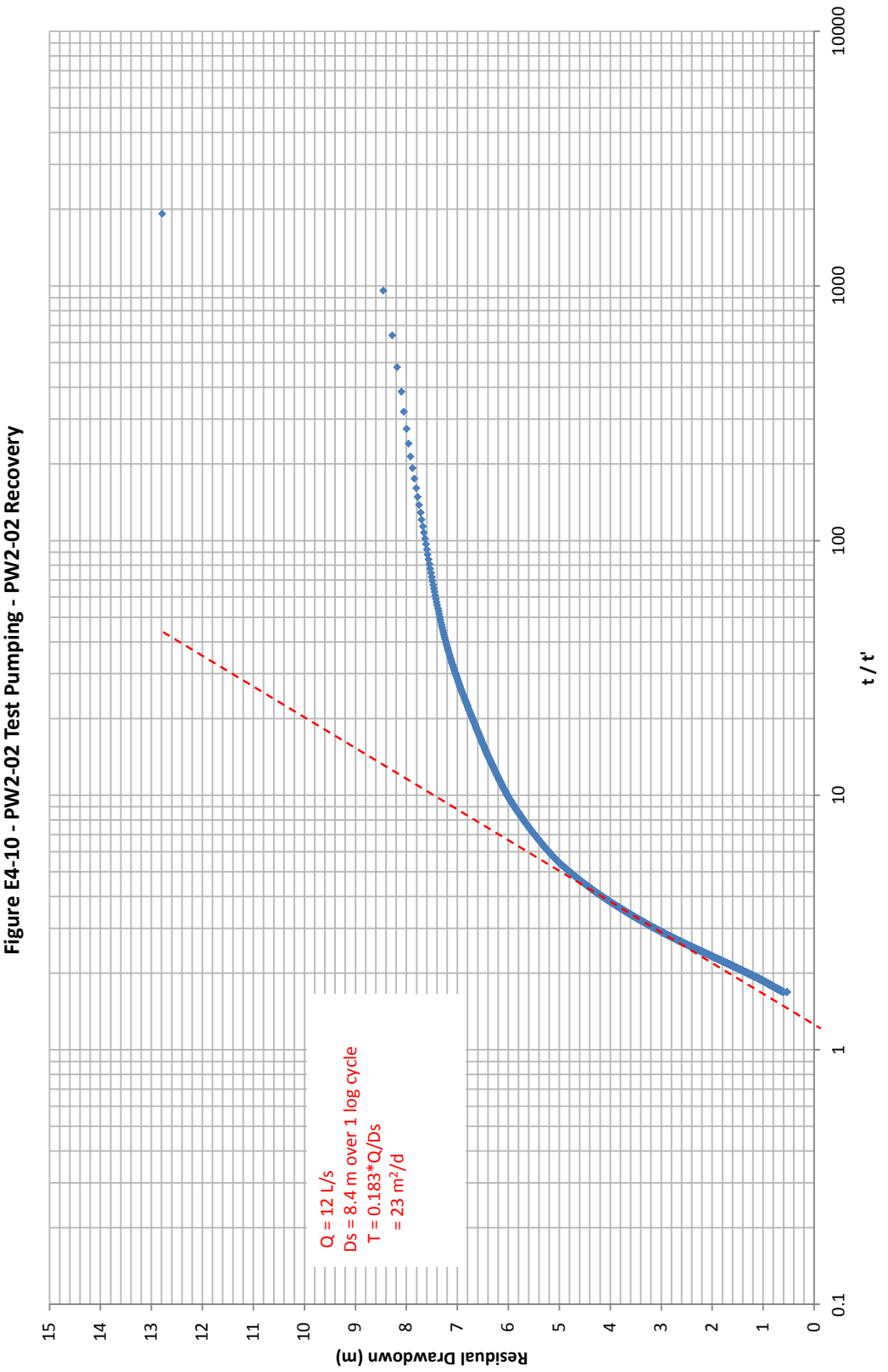


Figure E4-10 - PW2-02 Test Pumping - PW2-02 Recovery

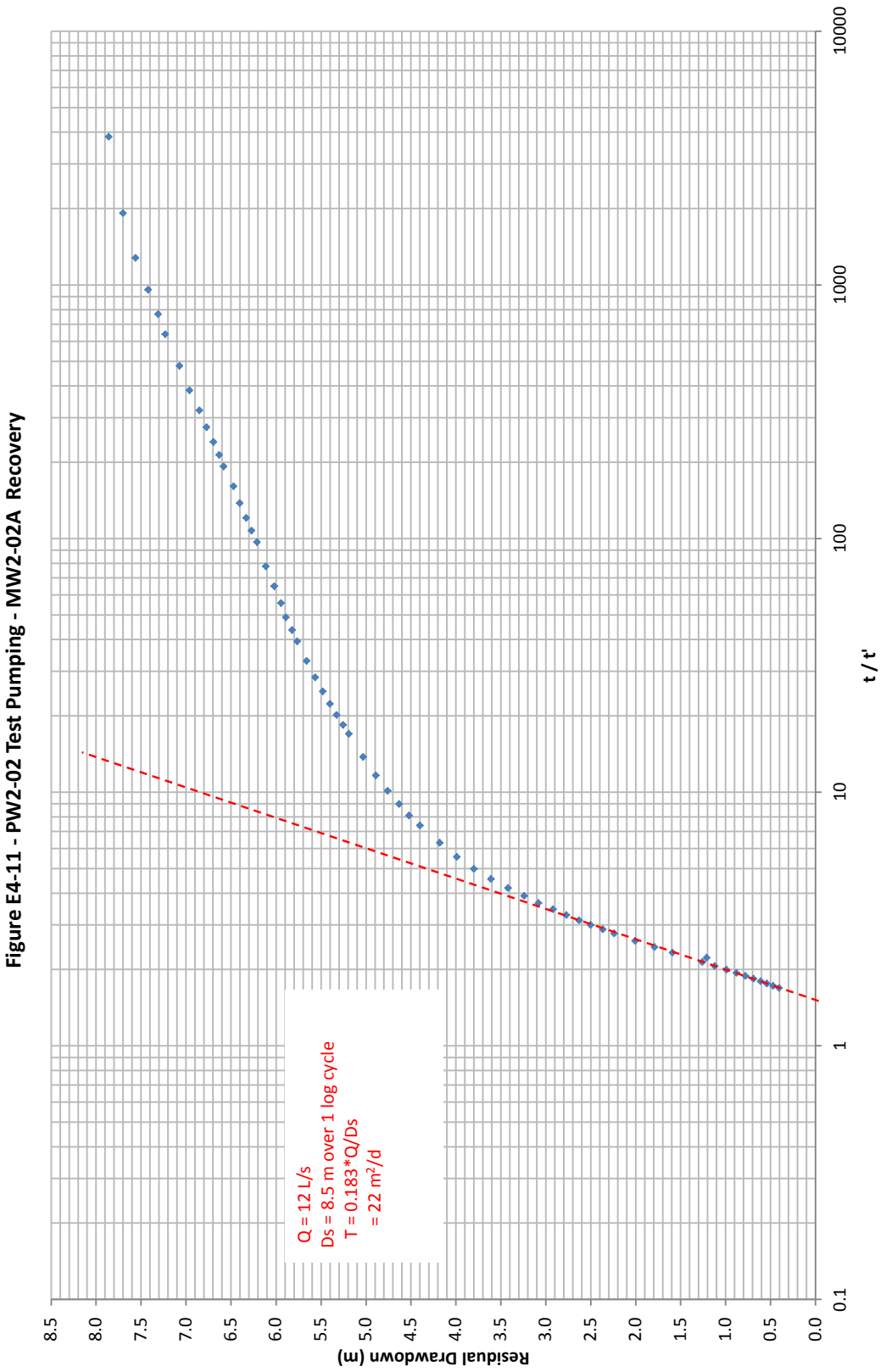


Figure E4-11 - PW2-02 Test Pumping - MW2-02A Recovery
Residual Drawdown (m)
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
1
10
100
1000
10000
t / t'
Q = 12 L/s
Ds = 8.5 m over 1 log cycle
T = 0.183*Q/Ds
= 22 m²/d

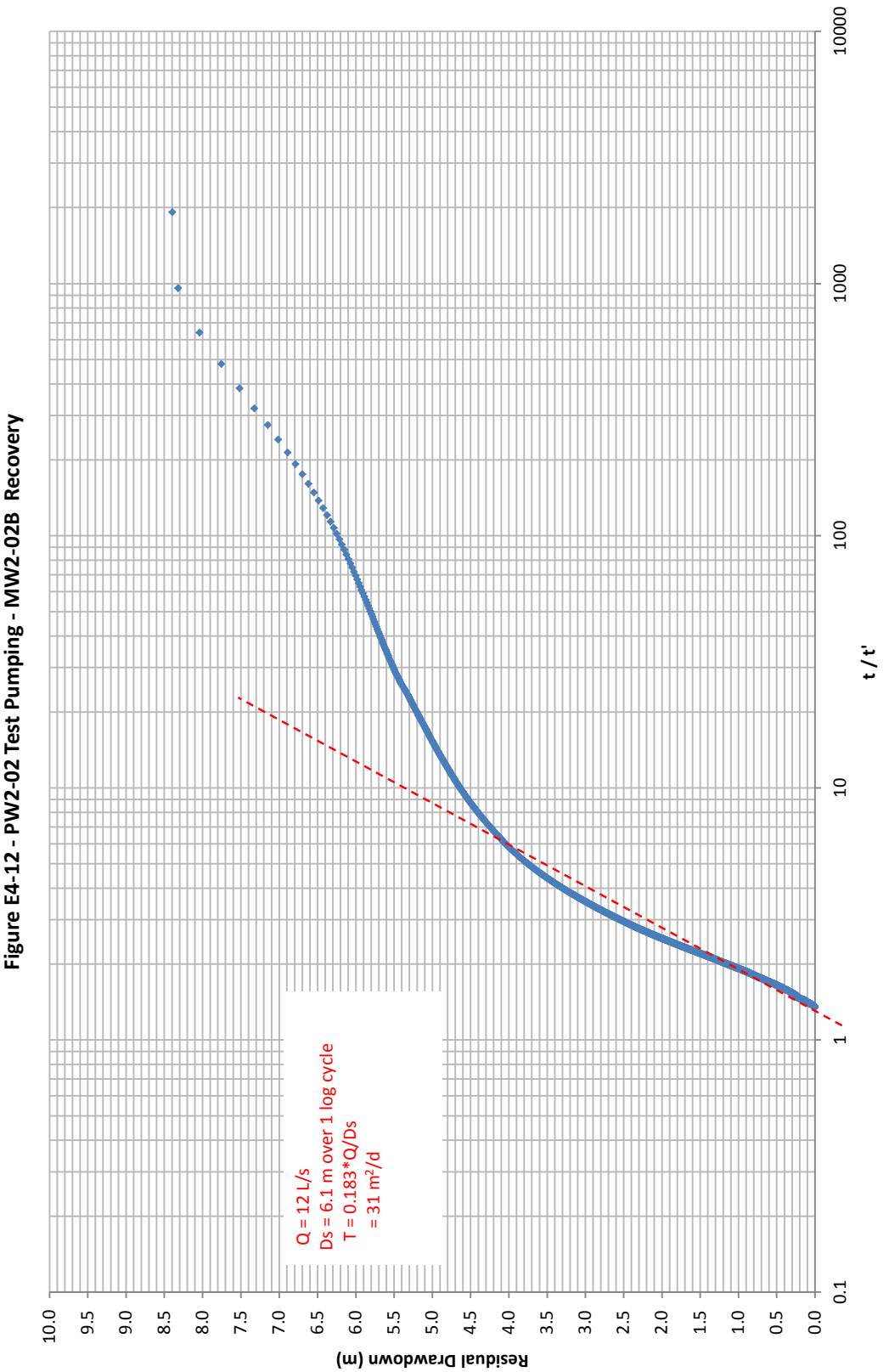


Figure E4-12 - PW2-02 Test Pumping - MW2-02B Recovery
Residual Drawdown (m)
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.1
1
10
100
1000
10000
t / t'
Q = 12 L/s
Ds = 6.1 m over 1 log cycle
T = 0.183*Q/Ds
= 31 m²/d

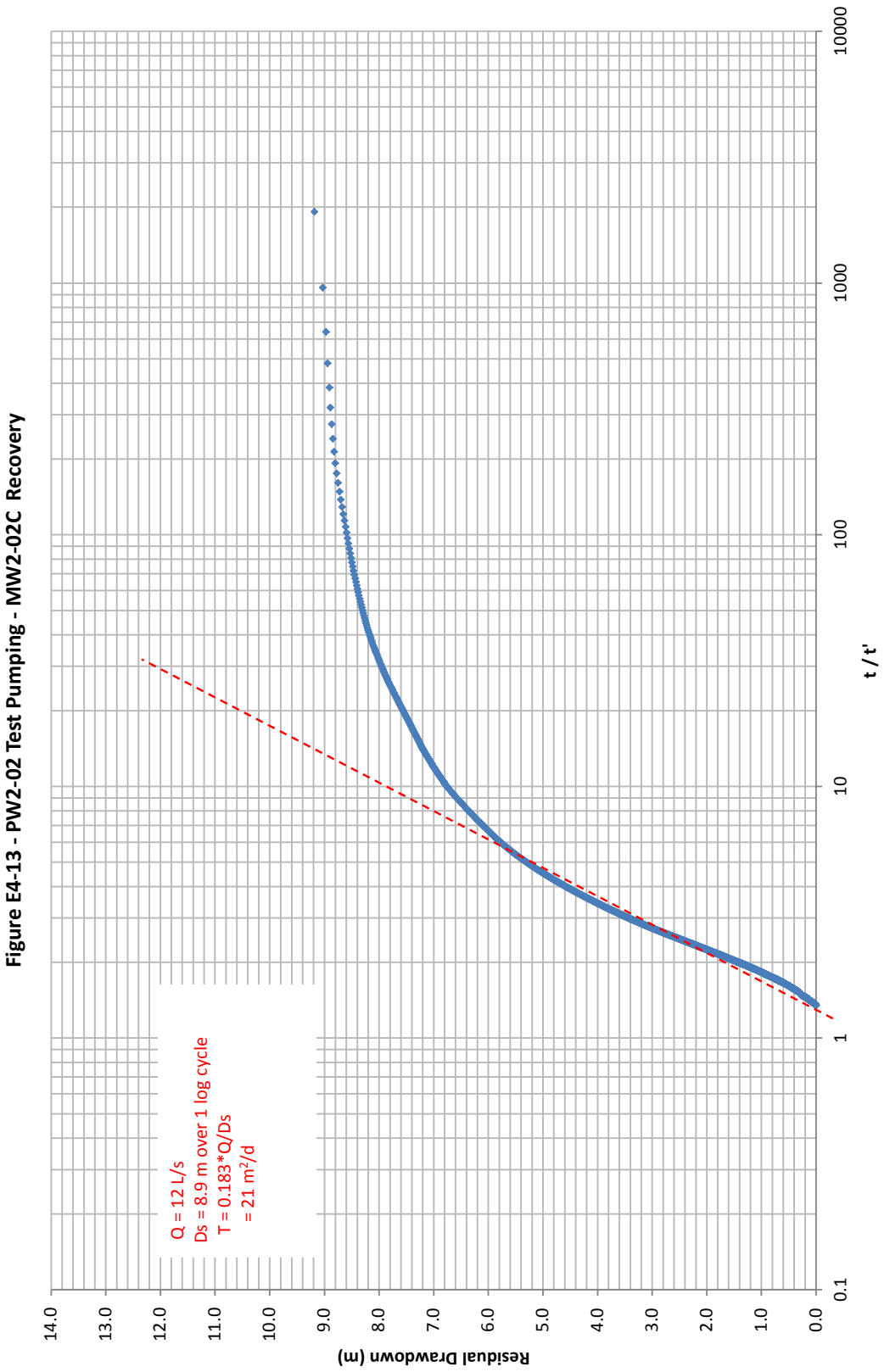


Figure E4-13 - PW2-02 Test Pumping - MW2-02C Recovery

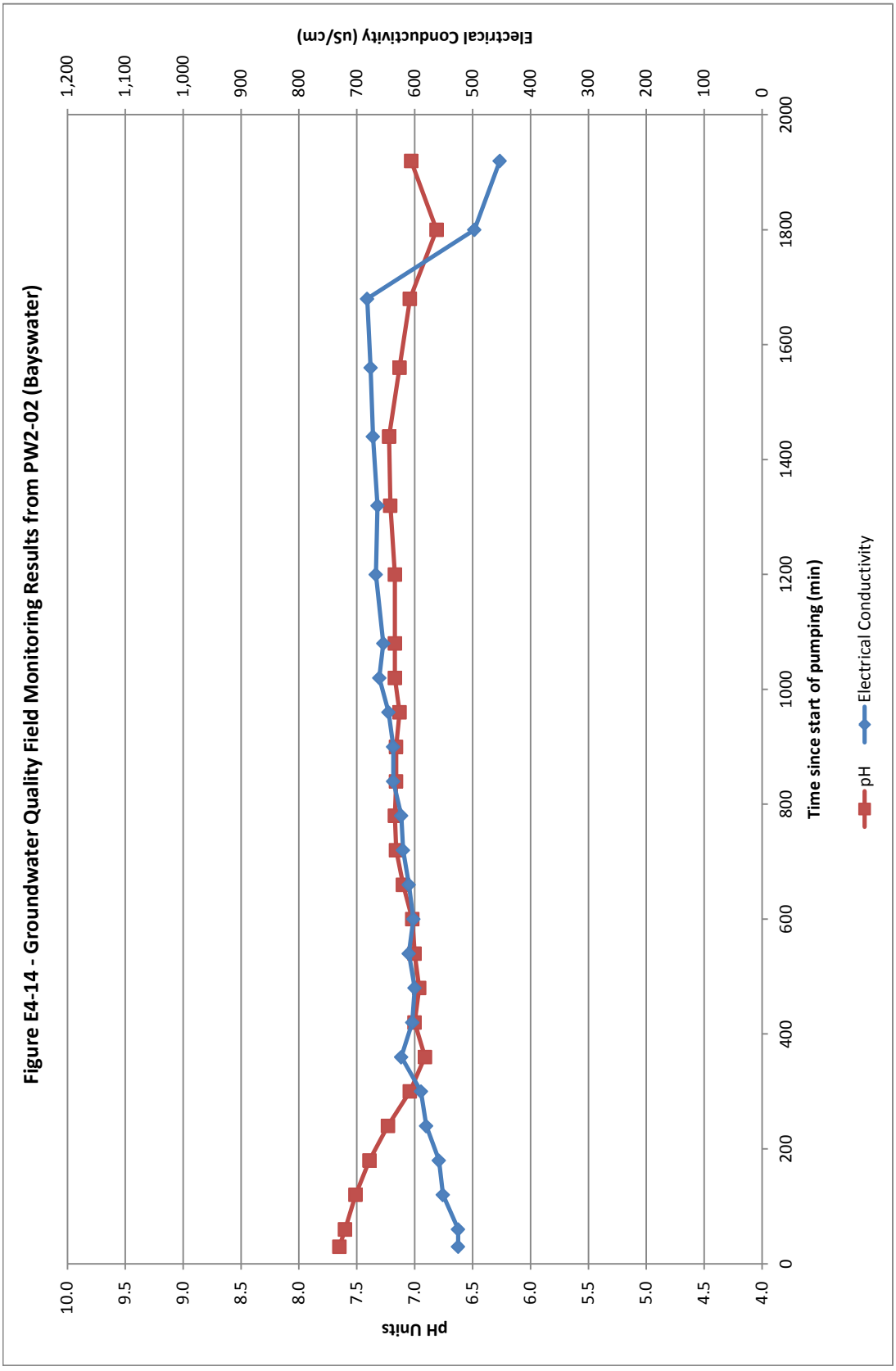


Figure E4-14 - Groundwater Quality Field Monitoring Results from PW2-02 (Bayswater)
pH Units
Electrical Conductivity (uS/cm)
Time since start of pumping (min)
pH
Electrical Conductivity
10.0
9.5
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
1,200
1,100
1,000
900
800
700
600
500
400
300
200
100
0
0
200
400
600
800
1000
1200
1400
1600
1800
2000

APPENDIX F

Description of Preliminary Groundwater Model Setup

May 2015
Report No. 147642129-077-R-Rev0

Golder Associates

APPENDIX F
Groundwater Model Construction and Calibration

1.0 INTRODUCTION

This appendix provides details on the construction and assumptions used in the 3D numerical groundwater model. Golder prepared a basic groundwater model in 2014 (Golder, 2014b). This model was used as the platform and updated to include more detailed information on the geology and hydrogeological properties.

The modelling development of the groundwater has been carried out in general accordance with the guiding principles in the Australian Groundwater Modelling Guidelines (NWC, 2012).

The objective of the preliminary groundwater model is to estimate the effect the construction dewatering could have on the groundwater level and piezometric levels and to provide preliminary dewatering rates by modelling conceptual dewatering designs. The model is still considered preliminary (Class 1 according to NWC, 2012) and further detailing of the groundwater model could be required during the dewatering design (Class 2 according to NWC, 2012).

2.0 MODEL ASSUMPTIONS

The following is a list of assumptions used in the design of the preliminary 3D groundwater model:

- Geology beneath the proposed alignment is representative of the geology to the north and south of the alignment and is relatively consistent across the modelled area. Publicly available information, where available, has been utilised to assist with creating the geological surfaces within the model domain outside of the tunnel alignment (e.g. extent and depth of the Mirrabooka Aquifer). However, this data can only be considered approximate.
- Each geological unit is homogenous and isotropic in terms of hydraulic properties with the exception of:
  - The ratio of vertical to horizontal hydraulic conductivity ranges between 1:1 and 1:10 (i.e. $k_v/k_h = 1$ to $k_v/k_h = 0.1$). These ratios were considered appropriate because:
    - The majority of the geological units represented in the model are sedimentary and have been deposited in fluvial environments.
    - Our professional experience on other projects in the Perth Metropolitan area indicates similar ratios.
    - Calibration of pumping test data indicated a best fit using such ratios.
  - Ascot Formation was divided into two areas of different hydraulic conductivity based on the results of the pumping tests which indicate higher hydraulic conductivity in the eastern part of this unit.
  - The Mirrabooka Member was divided into two layers of different hydraulic conductivity to simulate an interpreted decrease in hydraulic conductivity with depth based on bore logs and results from the pumping test.
- Diaphragm walls (D-walls) will be constructed around the perimeter (except for the entrance to the dive structures) of the excavations prior to the start of any excavation and dewatering. The proposed D-walls have been simulated using a low hydraulic conductivity, which allows for simulation of some seepage through the walls.
- Recharge and evapotranspiration are uniform spatially and temporally across the model. Seasonal variation has not been included in the model during transient model runs as this would mask the objective of assessing the changes in groundwater level and piezometric level caused by the construction dewatering. There is no recharge within the D-walls during the dewatering operation as it is assumed that any rainfall over the site will be pumped off site immediately.
- The model does not consider off-site influences, such as groundwater abstraction from other sites, which could affect the groundwater behaviour within the model.

2.1 Model Setup

The model setup and main input parameters are described in the sections below.

2.1.1 Extent and Grid Sizing

The model domain is 10.8 km long and 8.7 km wide (Figure F1). The model extent was selected so that the boundaries would be set at distances outside the influence of groundwater drawdown caused by the dewatering.

The grid sizing within the model ranges from 50 m by 50 m (mainly at the model peripheral boundaries) to 5 m by 5 m around the main structures where dewatering is required. This variation in grid size allows for greater resolution near the proposed structures. The model consists of a total of 438 rows and 493 columns.

2.1.2 Layers

The model has been divided into 13 layers to represent the main geological units. The geological surfaces have been obtained from the geological long section (Golder, 2015b) and geological control points away from the tunnel alignment. The reason for the additional model layers is to allow for the termination of the D-walls within a geological unit and to model variability of hydraulic conductivity with depth. Figure F2 shows a cross-section of the model along Row 111 (refer to Section A – A' on Figure F1 for alignment - given the orientation of the model the software does not allow for a cross-sectional representation along the alignment). Table F1 presents the different geological units per layer.

Table F1: Summary of Groundwater Model Layers.

Layers	West Zone			East Zone	
1	Bassendean Sand/Perth Formation Sand	Swan River Formation		Bassendean Sand, Guildford Formation, Gnangara Sand	Guildford Formation
2					
3	Perth Formation Clay				
4					
5					
6	Perth Formation Sand			Ascot Formation	
7					
8	Kardinya Shale (Osborne Formation)			Mirrabooka Member (Osborne Formation)	Kardinya Shale (Osborne Formation)
9					
10				Kardinya Shale (Osborne Formation)	
11					
12					
13					

The elevation of the Layer 1 is based on the current smoothed topography ranging from approximately RL 0 m AHD at the Swan River to RL 50 m AHD at the eastern boundary of the model towards the Darling Scarp. The bottom of Layer 13 is flat with an elevation of RL -100 m AHD, based on the inferred depth of the Kardinya Shale in Davidson (1995).

2.2 Steady-State Model Boundaries

Boundary conditions are assigned in the model to control how water will flow into and out of the model. It is preferable that natural boundaries are used where they exist, otherwise boundaries should be set far enough away that they do not influence what is occurring within the model. Figure F1 shows the assigned boundary locations.

2.2.1 General Head Boundaries

General head boundaries (GHB) were assigned at the north-western and south-eastern boundaries of the model in all layers. The boundaries were set using off-set groundwater level contours estimated from both the Perth Groundwater Atlas (WRC, 1997) and observed groundwater levels.

2.2.2 Constant Head Boundaries

A constant head boundary at an elevation of RL 0.2 m AHD was assigned to the portions of the model representing the Swan River.

2.2.3 Drain Boundaries

A river function was applied to represent the Bayswater Main Drain and the Belmont Main Drain; the locations of these drains are shown in Figure F1. River functions were used in preference to drain functions as the two drains modelled contain a phreatic surface for a significant proportion of the year. In the event that dewatering results in drawdown which extends to these drains, the use of a river function will allow water to flow from the drain into the aquifer (i.e. the drains could start giving water to the aquifer rather than draining it from the aquifer).

2.2.4 No-flow Boundaries

Inactive cells or no-flow boundaries were assigned to large portions of the north-eastern and south-western portions of the model because of the model geometry relative to the alignment. The areas within these inactive cells are unlikely to experience any influence from the proposed dewatering and are roughly perpendicular to groundwater flow direction. Assigning inactive flow to these large areas helps to improve accuracy and increase speed of computation. The bottom of the model was also considered a no flow boundary.

2.2.5 Recharge and Evapotranspiration

A spatially and temporally uniform recharge value of 100 mm/year (approximately 12% of Mean Annual Rainfall) was applied across the whole model. Evapotranspiration was applied to the model with an extinction depth of 1.5 m (i.e. the evapotranspiration function will only be active when the groundwater level comes within 1.5 m of the model surface, which mainly occurs in the Perth Airport area. Given that the whole of Perth Airport contains a large network of closely spaced drains to control the groundwater level at the airport estate (the Perth Airport area used to be a wetland area), the evapotranspiration function was used to simulate the dense network of drains rather than incorporating all the drains in the model using drain functions.

2.3 Transient Model (Non-steady State) Boundaries

The following input parameters are assigned for transient modelling components of the groundwater model.

2.3.1 Diaphragm Walls (D-walls)

D-walls were introduced into the model at varying depths for each dive structure and at the two stations and were based on the Rev F alignment plan and Reference Design Summary Table RevB:

- The D-walls will terminate 3 m into the Osborne Formation at the Airport West Station and Consolidated Airport Terminal Station.
- At the Bayswater and Forrestfield Dive Structures the D-Walls will extend to a depth that is about 1.9 and 2.3 times the depth of excavation, respectively.

The D-walls are introduced around the structures as cells with a low hydraulic conductivity. The hydraulic conductivity of the walls was varied between 0.001 m/d and 0.01 m/d to simulate some leakage through the D-walls.

2.3.2 Dewatering Method

Dewatering will likely be undertaken using a combination of dewatering wells, dewatering spears and sump pumping. The dewatering in the groundwater model was simulated by using drain cells assigned a particular drain elevation equivalent to 1m below the excavation base. For dive structures the drain elevation was inserted along a linear gradient from the surface (no dewatering) to the maximum dewatering elevation at the portal of the dive structure.

2.3.3 Recharge Wells

To simulate water being re-injected back into the aquifer, the water was introduced back into the groundwater model using the well function. The recharge wells were generally screened across the whole Superficial Aquifer to a similar depth as the D-walls. It was assumed in the model that the long-term recharge rate of a recharge well would not exceed 1 L/s.

3.0 MODEL CALIBRATION

3.1 Steady-State Calibration

The steady state model was run to simulate the estimated average seasonal groundwater level contours and groundwater flow direction based on the March 2015 groundwater level survey. The groundwater level results from the steady state calibration were then used as input into the transient (non-steady state) simulation.

The model boundaries and hydraulic conductivities were changed until groundwater and piezometric level contours across the model were approximately equal to the estimated average seasonal groundwater levels along the alignment (main focus was given to the groundwater and piezometric levels at the four main structures) and the observed hydraulic gradients and flow direction from the March 2015 groundwater level survey and Perth Groundwater Atlas (WRC, 1997).

Table F2 presents a comparison between estimated average seasonal groundwater and piezometric levels and modelled groundwater and piezometric levels at the four main structures along the alignment based on the March 2015 groundwater level survey and the 2014/2015 seasonal variation from the closest VWPs. The modelled groundwater levels were consistently 0.6 m to 0.7 m higher than the groundwater levels observed in March 2015, which is considered a reasonable representation of the average seasonal groundwater level, given that the March 2015 groundwater levels represent close to the seasonal low levels and that seasonal fluctuations have been observed to range between 0.7 m and 1.2 m over majority of the alignment and 3.0 m in the Forrestfield area.

Table F2: Comparison of Modelled Groundwater and Piezometric Levels (m AHD).

Structure	March 2015 Measure Groundwater and Piezometric Level	Estimated Average Seasonal Groundwater and Piezometric Level	Modelled Groundwater and Piezometric Level
Bayswater Structures – Upper Aquifer	5.3	5.9	6.0
Bayswater Structures – Lower Aquifer	1.6	2.0	2.0
Airport West Station and Structure	8.1	9.1	8.9
Consolidated Terminal Station	17.1	17.6	17.5
Forrestfield Structures	23.0	24.5	24.4

3.2 Transient Calibration

There is currently insufficient data to carry out a transient calibration on a regional scale. Instead the groundwater model was used to simulate the short-term pumping tests carried out at the four main structures by using the regional groundwater model structure and layers in four sub-models, one for each structure, with a greater discretisation (minimum cell size of 0.5 m) to better simulate the pumping tests. The results from modelling of the pumping tests are discussed in detail in Appendix E, but the main conclusions were:

- The hydrogeological units below Bassendean Sand were found to be semi-confined to confined, likely caused by the lower permeability layers in the Guildford Formation and Perth Formation.
- The vertical to horizontal hydraulic conductivity ratio (k_v/k_h) was found to range between 0.1 and 1 for all hydrogeological layers with the ratio of 0.1 generally providing the best fit with the monitoring data from the pumping tests.

Based on the favourable comparison between estimated average seasonal and modelled groundwater levels, similar hydraulic gradients and good comparison between the observed and modelled pumping test data (in the sub-models), Golder judge that the preliminary groundwater model is an adequate representation of actual groundwater conditions for the purposes of the study reported here. We therefore conclude that the preliminary groundwater model can be used for predictive purposes of dewatering design concepts to assess the effect the construction dewatering could have on the groundwater level and piezometric levels and to provide preliminary dewatering rates for the assumed design and construction methodology.

3.3 Hydraulic Parameters Used in the Calibrated Model

Table 1 presents the hydraulic input parameters of the calibrated preliminary groundwater steady-state model which was used for the transient scenario model runs.

Table 1: Summary of Hydraulic Parameters Used in the Groundwater Model

Unit	Horizontal Hydraulic Conductivity (k_h) (m/d)	Vertical Hydraulic Conductivity (k_v) (m/d)	Specific Yield	Specific Storage (1/m)
Swan River Formation	1	0.1	0.05	2×10^{-5}
Combined Bassendean Sand, Guildford Formation and Gnangara Sand	5	0.5	0.1	2×10^{-5}
Perth Formation – sandy unit	7.5	0.75 *	0.1	5×10^{-6}
Perth Formation – clay unit	0.0001	0.00001	0.02	2×10^{-5}
Guildford Formation	5	0.5 *	0.1	1×10^{-4}
Ascot Formation	8 – 14	0.8 – 1.4	0.15	2×10^{-5}
Mirrabooka Member (Osborne Formation)	1 – 3	0.1 – 0.3 *	0.1	2×10^{-5}
Kardinya Shale (Osborne Formation)	0.0005	0.00005	0.02	2×10^{-5}

* Sensitivity runs were carried out for the parameters by increasing the vertical hydraulic conductivity values 10 times (giving a K_v/K_h ratio of 1).

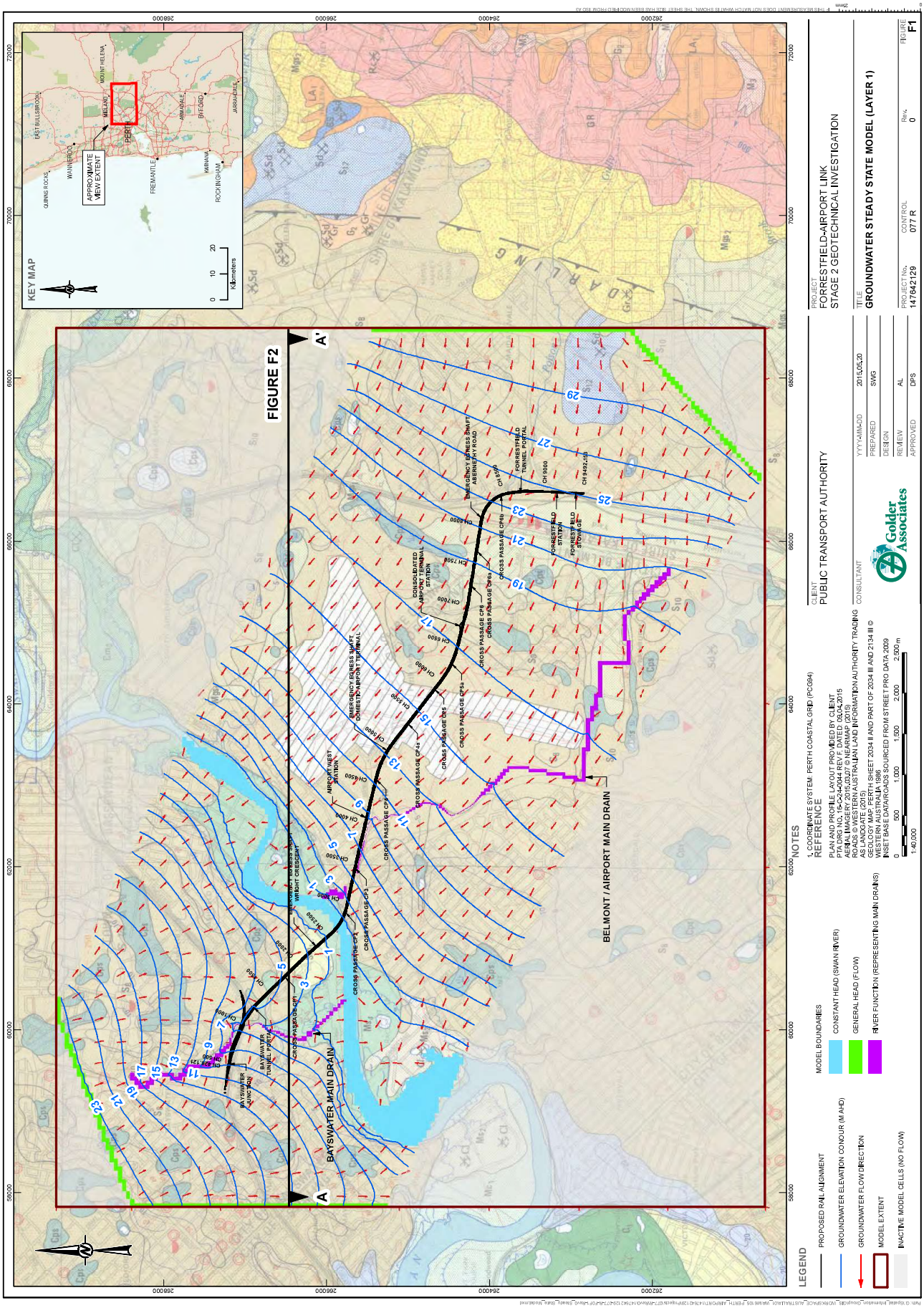
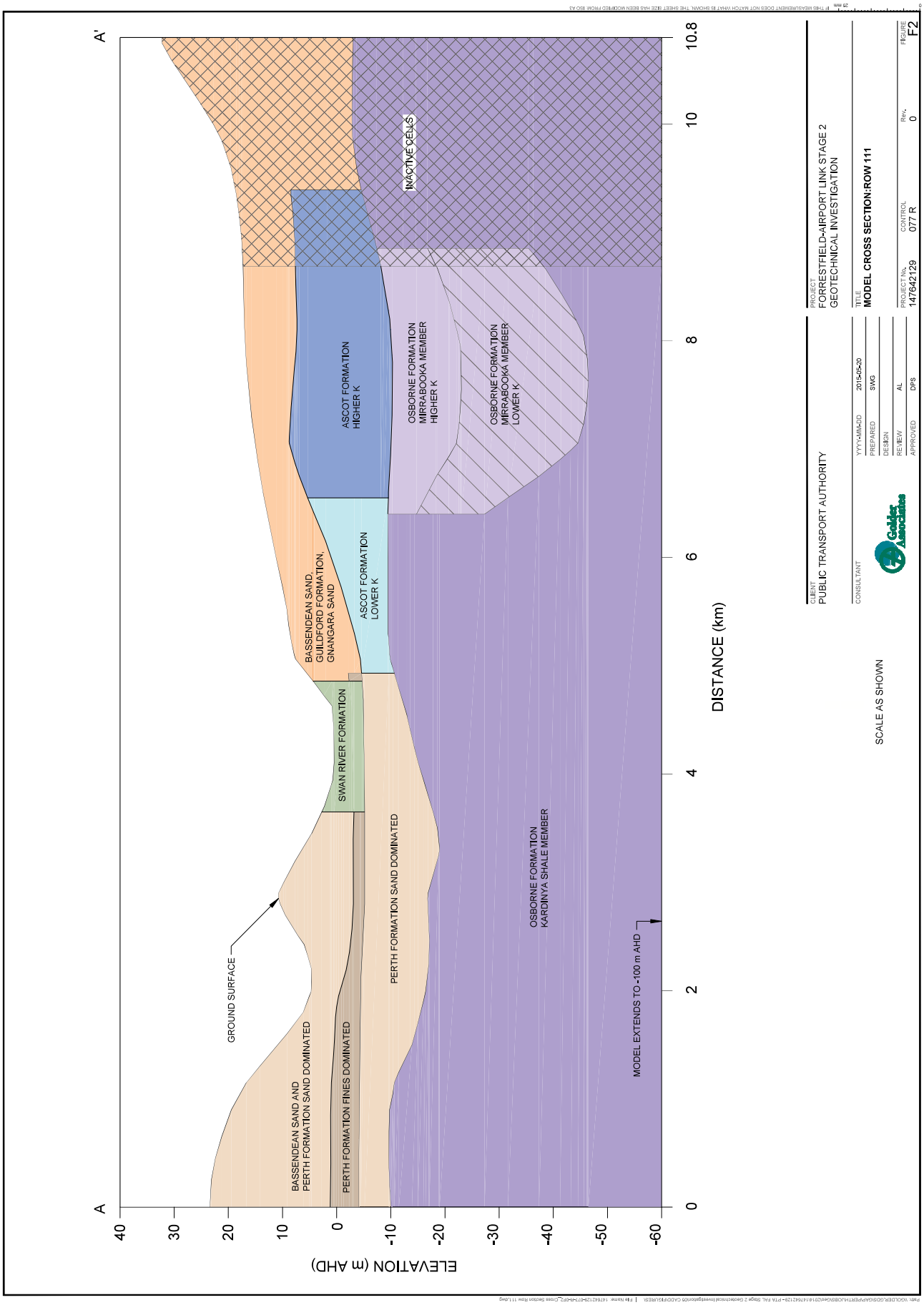


FIGURE F2
A
A'
BAYSWATER MAIN DRAIN
BEL MONT / AIRPORT MAIN DRAIN
KEY MAP
APPROXIMATE VIEW EXTENT
PERTH
MIDLAND
MOUNT HELENA
EAST BULLSBROOK
WANNEROO
QUINNS ROCKS
ARMADALE
BYFORD
FREMANTLE
KWINANA
ROCKINGHAM
JARRAHDALE
0 10 20 Kilometers
LEGEND
PROPOSED RAIL ALIGNMENT
GROUNDWATER ELEVATION CONTOUR (MAHD)
GROUNDWATER FLOW DIRECTION
MODEL EXTENT
INACTIVE MODEL CELLS (NO FLOW)
MODEL BOUNDARIES
CONSTANT HEAD (SWAN RIVER)
GENERAL HEAD (FLOW)
RIVER FUNCTION (REPRESENTING MAIN DRAINS)
NOTES
1. COORDINATE SYSTEM: PERTH COASTAL GRID (PCG94)
REFERENCE
PLAN AND PROFILE LAYOUT PROVIDED BY CLIENT
PTA DRG NO. 15-C-244044 REV F, DATED 03.04.2015
AERIAL IMAGERY 2015 (SIGIR 7.5 NEARMAP (2015)
PARCEL BOUNDARIES AUSTRALIAN LAND INFORMATION AUTHORITY TRACKING AS LANDGATE (2015)
GEOLOGY MAP: PERTH SHEET 2034 III AND PART OF 2034 III AND 2134 III © WESTERN AUSTRALIA 1986
INSET BASE DATA/ROADS SOURCED FROM STREET PRO DATA 2009
0 500 1,000 1,500 2,000 2,500 m
1:40,000
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
Golder Associates
PROJECT
FORRESTFIELD-AIRPORT LINK
STAGE 2 GEOTECHNICAL INVESTIGATION
TITLE
GROUNDWATER STEADY STATE MODEL (LAYER 1)
YYYY-MM-DD 2015.05.20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE F1


A
A'
ELEVATION (m AHD)
DISTANCE (km)
GROUND SURFACE
BASSENDEAN SAND AND
PERTH FORMATION SAND DOMINATED
PERTH FORMATION FINES DOMINATED
SWAN RIVER FORMATION
BASSENDEAN SAND,
GUILDFORD FORMATION,
GNANGARA SAND
ASCOT FORMATION
LOWER K
PERTH FORMATION SAND DOMINATED
ASCOT FORMATION
HIGHER K
OSBORNE FORMATION
MIRRABOOKA MEMBER
HIGHER K
OSBORNE FORMATION
MIRRABOOKA MEMBER
LOWER K
INACTIVE CELLS
OSBORNE FORMATION
KARDINYA SHALE MEMBER
MODEL EXTENDS TO -100 m AHD
CLIENT
PUBLIC TRANSPORT AUTHORITY
CONSULTANT
SCALE AS SHOWN
PROJECT
FORRESTFIELD-AIRPORT LINK STAGE 2
GEOTECHNICAL INVESTIGATION
TITLE
MODEL CROSS SECTION:ROW 111
YYYY-MM-DD 2015-05-20
PREPARED SWG
DESIGN
REVIEW AL
APPROVED DPS
PROJECT No. 147642129
CONTROL 077 R
Rev. 0
FIGURE F2

APPENDIX G

Summary of Analysis of Inverse Auger Hole Testing

May 2015
Report No. 147642129-077-R-Rev0

APPENDIX G
Inverse Auger Hole Testing Results

Shallow infiltration testing using the "inverse auger hole method" (Cocks, 2007) was carried out at six locations I1-01, I1-02, I1-03 (Forrestfield Area) and I1-04, I1-05 and I1-06 (Consolidated Airport Terminal Station). Attachment G1 shows the infiltration locations. All tests were carried out in Bassendean Sand except for I1-05 which was carried out in Fill.

The inverse auger hole method uses a machine slotted PVC casing sealed into an open auger hole followed by numerous falling head tests carried out by introducing water into the PVC casing. The decline in head inside the PVC casing is recorded at regular intervals and used to calculate the permeability as described by Cocks (2007).

The permeability values calculated from each test are summarised in Table 1 and the analysis sheets are presented in Attachment G2.

Table 1: Infiltration Tests Results

Test ID	Location	Soil Description*	Average Calculated Permeability (m/d)
I1–01	Forrestfield	Sand (*f-m*), some clay	12
I1–02		Sand (*f-m*), trace silt and clayey gravelly sand (*f-c*)	3
I1–03		Sand (*f-m*) and sandy gravel (*f-m*)	10
I1–04	Consolidated Airport	Sand (*f-m*)	10
I1–05		Fill/Topsoils: Sand (*f-m*)	5
I1–06		Sand (*f-m*)	4

*Material description based on upper 1 metre of soil profile encountered in closest test pit or borehole, *f*,- fine grained, *m* – medium grained, *c* – coarse grained,

ATTACHMENT G1
Infiltration Test Locations

May 2015
Reference No. 147642129-077-R-Rev0

APPENDIX G
Inverse Auger Hole Testing Results



Infiltration Test Locations at Consolidated Airport Terminal Station

APPENDIX G
Inverse Auger Hole Testing Results



Infiltration Test Locations in Forrestfield



APPENDIX G
Inverse Auger Hole Testing Results

ATTACHMENT G2
Inverse Auger Hole Permeability Testing Analysis

May 2015
Reference No. 147642129-077-R-Rev0

Golder Associates

Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

$$K = 1.15r\frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}(h_t)}{t - t_0}$$

Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-01

Spreadsheet Legend

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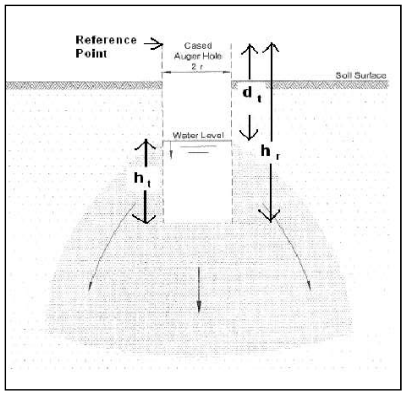
	Calculated field
	Comment field
X	Field not used
	Fixed field

Parameter	Description	Value	Units
K	Permeability	X	m/s
r	radius of test hole	0.043	m
t	time since start of measurement	X	s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t		m
h_t	Water column height at time t	X	m
h_0	h_t at t=0	X	m



Test 1

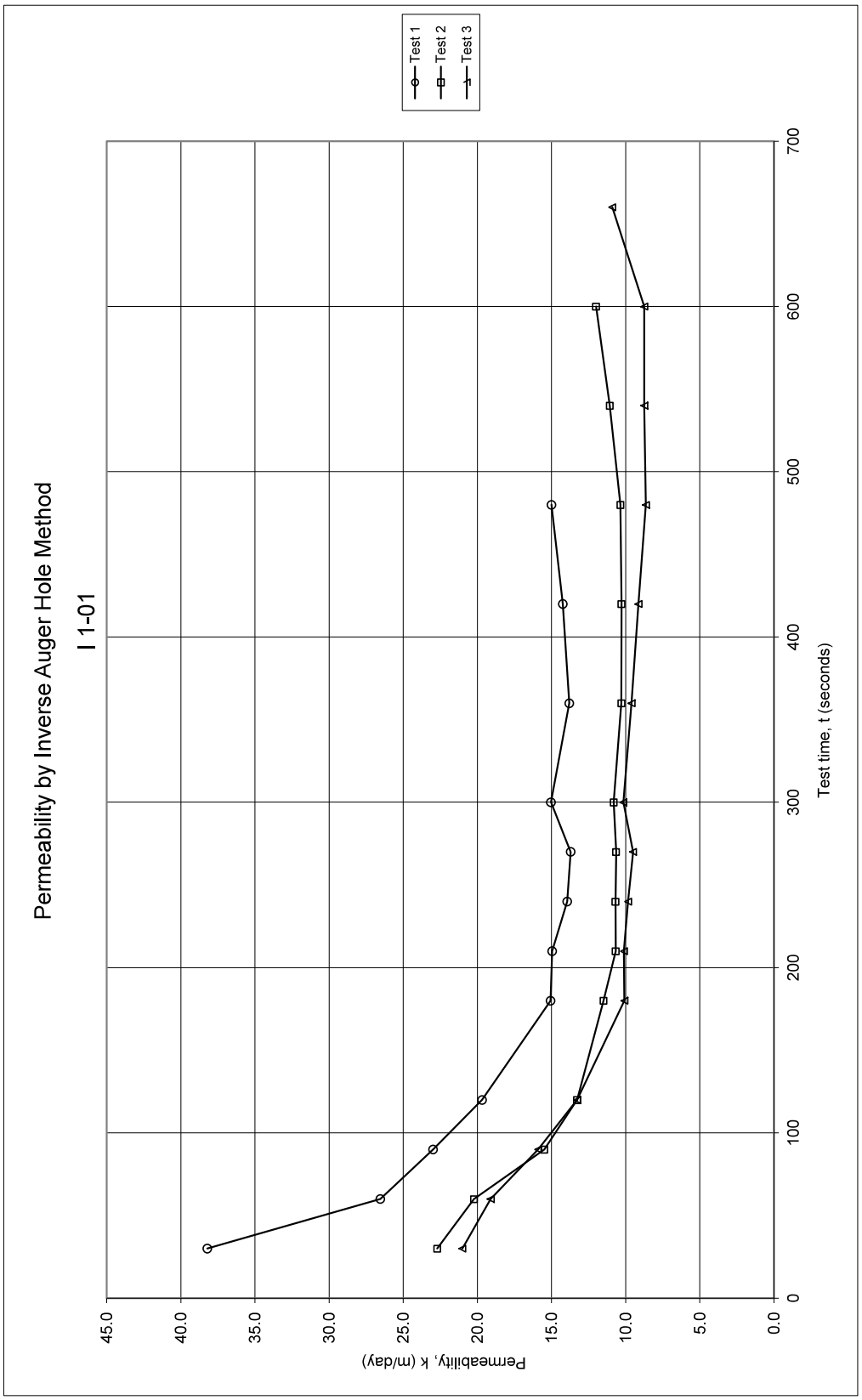
t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02	X	X
30	0.48	0.54	4.4E-04	38.2
60	0.6	0.42	3.1E-04	26.5
90	0.7	0.32	2.7E-04	23.0
120	0.75	0.27	2.3E-04	19.7
180	0.8	0.22	1.7E-04	15.1
210	0.85	0.17	1.7E-04	15.0
240	0.87	0.15	1.6E-04	13.9
270	0.9	0.12	1.6E-04	13.7
300	0.95	0.07	1.7E-04	15.0
360	0.97	0.05	1.6E-04	13.8
420	1	0.02	1.6E-04	14.2
480	1.02	0	1.7E-04	15.0
		AVERAGE	2.2E-04	18.6

Test 2

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02	X	X
30	0.32	0.7	2.6E-04	22.7
60	0.5	0.52	2.3E-04	20.2
90	0.55	0.47	1.8E-04	15.5
120	0.6	0.42	1.5E-04	13.3
180	0.7	0.32	1.3E-04	11.5
210	0.73	0.29	1.2E-04	10.7
240	0.78	0.24	1.2E-04	10.7
270	0.82	0.2	1.2E-04	10.6
300	0.86	0.16	1.3E-04	10.8
360	0.9	0.12	1.2E-04	10.3
420	0.94	0.08	1.2E-04	10.3
480	0.97	0.05	1.2E-04	10.4
540	1	0.02	1.3E-04	11.1
600	1.02	0	1.4E-04	12.0
		AVERAGE	1.5E-04	12.9

Test 3

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02	X	X
30	0.3	0.72	2.4E-04	21.0
60	0.48	0.54	2.2E-04	19.1
90	0.56	0.46	1.8E-04	15.9
120	0.6	0.42	1.5E-04	13.3
180	0.65	0.37	1.2E-04	10.1
210	0.71	0.31	1.2E-04	10.1
240	0.75	0.27	1.1E-04	9.8
270	0.78	0.24	1.1E-04	9.5
300	0.84	0.18	1.2E-04	10.2
360	0.88	0.14	1.1E-04	9.6
420	0.91	0.11	1.1E-04	9.1
480	0.93	0.09	1.0E-04	8.6
540	0.96	0.06	1.0E-04	8.8
600	0.98	0.04	1.0E-04	8.7
660	1.02	0	1.3E-04	10.9
		AVERAGE	1.3E-04	11.7



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Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

$$K = 1.15r\frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}(h_t)}{t - t_0}$$

Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-02

Spreadsheet Legend

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	Comment field
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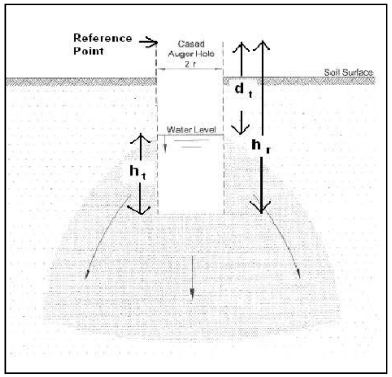
	Fixed field

Parameter	Description	Value	Units
K	Permeability		m/s
r	radius of test hole	0.043	m
t	time since start of measurement		s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t		m
h_t	Water column height at time t		m
h_0	h_t at t=0		m



Test 1

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02		
30	0.3	0.72	2.4E-04	21.0
60	0.33	0.69	1.4E-04	11.8
90	0.37	0.65	1.0E-04	9.0
120	0.39	0.63	8.4E-05	7.3
180	0.41	0.61	6.0E-05	5.2
210	0.43	0.59	5.4E-05	4.7
240	0.45	0.57	5.1E-05	4.4
270	0.46	0.56	4.6E-05	4.0
300	0.47	0.55	4.3E-05	3.7
360	0.5	0.52	3.9E-05	3.4
420	0.52	0.5	3.5E-05	3.1
480	0.53	0.49	3.2E-05	2.7
540	0.535	0.485	2.9E-05	2.5
600	0.55	0.47	2.7E-05	2.3
660	0.56	0.46	2.5E-05	2.2
720	0.57	0.45	2.4E-05	2.0
780	0.59	0.43	2.3E-05	2.0
840	0.61	0.41	2.3E-05	1.9
900	0.62	0.4	2.2E-05	1.9
		AVERAGE	5.8E-05	5.0

Test 2

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02		
30	0.21	0.81	1.6E-04	13.9
60	0.24	0.78	9.4E-05	8.1
90	0.27	0.75	7.2E-05	6.2
120	0.3	0.72	6.1E-05	5.3
180	0.31	0.71	4.2E-05	3.6
210	0.33	0.69	3.9E-05	3.4
240	0.345	0.675	3.6E-05	3.1
270	0.36	0.66	3.4E-05	2.9
300	0.385	0.635	3.3E-05	2.9
360	0.42	0.6	3.1E-05	2.7
420	0.44	0.58	2.8E-05	2.4
480	0.46	0.56	2.6E-05	2.3
540	0.48	0.54	2.5E-05	2.1
600	0.5	0.52	2.3E-05	2.0
660	0.51	0.51	2.2E-05	1.9
720	0.52	0.5	2.1E-05	1.8
780	0.53	0.49	2.0E-05	1.7
840	0.545	0.475	1.9E-05	1.6
900	0.555	0.465	1.8E-05	1.6
		AVERAGE	4.2E-05	3.7

Test 3

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02		
30	0.2	0.82	1.5E-04	13.2
60	0.22	0.8	8.5E-05	7.3
90	0.25	0.77	6.5E-05	5.7
120	0.27	0.75	5.4E-05	4.6
180	0.29	0.73	3.9E-05	3.4
210	0.3	0.72	3.5E-05	3.0
240	0.32	0.7	3.3E-05	2.8
270	0.34	0.68	3.1E-05	2.7
300	0.36	0.66	3.0E-05	2.6
360	0.4	0.62	2.9E-05	2.5
420	0.42	0.6	2.6E-05	2.3
480	0.44	0.58	2.5E-05	2.1
540	0.46	0.56	2.3E-05	2.0
600	0.48	0.54	2.2E-05	1.9
660	0.5	0.52	2.1E-05	1.8
720	0.51	0.51	2.0E-05	1.7
780	0.52	0.5	1.9E-05	1.6
840	0.53	0.49	1.8E-05	1.6
900	0.54	0.48	1.7E-05	1.5
1200	0.6	0.42	1.5E-05	1.3
1500	0.65	0.37	1.4E-05	1.2
1800	0.7	0.32	1.3E-05	1.1
		AVERAGE	3.6E-05	3.1

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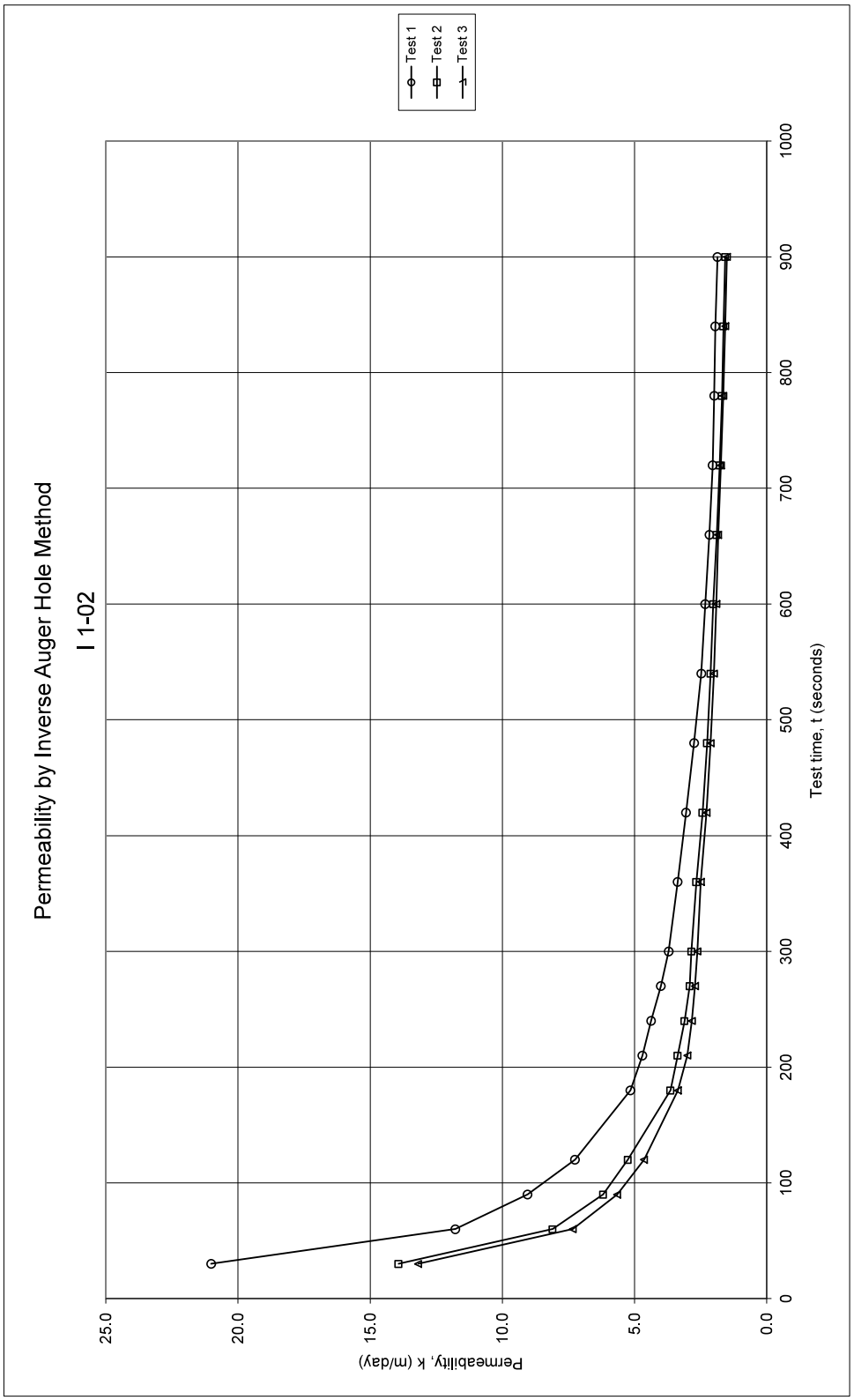

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Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

$$K = 1.15r\frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}(h_t)}{t - t_0}$$

Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-03

Spreadsheet Legend

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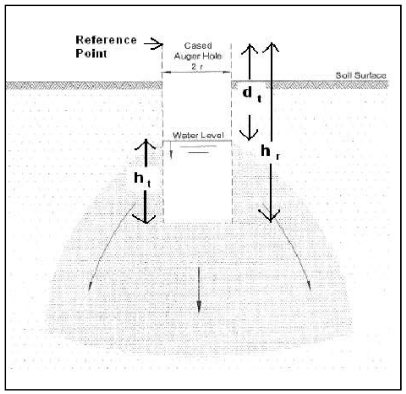
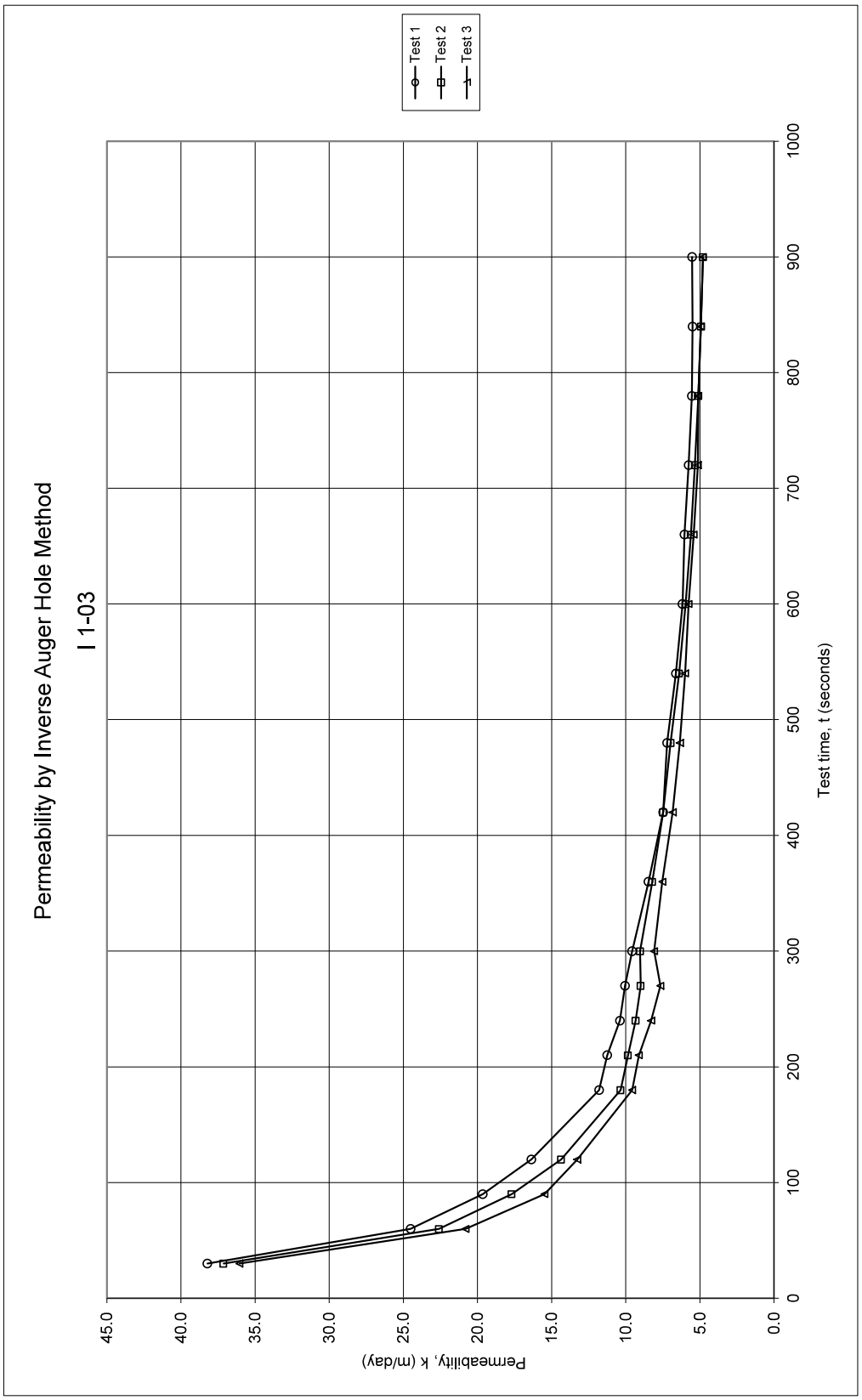
	Calculated field
	Comment field
X	Field not used
	Fixed field

Parameter	Description	Value	Units
K	Permeability	X	m/s
r	radius of test hole	0.043	m
t	time since start of measurement	X	s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t		m
h_t	Water column height at time t	X	m
h_0	h_t at t=0	X	m



Test 1					Test 2					Test 3				
t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02	X	X	0	0	1.02	X	X	0	0	1.02	X	X
30	0.48	0.54	4.4E-04	38.2	30	0.47	0.55	4.3E-04	37.1	30	0.46	0.56	4.2E-04	36.0
60	0.57	0.45	2.8E-04	24.5	60	0.54	0.48	2.6E-04	22.6	60	0.51	0.51	2.4E-04	20.8
90	0.64	0.38	2.3E-04	19.7	90	0.6	0.42	2.0E-04	17.7	90	0.55	0.47	1.8E-04	15.5
120	0.68	0.34	1.9E-04	16.4	120	0.63	0.39	1.7E-04	14.4	120	0.6	0.42	1.5E-04	13.3
180	0.71	0.31	1.4E-04	11.8	180	0.66	0.36	1.2E-04	10.4	180	0.63	0.39	1.1E-04	9.6
210	0.75	0.27	1.3E-04	11.3	210	0.7	0.32	1.1E-04	9.9	210	0.67	0.35	1.1E-04	9.1
240	0.77	0.25	1.2E-04	10.4	240	0.73	0.29	1.1E-04	9.3	240	0.685	0.335	9.6E-05	8.3
270	0.8	0.22	1.2E-04	10.0	270	0.76	0.26	1.0E-04	9.0	270	0.7	0.32	8.9E-05	7.7
300	0.82	0.2	1.1E-04	9.6	300	0.8	0.22	1.0E-04	9.0	300	0.76	0.26	9.4E-05	8.1
360	0.84	0.18	9.8E-05	8.5	360	0.83	0.19	9.5E-05	8.2	360	0.8	0.22	8.7E-05	7.5
420	0.85	0.17	8.7E-05	7.5	420	0.85	0.17	8.7E-05	7.5	420	0.82	0.2	7.9E-05	6.8
480	0.88	0.14	8.3E-05	7.2	480	0.87	0.15	8.1E-05	7.0	480	0.84	0.18	7.3E-05	6.3
540	0.89	0.13	7.7E-05	6.6	540	0.88	0.14	7.4E-05	6.4	540	0.86	0.16	6.9E-05	6.0
600	0.9	0.12	7.1E-05	6.2	600	0.89	0.13	6.9E-05	6.0	600	0.88	0.14	6.7E-05	5.8
660	0.92	0.1	7.0E-05	6.0	660	0.9	0.12	6.5E-05	5.6	660	0.89	0.13	6.3E-05	5.4
720	0.93	0.09	6.7E-05	5.8	720	0.91	0.11	6.2E-05	5.3	720	0.9	0.12	6.0E-05	5.1
780	0.94	0.08	6.4E-05	5.5	780	0.92	0.1	5.9E-05	5.1	780	0.92	0.1	5.9E-05	5.1
840	0.955	0.065	6.4E-05	5.5	840	0.93	0.09	5.7E-05	4.9	840	0.93	0.09	5.7E-05	4.9
900	0.97	0.05	6.4E-05	5.5	900	0.94	0.08	5.6E-05	4.8	900	0.94	0.08	5.6E-05	4.8
		AVERAGE	1.3E-04	11.4			AVERAGE	1.2E-04	10.5			AVERAGE	1.1E-04	9.8



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Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

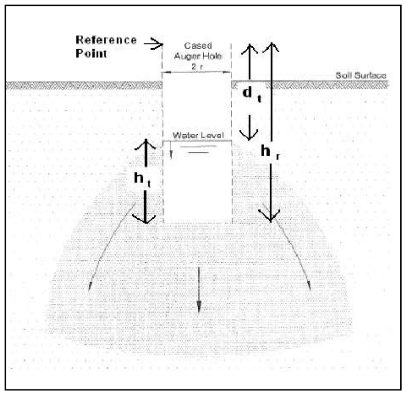
$$K = 1.15r\frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}}{t - t_0}$$

Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-04

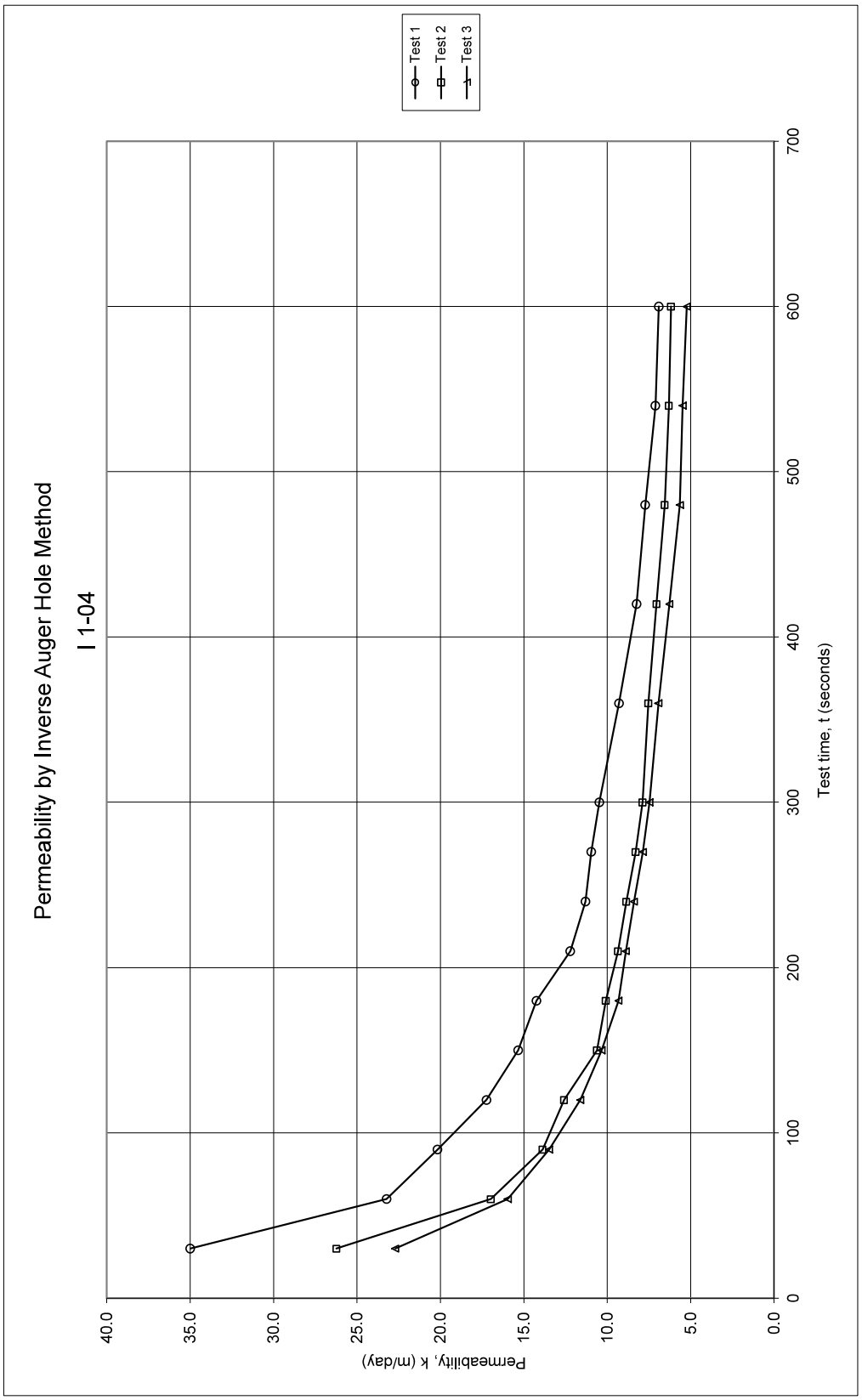
Spreadsheet Legend

	Required input
	Calculated field
	Comment field
X	Field not used
	Fixed field

Parameter	Description	Value	Units
K	Permeability		m/s
r	radius of test hole	0.043	m
t	time since start of measurement		s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t		m
h_t	Water column height at time t		m
h_0	h_t at t=0		m



Test 1					Test 2					Test 3				
t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02			0	0	1.02			0	0	1.02		
30	0.45	0.57	4.1E-04	35.0	30	0.36	0.66	3.0E-04	26.2	30	0.32	0.7	2.6E-04	22.7
60	0.55	0.47	2.7E-04	23.2	60	0.44	0.58	2.0E-04	17.0	60	0.42	0.6	1.8E-04	16.0
90	0.65	0.37	2.3E-04	20.2	90	0.51	0.51	1.6E-04	13.9	90	0.5	0.52	1.6E-04	13.5
120	0.7	0.32	2.0E-04	17.2	120	0.58	0.44	1.5E-04	12.6	120	0.55	0.47	1.3E-04	11.6
150	0.74	0.28	1.8E-04	15.3	150	0.6	0.42	1.2E-04	10.6	150	0.59	0.43	1.2E-04	10.3
180	0.78	0.24	1.6E-04	14.2	180	0.65	0.37	1.2E-04	10.1	180	0.62	0.4	1.1E-04	9.3
210	0.8	0.24	1.4E-04	12.2	210	0.68	0.34	1.1E-04	9.3	210	0.66	0.36	1.0E-04	8.9
240	0.83	0.22	1.3E-04	11.3	240	0.71	0.31	1.0E-04	8.9	240	0.69	0.33	9.7E-05	8.4
270	0.85	0.19	1.3E-04	11.0	270	0.73	0.29	9.6E-05	8.3	270	0.71	0.31	9.1E-05	7.9
300	0.87	0.17	1.2E-04	10.5	300	0.75	0.27	9.1E-05	7.9	300	0.73	0.29	8.6E-05	7.5
360	0.88	0.15	1.1E-04	9.3	360	0.8	0.22	8.7E-05	7.5	360	0.77	0.25	8.0E-05	6.9
420	0.9	0.14	9.5E-05	8.2	420	0.83	0.19	8.2E-05	7.0	420	0.79	0.23	7.3E-05	6.3
480	0.91	0.12	8.9E-05	7.7	480	0.85	0.17	7.6E-05	6.5	480	0.8	0.22	6.5E-05	5.6
540	0.93	0.11	8.2E-05	7.1	540	0.875	0.145	7.3E-05	6.3	540	0.83	0.19	6.3E-05	5.5
600	0.95	0.09	8.0E-05	6.9	600	0.9	0.12	7.1E-05	6.2	600	0.85	0.17	6.1E-05	5.2
		AVERAGE	1.6E-04	14.0			AVERAGE	1.2E-04	10.6			AVERAGE	1.1E-04	9.7



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Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

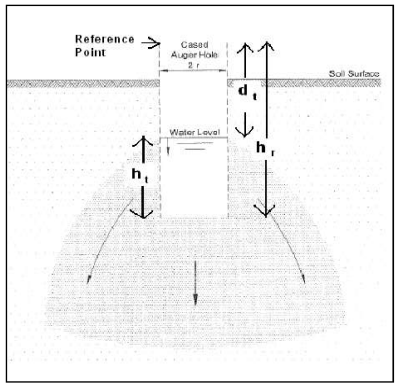
$$K = 1.15r\frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}(h_t)}{t - t_0}$$

Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-05

Spreadsheet Legend

	Required input
	Calculated field
	Comment field
X	Field not used
	Fixed field

Parameter	Description	Value	Units
K	Permeability	X	m/s
r	radius of test hole	0.043	m
t	time since start of measurement	X	s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t	X	m
h_t	Water column height at time t	X	m
h_0	h_t at t=0	X	m



Test 1

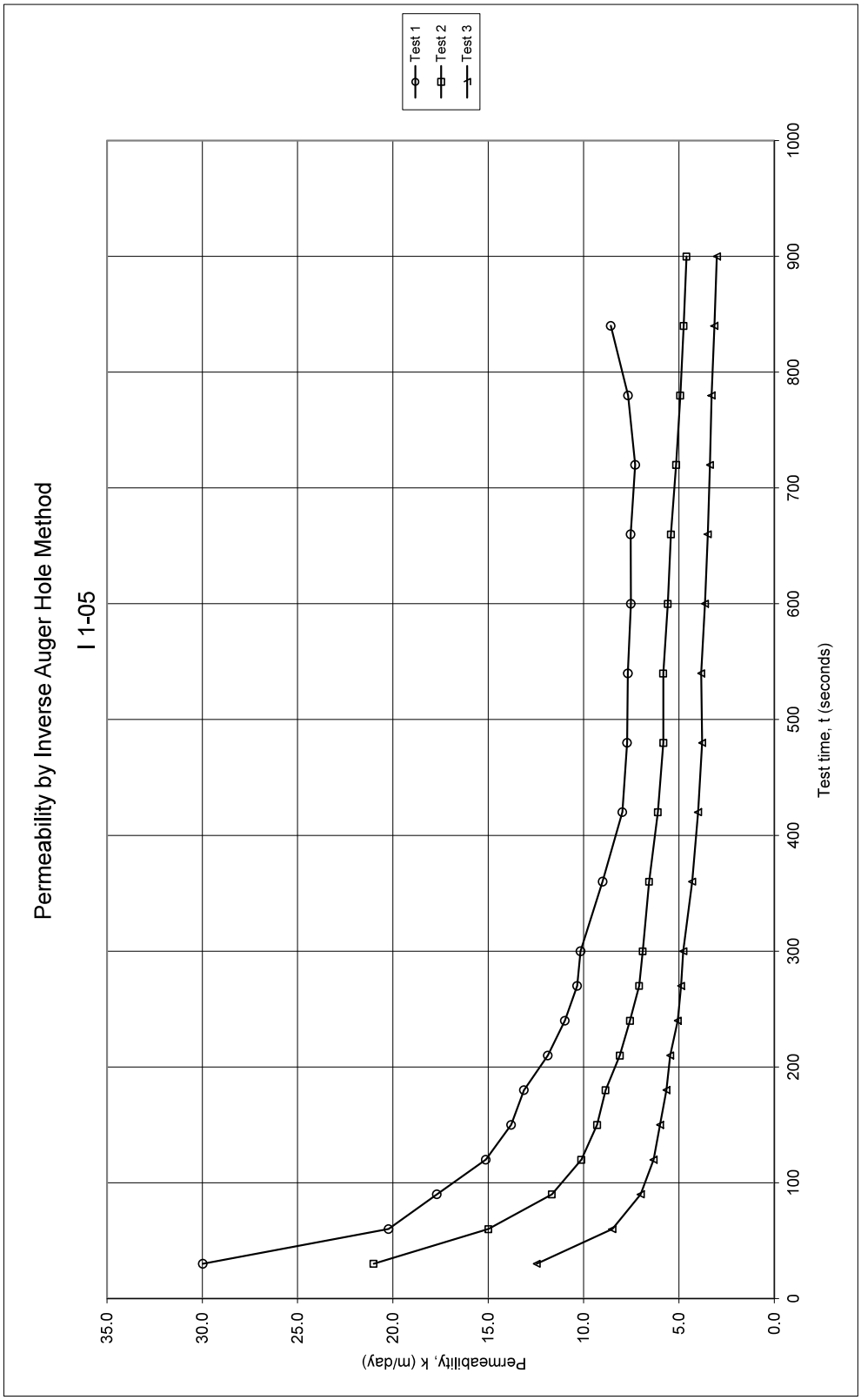
t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02	X	X
30	0.4	0.62	3.5E-04	30.0
60	0.5	0.52	2.3E-04	20.2
90	0.6	0.42	2.0E-04	17.7
120	0.65	0.37	1.8E-04	15.1
150	0.7	0.32	1.6E-04	13.8
180	0.75	0.27	1.5E-04	13.1
210	0.77	0.25	1.4E-04	11.9
240	0.79	0.23	1.3E-04	11.0
270	0.81	0.21	1.2E-04	10.3
300	0.84	0.18	1.2E-04	10.2
360	0.86	0.16	1.0E-04	9.0
420	0.87	0.15	9.2E-05	8.0
480	0.9	0.12	8.9E-05	7.7
540	0.93	0.09	8.9E-05	7.7
600	0.95	0.07	8.7E-05	7.5
660	0.97	0.05	8.7E-05	7.5
720	0.98	0.04	8.4E-05	7.3
780	1	0.02	8.9E-05	7.7
840	1.02	0	9.9E-05	8.6
		AVERAGE	1.4E-04	11.8

Test 2

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02	X	X
30	0.3	0.72	2.4E-04	21.0
60	0.4	0.62	1.7E-04	15.0
90	0.45	0.57	1.4E-04	11.7
120	0.5	0.52	1.2E-04	10.1
150	0.55	0.47	1.1E-04	9.3
180	0.6	0.42	1.0E-04	8.8
210	0.625	0.395	9.4E-05	8.1
240	0.65	0.37	8.8E-05	7.6
270	0.67	0.35	8.2E-05	7.1
300	0.7	0.32	8.0E-05	6.9
360	0.75	0.27	7.6E-05	6.6
420	0.78	0.24	7.1E-05	6.1
480	0.81	0.21	6.7E-05	5.8
540	0.85	0.17	6.7E-05	5.8
600	0.87	0.15	6.5E-05	5.6
660	0.89	0.13	6.3E-05	5.4
720	0.9	0.12	6.0E-05	5.1
780	0.91	0.11	5.7E-05	4.9
840	0.92	0.1	5.5E-05	4.7
900	0.93	0.09	5.3E-05	4.6
		AVERAGE	9.3E-05	8.0

Test 3

t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02	X	X
30	0.19	0.83	1.4E-04	12.5
60	0.25	0.77	9.8E-05	8.5
90	0.3	0.72	8.1E-05	7.0
120	0.35	0.67	7.3E-05	6.3
150	0.4	0.62	6.9E-05	6.0
180	0.44	0.58	6.6E-05	5.7
210	0.48	0.54	6.3E-05	5.5
240	0.5	0.52	5.9E-05	5.1
270	0.53	0.49	5.7E-05	4.9
300	0.56	0.46	5.5E-05	4.8
360	0.59	0.43	5.0E-05	4.3
420	0.62	0.4	4.6E-05	4.0
480	0.65	0.37	4.4E-05	3.8
540	0.7	0.32	4.4E-05	3.8
600	0.72	0.3	4.2E-05	3.6
660	0.74	0.28	4.0E-05	3.5
720	0.76	0.26	3.9E-05	3.4
780	0.78	0.24	3.8E-05	3.3
840	0.79	0.23	3.6E-05	3.1
900	0.8	0.22	3.5E-05	3.0
		AVERAGE	5.9E-05	5.1



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Permeability Calculation - Inverse Auger Hole Method

Golder Associates

Spreadsheet author: 4-Jun-08

REFERENCE: Cocks, G. *Disposal of Stormwater Runoff by Soakage in Perth Western Australia*, Journal and News of the Australian Geomechanics Society, Volume 42 No 3 September 2007, pp101-114

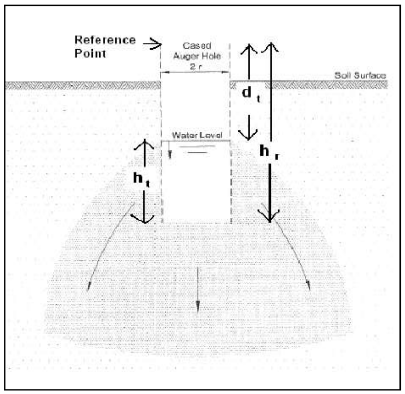
$$K = 1.15r\frac{\log_{10}(h_0 + \frac{1}{2}r) - \log_{10}(h_t)}{t - t_0}$$

Job No:	147642129
Client:	PTA
Site:	STAGE 2 FAL PROJECT
Location:	Forrestfield to Perth Airport
Calc by:	PSM 17-Mar-15
BH Name:	I 1-06

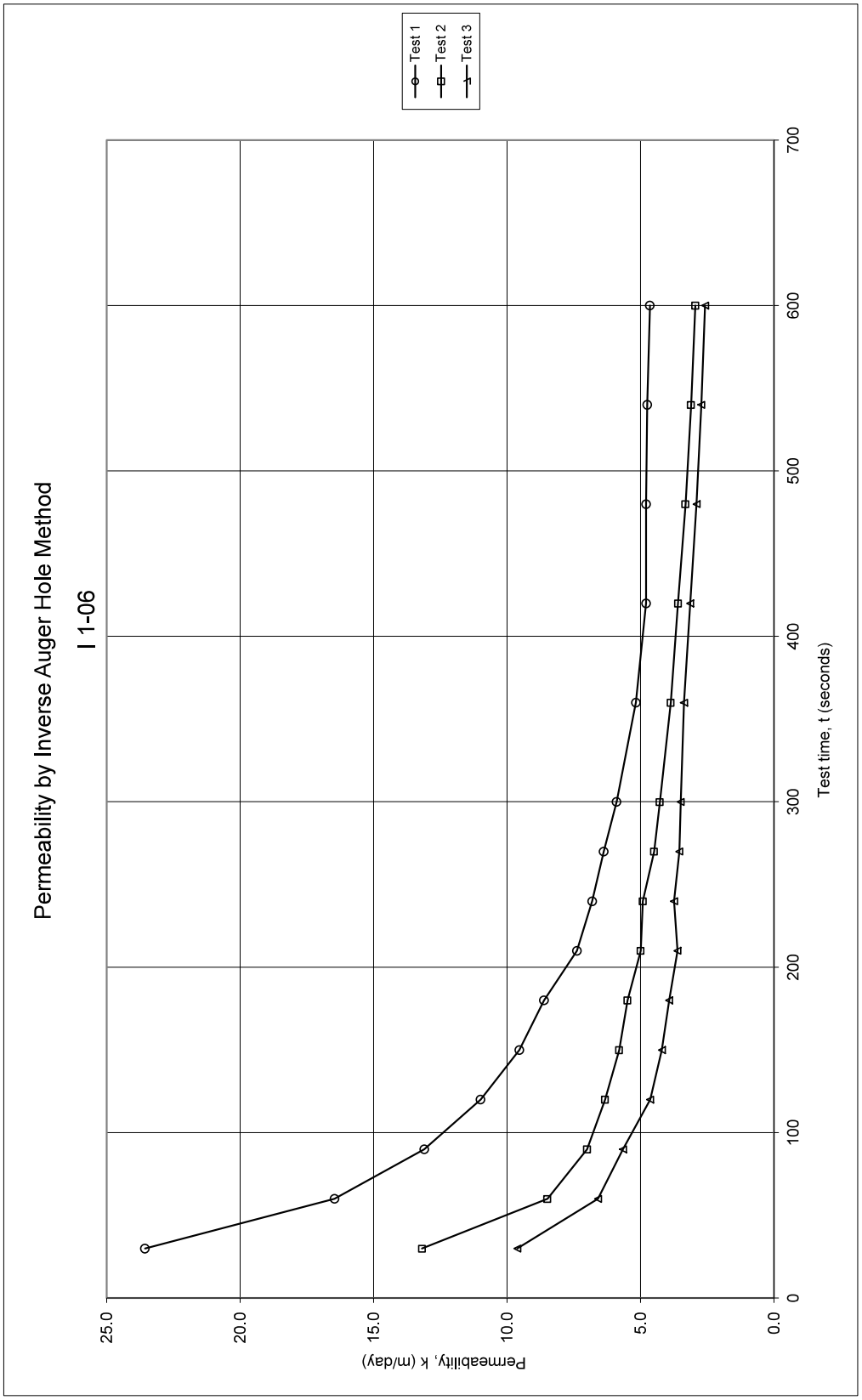
Spreadsheet Legend

	Required input
	Calculated field
	Comment field
X	Field not used
	Fixed field

Parameter	Description	Value	Units
K	Permeability		m/s
r	radius of test hole	0.043	m
t	time since start of measurement		s
h_r	reference point height above base	1.02	m
d_t	depth from reference point to water at time t		m
h_t	Water column height at time t		m
h_0	h_t at t=0		m



Test 1					Test 2					Test 3				
t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)	t (s)	d_w (m)	h_t (m)	K (m/s)	K (m/day)
0	0	1.02			0	0	1.02			0	0	1.02		
30	0.33	0.69	2.7E-04	23.6	30	0.2	0.82	1.5E-04	13.2	30	0.15	0.87	1.1E-04	9.6
60	0.43	0.59	1.9E-04	16.5	60	0.25	0.77	9.8E-05	8.5	60	0.2	0.82	7.6E-05	6.6
90	0.49	0.53	1.5E-04	13.1	90	0.3	0.72	8.1E-05	7.0	90	0.25	0.77	6.5E-05	5.7
120	0.53	0.49	1.3E-04	11.0	120	0.35	0.67	7.3E-05	6.3	120	0.27	0.75	5.4E-05	4.6
150	0.56	0.46	1.1E-04	9.5	150	0.39	0.63	6.7E-05	5.8	150	0.3	0.72	4.9E-05	4.2
180	0.59	0.43	1.0E-04	8.6	180	0.43	0.59	6.4E-05	5.5	180	0.33	0.69	4.5E-05	3.9
210	0.61	0.43	8.5E-05	7.4	210	0.45	0.57	5.8E-05	5.0	210	0.35	0.67	4.2E-05	3.6
240	0.63	0.41	7.9E-05	6.8	240	0.49	0.53	5.7E-05	4.9	240	0.4	0.62	4.3E-05	3.7
270	0.64	0.39	7.4E-05	6.4	270	0.5	0.52	5.2E-05	4.5	270	0.42	0.6	4.1E-05	3.5
300	0.66	0.38	6.8E-05	5.9	300	0.52	0.5	5.0E-05	4.3	300	0.45	0.57	4.1E-05	3.5
360	0.69	0.36	6.0E-05	5.2	360	0.55	0.47	4.5E-05	3.9	360	0.5	0.52	3.9E-05	3.4
420	0.74	0.33	5.6E-05	4.8	420	0.58	0.44	4.2E-05	3.6	420	0.53	0.49	3.6E-05	3.1
480	0.78	0.28	5.5E-05	4.8	480	0.6	0.42	3.8E-05	3.3	480	0.55	0.47	3.4E-05	2.9
540	0.81	0.24	5.5E-05	4.7	540	0.62	0.4	3.6E-05	3.1	540	0.57	0.45	3.2E-05	2.7
600	0.85	0.21	5.4E-05	4.7	600	0.64	0.38	3.4E-05	2.9	600	0.59	0.43	3.0E-05	2.6
		AVERAGE	1.0E-04	8.9			AVERAGE	6.3E-05	5.5			AVERAGE	4.9E-05	4.3



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APPENDIX H

Summary of Analysis of Infiltration Testing

May 2015
Report No. 147642129-077-R-Rev0

Golder Associates

APPENDIX H
Infiltration Testing

1.0 INTRODUCTION

This appendix presents the results from the infiltration testing carried out during the development of the pumping test wells at Bayswater Dive Structure, Airport West Station, Consolidated Airport Terminal Station and Forrestfield Dive Structure. In addition the results from monitoring the infiltration basin at the Bayswater Dive Structure during the pumping test are described.

2.0 BAYSWATER DIVE STRUCTURE

2.1 Infiltration Test During Well Development

The pumping test well (PW2-02) at Bayswater Dive Structure was developed on 4 and 5 November 2014. The groundwater from the development was pumped into two different infiltration pits excavated with a backhoe. Attachment H1 shows the location of the infiltration basins while Figure H1 shows photos of the two infiltration pits.



Figure H1: Infiltration Testing at Bayswater Dive Structure (Pit 1 left, Pit 2 right)

The dimensions of the two infiltration pits were approximately 5 m long by 5 m wide and 1 m deep, giving a volume of 25 kL. Given the loose sandy conditions it was not possible to form steep batter slopes and the pits kept collapsing during excavation. The dimensions are therefore approximate only.

PW2-02 was developed at a rate of approximately 4 L/s for a total period of 12 hours (over 2 days). Generally, it was found that the infiltration pits were able to infiltrate all of the discharged water by alternating the pumping into the pits.

The infiltration test consisted of filling the infiltration pit with water from the well development process and then measuring the water level decline in the pit over time. Infiltration Pit 1 was tested three times while Infiltration Pit 2 was only tested once.

Figure H2 shows the resulting water level decline while Table H1 presents the estimated infiltration rate for each test.

APPENDIX H
Infiltration Testing

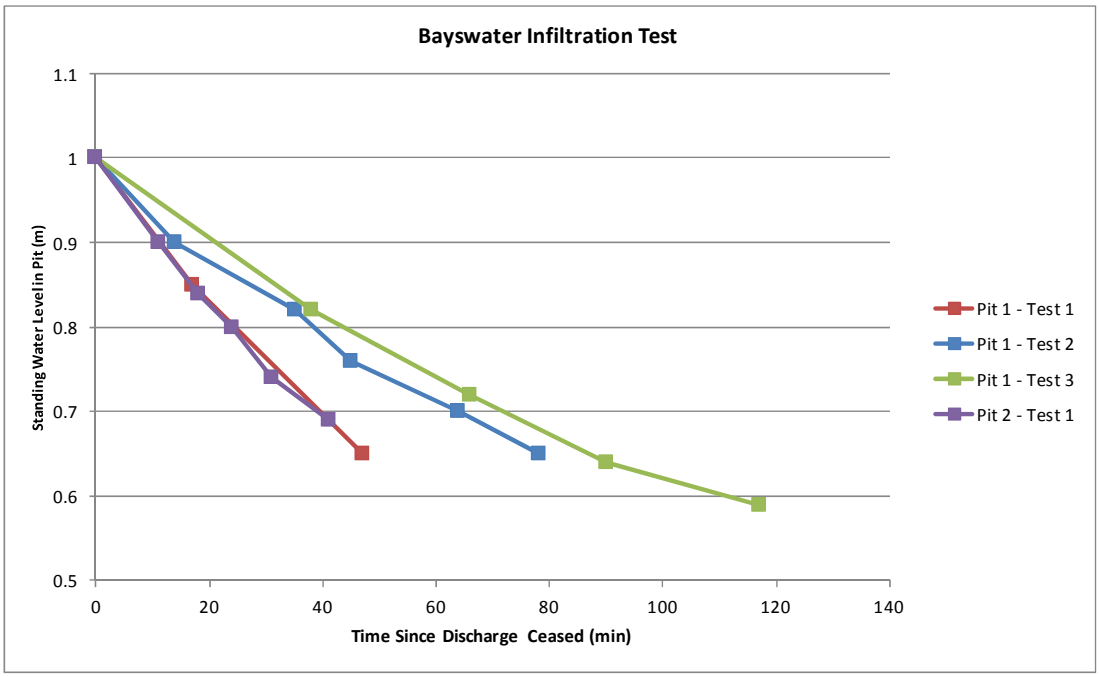

Figure H2: Infiltration Testing Data at Bayswater Dive Structure

Table H1: Estimated Infiltration Rate at Bayswater Dive Structure

Test	Approximate Infiltration Rate	
	L/s	m/d *
Pit 1 – Test 1	3.1	11
Pit 1 – Test 2	1.9	7
Pit 1 – Test 3	1.6	6
Pit 2 – Test 1	3.1	11

* Assuming all water is infiltrating through the base of the Infiltration Pit, which is a conservative assumption.

The results indicate:

- The derived infiltration rate ranges from 6 to 11 m/d assuming that all water infiltrated through the 25 m^2 base of the pits.
- Similar water level decline was observed in both pits for Test 1, resulting in similar infiltration rate at the two locations.
- The infiltration rates for Infiltration Pit 1 was found to decrease in Test 2 and Test 3, which is likely due to progressive saturation of the soil profile and fines being deposited at the bottom of the pit during the development (the development water contained some fines from the well).

2.2 Infiltration During Pumping Test

Due to restrictions on the groundwater discharge disposal, the pumped groundwater was also infiltrated on site during the pumping test carried out from 3 to 6 February 2015. Attachment H1 shows the location of the

main infiltration basin and a contingency infiltration basin. The footprints of the basins were approximately 250 m^2 and 25 m^2. The main basin was only excavated to approximately 1 m depth at the periphery of the basin to create the bund walls while the central area was not excavated. The estimated volume of the main basin is approximately 115 kL.

A monitoring well (unknown construction details but with a depth of 4.5 m) existed inside the main infiltration basin. The groundwater level was measured to be 1.8 m below ground surface prior to start of the infiltration. A groundwater level logger was installed in the well to measure the change in groundwater level during the infiltration. The results from the groundwater level logger indicate that the groundwater level rose to the surface, indicating that fully saturated conditions occurred below the infiltration basin.

The discharge rate from the pumping test was approximately 12 L/s, but after 6 hours the basin was full and the discharge was therefore diverted into the contingency basin. The total volume of water pumped to the main basin within the first 6 hours was approximately 260 kL. With a storage volume of 115 kL, a total of 145 kL would have infiltrated over the 6 hour period, which corresponds to an average infiltration rate of approximately 2 m/d.

3.0 AIRPORT WEST STATION

The pumping test well (PW2-01) at Airport West Station was developed on 25 November 2014. The groundwater from the development was pumped into an infiltration pit excavated with a backhoe. Attachment H2 shows the location of the infiltration pit while Figure H3 shows a photo of the infiltration pit.



Figure H3: Infiltration Pit at Airport West Station

The dimensions of the infiltration pit were measured to be 10.3 m long by 2.9 m wide and 1.1 m deep, giving a volume of 33 kL. The ground was fairly consolidated and steep batter slopes were therefore achievable.

PW2-01 was developed at a rate of approximately 4 L/s. Generally, it was found that the infiltration pit would fill up and that well development had to cease until the water had infiltrated in the infiltration pit. This indicates that the infiltration rate is less than 10 m/d.

The infiltration tests consisted of pumping water into the pits until the standing water level in the pit was approximately 0.8 m. The pumping was then ceased and the decline in water level in the infiltration pit was measured over time.

It took approximately 110 minutes of pumping at 4 L/s (total volume of 26 kL) to fill the infiltration pit to 0.8 m (total volume in pit of 20 kL), indicating that approximately 6 kL infiltrated during the 110 minutes, corresponding to an average infiltration rate of 0.9 L/s or approximately 2.6 m/d during the time of filling.

Figure H4 shows the resulting water level decline and indicates that the groundwater declined by approximately 0.13 m over a period of 55 minutes, corresponding to an infiltration rate of 1.2 L/s or approximately 3.4 m/d (assuming that all water is infiltrating through the base of the Infiltration Pit, which is a conservative assumption).

It is noted that the calculated infiltration rate for the emptying of the pit is more precise than for filling due to the accumulation of potential measurement errors.

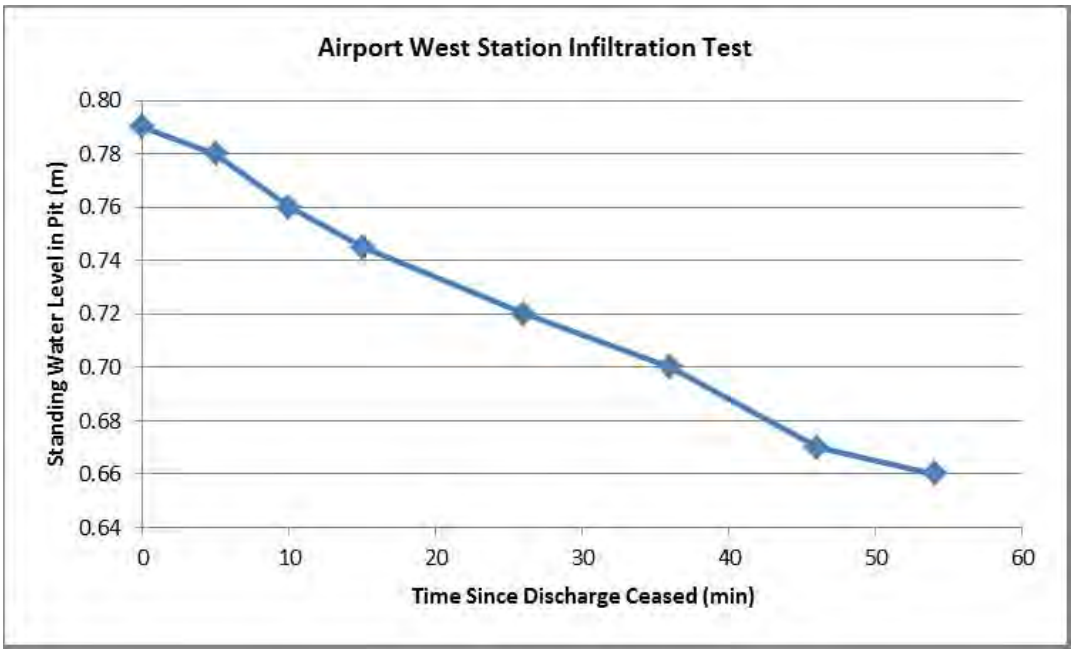

Figure H4: Infiltration Test Data at Airport West Station

4.0 CONSOLIDATED AIRPORT TERMINAL STATION

The pumping test well (PW1-02) at Consolidated Airport Terminal Station was developed on 25 November 2014. The groundwater from the development was pumped into an infiltration pit excavated with a backhoe. Attachment H3 shows the location of the infiltration pit while Figure H5 shows a photo of the infiltration pit.

APPENDIX H
Infiltration Testing



Figure H5: Infiltration Pit at Consolidated Station (left before discharge, right after start of discharge)

The dimensions of the infiltration pit were approximately 6 m long by 2 m wide and 1 m deep, giving a volume of 12 kL. The ground was fairly consolidated and steep batter slopes were therefore achievable. The red colour on the left photo in Figure H5 indicates that coffee rock was present at approximately 1 m depth at the infiltration pit location.

PW1-02 was developed at a rate of approximately 8 L/s. The infiltration test consisted of filling the infiltration pit with water from the development process and then measuring the water level decline in the pit over time.

It took approximately 27 minutes of pumping at approximately 8 L/s (total volume of 13 kL) to fill the infiltration basin, indicating that approximately 1 kL infiltrated during the 27 minutes, corresponding to an average infiltration rate of 0.6 L/s or approximately 4 m/d during the time of filling.

Figure H6 shows the resulting water level decline and indicates that the groundwater declined by approximately 0.03 m over a period of 33 minutes, corresponding to an infiltration rate of 0.2 L/s or approximately 1 m/d (assuming that all water is infiltrating through the base of the Infiltration Pit, which is a conservative assumption).

It is noted that the calculated infiltration rate for the emptying of the pit is more precise than for filling due to the accumulation of potential measurement errors.



APPENDIX H
Infiltration Testing

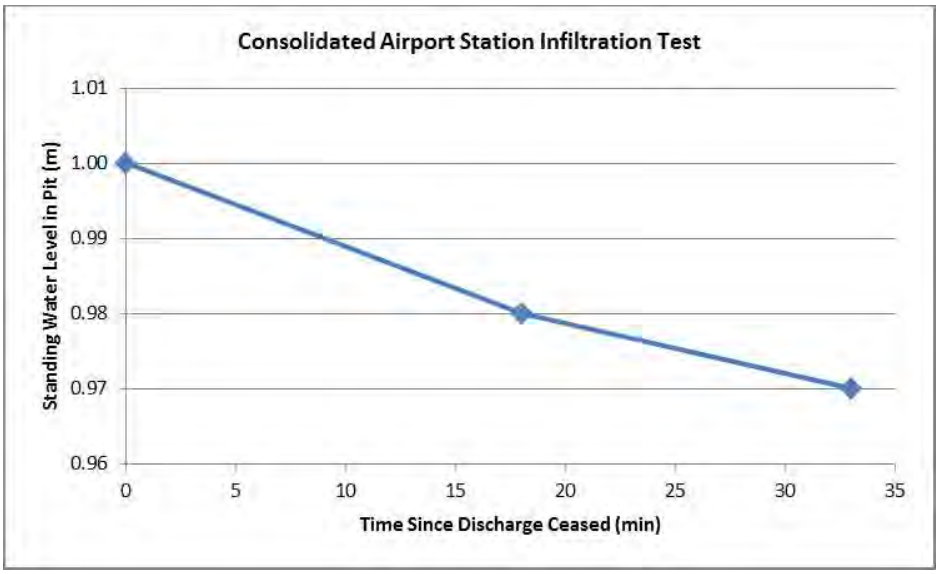

Figure H6: Infiltration Test Data at Consolidated Airport Terminal Station

5.0 FORRESTFIELD DIVE STRUCTURE

The pumping test well (PW1-01) at Forrestfield Dive Structure was developed on 7 November 2014. Some of the groundwater from the well development was pumped into an infiltration pit excavated with a backhoe. Attachment H4 shows the location of the infiltration pit while Figure H7 shows a photo of the infiltration pit.



Figure H7: Infiltration Pit at Forrestfield Dive Structure

The dimensions of the infiltration pit were approximately 3 m long by 1 m wide and 0.5 m deep, giving a volume of 1.5 kL. The ground was hard and steep batter slopes were therefore achievable.

APPENDIX H
Infiltration Testing

PW1-01 was developed at a rate of approximately 6 to 10 L/s. The infiltration test consisted of filling the infiltration pit with water from the well development and then measuring the water level decline in the pit over time.

The capacity of the pit was reached within five minutes of the commencement of development of PW1-01. The change in water level in the pit was found to negligible between ceasing pumping into the pit and the end of the day (approximately 6 hours). The water level in the test pit was observed again the following day and the water level had not declined noticeably, indicating that the infiltration capacity of the soil at the infiltration location is negligible.

ATTACHMENT H1
Bayswater Infiltration Test Locations

May 2015
Reference No. 147642129-077-R-Rev0


Properties
Infiltration Pit 1
Infiltration Pit 2
PW2-02
Contingency Infiltration Basin
Main Infiltration Basin

ATTACHMENT H2
Airport West Infiltration Test Locations

May 2015
Reference No. 147642129-077-R-Rev0

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Infiltration Pit
PW2-02

ATTACHMENT H3
Consolidated Infiltration Test Locations

May 2015
Reference No. 147642129-077-R-Rev0


I1-06
I1-04
PW1-02
Infiltration Pit
I1-05

ATTACHMENT H4
Forrestfield Infiltration Test Locations

May 2015
Reference No. 147642129-077-R-Rev0

Golder Associates


Infiltration Pit 1
PW1-01
I1-03
I1-02
I1-01
Maida Vale Rd
Ibis Pl
Palmer Crescent
Durant Way
Dundas Rd
Imperial St
Eureka St
Milner Rd
Avonside Crescent
55

APPENDIX I

Limitations

May 2015
Report No. 147642129-077-R-Rev0

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Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 44 1628 851851
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Pty Ltd
Level 3, 1 Havelock Street
West Perth, Western Australia 6005
Australia
T: +61 8 9213 7600

Golder Associates

APPENDIX 4 MONITORING DATA (STRATEGEN, 2020)

						Physical							Metals and Metaloids							Nutrients								
	Month	Date	Time	GWL (mBTOC)	GWL (mAHD)	pH	Change in pH	EC	Temp	Diss Oxy (mg/L)	Diss Oxy (%)	Redox (mV)	Arsenic	Cadmium	Copper	Chromium	Nickel	Lead	Zinc	Ammonia as N	Nitrite as N	Nitrate as N	Nitrite + Nitrate as N	TKN	TN	TP	FRP	Comments
Swan Canning WQIP (Short Term Limits)																								2.0	0.2		
Swan Canning WQIP (Long Term Limits)																								0.5	0.05		
Australian and NZ Guidelines for Fresh and Marine Water Quality						7.0-8.5							0.013	0.0002	0.003	0.0031	0.011	0.0034	0.008								
Australian Drinking Water Guidelines (2000)																						11.3					
MB01	September	28/09/2011	12:25	4.59	26.05	6.61	0.31		21.3	3.42	39	188															Bailed 30L, bore still not dry
	October	19/10/2011	11:40	4.21	26.43	6.62	0.01	0.45	21.5	2.76	31	188	0.009	<0.0001	0.024	0.0020	0.011	0.0620	0.170	0.65	0.27	1.20	1.47	1.00	2.40	0.61	0.003	Bailed 10L, bore still not dry, samples taken
	November	14/11/2011	8:45	5.23	25.41																						
	December	22/12/2011	9:25	5.86	24.78																						
	January	16/01/2012		6.41	24.23	5.87	-0.75	0.32	23.2	2.78	87	133	0.007	<0.0001	0.044	0.0070	0.022	0.1000	0.520	1.90	0.03	0.97	1.00	4.90	5.90	1.70	<0.002	Bailed 15L, Bore still not dry, samples taken
	February	22/02/2012		6.87	23.77																						
	March	27/03/2012		7.20	23.44																						
	April	23/04/2012		7.31	23.33	5.68	-0.19	0.25	22.2	2.72	32	67								0.07	<0.05	1.12	5.25	0.66	1.80	0.41	<0.002	Bailed 10L, Bore still not dry, samples taken
	May	24/05/2012		7.00	23.64																						
	June	27/06/2012		6.01	24.63																						
	July	26/07/2012		6.19	24.45	5.84	0.16	0.24	22.2	2.24	26	114								0.09	<0.05	0.63	0.66	0.45	1.10	0.21	0.008	Bailed 10L, Bore still not dry, samples taken
	August	29/08/2012		5.70	24.94																						
	September	26/09/2012		5.51	25.13																						
	October	24/10/2012		5.95	24.69																						
	November	21/11/2012		6.25	24.39	6.07	-0.23	0.23	22.3	3.10	32	145								0.05	0.20	0.25	0.45	0.66	0.93	0.68	<0.005	Bailed 10L, Bore still not dry, samples taken
	December	20/12/2012		6.32	24.32																						
	Minimum			4.21	23.33	5.68		0.23	21.3	2.24	26	67	0.007	0.0001	0.024	0.0020	0.011	0.0620	0.170	0.05	0.03	0.25	0.45	0.45	0.93	0.21	0.001	
	Maximum			7.31	26.43	6.62		0.45	23.2	3.42	87	188	0.009	0.0001	0.044	0.0070	0.022	0.1000	0.520	1.90	0.27	1.20	5.25	4.90	5.90	1.70	0.008	
	Median					5.97		0.25	22.2	2.77	32	139	0.008	0.0001	0.034	0.0045	0.017	0.0810	0.345	0.09	0.03	0.97	1.00	0.66	1.80	0.61	0.003	
MB07	September	28/09/2011		9.98	25.11	6.2	0.00	0.26	21.2	5.66	63	235																Bailed 30L, bore still not dry
	October	19/10/2011		10.21	24.88	6.15	-0.05	0.29	21.0	6.69	76	184	0.001	<0.0001	0.013	0.0020	0.004	0.0060	0.066	0.04	<0.005	7.50	7.50	0.43	8.00	0.38	<0.002	Bailed 10L, bore still not dry, samples taken
	November	14/11/2011		10.15	24.94																						
	December	22/12/2011		10.76	24.33																						
	January	16/01/2011		11.00	24.09	5.84	-0.31	0.25	21.8	15.18	134	188	0.001	<0.0001	0.018	0.0080	0.003	0.0070	0.058	0.10	<0.05	6.32	28.00	0.91	7.30	0.77	<0.002	Bailed 15L, Bore still not dry, samples taken
	February	22/02/2011		11.23	23.86																						
	March	27/03/2012		11.55	23.55																						
	April	23/04/2012		11.84	23.25	6.05	0.21	0.31	21.9	6.60	75	159								0.10	<0.05	7.67	34.03	0.25	8.00	0.20	<0.002	Bailed 8L, slow recharge, samples taken
	May	24/05/2012		11.45	23.64																						
	June	27/06/2012		10.99	24.11																						
	July	26/07/2012		11.14	23.95	6.34	0.29	0.14	20.8	4.52	50	164								0.08	<0.005	1.60	1.60	5.50	7.10	1.60	0.003	Bailed 15L, Bore still not dry, samples taken
	August	29/08/2012		10.74	24.35																						
	September	26/09/2012		10.54	24.55																						
	October	24/10/2012		10.86	24.23																						
	November	21/11/2012		10.96	24.13	6.25	0.09	0.20	21.4	4.53	49	156								<0.005	<0.005	6.10	6.10	4.10	10.00	1.10	<0.005	Bailed 10L, Bore still not dry, samples taken
	December	20/12/2012		11.12	23.97																						
	Minimum			9.98	23.25	5.84		0.14	20.8	4.52	49	156	0.001	0.0001	0.013	0.0020	0.003	0.0060	0.058	0.04	<0.005	1.60	1.60	0.25	7.10	0.20	0.001	
	Maximum			11.84	25.11	6.34		0.31	21.9	15.18	134	235	0.001	0.0001	0.018	0.0080	0.004	0.0070	0.066	0.10	0.03	7.67	34.03	5.50	10.00	1.60	0.003	
	Median					6.18		0.25	21.3	6.13	69	174	0.001	0.0001	0.016	0.0050	0.004	0.0065	0.062	0.09	0.00	6.32	7.50	0.91	8.00	0.77	0.001	
MB08	September	28/09/2011		3.95	28.49	6.15	0.00	0.69	20.4	6.88	77	229																Bailed 30L, bore still not dry
	October	19/10/2011		4.57	27.87	6.03	-0.12	0.92	20.1	6.65	76	202	0.001	<0.0001	0.006	<0.001	<0.001	0.0080	0.014	0.02	<0.005	22.00	22.00	0.81	23.00	0.18	<0.002	Bailed 10L, bore still not dry, samples taken
	November	14/11/2011		4.68	27.76																						
	December	22/12/2011		4.87	27.57																						
	January	16/01/2011		5.78	26.67	5.83	-0.20	0.71	22.8	24.73	271	204	0.002	<0.0001	0.047	0.0100	0.005	0.2400	0.170	0.19	<0.005	20.32	90.00	4.30	25.00	0.72	<0.002	Bailed 6L and bore ran dry, replenished in 15-20 mins, sample taken
	February	22/02/2012		6.55	25.89																						
	March	27/03/2012		dry																							dry
	April	23/04/2012		dry																							dry
	May	24/05/2012		dry																							dry
	June	27/06/2012		5.31	27.13																						
	July	26/07/2012		5.31	27.13															0.03	<0.005	18.00	18.00	2.50	20.00	0.52	0.017	Bailed 1L and bore ran dry, replenished in 25mins, samples taken
	August	29/08/2012		5.77	26.67																						
	September	26/09/2012		5.50	26.94																						
	October	24/10/2012		6.69	25.75																						
	November	21/11/2012		6.30	26.14	6.58	-0.75	0.01	23.0	6.23	73	115								0.15	<0.005	13.00	13.00	5.70	18.00	1.20	0.013	Bailed 10L, bore still not dry, samples taken
	December	20/12/2012		6.97	25.47																						
	Minimum			3.95	25.47	5.83		0.01	20.1	6.23	73	115	0.001	0.0001	0.006	0.0005	0.001	0.0080	0.014	0.02	<0.005	13.00	13.00	0.81	18.00	0.18	<0.002	
	Maximum			6.97	28.49	6.58		0.92	23.0	24.73	271	229	0.002	0.0001	0.047	0.0100	0.005	0.2400	0.170	0.19	<0.005	22.00	90.00	5.70	25.00	1.20	0.017	
	Median					6.09		0.70	21.6	6.77	76	203	0.001	0.0001	0.027	0.0053	0.003	0.1240	0.092	0.09	<0.005	19.16	20.00	3.40	21.50	0.62	0.007	

		Physical	Metals and Metaloids	Nutrients	

	Month	Date	Time	GWL (mBTOC)	GWL (mAHD)	pH	Change in pH	EC	Temp	Diss Oxy (mg/L)	Diss Oxy (%)	Redox (mV)	Arsenic	Cadmium	Copper	Chromium	Nickel	Lead	Zinc	Ammonia as N	Nitrite as N	Nitrate as N	Nitrite + Nitrate as N	TKN	TN	TP	FRP	Comments
Swan Canning WQIP (Short Term Limits)																								2.0	0.2			
Swan Canning WQIP (Long Term Limits)																								0.5	0.05			
Australian and NZ Guidelines for Fresh and Marine Water Quality						7.0-8.5							0.013	0.0002	0.003	0.0031	0.011	0.0034	0.008									
Australian Drinking Water Guidelines (2000)																						11.3						
MB09	September	28/09/2011		0.99	32.97	6.02	0.00	0.17	16.8	4.18	44	232																	Bailed 30L, bore still not dry
	October	19/10/2011		1.55	32.41	6.68	0.66	0.19	17.3	5.89	62	205	0.001	<0.0001	0.004	<0.001	<0.001	0.0030	0.020	0.03	0.00	0.54	0.54	0.76	1.30	0.08	0.001	Bailed 10L, bore still not dry, samples taken
	November	14/11/2011		2.08	31.89																							
	December	22/12/2011		dry																								Dipper muddy? May just be moisture at bottom of bore
	January	16/12/2011		dry																								Bore was dry, water level obtained by moist mud level, no sample taken
	February	22/02/2012		dry																								Bore was dry, water level obtained by moist mud level
	March	27/03/2012		dry																								dry
	April	23/04/2012		dry																								dry
	May	24/05/2012		dry																								dry
	June	27/06/2012		dry																								dry
	July	26/07/2012		dry																								dry
	August	29/08/2012		dry																								dry
	September	26/09/2012		dry																								Bore was dry, water level obtained by moist mud level, no sample taken
	October	24/10/2012		5.61	28.35																							Water level obtained by moist mud level, no sample taken
	November	21/11/2012		5.63	28.33																							Water level obtained by moist mud level, no sample taken
	December	20/12/2012																										
	Minimum			0.99	28.33	6.02		0.17	16.8	4.18	44	205	0.001	0.0001	0.004	0.0005	0.001	0.0030	0.020	0.03	0.00	0.54	0.54	0.76	1.30	0.08	0.001	
	Maximum			5.63	32.97	6.68		0.19	17.3	5.89	62	232	0.001	0.0001	0.004	0.0005	0.001	0.0030	0.020	0.03	0.00	0.54	0.54	0.76	1.30	0.08	0.001	
	Median					6.35		0.18	17.1	5.04	53	219	0.001	0.0001	0.004	0.0005	0.001	0.0030	0.020	0.03	0.00	0.54	0.54	0.76	1.30	0.08	0.001	
Poison Gully Creek (at Milner Road)	September	28/09/2011				7.21	0.64	14.80	9.0	87.30	181																	
	October	19/10/2011		<0.20cm		6.75	-0.46	0.70	16.6	8.70	89	209	0.001	<0.0001	<0.001	<0.001	<0.001	<0.001	0.022	0.02	0.00	0.15	0.15	0.28	0.42	0.01	0.001	Less flow than September
	November	14/11/2011		<0.20cm																								Similar flow to October
	December	22/12/2011																										Creek is dry
	January	16/01/2012																										Creek is dry
	February	22/02/2012																										Creek is dry
	March	27/03/2012																										dry
	April	23/04/2012																										dry
	May	24/05/2012																										dry
	June	27/06/2012		50 cm																								No samples taken
	July	26/07/2012		25 cm																								No samples taken
	August	29/08/2012		50 cm																								No samples taken
	September	26/09/2012		50 cm																								No samples taken
	October	24/10/2012		45 cm																0.00	0.00	0.00	0.00	0.36	0.36	0.01	0.010	Samples taken
	November	21/11/2012		Dry																								Creek
	December	20/12/2012																										
	Minimum			0.00	0.00	6.75		0.70	9.0	8.70	89	209	0.001	0.0001	0.001	0.0005	0.001	0.0005	0.022	0.00	0.00	0.00	0.00	0.28	0.36	0.01	0.001	
	Maximum			0.00	0.00	7.21		14.80	16.6	87.30	181	209	0.001	0.0001	0.001	0.0005	0.001	0.0005	0.022	0.02	0.00	0.15	0.15	0.36	0.42	0.01	0.010	
	Median					6.98		7.75	12.8	48.00	135	209	0.001	0.0001	0.001	0.0005	0.001	0.0005	0.022	0.01	0.00	0.08	0.08	0.32	0.39	0.01	0.006	



Forrestfield Groundwater Level Monitoring - Forrestfield -Airport Link

Desktop Review

Prepared for
Public Transport Authority
by Strategen

May 2016

strategen ENVIRONMENTAL

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Forrestfield Groundwater Level Monitoring - Forrestfield -Airport Link

Desktop Review

Strategen is a trading name of
Strategen Environmental Consultants Pty Ltd
Level 1, 50 Subiaco Square Road Subiaco WA 6008
ACN: 056 190 419

May 2016

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Limitations

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Environmental conclusions

Within the limitations imposed by the scope of services, the preparation of this report has been undertaken and performed in a professional manner, in accordance with generally accepted environmental consulting practices. No other warranty, whether express or implied, is made.

Client: Public Transport Authority

Report Version	Revision No.	Purpose	Strategen author/reviewer	Submitted to Client	
				Form	Date
Preliminary Draft Report	A	Client review	M Dunlop, D Jarvis/ P Bourgault	Electronic	3/3/2016
Draft Report	B	Client submission	M Dunlop, D Jarvis/ P Bourgault	Electronic	4/4/2016
Final Report	C	Client submission	M Dunlop, D Jarvis/ P Bourgault	Electronic	7/4/2016
Final for agency submission	0	Agency submission	M Dunlop, D Jarvis/ P Bourgault	Electronic	5/5/2016
Revised final for agency submission	1	Agency submission	M Dunlop, D Jarvis/ P Bourgault	Electronic	9/5/2016

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Appendix 2 Test pit locations and logs

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1. Introduction

The Forrestfield-Airport Link Project (the Project) is a new rail line to the eastern suburbs of Perth. The project will be delivered by a lead contractor who will be appointed in early to mid-2016.

Preliminary groundwater modelling of the Forrestfield Development Envelope (FDE) identified that the cone of depression from dewatering activities at the FDE (Dive Structure) potentially extends into surrounding Threatened Ecological Community (TEC) vegetation within Poison Gully and Lot 12 Ibis Place (Figure 1-1, Figure 1-2). The drawdown shown on Figure 1-2 is considered to represent the current worst case scenario (Golder 2015b). The more likely scenario is shown in Figure 1-3.

Advice from Department of Parks and Wildlife (DPaW) suggests that a shallow perched water table is present within Ibis Place and potentially the Poison Gully TEC.

Following the Environmental Protection Authority (EPA) assessing and recommending approval of the Project, the Minster for Environment approved the project on 13 November 2015 with the issue of Ministerial Statement 1022.

Condition 6-2 of Ministerial Statement 1022 requires that a Flora and Vegetation Monitoring Plan be prepared that "shall…

- detail baseline groundwater levels as close as practicable to Poison Gully and Lot 12 Ibis Place
- attach the results of the groundwater level baseline survey, include a report on the extent of perched groundwater levels at Poison Gully Creek and Lot 12 Ibis Place…"

Groundwater level monitoring is recommended to determine the level of connectivity between the regional aquifer, which will be subject to temporary construction dewatering, and any perched groundwater in the immediate surrounds of the dive structure at Forrestfield. The objective of this report is to review the existing geological and hydrogeological information and develop a baseline groundwater level monitoring program. This report forms Phase 1 of the Scope of Services set out in RFQ160300.

1.1 Scope

This report presents a desktop review of the existing bore network and site investigation data for the FDE and surrounds, specifically surrounding the proposed dive structure (site of dewatering works) and the neighbouring TECs at Poison Gully and Ibis Place.

Based on the Scope of Services, this desktop review:

- assesses the existing information, including available geotechnical and hydrogeological data, to determine the existence or potential presence of perched groundwater within the Forrestfield Development Envelope and adjacent areas of TEC
- assess the suitability of the monitor bore network with regard to well distribution and depth
- where perched groundwater is identified, determine whether sufficient information is currently available to determine connectivity between the regional groundwater table and perched groundwater, and assess the potential for the dewatering to draw down the perched groundwater
- where insufficient information is available, develop a monitoring program to identify the existence/absence of shallow perched groundwater in the zone of influence of dewatering.

The Scope of Services requires that the proposed monitoring program includes:

- location of existing wells and access constraints
- location of new wells to ensure adequacy of data
- the specifications required for the installation of the wells
- post installation survey of the wells
- frequency and type of monitoring, e.g. bore loggers or frequency of dipping.



Figure 1-1: Site location and Threatened Ecological Communities

Scale 1:6,500 at A4

0 100 200 m

N

Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 26/02/2016
Author: DWhite
Source: Aerial image: ESRI online , approx. 2010.
All other data: Client 2015.
Path: Q:\Consult\2016\PTA\PTA16057\ArcMap_documents\R001\Rev A\PTA16057_01_R001_RevA_F001_1.mxd

Legend

- Poison Gully Creekline
- Development Envelope
- Stations
- Surface disturbance

Threatened Ecological Community

- Poison Gully TEC
- Ibis Place TEC

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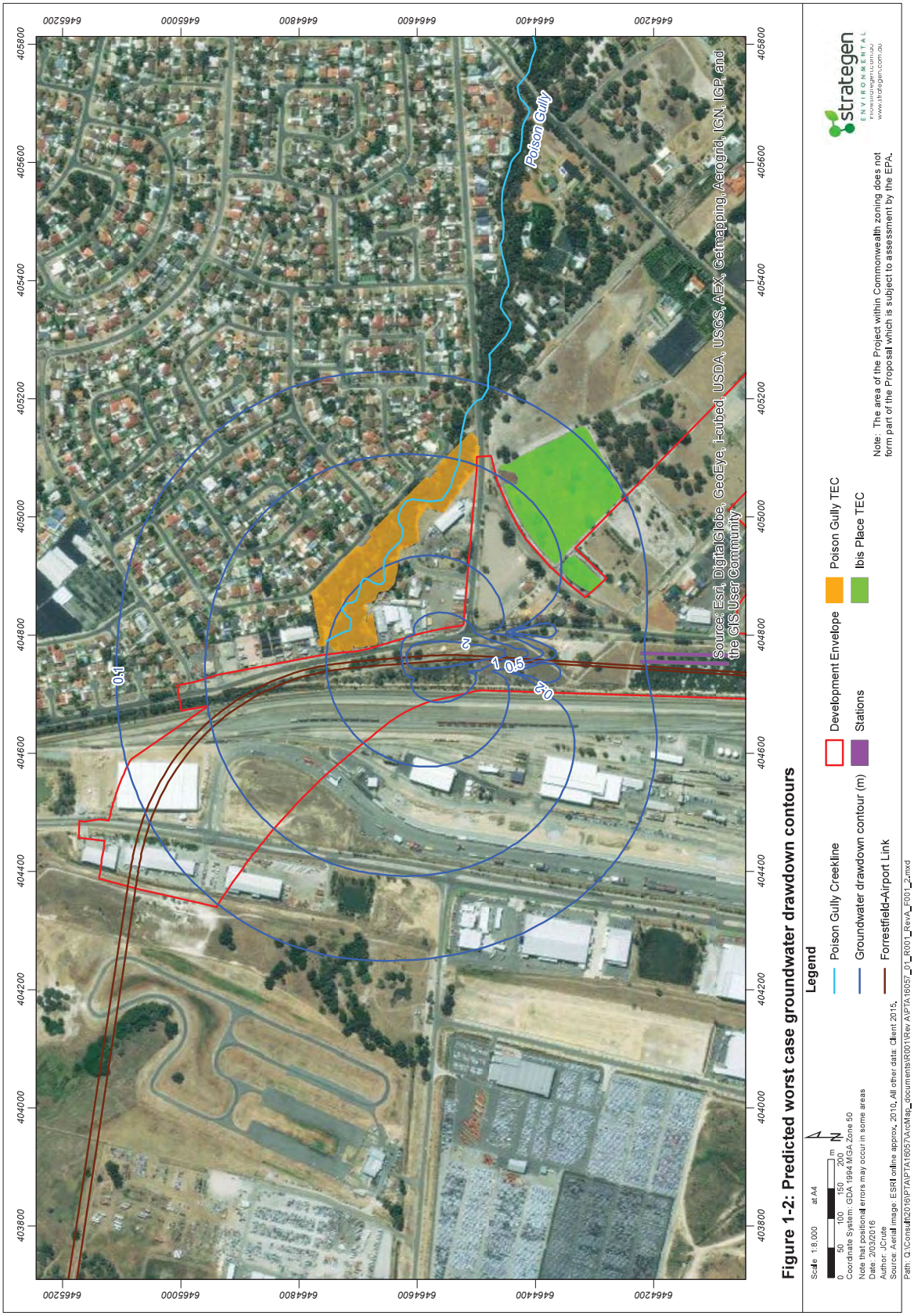
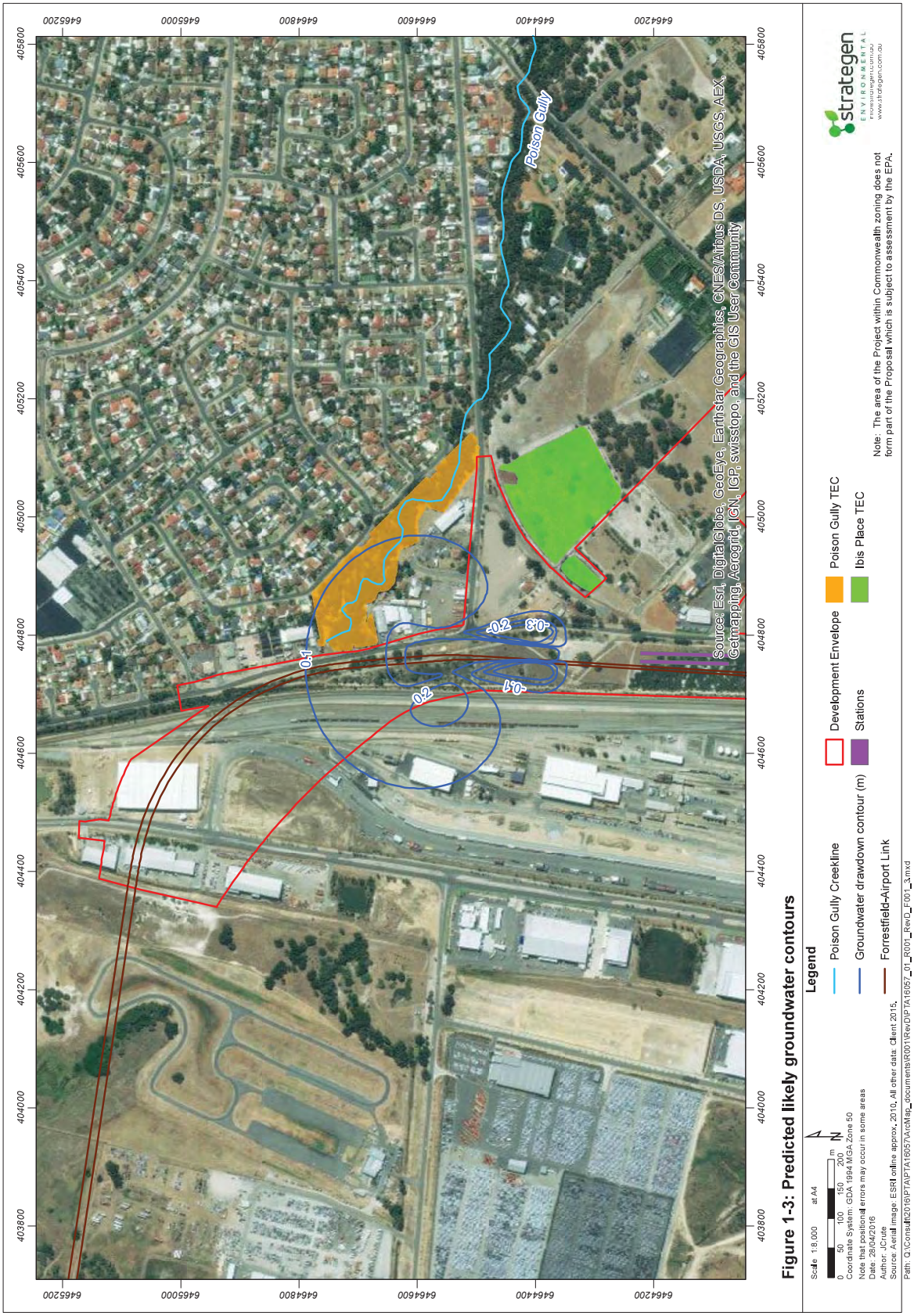

Poison Gully
Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, and the GIS User Community
0.1
1
0.5
0.2
Figure 1-2: Predicted worst case groundwater drawdown contours
Legend
Poison Gully Creekline
Groundwater drawdown contour (m)
Forrestfield-Airport Link
Development Envelope
Stations
Poison Gully TEC
Ibis Place TEC
Note: The area of the Project within Commonwealth zoning does not form part of the Proposal which is subject to assessment by the EPA.
Scale 1:8,000 at A4
0 50 100 150 200 m
Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 2/03/2016
Author: JCrute
Source: Aerial image - ESRI online approx. 2010, All other data - Client 2015.
Path: Q:\Consult\2016\PTA\PTA16057\ArcMap_documents\R001\Rev A\PTA16057_01_R001_RevA_F001_2.mxd


Poison Gully
0.1
0.2
0.3
0.4
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community
Figure 1-3: Predicted likely groundwater contours
Legend
Poison Gully Creekline
Groundwater drawdown contour (m)
Forrestfield-Airport Link
Development Envelope
Stations
Poison Gully TEC
Ibis Place TEC
Note: The area of the Project within Commonwealth zoning does not form part of the Proposal which is subject to assessment by the EPA.
Scale 1:8,000 at A4
Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 28/04/2016
Author: JCrute
Source: Aerial image - ESRI online approx. 2010, All other data - Client 2015.
Path: Q:\Consult\2016\PTA\PTA16057\ArcMap_documents\R001\Rev0\PTA16057_01_R001_Rev0_F001_3.mxd
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ENVIRONMENTAL
melbourne@strategen.com.au
www.strategen.com.au

1.2 Documents reviewed

Documents reviewed for this report include:

- *Contamination and Acid Sulphate Soils Investigation* (Golder 2015a)
- *Groundwater Conditions Report* (Golder 2015b)
- *Geotechnical Baseline Report* (PTA 2015)
- *Stage 2 Geotechnical Investigations - Factual Geotechnical Report* (Golder 2015c)
- *Stage 2 Geotechnical Investigations - Factual Geological and Hydrogeological Model Report* (Golder 2015d)
- *Stage 3 Geotechnical Investigation - Monthly Vibrating Wire Piezometer Monitoring Report no. 06* (Worley Parsons 2016)
- *FAL Project: Forrestfield Station Groundwater Levels Investigation* (Western 2015)
- *Summary of advice from other agencies* 25 February 2015 (OEPA 2015)
- *Forrestfield Airport Link Assessment on Proponent Information - Environmental Review Document* (Strategen 2015a)
- *Forrestfield Airport Link Surface Water Monitoring Report* (Strategen 2016)
- *Forrestfield North District Structure Plan – District Water Management Strategy* (Strategen 2015b).

DPaW has raised questions about the presence of perched groundwater at Ibis Place and in the surrounding areas to the Office of the Environmental Protection Authority (OEPA 2015). The evidence of perched groundwater is based on a report by Ecoscape (2010) titled *Threatened Ecological Community (TEC) Assessment of Lot 12 Ibis Place, High Wycombe.* This report included information obtained from installing five piezometers in the Ibis Place TEC. DPaW has provided PTA with a one page extract of this report but this does not include information regarding piezometer installation and geology.

2. Desktop review

2.1 Conditions required for perched groundwater

Rainfall infiltrating into a soil profile will percolate downwards with gravity at a rate governed by the permeability of the surface soils. In circumstances where the percolating rainfall encounters a soil layer of lower permeability, the downward movement of water is impeded and it is possible the soil above the layer of lower permeability becomes saturated. If a borehole or equivalent is placed into this zone of saturation, free water will flow into the void made by the hole. In many instances this free water is referred to as perched groundwater. Figure 2.1 illustrates this concept.

Perched groundwater can occur at any point in the soil profile where an impeding layer occurs above the permanent superficial aquifer. When present the perched groundwater reflects a localised zone of saturation within the vadose (unsaturated) zone above the permanent watertable. In some circumstances where there is sufficient recharge and sufficient thickness of permeable soils over less permeable soils the presence of perched groundwater can be semi-permanent to permanent. Soils beneath the impeding layer remain unsaturated and are not directly hydraulically connected to the underlying permanent superficial aquifer.

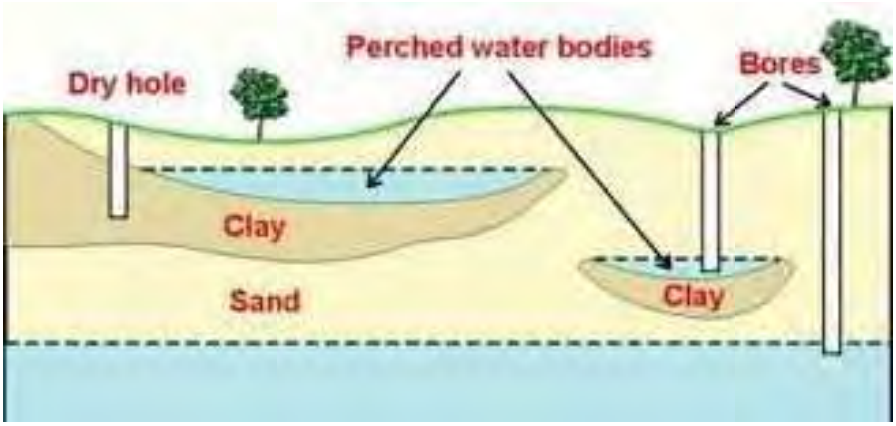

Figure 2-1: Conceptual cross section of perched water body

The presence of perched water can be seasonal. In this case, perched water may be present during the wetter months but dried out during summer as a result of evapotranspiration by vegetation and slow percolation of water through the impeding layer into the regional aquifer.

Understanding the potential for perched water requires an understanding of the soil type, soil texture, underlying geology and hydrology of a site. For the purposes of this report, the term 'impeding layer' has been used to describe less permeable layers in the soil profile above which perched groundwater may occur. Until information regarding the relative hydraulic conductivities of soil strata has been gathered, it is difficult to determine if any soil layers are sufficiently impermeable to impede the vertical percolation of infiltrating rainwater to allow saturation above the impeding layer to occur and perched groundwater to be present.

2.2 Soil investigations

2.2.1 Surface environmental geology mapping

Surface environmental geology mapping for the Perth Metropolitan Region (Gozzard 1986) indicates that the soils at the site are from the S_{10} soil unit, being thin Bassendean Sand over Guildford Formation (Qpb/Qpa) (Figure 2-2). Bassendean Sand is defined as the S_8 soil unit, very light grey at surface, yellow at depth, fine to medium grained, sub-rounded quartz, moderately well sorted of eolian origin.

The upper profile of soils from the S_{10} soil unit are typically layers of Bassendean Sand with interbedded lenses of clay derived from the Guildford Formation. The sands from the Guildford Formation may also be expressed at the top surface of the soil profile. Seasonal perched groundwater is common in this soil type.

2.2.2 Soil investigations

Geological investigations have been undertaken within the area of surface disturbance and have included test pitting and the construction of bores at the locations shown on Figure 2-3. The following represents a summary of the geological conditions based on the bore logs provided to Strategen for the MW 3 series of bores shown in Figure 2-3 and test pits as presented in Appendix 2. Information was available for 22 bores installed between 20 October 2014 and 11 May 2015. Bore logs are presented in Appendix 1.

Surficial fill

At MW3-001, MW3-009, MW3-034 and MW3-038 a layer of sand to sandy gravel fill has been placed on the site. The Golder logs identify that the fill depth ranges from 0.8 m at MW3-034 to 4 m at MW3-009. On review of the logs following observations of soils in the TEC, Strategen considers that the brown orange and yellow orange sands to sandy gravel in the Golder logs are natural soils and represent the surficial layer of the Guildford Formation. On this basis, Strategen considers the actual fill depth at these bores to be more likely 0.2 m (MW 3-001) to 0.9 m (MW 3-009) with the possibility that some of the grey sands above these layers being misidentified Bassendean Sand.

Bassendean Sand

A layer of sand is present at the top of the profile at most locations. This sand was confirmed as being from the S_{10} Formation, being “light grey, yellow, dark brown, fine to medium grained sand, loose to dense” (Golder 2015d). Some of this sand layer contains gravel or had orange to red colouring, indicating that these gravels may actually be derived from the reworked Guildford Formation that occurs below the sand. For the purposes of this report, this sand layer with intermittent gravels is referred to as Bassendean Sand. Where present, the material ranges from 0.3 to 3.5 m in thickness, with base levels between 24.18 mAHD (MW 3-001) and 29.67 mAHD (MW 1-01 A).

This top layer of sand was not present at MW 3-013, MW3 -038, MW 3-044, MW 3-051, MW 3-052 or MW 3-055 (Table 2-1). Apart from MW 3-013, these are all located in the north of the Site near Poison Gully (Figure 2-3). MW 3-013 is located near Milner Road, to the south of Ibis Place.

Guildford Formation

The gravelly and clayey soils of the Site are primarily associated with the Guildford Formation. Golder 2015 (d) described the Guildford Formation in the Forrestfield portion of the Project as consisting of “pale grey and brown, fine to coarse grained, rounded to sub-angular, medium dense to very dense sand, silty sand and clayey sand with trace quartz gravel generally less than 5 mm of size. The Guildford Formation also contains a sequence of mottled orange, low plasticity, stiff to hard clays in the Forrestfield area that are generally present within 10 m of ground surface and being up to 6 m thick”.

Table 2-1: Summary of geology and potential for perched groundwater

Name	Ground level (mAHD)	Bottom of sand and fill (mbgl)	Bottom of sand (mAHD)	Top of potentially impeding layer (mbgl)	Top of potentially impeding layer (mAHD)	Type of impeding material	Moisture present over dry soil?	Top of screen (mbgl)
MW 3-001	27.78	3.60	24.18	Not present	Not present	Not present, clayey sand only	No	3.00
MW 3-004	30.00	2.10	27.90	Not present	Not present	Not present, silty clayey sand only	No	5.00
MW 3-009	30.20	5.50	24.70	5.50	24.70	sandy clay	No	3.90
MW 3-012	29.24	0.30	28.94	0.30	28.94	silt then silty clay at 4.8 mbgl	No	7.50
MW 3-013	34.68	Not present	Not present	2.00	32.68	clayey silt	No	16.30
MW 3-019	30.61	1.00	29.61	Not present	Not present	Not present, clayey sand only	No	3.50
MW 3-026	28.39	0.60	27.79	3.00	25.39	sandy gravelly clay, then sandy clay	No	5.00
MW 3-034	31.23	2.20	29.03	2.20	29.03	clayey silt	No	4.00
MW 3-038	29.33	1.50	27.83	1.50	27.83	sandy clay	No	4.50
MW 3-044 D	29.54	Not present	Not present	1.40	28.14	clayey silt	No	11.00
MW 3-044 S	29.50	Not present	Not present	1.40	28.14	clayey silt	No	3.30
MW 3-047	30.36	1.50	28.86	1.50	28.86	clayey silt	No	9.00
MW 3-051 D	29.41	Not present	Not present	2.40	27.01	clayey silt	No	15.50
MW 3-051 S	29.39	Not present	Not present	2.40	28.19	clayey silt	No	4.00
MW 3-052	28.35	Not present	Not present	2.00	26.35	clayey silt	No	7.00
MW 3-055	28.81	Not present	Not present	1.50	27.31	silty clay	No	4.00
MW 1-01 A	30.47	3.50	26.97	5.30	25.17	Clay	Moisture not logged	8.00
MW 1-01B	29.33	1.00	28.33	Not present	Not present	Not present	Moisture not logged	8.50
MW 1-01 C	30.47	2.15	28.32	Not present above 0 mAHD (i.e. above sea level)	Not present above 0 mAHD	Not present above 0 mAHD	Moisture not logged	7.00
MW 1-01 D	30.19	5.00	25.19	Not present above 0 mAHD (i.e. above sea level)	Not present above 0 mAHD	Not present above 0 mAHD	Moisture not logged	7.00
MW 1-05	28.42	1.00	27.42	Not present	Not present	Not present	Moisture not logged	7.00
PW 1-01	30.32	2.00	28.32	Not present	Not present	Not present	Moisture not logged	7.00

City of Kalamunda
Ordinary Council Meeting 12 October 2021 Attachments
Attachment 10.5.2.3
1825

The geology of the Guildford Formation as present in the bore logs is highly variable and includes units from sandy gravel to clays. From a perched groundwater perspective, the key factors are the presence of fine grained layers which may impede downward percolation of water. Where the majority component of a soil unit is described as silt or clay and the material above is predominantly sand, the difference in permeability may be sufficient for seasonal perched groundwater to occur. A silty or clayey sand was not considered to offer adequate difference in permeability to significantly impede the downwards movement of water and thus cause perched groundwater on top of the layer.

Test pits

The test pits along the south-western edge of the Ibis Place TEC indicate:

- TP 1-46: sand then sandy clayey gravel 'weakly to very weakly iron cemented mass' at 0.4 m then refusal at 1.3 m (no groundwater encountered, excavated on 2/12/14)
- TP1-49: sand over moist gravelly clayey sand 'weakly to well cemented' at 0.7 m then refusal at 2.4 m (no groundwater encountered, excavated on 2/12/14)
- TP1-20: sand then moist interbedded sand and clayey sand 'weakly to well cemented lateritic gravel at 1.3m, no refusal (no groundwater encountered, excavated on 12/12/14) (Appendix 2).

The material causing refusal at TP1-46 and TP1-49 could possibly be a cemented perching layer, but there no evidence of actual perching (i.e. no saturation was encountered).

TP1-28 and TP1-29 are located in the vicinity of Poison Gully

- TP1-28: gravelly sand, then sand over clayey sand and then sandy clay at 1.5 mbgl. No evidence of groundwater to 2.1 mbgl during excavation on 15/10/14. Base of hole at approximately 27.15 mAHD
- TP1-29: sandy clay over sandy gravel then clayey sand to 2.6 m. No evidence of groundwater to this depth during excavation on 27/10/14. Base of hole at approximately 26.62 mAHD (Appendix 2).

The bases of the holes appear to be above the invert of the creek at this location. As such, this does not provide any indication of whether the creek is intersecting perched groundwater, which could be present below the base of the holes.

Data gaps for the assessment of perched groundwater

Based on the observed soil profiles, layers with the potential to impede flow were considered to occur in 14 of the 22 bores. Where impeding layers were present, the top of the impeding layer varied from 0.3 m (MW 3-012) to 5.5 mbgl (MW 3-009), with the majority being less than 2.5 m below the surface. The elevation (in mAHD) of these impeding layers varied from 24.7mAHD (MW 3-009) to 32.68 mAHD (MW 3-013). In all cases, the bores were only screened below the impeding layer (Table 2-1).

Logs from four bores installed by Strategen in September 2011 for the Shire of Kalamunda (SoK) in the broader Forrestfield area (MB01, MB07, MB08 and MB09) did not record any potential impeding layers, with sand or gravelly sand/sandy gravel at the surface and then clayey sand at depths between 2.25 and 3.5 mbgl to the end of the hole (Strategen 2015b). Locations of these bores are shown on Figure 2-3.

The geology quoted by DPaW for Ibis Place is consistent with very shallow Bassendean Sand over the Guildford Formation, however the bore logs for these bores have not been provided to Strategen.

The bores and test pits are focussed on investigating the upper soil profile of the regional superficial aquifer along the proposed alignment. One bore (MW 3-034) has been located adjacent to Ibis Place and one on Milner Road (MW 3-013) (Figure 2-3). Only MW 3-055 is located to the north of Poison Gully. Given the complex geology of the subject area, the current geological information is considered limited within and surrounding the TECs. Information in bore logs is variable and determining the presence or otherwise of impeding layers is difficult. Additional geological information would be beneficial.

2.3 Hydrogeology

Ecoscape (2010) report

Ecoscape (2010) reported a depth to groundwater in August 2008 of less than 0.4 m in five piezometers installed within the Ibis Place TEC. The soil samples taken from the installation of the bores showed a "shallow layer of wet sand approximately 10 cm thick over a wet clayey sand, to wet sandy clay, with dry mottled clay at a depth of approximately 1 m" (Ecoscape 2010, quoted in DPaW undated). These readings were taken in winter and it is possible that the perched water was ephemeral and resulted from a period of rain that occurred leading up to the monitoring event. The exact date of the monitoring event has not been provided to allow comparison to rainfall records.

These water levels are in contrast to the groundwater levels at the adjacent PTA bore MW 3-034, which was dry during monitoring by Western Environmental in July and October 2015 (Western 2015). MW 3-034 was installed to a depth of 6.3 metres below ground level (mbgl) and was screened between 3.3 and 6.3 mbgl. Water may have been present as a perched layer above this screened depth.

Evidence from bore installation and test pits

If perched groundwater was present during bore installation or test pitting, a layer of wet soil would have been found over a dry impeding layer. Such perched groundwater conditions were not encountered during drilling or test pitting, indicating that permanent perched groundwater is unlikely to be present. Bore logs were available to Strategen for the bores listed in Table 2-1 and four bores installed for SoK by Strategen (Figure 2-3). Test pit logs were available for locations in the vicinity of both the Ibis Place TEC and Poison Gully (Appendix 2).

These bore logs show that perched groundwater was not present during bore or test pit installation. Groundwater levels on the Swan Coastal Plain generally peaks in October, and as such perched groundwater would be more likely to be present at this time of year. In terms of timing of installation:

1. Installation of the Golder bores for which soil moisture was logged occurred during January and February 2015, when ephemeral perching would be unlikely to be present.
2. Strategen bores were installed in September 2011, close to peak groundwater when ephemeral perching would be more likely to be expressed.
3. Of the Golder test pits, two were installed in October 2014 (around peak groundwater) and three in December 2014. Perched groundwater is likely to be found if present during October but is less likely to be present in December.

The lack of perching in the pits and bores constructed in September and October indicates that perching is not present at these locations.

Screened depth

Of the bores installed to date, none have been screened in the top 3 m of the profile. Top of screen levels vary from 3.0 mbgl at MW 3-001 to 16.3 mbgl at MW 3-013 with bottoms of screens located from 5.9 to 32.5 mbgl. All of the bores that had intersected impeding layers were screened below the top of the impeding material.

Groundwater levels encountered during the summer installation period were below this level and thus the shallowest bores were screened across the groundwater table, as is standard practice. A minimum of 0.5 m of bentonite was placed above the screens to ensure that water from aquifers above the aquifer being monitored did not affect water levels in the bore. Assuming this process was effective and the top of all screens are located below the top of potentially impeding layers, groundwater levels in these bores will not accurately reflect water levels occurring above the impeding layer.

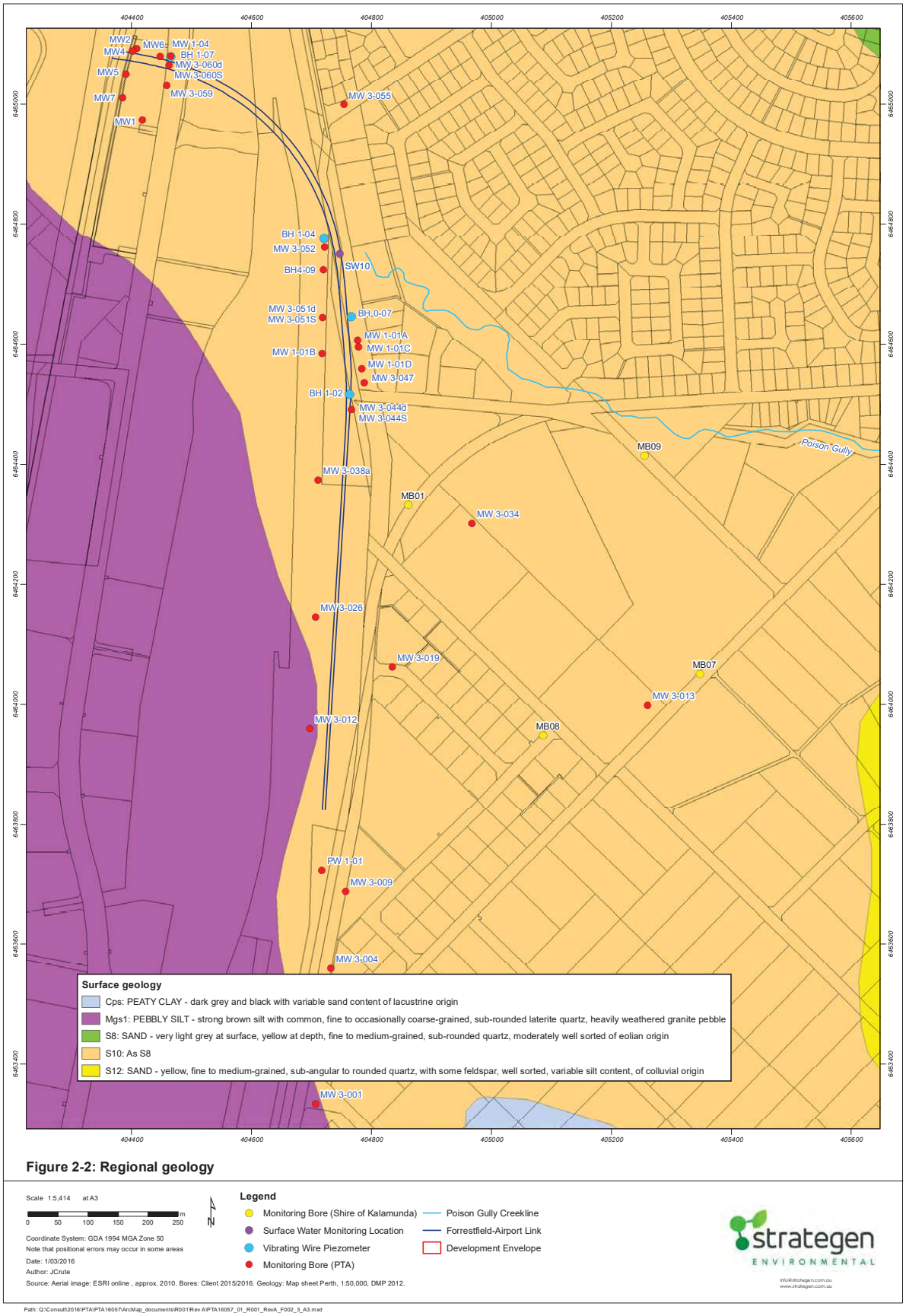

Figure 2-2: Regional geology

Scale 1:5,414 at A3

0 50 100 150 200 250 m

Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 1/03/2016
Author: JCrute

Source: Aerial image: ESRI online , approx. 2010. Bores: Client 2015/2016. Geology: Map sheet Perth, 1:50,000, DMP 2012.

Legend

- Monitoring Bore (Shire of Kalamunda)
- Surface Water Monitoring Location
- Vibrating Wire Piezometer
- Monitoring Bore (PTA)
- Poison Gully Creekline
- Forrestfield-Airport Link
- Development Envelope

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Figure 2-3: Groundwater and surface water monitoring locations

Scale 1:5,414 at A3

0 50 100 150 200 250 m

Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 31/03/2016
Author: JCrute
Source: Aerial image: ESRI online , approx. 2010. All other data: Client 2015/2016.

Legend

- Monitoring Bore (Shire of Kalamunda)
- Surface Water Monitoring Location
- Vibrating Wire Piezometer
- Monitoring Bore (PTA)
- Open Pit Observations
- Poison Gully Creekline
- Forrestfield-Airport Link
- Development Envelope
- Poison Gully TEC
- Ibis Place TEC

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Vibrating wire piezometers

Vibrating wire piezometers (VWPs) have been installed at three locations to monitor groundwater pressure/levels at depth (Figure 2-3). The VWPs record water levels on a minimum of daily frequency. VWPs are located at:

- BH0-07 contains at 12.5 mbgl, monitored since April 2014
- BH1-02 at 12.5 mbgl and 24.5 mbgl, monitored since October 2014
- BH1-04 at 9.8 mbgl and 13.7 mbgl, monitored since October 2014.

Strategen understands that the VWP monitoring program will continue until May/June 2016. The VWPs have been installed too deep to represent any perched groundwater but will provide useful historical water level data.

Groundwater levels

The PTA bores were monitored for groundwater levels on:

- 10 March 2015 by Golder
- 23 July 2015 by Western Environmental
- 20 October 2015 by Western Environmental
- 26 February 2016 by Strategen.

The SoK bores were monitored on 12 October 2015 by Gecko Environmental. The only SoK bore located by Gecko (MB09) was dry. Strategen attempted to locate the SoK bores on 26 February 2016, but only MB09 could be located. The SoK bores were installed in the road reserve, so it is likely that these have been either destroyed or covered over with material (e.g. concrete) where flush covers were used.

The only bore showing water levels clearly above an impeding layer is MW 3-009. The water level on 20 October 2015 was 24.86 mAHD, with an impeding layer logged as being present at 24.70 mAHD. Groundwater levels at this bore varied from 24.54 mAHD to 24.86 mAHD during the monitoring events, and thus has been both above and below the top of the impeding layer. This is consequently not considered to be evidence of consistent perching.

Data gaps for assessment of perched groundwater

The bores installed by Golder have been designed to monitor the deeper parts of the superficial aquifer that will be directly affected by dewatering, close to the proposed alignment. Additional monitoring is recommended in the vicinity of the TECs to better understand the hydrology of these areas.

2.4 Surface water

Poison Gully (Plate 1 to Plate 3) is an ephemeral creek which collects water from elevated areas to the east of the Site including parts of Kalamunda and flows in a westerly direction via the Perth Airport Northern Main Drain and Limestone Creek into the Swan River. Between Maida Vale Road and the location to the west of Dundas Road, where Poison Gully becomes a piped drain, the creek is incised up to approximately 2 m below the normal ground surface. No surface water was observed in this section of Poison Gully during a site visit on 26 February 2016. During 2015, the creek was dry at SW10 in the vicinity of the Site during two of four surface water monitoring events (March and December) (Strategen 2016). The location of SW10 is shown on Figure 2-3.

Groundwater monitoring undertaken by Strategen for Shire of Kalamunda in the Forrestfield area indicated a southerly regional groundwater flow direction in the area to the south of Poison Gully in October 2011 (Strategen 2014). The study area and bores for this project were predominantly located to the east of FDE, with the easternmost bore located in Ibis Place. The implication of this flow direction is that Poison Gully may be a losing stream (i.e. one that recharges groundwater) during the period over which it flows. In saying this, the study was undertaken at a regional level and the majority of the project area was located to the east of the area of interest for this study, within soils mapped as deep Yoganup Sands rather than the Guildford Formation.



Plate 1: Poison Gully, February 2016



Plate 2: Poison Gully, February 2016



Plate 3: Poison Gully, February 2016 showing erosion

Surface water levels recorded by Strategen (2016) at SW10 were approximately 26.5 mAHD on 3 June and 3 September 2015. The maximum groundwater level recorded by Western Environmental during their July and October 2015 groundwater monitoring events was 25.31 mAHD in MW 1-01A on 23 June. This implies that the water level in Poison Gully when flowing is generally above the regional groundwater level. This implies that the general direction of water flow when Poison Gully is flowing would be from Poison Gully to the regional groundwater (i.e. this section of Poison Gully is a losing stream).

Poison Gully has previously been surveyed by Water Corporation (2010) as part of a drainage study of the area. Strategen has been provided with this information for another job under the condition that it is used for that purpose. Permission to use this information should be obtained by PTA to assist in determining the interaction between Poison Gully and any perched groundwater.

2.5 Observation of open pits

On the 24 March, Strategen inspected several open pits ranging in depth from 1-2.5m to assess the textural differences within the soil profile. Two pits to approximately one metre depth where inspected along Raven Road on the north-eastern side of the Ibis Road TEC (Plate 4). Another large excavation along the alignment of the gas pipeline running parallel with Dundas Road on the western side was inspected (Plate 5). This excavation was close to the culvert on Dundas Road, through which Poison Gully flows. The approximate locations of the observation pits are shown on Figure 2-3.

The soil profile along Raven Road was found to be grey fine-medium sand to about 0.3m depth over yellow to orange clayey sand (~5% clay) with weathered coffee rock. This soil, which also contains a large percentage of gravel, is unlikely to retard downward percolation of rainwater sufficient to result in perching during normal flows. High intensity rainfall may be sufficiently retarded above the clayey sand for short periods to cause isolated pockets of water saturation, but this would be of very short duration.

The soil profile along the gas pipeline alignment had three distinct horizons. The surface 0.3-0.4m is grey fine-medium sand, typical of Bassendean sand. The underlying 0.4-1.0m of soil profile textured as a clayey sand and was similar to the soil encountered in Raven Road. At a depth of ~0.8m there is a distinct textural change from the clayey sand to sandy clay. The sandy clay is quite plastic but the sand grains are obvious. Based on the bolus formed by wetting some of the sandy clay material and manipulating it in the hand and forming a ribbon by squeezing between the thumb and forefinger (The National Committee on Soil and Terrain, 2009), it is estimated there is about 25% clay. During normal rainfall events, this textural difference at about 1m is possibly sufficient to retard downward percolation of rainfall for long enough to result in minor amounts of water saturation within the soil profile (perched groundwater). However it was noticed there were regular macropores (cracks and old root channels greater than 2 mm in diameter) that would conduct any perched groundwater deeper into the soil profile (Plate 6). Macropores can rapidly convey water through the unsaturated zone of the soil profile, draining areas of temporary saturation (Beven and Germann 1982; Bourgault Du Coudray 1996).

At all inspected locations, the soil profile was dry and showed no evidence of perching. It is highly unlikely there is any connectivity between potential perched groundwater layers and the regional superficial groundwater table.

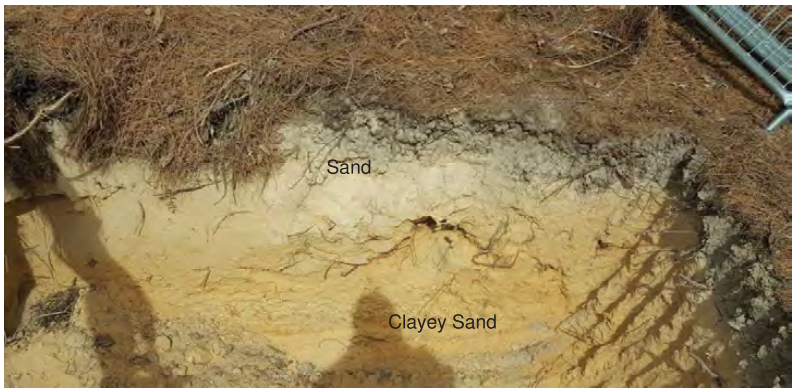

Plate 4: Soil profile at OP2



Plate 5: Soil observations at OP1

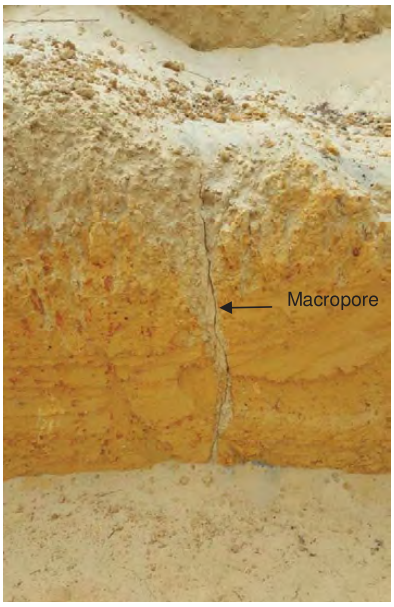

Plate 6: Macropore through sandy clay layer at OP1 and natural gravel within the profile

2.6 Potential for perched groundwater

Apart from the Ecoscape (2010) report, there is no conclusive evidence that perched water occurs near the proposed Forrestfield area. It is possible that the perched groundwater seen by Ecoscape is temporary perched groundwater following the retardation of percolating rainfall by a impeding soil layer of lower hydraulic conductivity, following a short period of relatively high intensity rainfall. Based on the inspection of the soil profile near Raven Road and adjacent to Dundas Road, there is sufficient textural difference between overlying sands and underlying sandy clay to give rise to the possibility of localised temporary perched groundwater following rainfall. The refusal in test pits adjacent to Ibis Place TEC is also indicative of a cemented layer that may cause perching.

Soil logs show potential impeding layers that may cause perched groundwater after rainfall are present at 13 out of 16 Golder bores and at TP 1-46 and TP 1-49 on the southwestern boundary of the Ibis Place TEC. However, perched water was not encountered during bore installation in January/February 2015 or test pitting. This suggests that if there is any perched groundwater, it is not permanent. A potentially impeding layer was noted at a depth of 2.2 mbgl at the bore closest to Ibis Place and in the form of refusal during test pitting at 1.3 m and 2.4 mbgl at TP1-46 and TP 1-49 respectively. There is no evidence of hydraulic connection between the perching layers and the underlying superficial aquifer.

Where potential impeding layers were found, they were of variable texture, depth and thickness. This implies that temporary perched groundwater may exist in some areas but not others.

If permanent perched groundwater occurred and was above the level of Poison Gully, springs or seeps where lateral flow of the perched water intercepted the creek would be expected. If significant volumes of perched water were present and seeping into Poison Gully permanent to semi-permanent pools would be expected along the creek line. There is no evidence of subsurface seepage into Poison Gully from perched groundwater, even in areas where the creek is deeply incised.

2.7 Conclusions

The groundwater level monitoring to date has been focussed on the regional superficial aquifer. While more information regarding the hydrology of the TECs would be beneficial, the evidence available indicates that any perched groundwater that may occur is likely to be ephemeral and limited to isolated areas. Hence temporary lowering of the regional groundwater table is unlikely to impact the TECs adjacent to the FDE.

3. Proposed future works program

3.1 Groundwater monitoring

3.1.1 Bore specifications

The proposed additional monitoring bore locations are shown in Figure 3-1. Bore depth will depend on the geology and presence of groundwater at the drilling location.

Monitoring bores will be installed as a pair or shallow bore only on the following basis:

1. As a pair of monitoring bores, typically 2 to 5 metres apart, where a deep bore is not currently present and a better understanding of the interaction between the shallower and regional (deeper) superficial aquifer is required. The shallow bore will be screened so that it is above the first impeding layer that is encountered in the profile (typically from 0 to 3 mbgl) to intersect any seasonal perched groundwater that may occur in that location. The deepest of the pair will be screened below any confining layer (typically at depths of more than 4 mbgl, but not more than 8 mbgl) so as to intersect the regional groundwater table. If no impeding layer is identified, the shallow bore will still be installed to a depth of 2mbgl.
2. As a shallow bore on top of the impeding layer at a location where a deeper bore already exists or as a supplementary shallow bore located adjacent to the TEC. Where this is not located adjacent to a deeper bore, the hole will extend a minimum of 300 mm into the potential impeding layer to confirm that perched groundwater is likely to be present and then the hole refilled to the top of the impeding layer. If no impeding layer is identified, then the bore will be installed to depth of 2mbgl.

Bores will be constructed in accordance with the requirements outlined in the Land and Water Biodiversity Committee (2003) guidance document *Minimum Construction Requirements for Water Bores in Australia.* Minimum bore internal diameter shall be 50 mm based on this guidance and to allow installation of 41 mm diameter data loggers in any bore.

The screening intervals for all bores will be carefully selected based on the soil profiles encountered at each location. A screened interval of 2-3 m will be installed either completely above (shallow bores) or completely below (deep bores) any impeding layer that is identified.

The soil lithology will be logged for all new bores proposed for installation at the locations shown in Figure 3-1 and Table 3-1 by a suitably qualified environmental scientist. Where bores are paired as a deep and shallow set, logging is required for both bores. The deep bore shall be installed first and this geological information will be used to determine the depth of the shallow well. This will include logging of soil type, texture and moisture status at a minimum of every 20 cm down the hole. Bores will be installed using a push core drilling rig to ensure a clean core is obtained for accurate geological logging.

The proposed locations for the additional bores are provided in Table 3-1. A deep bore screened below the perching layer to access the regional groundwater table will be paired with MW3-009 as a potential control bore set (Figure 3-1). This location has been selected as a control bore as:

- a perched groundwater layer is known to occur in this area
- the bores are located outside the drawdown area shown on Figure 1-2.

Table 3-1: Additional monitoring bore locations

Label	Type	E (MGA94z50)	N (MGA94z50)
MW 8-001	A, D, K	404846.2	6464670
MW 8-002	B	404918.1	6464733.1
MW 8-003	A	404929.3	6464604.2
MW 8-004	B, K	405114.4	6464477.6
MW 8-005	A	405021.3	6464444.5
MW 8-006	A, D, K	404874.8	6464295.5
MW 8-007	A, D	405144.3	6464305.7
MW 3-009 (D)	R	404846.2	6464670

A: New Bore Pair
B: Shallow Bore Only
D: Data Logger Pair
K: Permeability Test
R: Deeper bore to Access Regional Groundwater Table

All new monitoring bores will be surveyed for Easting, Northing and Elevation (surface and top of casing in mAHD) to allow accurate assessment of the groundwater at the site.

3.1.2 Proposed monitoring program

Groundwater water levels will be measured on a monthly basis for all bores shown on Figure 3-1. It is anticipated that monitoring will commence in May 2016 and be ongoing. The amount of data collected should be maximised.

In addition to the monthly measurements, opportunistic monitoring immediately following daily rainfall greater than 10 mm, or following high-intensity rainfall, such as following a thunderstorm, even though the duration and total amount of rainfall is relatively modest. It may be necessary to visit the shallow bores several times following a rain event to ensure any temporary perched groundwater is captured.

The period from March to July is associated with seasonal low groundwater levels (typically March/April) followed by the subsequent rise of the watertable as the winter months approach. Heavy falls are likely to occur in the proposed monitor period, and the monthly and opportunistic monitoring of the shallow bores during these periods should provide sufficient data to help assess any potential impact to the TECs from drawdown in the regional water table.

Data loggers

Data loggers will be installed in three paired sets (i.e. six loggers in total) at the locations shown on Figure 3-1 (two associated with the Ibis Place TEC and one with the Poison Gully TEC). If possible, a data logger should be placed in one of the Ecoscape piezometers. These data loggers will log water levels on an hourly basis to observe the behaviour of water levels following rain. At this logging frequency, a battery life of at least four months is anticipated.

Ecoscape bores

Access to the Ibis Place piezometers installed by Ecoscape should be obtained if possible for monitoring purposes. These piezometers would need to be assessed prior to use to determine that they are still present and in a suitable condition for monitoring. If these bores are accessible and suitable, they should be included in the monitoring program and a data logger placed in one of the bores.

Vibrating wire piezometers

Strategen understands that the VWPs are intended to be monitored until May/June 2016. If possible, this program should be continued as long as practical up until the period of maximum groundwater levels (typically September). Data from the data loggers and dipping program should be compared to data from the VWPs to assist in investigating interactions between any perched and deeper superficial groundwater.

3.2 Soil investigations

3.2.1 Permeability testing

Permeability testing will be undertaken adjacent in hand augered holes adjacent to bores where impeding layers are identified using a falling head (Talsma-Hallam) permeameter based on methodology described in Appendix G of Australian Standard AS/NZS 1547:2012. The permeability tests will be undertaken in hand augered holes:

- within the permeable layer
- within the potentially impeding layer.

3.2.2 Poison Gully creekline

Obtaining soil logs from the Poison Gully creekline could be useful to determine the interaction between surface water in Poison Gully and groundwater. The creek is highly incised (approximately 2 m) and provides a useful geological cross section. During high flow periods, the sides of the creek are covered in clayey sediment, which sticks to the soil and can make such observations difficult. This section of Poison Gully is a significant site to Aboriginal people, and as such the sides of the creek should not be altered or damaged. Visual inspection should be undertaken and logging undertaken where it is considered that this will provide useful data (e.g. freshly eroded areas). This should ideally be done at a minimum of six locations between where the creek becomes a drain to the west of Dundas Road and Maida Vale Road, with the locations recorded by GPS and surveyed.

For safety reasons, any work in Poison Gully creek line should be done when the creek is dry.

3.3 Surface water

Monitoring of surface water levels in Poison Gully should continue at SW10 on the same frequency as the groundwater monitoring (Section 3.1.2). Data loggers are not proposed for Poison Gully.

As discussed in Section 2.4, survey information for Poison Gully should be obtained from Water Corporation to assist in determining the interaction between Poison Gully and any perched groundwater.

3.4 Potential ethnographic constraints

Investigations should be undertaken at the locations shown on Figure 3-1. Strategen notes that cultural reasons may prevent ground disturbing activities within vegetation areas adjacent to Poison Gully as well as the creek line itself. The cultural sensitivity of these areas should be confirmed with PTA prior to work commencing.

Should this be the case, locations shall be moved to a nearby location without such sensitivities. The movement of locations may impact on the relevance of the data obtained to the Poison Gully TEC. Where such movement of locations is required, this shall be identified within subsequent reporting and the reason for such movements identified.



Figure 3-1: Proposed monitoring locations

Scale 1:5,500 at A3

0 50 100 150 200 250 m

Coordinate System: GDA 1994 MGA Zone 50
Note that positional errors may occur in some areas
Date: 6/05/2016
Author: JCrute
Source: Aerial image: ESRI online , approx. 2010.
All other data: Client 2015/2016.

Legend

- New Monitoring Location
- Monitoring Bore (Shire of Kalamunda)
- Existing Surface Water Monitoring Location
- Vibrating Wire Piezometer monitored by Worley Parsons
- Existing Monitoring Bore (PTA) to be monitored
- Poison Gully Creekline
- Forrestfield-Airport Link
- Development Envelope
- Poison Gully TEC
- Ibis Place TEC

A: New Bore Pair
B: Shallow Bore Only
D: Data Logger Pair
K: Hydraulic Conductivity Testing
R: Deeper Bore to Access Regional Groundwater Table

strategen ENVIRONMENTAL
info@strategen.com.au
www.strategen.com.au

Path: Q:\Consult\2016\PTA\PTA 16057\ArcMap_documents\R001\RevD\PTA16057_01_R001_RevD_F003_1_A3.mxd

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4. Conclusion

The groundwater level monitoring to date has been focussed on the regional superficial aquifer. While more information regarding the hydrology of the TECs would be beneficial, the evidence available indicates that any perched groundwater that may occur is likely to be ephemeral and limited to isolated areas. Hence temporary lowering of the regional groundwater table is unlikely to impact the TECs adjacent to the FDE.

The data gathered from the additional groundwater level monitoring program outlined in this report will form the basis of further assessment of the seasonal behaviour of the Superficial Aquifer and localised areas of perched groundwater. This assessment will assist in the investigation of potential impacts to TECs in the area due to temporary drawdown of the regional watertable due to dewatering.

5. References

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APPENDIX 5 WATER CORPORATION CATCHMENTS (2010)

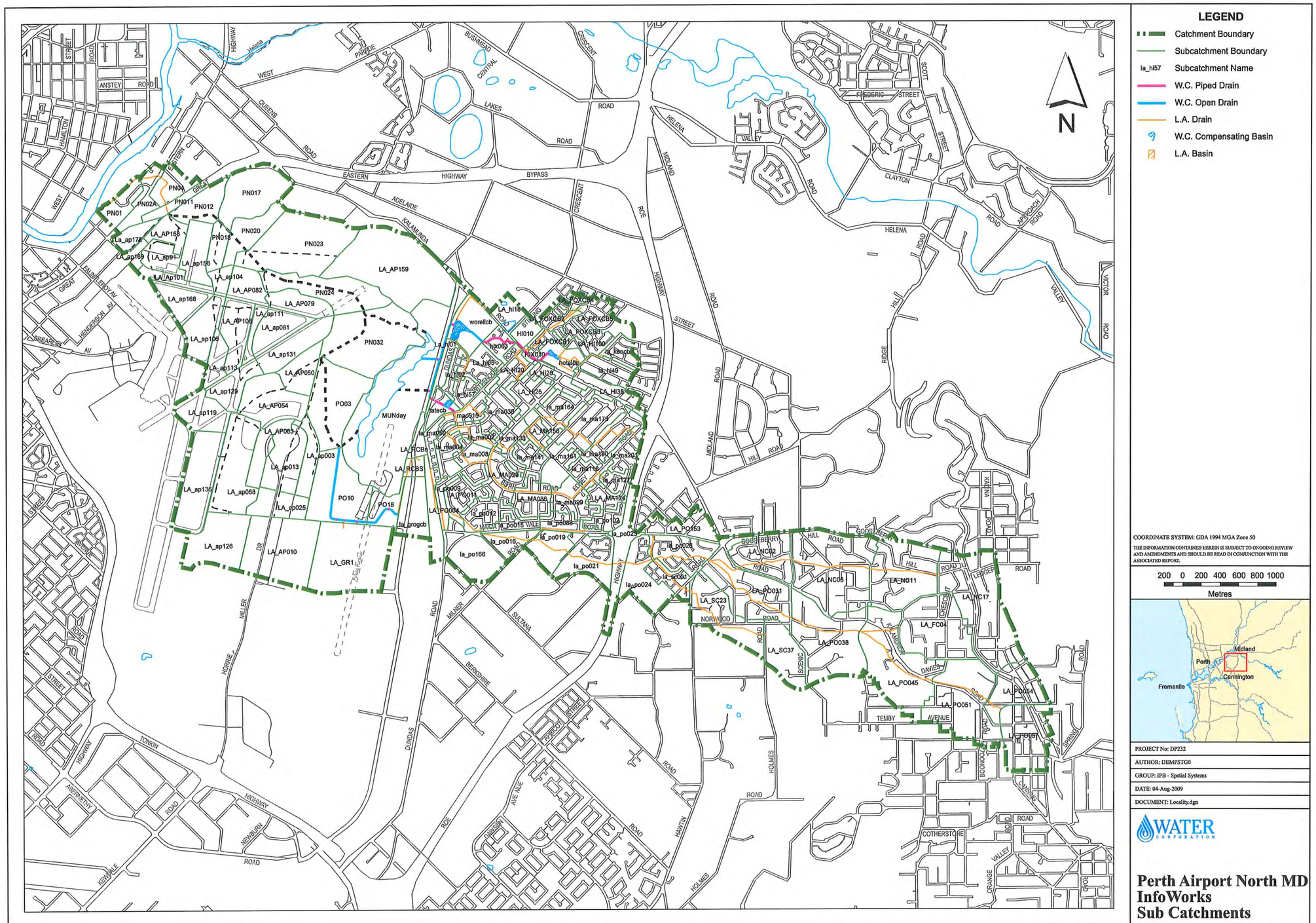

LEGEND
Catchment Boundary
Subcatchment Boundary
la_hl57 Subcatchment Name
W.C. Piped Drain
W.C. Open Drain
L.A. Drain
W.C. Compensating Basin
L.A. Basin
N
COORDINATE SYSTEM: GDA 1994 MGA Zone 50
THE INFORMATION CONTAINED HEREIN IS SUBJECT TO ONGOING REVIEW AND AMENDMENTS AND SHOULD BE READ IN CONJUNCTION WITH THE ASSOCIATED REPORT.
200 0 200 400 600 800 1000
Metres
Perth
Midland
Cannington
Fremantle
PROJECT No: DP232
AUTHOR: DEMPSTG0
GROUP: IPB - Spatial Systems
DATE: 04-Aug-2009
DOCUMENT: Locality.dgn
WATER CORPORATION
Perth Airport North MD
InfoWorks
Sub Catchments

Figure A.1

APPENDIX 6 UNDO RESULTS PRE-DEVELOPMENT

Government of Western Australia
Department of Water

Project: HW Pre-Dev

Date: 12-Dec-20

Version: Version 1.1.0.16333

Subregion name: Subregion 1

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	0	0.00	0.00	0.00
Industrial, commercial & schools	41	22.92	327.74	66.47
Rural living	31	17.33	852.59	58.92
Public open space	7	3.91	13.30	0.00
Road reserve	21	11.74	65.74	8.80

Total area (ha)	Total percent (%)
55.90	100

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
1551.73	142.57

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
91.47	2.01

Commercial, Industry and Schools

Landuse	Percent (%)	Area (ha)
Light industrial	100	22.92
Heavy industrial	0	0.00
Commercial / Offices	0	0.00
Schools	0	0.00
Public buildings	0	0.00

Total area (ha)	Total percent (%)
22.92	41

Nitrogen input (kg)	Phosphorus input (kg)
327.74	66.47

Rural living

Landuse	Percent (%)	Area (ha)
Unrestricted	0	0.00
No livestock	100	17.33
No clearing apart from the housing pad	0	0.00

Total area (ha)	Total percent (%)
17.33	31

Nitrogen input (kg)	Phosphorus input (kg)
852.59	58.92

Note: Commercial horticultre is not permitted in the rural living zone, due to spray drift buffers.

Public Open Space (POS)

Landuse	Percent (%)	Area (ha)
Native gardens	0	0.00
Non-native gardens	0	0.00
Not fertilised	15	0.59
Nature	85	3.33
Sport	0	0.00
Recreation	0	0.00
Golf course	0	0.00
Bowling green	0	0.00
Impervious	0	0.00
Water body	0	0.00

Total area (ha)	Total percent (%)
3.91	7

Nitrogen input (kg)	Phosphorus input (kg)
13.30	0.00

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	8.22
Road reserve - impervious	0	0.00
Road reserve - native garden	0	0.00
Road reserve - non-native garden	0	0.00
Road reserve - turf	5	0.59
Road reserve - not fertilised	25	2.93

Total area (ha)	Total percent (%)
11.739	21

Nitrogen input (kg)	Phosphorus input (kg)
65.74	65.74

Soil and drainage information

Type of drainage	Open channel drains	Does it contain imported fill?	No
Soil type	Pinjarra	Does subregion contain onsite sewage diposal system?	N
Depth to groundwater (m)	2		
Groundwater slope (%)	1		
Soil PRI	19.3		

Note: Please attach the results of soil tests to this report when submitting.

Summary: Nutrient stripping devices

Treatment	Name	Size (m²)	Treated area (ha)	Treating	N removed (kg/yr)	P removed (kg/yr)
Load removed					0.00	0.00
Net export					91.47	2.01

Summary: Nutrient load exports

Region	Area (ha)	P export (kg/yr)	N export (kg/yr)
Subregion 1	55.90	2.01	91.47

PRE-TREATMENT LOAD (kg/yr)		LOAD REMOVED (kg/yr)		NET LOAD EXPORT (kg/yr)	
NITROGEN	PHOSPHORUS	NITROGEN	PHOSPHORUS	NITROGEN	PHOSPHORUS
91.47	2.01	0.00	0.00	91.47	2.01

APPENDIX 7 UNDO RESULTS POST-DEVELOPMENT

Government of Western Australia
Department of Water

Project: High Wycombe

Date: 12-Dec-20

Version: Version 1.1.0.16333

Subregion name: Catchment 1

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	0	0.00	0.00	0.00
Industrial, commercial & schools	65	2.36	33.78	6.85
Rural living	0	0.00	0.00	0.00
Public open space	0	0.00	0.00	0.00
Road reserve	35	1.27	26.71	3.03

Total area (ha)	Total percent (%)
3.63	7

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
79.49	10.43

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
5.08	0.49

Commercial, Industry and Schools

Landuse	Percent (%)	Area (ha)
Light industrial	100	2.36
Heavy industrial	0	0.00
Commercial / Offices	0	0.00
Schools	0	0.00
Public buildings	0	0.00

Total area (ha)	Total percent (%)
2.36	65

Nitrogen input (kg)	Phosphorus input (kg)
33.78	6.85

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	0.89
Road reserve - impervious	0	0.00
Road reserve - native garden	15	0.19
Road reserve - non-native garden	0	0.00
Road reserve - turf	15	0.19
Road reserve - not fertilised	0	0.00

Total area (ha)	Total percent (%)
1.271795	35

Nitrogen input (kg)	Phosphorus input (kg)
26.71	26.71

Soil and drainage information

Type of drainage	**Piped drainage**	**Does it contain imported fill?**	**Yes**
Soil type	**Pinjarra**	Type of fill imported	**White sands (Bassendean)**
Depth to groundwater (m)	**3**	Fill depth (m)	**1**
Groundwater slope (%)	**1**	Approximate PRI of imported fill	**5**
Soil PRI	**14.6**	**Does subregion contain onsite sewage diposal system?**	**No**

Note: Please attach the results of soil tests to this report when submitting.

Subregion name: **Catchment 2**

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	37	1.54	32.08	9.75
Industrial, commercial & schools	13	0.54	14.25	3.46
Rural living	0	0.00	0.00	0.00
Public open space	36	1.50	5.98	0.00
Road reserve	14	0.58	12.21	1.39

Total area (ha)	Total percent (%)
4.15	8

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
86.24	15.21

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
5.07	0.21

Residential

Input load

Size (m²)	Percent (%)	Area (ha)	Nitrogen (kg)	Phosphorus (kg)
<400	0	0.00	0.00	0.00
400-500 m²	0	0.00	0.00	0.00
501-600 m²	0	0.00	0.00	0.00
601-730 m²	0	0.00	0.00	0.00
>730 m²	0	0.00	0.00	0.00
Multiple dwellings	100	1.54	32.08	9.75

Total area (ha)	Total percent (%)
1.536536	37

Nitrogen input (kg)	Phosphorus input (kg)
32.08	9.75

Commercial, Industry and Schools

Landuse	Percent (%)	Area (ha)
Light industrial	0	0.00
Heavy industrial	0	0.00
Commercial / Offices	0	0.00
Schools	0	0.00
Public buildings	100	0.54

Total area (ha)	Total percent (%)
0.54	13

Nitrogen input (kg)	Phosphorus input (kg)
14.25	3.46

Public Open Space (POS)

Landuse	Percent (%)	Area (ha)
Native gardens	0	0.00
Non-native gardens	0	0.00
Not fertilised	0	0.00
Nature	100	1.50
Sport	0	0.00
Recreation	0	0.00
Golf course	0	0.00
Bowling green	0	0.00
Impervious	0	0.00
Water body	0	0.00

Total area (ha)	Total percent (%)
1.50	36

Nitrogen input (kg)	Phosphorus input (kg)
5.98	0.00

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	0.41
Road reserve - impervious	0	0.00
Road reserve - native garden	15	0.09
Road reserve - non-native garden	0	0.00
Road reserve - turf	15	0.09
Road reserve - not fertilised	0	0.00

Total area (ha)	Total percent (%)
0.581392	14

Nitrogen input (kg)	Phosphorus input (kg)
12.21	12.21

Soil and drainage information

Type of drainage	**Open channel drains**	**Does it contain imported fill?**	**Yes**
Soil type	**Pinjarra**	Type of fill imported	**White sands (Bassendean)**
Depth to groundwater (m)	**2**	Fill depth (m)	**1**
Groundwater slope (%)	**1**	Approximate PRI of imported fill	**5**
Soil PRI	**12.2**	**Does subregion contain onsite sewage diposal system?**	**N**

Note: Please attach the results of soil tests to this report when submitting.

Subregion name: Catchment 3

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	83	3.45	72.36	22.00
Industrial, commercial & schools	0	0.00	0.00	0.00
Rural living	0	0.00	0.00	0.00
Public open space	0	0.00	0.00	0.00
Road reserve	17	0.71	14.83	1.68

Total area (ha)	Total percent (%)
4.15	8

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
108.90	24.31

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
6.95	1.14

Residential

Size (m²)	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
<400	0	0.00	0.00	0.00
400-500 m²	0	0.00	0.00	0.00
501-600 m²	0	0.00	0.00	0.00
601-730 m²	0	0.00	0.00	0.00
>730 m²	0	0.00	0.00	0.00
Multiple dwellings	100	3.45	72.36	22.00

Total area (ha)	Total percent (%)
3.446824	83

Nitrogen input (kg)	Phosphorus input (kg)
72.36	22.00

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	0.49
Road reserve - impervious	0	0.00
Road reserve - native garden	15	0.11
Road reserve - non-native garden	0	0.00
Road reserve - turf	15	0.11
Road reserve - not fertilised	0	0.00

Total area (ha)	Total percent (%)
0.705976	17

Nitrogen input (kg)	Phosphorus input (kg)
14.83	14.83

Soil and drainage information

Type of drainage	**Piped drainage**	**Does it contain imported fill?**	**No**
Soil type	**Pinjarra**	**Does subregion contain onsite sewage diposal system?**	**No**
Depth to groundwater (m)	**3**		
Groundwater slope (%)	**1**		
Soil PRI	**19.3**		

Note: Please attach the results of soil tests to this report when submitting.

Subregion name: **Catchment 4**

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	0	0.00	0.00	0.00
Industrial, commercial & schools	77	2.80	40.01	8.11
Rural living	0	0.00	0.00	0.00
Public open space	7	0.25	16.79	0.51
Road reserve	16	0.58	12.21	1.39

Total area (ha)	Total percent (%)
3.63	7

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
88.01	10.55

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
5.53	0.49

Commercial, Industry and Schools

Landuse	Percent (%)	Area (ha)
Light industrial	100	2.80
Heavy industrial	0	0.00
Commercial / Offices	0	0.00
Schools	0	0.00
Public buildings	0	0.00

Total area (ha)	Total percent (%)
2.80	77

Nitrogen input (kg)	Phosphorus input (kg)
40.01	8.11

Public Open Space (POS)

Landuse	Percent (%)	Area (ha)
Native gardens	0	0.00
Non-native gardens	0	0.00
Not fertilised	0	0.00
Nature	0	0.00
Sport	0	0.00
Recreation	100	0.25
Golf course	0	0.00
Bowling green	0	0.00
Impervious	0	0.00
Water body	0	0.00

Total area (ha)	Total percent (%)
0.25	7

Nitrogen input (kg)	Phosphorus input (kg)
16.79	0.51

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	0.41
Road reserve - impervious	0	0.00
Road reserve - native garden	15	0.09
Road reserve - non-native garden	0	0.00
Road reserve - turf	15	0.09
Road reserve - not fertilised	0	0.00

Total area (ha)	Total percent (%)
0.581392	16

Nitrogen input (kg)	Phosphorus input (kg)
12.21	12.21

Soil and drainage information

Type of drainage	**Piped drainage**	**Does it contain imported fill?**	**No**
Soil type	**Pinjarra**	**Does subregion contain onsite sewage diposal system?**	**No**
Depth to groundwater (m)	**3**		
Groundwater slope (%)	**0.5**		
Soil PRI	**19.3**		

Note: Please attach the results of soil tests to this report when submitting.

Subregion name: Catchment 5

Landuse	Percent (%)	Area (ha)	Input load Nitrogen (kg)	Input load Phosphorus (kg)
Residential	0	0.00	0.00	0.00
Industrial, commercial & schools	74	26.89	384.52	77.98
Rural living	0	0.00	0.00	0.00
Public open space	5	1.82	119.91	3.63
Road reserve	21	7.63	160.25	18.20

Total area (ha)	Total percent (%)
36.34	70

Nitrogen input (kg/yr)	Phosphorus input (kg/yr)
854.72	105.26

Nitrogen export (kg/yr)	Phosphorus (kg/yr)
47.26	1.43

Commercial, Industry and Schools

Landuse	Percent (%)	Area (ha)
Light industrial	100	26.89
Heavy industrial	0	0.00
Commercial / Offices	0	0.00
Schools	0	0.00
Public buildings	0	0.00

Total area (ha)	Total percent (%)
26.89	74

Nitrogen input (kg)	Phosphorus input (kg)
384.52	77.98

Public Open Space (POS)

Landuse	Percent (%)	Area (ha)
Native gardens	0	0.00
Non-native gardens	0	0.00
Not fertilised	0	0.00
Nature	0	0.00
Sport	0	0.00
Recreation	100	1.82
Golf course	0	0.00
Bowling green	0	0.00
Impervious	0	0.00
Water body	0	0.00

Total area (ha)	Total percent (%)
1.82	5

Nitrogen input (kg)	Phosphorus input (kg)
119.91	3.63

Road reserve

Landuse	Percent (%)	Area (ha)
Roads	70	5.34
Road reserve - impervious	0	0.00
Road reserve - native garden	15	1.14
Road reserve - non-native garden	0	0.00
Road reserve - turf	15	1.14
Road reserve - not fertilised	0	0.00

Total area (ha)	Total percent (%)
7.63077	21

Nitrogen input (kg)	Phosphorus input (kg)
160.25	160.25

Soil and drainage information

Type of drainage	**Open channel drains**	**Does it contain imported fill?**	**No**
Soil type	**Pinjarra**	**Does subregion contain onsite sewage diposal system?**	**N**
Depth to groundwater (m)	**1.5**		
Groundwater slope (%)	**1**		
Soil PRI	**19.3**		

Note: Please attach the results of soil tests to this report when submitting.

Summary: Nutrient stripping devices

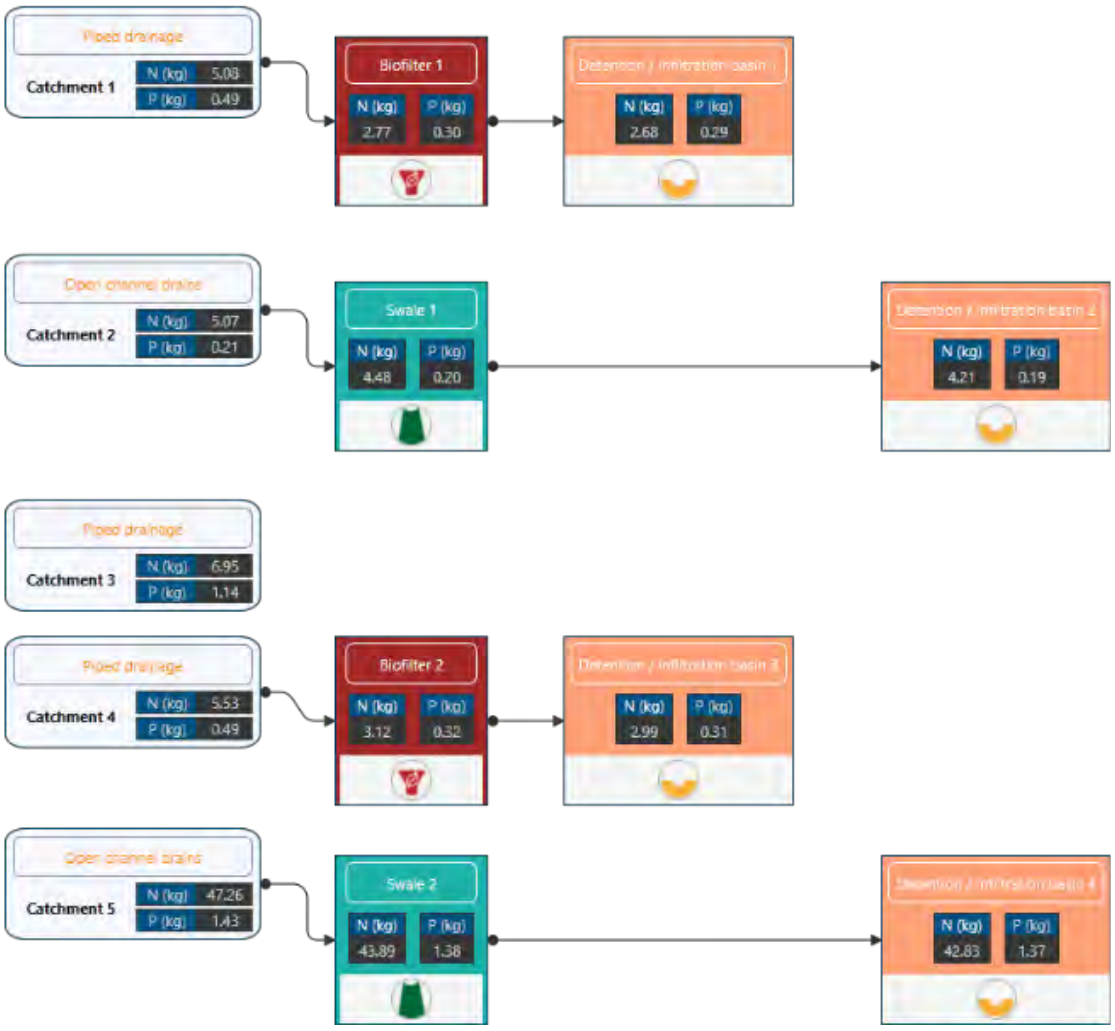
Treatment	Name	Size (m^2)	Treated area (ha)	Treating	N removed (kg/yr)	P removed (kg/yr)
Detention / infiltration basin	Detention / infiltration basin 1	1260.00	3.63	Sandy soils – Runoff only (infiltration on lots)	0.09	0.01
Biofilter	Biofilter 1	325.00	3.63	Sandy soils – Runoff and subsoil drains	2.31	0.19
Swale	Swale 1	2750.00	4.15	Sandy soils – Runoff only (infiltration on lots)	0.59	0.01
Detention / infiltration basin	Detention / infiltration basin 2	1200.00	4.15	Sandy soils – Runoff and subsoil drains	0.27	0.01
Biofilter	Biofilter 2	152.00	3.63	Sandy soils – Runoff and subsoil drains	2.40	0.18
Detention / infiltration basin	Detention / infiltration basin 3	2000.00	3.63	Sandy soils – Runoff only (infiltration on lots)	0.13	0.01
Detention / infiltration basin	Detention / infiltration basin 4	4500.00	36.34	Sandy soils – Runoff only (infiltration on lots)	1.06	0.02
Swale	Swale 2	2750.00	36.34	Sandy soils – Runoff only (infiltration on lots)	3.37	0.05
Load removed					**10.24**	**0.47**
Net export					**59.66**	**3.30**

Summary: Nutrient load exports

Region	Area (ha)	P export (kg/yr)	N export (kg/yr)
Catchment 1	3.63	0.49	5.08
Catchment 2	4.15	0.21	5.07
Catchment 3	4.15	1.14	6.95
Catchment 4	3.63	0.49	5.53
Catchment 5	36.34	1.43	47.26

PRE-TREATMENT LOAD (kg/yr)		LOAD REMOVED (kg/yr)		NET LOAD EXPORT (kg/yr)	
NITROGEN	PHOSPHORUS	NITROGEN	PHOSPHORUS	NITROGEN	PHOSPHORUS
69.90	3.76	10.24	0.47	59.66	3.30

Treatment diagram



Client: Development WA

Report	Version	Prepared by	Reviewed by	Submitted to Client	
				Copies	Date
Draft	V1	RP / YY	HB	Electronic	December 2020
Rev 1	V1	RMu / YY	REp	Electronic	March 2021
Rev 1	V2	YY	REp	Electronic	May 2021

Urbaqua
land & water solutions
Suite 4/226 Carr Place
p: 08 9328 4663 | f: 08 6316 1431
e: info@urbaqua.org.au
www.urbaqua.org.au

element.

Technical Appendix G – Urbis Population Forecast Summary


URBIS
HIGH WYCOMBE
SOUTH REVIEW
Population and
dwelling type review
Prepared for City of Kalamunda
5 September 2021

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PROJECT OVERVIEW

PROJECT PURPOSE

Urbis has been engaged by City of Kalamunda to provide a review of the High Wycombe South (formerly known as Forrestfield North) area and look at population projections, as well as forecasts of what type of development would be viable in this area.

High Wycombe South comprises two distinct precincts, the Residential Precinct Local Structure Plan area ('Residential Precinct') area as well as the Transit Orientated Development Precinct area ('TOD Precinct') which also encompasses the Activity Centre Precinct as identified under the Forrestfield North District Structure Plan. This report deals only with the yield analysis undertaken for the Residential Precinct.

This work is based on work undertaken for DevelopmentWA as part of a review and forecasting process for the TOD Precinct. As part of this work population projections and densities for the Residential Precinct were also considered and this report focuses primarily on the Residential Precinct, however, it is also important to consider how the TOD Precinct will develop as the two precincts need to compliment each other.

Residential Precinct Structure Plan



HIGH WYCOMBE SOUTH CONTEXT

Project Overview

The TOD Precinct is a 60 ha redevelopment area located directly east of the new High Wycombe Train Station, which is currently under construction and due to commence operations in late 2021 as part of the Forrestfield-Airport Link.

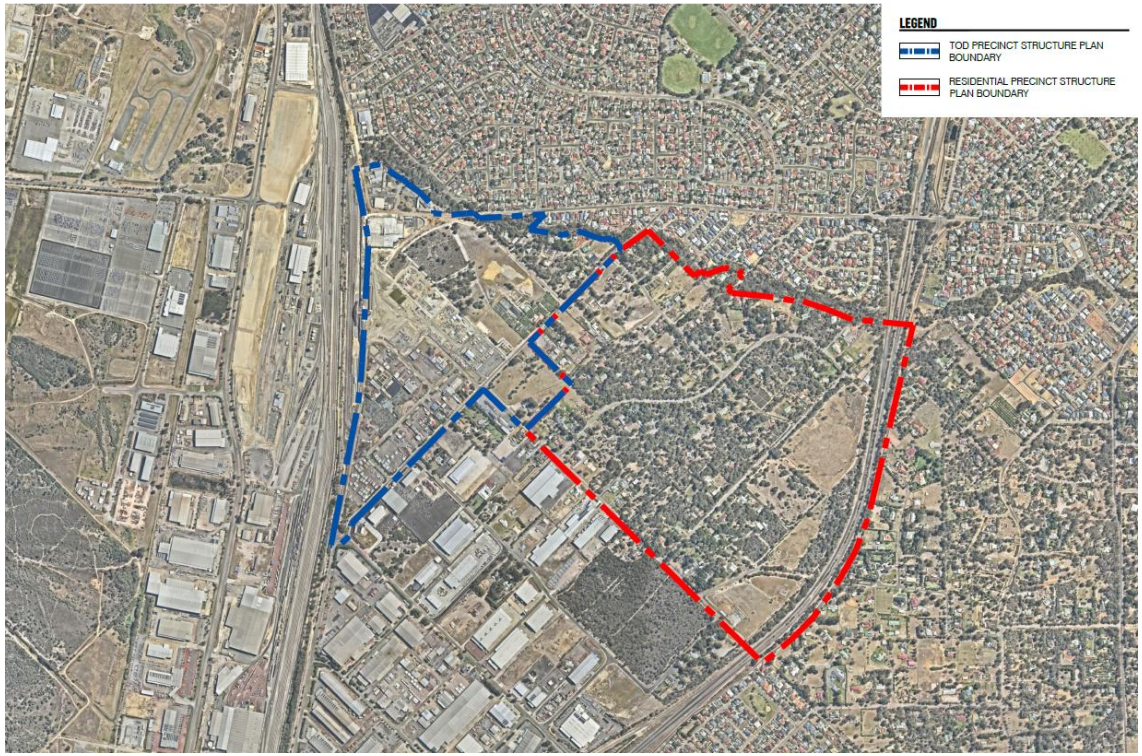
Current land uses in the TOD Precinct and Forrestfield North District DSP area are comprised of light industrial and rural residential land uses. The existing light industrial uses within the southern portion of the TOD Precinct are likely to be non-viable for redevelopment in the forecast timeframe (2050).

The Residential Precinct is to the east of the TOD precinct and comprises a total area of 123.0591 hectares (including the adjacent Roe Highway Reserve) with 47.5228 hectares of land designated for residential development.

Some key challenges identified which may impact development timing of North Forrestfield include:

- **Fragmented Nature of Land Ownership:** sites within High Wycombe South are in fragmented private ownership and this has the potential to slow development
- **Surrounding Property Values –** the median house price in the High Wycombe area ($440,750 as at June 2020) is well below the median price for Perth ($510,000). Whilst this will rise due to the infrastructure development in the area, low density development is likely to be the most viable form of development in the short to medium term.

Precincts



Source: Nearmap, Urbis, Element

PRECINCT OVERVIEW

Key Insights

Based on the work that Urbis prepared, key expected development scale and timeframe outcomes identified are:

- Due to land tenure, planning and environmental constraints, the TOD Precinct is likely to see an extended development timeframe, **estimated to commence as of 2026, and continue through to around 2064 based on modelling carried out.**
- The Residential Precinct has a significant yield and also allows for a significant proportion of medium density product that will be challenging to make commercially viable in the short to medium term and will be more financially viable in the medium to longer term. This means that the development period is **expected to be 44 years.**
- Based on the scenario analysed there is potential for around **2,417 new dwellings** within the residential precinct. The will be dependent on the density that the development takes place at and ultimate yield plan.
- The TOD Precinct is expected to have a greater focus on medium density product and in the longer term some higher density. This precinct has the potential for around **743 new dwellings and 1,770 new residents**.
- The residents in the TOD and Residential precincts and surrounding areas will support a total of **11,310 sq.m of retail and commercial floorspace,** with 5,160 sq.m of this being in the form of shop retail, comprised of a supermarket and associated specialties. Although, located in the TOD precinct, this will become important in providing amenity to the Residential Precinct and provide local jobs.

RESIDENTIAL PRECINCT

+2,417 DWELLINGS
Potential for over 2,000 new dwellings to be developed with a focus on medium density product.

+6,195 NEW RESIDENTS
To live in the Residential Precinct at final build out.

TOD PRECINCT

+5,160 SQ.M OF SHOP RETAIL FLOORSPACE
Accommodated within a neighbourhood shopping centre within the TOD Precinct by 2050.

+6,150 SQ.M OF NON-RETAIL FLOORSPACE
Accommodated within the TOD Precinct by 2050.

+743 DWELLINGS
To be Built in High Wycombe TOD Precinct, with densities ranging from R60 to R80 to be delivered by 2064

+1,770 NEW RESIDENTS
To live in the TOD Precinct over the development timeframe from 2026 through to 2064.

COMPETING SUPPLY AND POPULATION FORECASTS

SURROUNDING DEVELOPMENT AREA ANALYSIS

Review of Surrounding Development Area Assessment

The City of Kalamunda has made assessments for the area surrounding the Forrestfield North DSP area, to assist with the planning process. These assessments are largely based on the assumption that these areas will develop in line with comparable areas.

We have commented on the overall yields and uptake rates provided by the City in their report in the table below.

City of Kalamunda - Surrounding Area Analysis – New Dwellings

Area	2031	2041	2050
Maida Vale South	817	1,905	2,722
Cell-9 Wattle Grove	374	374	374
Dual Density R20/40	554	1,058	1,511
The Hales	250	250	250

Source: Urbis, City of Kalamunda Yields

Kalamunda Yield Analysis – Surrounding Development Area Yields

Area	Urbis Comments on Yield	Urbis Comments on Timing
Maida Vale South	The medium yield scenario which excludes 30% of land for roads and POS and selects an average lot size of 450sq.m per lot, yielding a total of 2,722 lots at build-out is a reasonable scenario.	The City comments that there is a planning process ahead for the development area, which is an urban investigation area, that will need a planning scheme amendment. However, things are well progressed with land amalgamation and the planning process.
Cell-9 Wattle Grove	374 lots are estimated to be remaining, based on the guided development scheme. This may be above the final realised number of lots, with the 2020/21 Urban Land Development Outlook (ULDO) for the region stating 150 lots remain in active and planned developments within the region. However, given the longer timeframe of this analysis, the guided development scheme yield assumption is a reasonable one.	The estimation that the area will be built out by 2031 is a reasonable assumption, given the number of lots and historical selling rates of estates in the region.
Dual Density R20/40	The 2% annual take-up of subdivisions, based on historical take-up rates is reasonable.	As the yields are estimated based on historical take-up rates, the timeframes applied to yields are reasonable.
The Hales	Yields are broadly in line with the ULDO assumptions and are reasonable.	The expectation that the estate will be sold out in 3-4 years is a reasonable one, given historical sales rates of 60-100 p.a for the estate

Source: Urbis

PUBLISHED FORECASTS

Comments on Forecasts

Forecast.id has recently completed a set of forecasts for the City of Kalamunda. As part of this process, Forecast.id has estimated dwelling construction activity in the suburbs of Kalamunda.

Broadly, these forecasts appear to be conservative, with the suburb of High Wycombe, which contains the development area of High Wycombe only expected to see 62 dwellings constructed p.a, during the development phase for the TOD and Residential Precincts. This uptake rate also includes infill development for established areas of High Wycombe. Indicatively, Rosehill Waters, the Hales, and Bushmead all commonly see sales rates in excess of this, and are smaller developments compared to Forrestfield North.

While .id states the active and planned developments in the area, it does not separate the sales acquired in development areas, and how their market shares adjust over time.

Forecast.id, Dwellings Constructed p.a

Suburb	2017-2021	2022-2026	2027-2031	2032-2036	2037-2041
High Wycombe	111	127	270	270	260
Forrestfield	264	276	236	115	105
Maida Vale	36	194	303	230	210
Wattle Grove	100	215	287	231	210

Source: Forecast.id

DWELLING YIELD AND POPULATION PROJECTIONS

YIELD ASSUMPTIONS

Yield Assumptions

Three dwelling yield scenarios were tested for the TOD Precinct (Low Yield, Medium Yield and High Yield).

The 'Medium' yield scenario was adopted and formed the basis for population projections.

Some of the key assumptions for the medium assumption is that the current commitments of car park, station and other community infrastructure are built, and lots are serviced with sewer.

The low and high assumptions reflected lower or higher levels of government intervention in the TOD Precinct. It was considered that a higher level of government intervention could potentially assist in achieving a faster take-up rate. They also considered allowing for lower levels of density that would reduce the TOD Precinct to a yield of 406 dwellings and the Residential Precinct to 1,973 dwellings. The high scenario allowed for similar yields but did allow for apartments in the Residential Precinct which was considered to be likely to be difficult to make viable financially even in the longer term.

Analysis of market viability carried out by Urbis indicated that in the short and medium term low and medium density development was most likely to be financially viable in the High Wycombe South area and the yields adopted reflected this.

The selection of the medium scenario was also informed by a review of dwelling preferences in the area (see Appendix 1) that indicated the future demand was likely to be focused on low density (68%) and a low preference for apartments.

Yields for the Residential Precinct have been based on work carried out by Element for the City of Kalamunda.

The yield assumptions tested for the TOD Precinct do not reflect the yields adopted in subsequent plans prepared for DevelopmentWA. However, they are still broadly in line with these subsequent plans and yields and relevant in terms of take up rates and population growth for the TOD Precinct.

TOD Precinct Yield Assumptions

Density	R30	R60	R80	R160	Total
Medium yield scenario	0	625	118	0	743

Source: Urbis, DevelopmentWA, '2020-11-10 High Wycombe Traffic Study Yields' Rev2'

Residential Precinct Yield Assumptions

Density	R30	R40	R60	R80	R100	Total
	360	578	1,056	423	0	2,417

Source: Urbis, City of Kalamunda (Element)

Combined Adopted Yield Assumptions, Dwelling Typologies

Lot Types	Residential Precinct	Station Precinct
Single Lot	938	0
Medium Density	1,329	625
Apartments	150	118
Total Dwellings	**2,417**	**743**

Source: Urbis, DevelopmentWA, City of Kalamunda (Element)

INDICATIVE DEVELOPMENT TIMEFRAMES

Staging Considerations

Staging considerations have been guided by the following:

- Observed staging in similar other developed TOD Precincts in Perth, such as Clarkson and Atwell, and;
- Fundamental market factors, such as the median house price, that will influence the viability of different types of dwelling typologies.

The potential development stage timing has been staggered to provide a guide as to potential ultimate build-out timing. However, ultimately build-out will be determined by market forces and there is potential that a shorter ultimate build-out timeframe in advance of this scenario could take place.

The terrace/townhouse product would likely begin in the Residential Precinct, before High Wycombe TOD Precinct semi-detached development begins. However, with significant government intervention to make the area more liveable and appealing, there is potential that medium density development could occur in the TOD Precinct simultaneously with medium density development in the Residential Precinct.

For the purposes of the medium scenario tested it was assumed that the R60 and R80 zoned land in the Residential Precinct were developed as small lot medium density product rather than apartments.

Development Timeframe, TOD Precinct Scenarios

STAGES	TOD PRECINCT SCENARIO	COMMENCES	2021-2026	2026-2031	2031-2036	2036-2041	2041-2046	2046+
Single Lot Detached Homes	Low	2028						
Terrace/ Townhouses	Low	2031						
	Medium	2031						
	High	2027						
Apartments	Medium	2043						
	High	2041						

Source: Urbis

Development Timeframe, Residential Precinct

STAGES	COMMENCES	2021-2026	2026-2031	2031-2036	2036-2041	2041-2046	2046+
Single Lot Detached Homes	2026						
Terrace/ Townhouse	2029						
Apartments (only in high scenario)	2041						

Source: Urbis

POPULATION FORECAST

Forecast Insights

The medium yield scenario for the Residential Precinct has been reviewed to look at expected sales rates for each product type. The review is based on looking at the market share that the broader area could achieve of all Perth sales, as well as of sales that the Residential Precinct will get compared to surrounding competition.

The Residential Precinct population is anticipated to increase from the existing rural residential base of 200 residents, to 6,195 at build out.

The modelling carried out is based on forecasting market shares. This modelling indicated that the residential precinct would be sold out in 2066 and TOD precinct in 2064. It is important to note that long term forecasting for when the project will sell out by is dependent on many factors including; overall economic conditions and Perth population growth, take up rates for single dwellings and medium density, the proportion of medium density vs other housing types sold in Perth in the future, the market share that this area gets of the overall Perth new dwelling sales and the developers involved and their marketing strategies. All of these factors are subject to change and can very greatly over a 30 year time frame and the selling period is an indication only based on the modelling carried out and assumptions made and could and probably will vary from this as the project develops over the next 30 years.

In the adopted scenario the dwelling types in the Residential Precinct will be focused on single residential lots and medium density terrace/townhouse product. Apartment development is expected to be difficult to make financially viable in the Residential Precinct even in the longer term and this type of dwelling is expected to be focused on the TOD Precinct that will offer greater amenity rather than the Residential Precinct. Even at the closest point the Residential Precinct is more than an 800 meter walk to the train station which is considered a walkable catchment.

Residential Precinct Dwelling Forecast Scenarios

Dwelling Type	2020	2025	2030	2035	2040	2045	2050	2055	2060	2065	2070
Total	**0**	**0**	**225**	**585**	**1,031**	**1,411**	**1,628**	**1,869**	**2,131**	**2,388**	**2,417**
Single Lot Dwellings	0	0	189	433	735	938	938	938	938	938	938
Terrace/Townhouses	0	0	36	152	296	473	690	931	1,194	1,450	1,479
Apartments	0	0	0	0	0	0	0	0	0	0	0

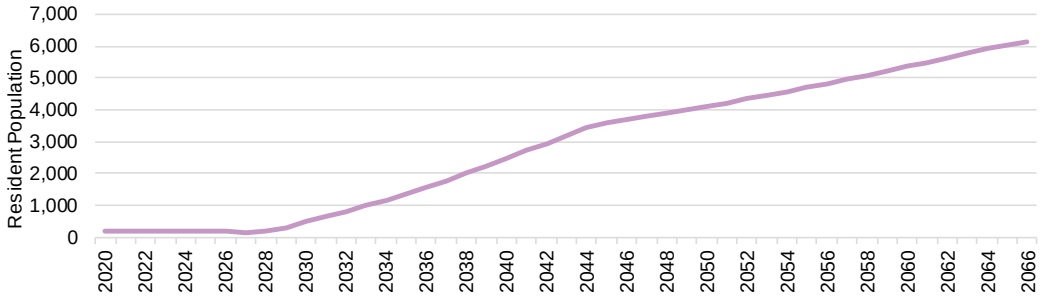
Source: Urbis

Residential Precinct Population Forecast Scenarios

	2020	2025	2030	2035	2040	2045	2050	2055	2060	2065	2070
Residential Precinct	200	200	497	1,374	2,479	3,579	4,109	4,458	5,346	6,051	6,195

Source: Urbis

Residential Precinct Population Forecast Chart



Source: Urbis

KEY ASSUMPTIONS FOR POPULATION CONVERSION

Assumption Discussion

Key assumptions that have been made to convert the dwelling forecast into population forecast are listed in the table on the right.

Key Assumptions for Population Forecasting

Assumption	Description
People Per Household	Based on the historical trends within the City of Kalamunda, we have assumed an initial persons per household ratio of 2.6, that trends downwards to 2.5 over the study period for single lot and medium density dwellings. This is in line with .id's assumptions. For apartments, we have assumed a static ratio of 2.2 persons per household, which is aligned with the rate for apartments across Perth, adjusted upwards for the demographics of the region (higher proportions of couples with children)
Lag Between Dwelling Approval/Sale and Population	We have assumed a 12 month gap between dwelling approval or sale, and the resident occupying the home to account for build time.
Existing Population within Development Area	Based on the Census ERP for the Forrestfield North DSP area (which corresponds with the 5113913 Statistical Area Level 1), we have assumed 113 residents currently reside in the TOD Precinct. This number will decline as lots are consolidated and these residents move elsewhere, and this will occur prior to residential development. This is expanded on in the appendix detailed forecasting components.
Infill Development in Established Suburb	We have assumed no subdivision/non-developer infill development will occur during the forecast period for the development area as it will be establishing. We have assumed a rate of subdivision/non-developer development within the suburb of High Wycombe of 27 dwellings p.a, in line with id's assumptions.

FORECAST COMPARISON – HIGH WYCOMBE SUBURB

Forecast Discussion

We have developed a projection of the High Wycombe Suburb (which correlates with the High Wycombe Statistical Area Level 2).

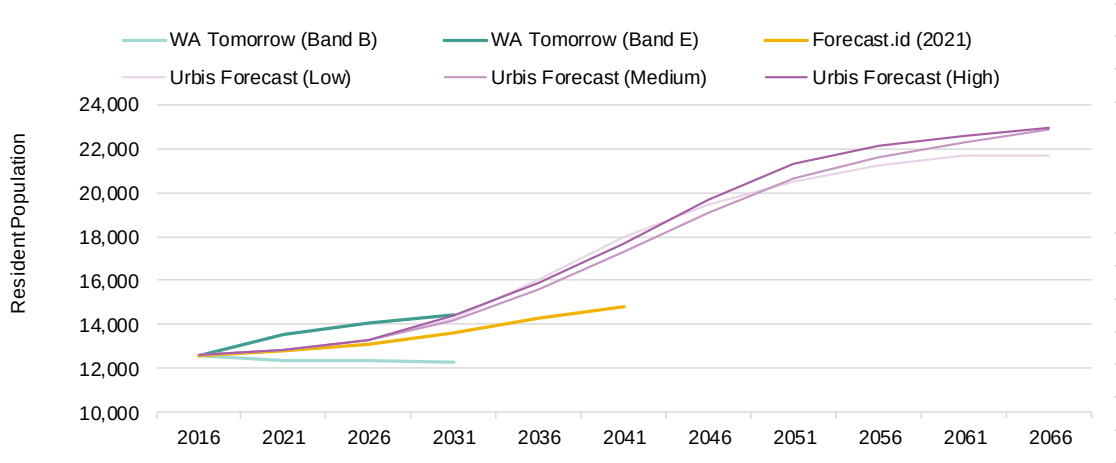
This forecast includes the Residential Precinct, as well as existing residents and infill development within the High Wycombe established area.

These projections allow for a significantly higher rate of development relative to the Forecast.id forecasts completed earlier this year. As discussed earlier in this report, we believe that the Forecast.id Forecasts do not allow for a high enough level of growth to cater to the High Wycombe South Precinct.

High Wycombe Suburb Population

Suburb Population Yield	2021	2026	2031	2036	2041	2046	2051	2056	2061	2066
Low	12,971	13,321	14,255	16,073	17,966	19,441	20,484	21,223	21,667	22,076
Medium	12,971	13,321	14,184	15,612	17,291	19,114	20,611	21,606	22,292	22,874
High	12,971	13,321	14,393	15,902	17,680	19,696	21,272	22,133	22,576	22,931

High Wycombe Suburb Population



APPENDIX 1

DWELLING PREFERENCE ANALYSIS

DWELLING PREFERENCES APPROACH

Dwelling Preferences Approach



1: Generate a study area to determine housing preference trends (Cannington, Swan, Kalamunda, Belmont) have been selected as they are appropriate aspirations for this area.



2: Analyse historical household trends and preferences from 2006, 2011 and 2016 Census.



3. Analyse population and household type projections using Forecast.id. and apply to dwelling preferences to determine forecast trend preference to 2041.



4. Apply dwelling preferences to household projections for the High Wycombe area to determine dwelling yields and implications for the development area.

DWELLING PREFERENCE TRENDS

Description of Study Area

We have chosen to use four adjacent local government areas to generate a dwelling preference trend for the Forrestfield North area. The Local Government areas selected are: Canning, Belmont, Kalamunda, and Swan. Canning & Belmont have been selected as they are established areas that are beginning to see denser development opportunities in certain areas.

Overall, we see a trend towards medium density units and townhouse dwellings since 2006, as well as higher density apartment product, though apartments are still a relatively low proportion of all dwellings.

The household types that see the highest level of growth towards apartments are couple families without children, lone persons, and group households.

There has been a shift towards medium to higher density living over time across Greater Perth. Residents living in units, townhouses and apartments increased from 20% in 2006 to 23% in 2016.

Housing Preference by Household Type, 2016, Study Area

2016	Couple family with no children	Couple family with children	One parent family	Other family	Lone person household	Group household	Total
Detached Houses	85%	92%	87%	88%	70%	72%	**87%**
Units & Townhouses	13%	8%	13%	11%	27%	26%	**12%**
Apartments (3+ storey)	1.6%	0.3%	0.5%	0.7%	3.2%	2.7%	**1%**
TOTAL	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**

Source: ABS 2016 Census Tablebuilder, Urbis
Note: Data Relates to Dwellings in the Study Area

Historical Housing Preference by Household Type, 2006 and 2011, Study Area

2011	Couple family with no children	Couple family with children	One parent family	Other family	Lone person household	Group household	Total
Detached Houses	88%	94%	87%	90%	72%	73%	**89%**
Units & Townhouses	11%	6%	12%	9%	26%	25%	**10%**
Apartments (3+ storey)	0.9%	0.2%	0.5%	0.4%	2.5%	1.9%	**1%**
TOTAL	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**

2006	Couple family with no children	Couple family with children	One parent family	Other family	Lone person household	Group household	Total
Detached Houses	90%	96%	90%	78%	72%	72%	**90%**
Units & Townhouses	10%	4%	10%	22%	26%	27%	**9%**
Apartments (3+ storey)	0.4%	0.2%	0.3%	0.4%	2.2%	1.5%	**0%**
TOTAL	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**

Source: ABS 2016 Census Tablebuilder, Urbis
Note: Data Relates to Dwellings in the Study Area
NB: 'Non-classifiable' and 'visitor households' have been excluded.

DWELLING PREFERENCE PROJECTIONS

Dwelling Preferences

There has been a favourable shift in couple families with no children living in apartments in the 5 years to 2016. This has increased by 12% p.a followed by other families living in apartments increasing by 10.3% p.a.

There has also been a favourable shift in couple families with children living in units or townhouses, seeing an increase by 6% between 2011 and 2016.

Overall there has been a 7% increase per annum in residents living in apartments and 3% increase in residents living in units and townhouses.

Generally there has been a decline across all households living in detached housing by 0.4%.

Projections past 2041 are likely to be unreliable due to the compounding of the trend over an extended time period. Overall approximately 67% of households will prefer to live in detached housing, 26% in units and townhouses and 7% in apartments by 2041.

The introduction of a train station in the local area would be likely to reduce the historic preference towards a detached house, as it would reduce the necessity for a two car garage. However, the volume of retail amenity, presence of entertainment options within walking distance, and the quality of public open space that can be used as a replacement for back yards, will have a more substantial effect on increasing the preference for semi-detached houses and apartments.

Annual % Change from 2011-2016, Study Area

Compound Annual Growth Change (2011-2016)	Couple family with no children	Couple family with children	One parent family	Other family	Lone person household	Group household	Total
Detached Houses	-0.6%	-0.4%	-0.1%	-0.4%	-0.5%	-0.4%	**-0.4%**
Units & Townhouses	3.3%	6.2%	1.0%	3.5%	0.8%	0.6%	**3%**
Apartments (3+ storey)	11.9%	2.2%	1.2%	10.3%	5.1%	7.3%	**7%**
TOTAL	**14.6%**	**8.0%**	**2.1%**	**13.3%**	**5.4%**	**7.4%**	**9%**

Source: 2016 Census ABS, Urbis
Note: Data Relates to Dwellings in the Study Area
NB: 'Non-classifiable' and 'visitor households' have been excluded.

Forecast Trend Preferences, 2041, Study Area

Forecast to 2041	Couple family with no children	Couple family with children	One parent family	Other family	Lone person household	Group household	Total
Detached Houses	55.6%	70.6%	83.0%	69.8%	58.5%	58.6%	**67.3%**
Units & Townhouses	23.3%	29.0%	16.3%	23.1%	31.3%	27.0%	**26.2%**
Apartments (3+ storey)	21.1%	0.4%	0.7%	7.0%	10.3%	14.4%	**6.5%**
TOTAL	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**

Source: 2016 Census ABS, Urbis
Note: Data Relates to Dwellings in the Study Area
NB: 'Non-classifiable' and 'visitor households' have been excluded.

IMPLICATIONS FOR DEVELOPMENT AREA

Key Findings

The suburb of High Wycombe consists largely of couple families with dependants (32%), followed by couples without dependants (27%) and lone person households (22%). By 2051 this is expected to change with a higher proportion of couples without dependants (30%) predicted to live in the suburb, followed by couple families with dependants (28%) and lone person households (25%).

Overall, it is likely that the dwelling types that would be most desirable to residents in the longer term would be lower density single lot dwellings, though there is likely to be an increasing demand for both medium density semi-detached dwellings, and a smaller proportion of apartments (6% of yield). It should be noted that these yields would be for an entire development area, so a smaller precinct could have a higher density focus, if the broader precinct it is a part of could be weighted towards the more desirable lower density product.

Forecasted household population by type, High Wycombe Suburb

Year	2016	2031	2041	2051
Couple families with dependants	31.8%	29.8%	29.2%	28.0%
Couples without dependants	26.9%	28.6%	28.7%	29.7%
Group households	3.3%	3.2%	3.1%	3.1%
Lone person households	22.2%	23.7%	24.2%	25.2%
One parent family	12.8%	12.0%	11.9%	11.4%
Other families	3.0%	2.8%	2.8%	2.7%

Source: Forecast.id, Urbis Analysis of Forecast.id to extend trend to 2051

Dwelling Yields, High Wycombe Suburb

Scenario	Low Density (R30)	Medium Density (R60)	2-3 Storey Apartments (R80-100)
Forrestfield North Full Build-Out	68%	26%	6%

Source: 2016 Census ABS, Urbis

COVID-19 AND THE POTENTIAL IMPACT ON DATA INFORMATION

The data and information that informs and supports our opinions, estimates, surveys, forecasts, projections, conclusion, judgments, assumptions and recommendations contained in this report (Report Content) are predominantly generated over long periods, and is reflective of the circumstances applying in the past. Significant economic, health and other local and world events can, however, take a period of time for the market to absorb and to be reflected in such data and information. In many instances a change in market thinking and actual market conditions as at the date of this report may not be reflected in the data and information used to support the Report Content.

The recent international outbreak of the Novel Coronavirus (COVID-19), which the World Health Organisation declared a global health emergency in January 2020 and pandemic on 11 March 2020, has and continues to cause considerable business uncertainty which in turn materially impacts market conditions and the Australian and world economies more broadly.

The uncertainty has and is continuing to impact the Australian real estate market and business operations. The full extent of the impact on the real estate market and more broadly on the Australian economy and how long that impact will last is not known and it is not possible to accurately and definitively predict. Some business sectors, such as the retail, hotel and tourism sectors, have reported material impacts on trading performance. For example, Shopping Centre operators are reporting material reductions in foot traffic numbers, particularly in centres that ordinarily experience a high proportion of international visitors.

The data and information that informs and supports the Report Content is current as at the date of this report and (unless otherwise specifically stated in the Report) does not necessarily reflect the full impact of the COVID-19 Outbreak on the Australian economy, the asset(s) and any associated business operations to which the report relates. It is not possible to ascertain with certainty at this time how the market and the Australian economy more broadly will respond to this unprecedented event and the various programs and initiatives governments have adopted in attempting to address its impact. It is possible that the market conditions applying to the asset(s) and any associated business operations to which the report relates and the business sector to which they belong has been, and may be further, materially impacted by the COVID-19 Outbreak within a short space of time and that it will have a longer lasting impact than we have assumed. Clearly, the COVID-19 Outbreak is an important risk factor you must carefully consider when relying on the report and the Report Content.

Were we have sought to address the impact of the COVID-19 Outbreak in the Report, we have had to make estimates, assumptions, conclusions and judgements that (unless otherwise specifically stated in the Report) are not directly supported by available and reliable data and information. Any Report Content addressing the impact of the COVID-19 Outbreak on the asset(s) and any associated business operations to which the report relates or the Australian economy more broadly is (unless otherwise specifically stated in the Report) unsupported by specific and reliable data and information and must not be relied on.

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All surveys, forecasts, projections and recommendations contained in or made in relation to or associated with this report are made in good faith and on the basis of information supplied to Urbis at the date of this report. Achievement of the projections and budgets set out in this report will depend, among other things, on the actions of others over which Urbis has no control.

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Urbis staff responsible for this report were:

Director	David Cresp
Project code	P0036536
Report number	DRAFT

Urbis acknowledges the important contribution that Aboriginal and Torres Strait Islander people make in creating a strong and vibrant Australian society.

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the art and science of place

Level 18, 191 St Georges Tce, Perth WA 6000
T. (08) 9289 8300 – **E.** hello@elementwa.com.au
elementwa.com.au