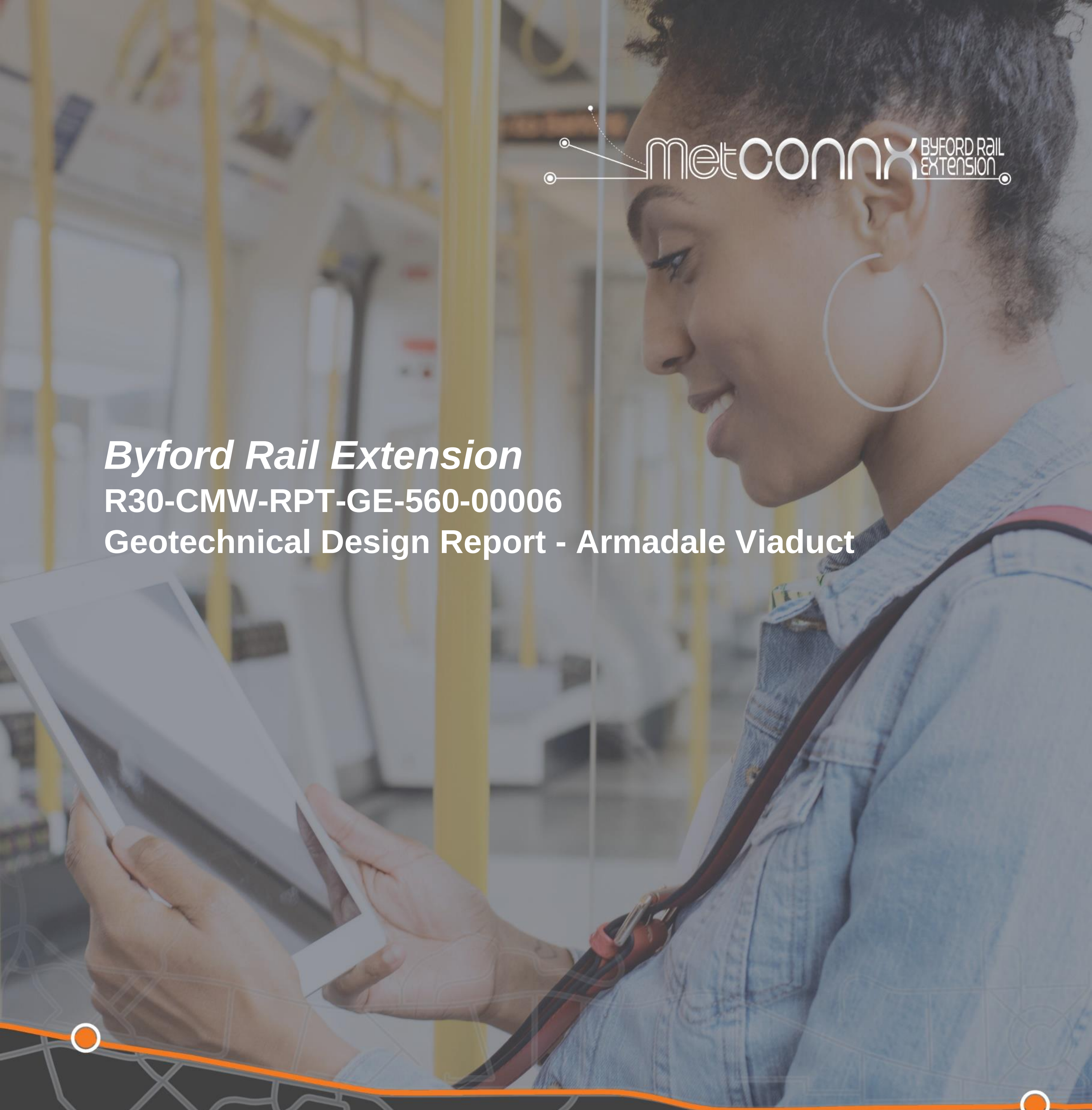
Appendix J: Geotechnical



metconnx BYFORD RAIL EXTENSION

Byford Rail Extension

R30-CMW-RPT-GE-560-00006

Geotechnical Design Report - Armadale Viaduct

Connecting communities.
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METRONET

Byford Rail Extension

Document details	
Title	Geotechnical Design Report - Armadale Viaduct
Project	Byford Rail Extension (BRE) Design and Construction Project
Laing O'Rourke Project No.	R30
Client	Public Transport Authority of Western Australia
Client contract No.	PTA200142
MetCONNX Document No.	R30-CMW-RPT-GE-560-00006

Rev	Date	Revision Description	Prepared by	Reviewed by	Approved by
A.	18-Oct-2022	Issued for Reference Design	Andrew Webspер	Craig Butterworth	Alyssa Edwards

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Figures

Tables

1. Executive Summary

An executive summary has not been provided as part of this Geotechnical Design Report but will be incorporated at IDD stage when additional Geotechnical information becomes available.

Design Package
GE-560– Geotechnical Report

Reference Design Submission 4	
AR-325	Byford Precinct - Architecture
CI-300	Byford Station – Civil
CI-420	SSJ Roads (Earthworks, Drainage, Roads, Pavements)
EG-355	Byford Precinct – Electrical Services
EG-500	Specialist Packages – Roads & ITS
FE-350	Byford Precinct – Fire Engineering
FS-351	Byford Precinct – Fire Protection Services
HY-345	Byford Precinct – Hydraulic Services
LA-330	Byford Precinct – Landscaping
ME-340	Byford Precinct – Mechanical Services
ST-335	Byford Precinct – Station Structures
TL-370	Byford Precinct – Comms & Security Services
GE-560	**Geotechnical Report (this package)**

Package Description

GE-560 Submission consists of the following reports:
R30-CMW-RPT-GE560-0002: GDR – Rail Alignment (At Grade)
R30-CMW-RPT-GE560-0003: GDR Wungong Rail and PSP Bridge
R30-CMW-RPT-GE560-0004: GDR Eleventh Road Bridge
R30-CMW-RPT-GE560-0005: GDR Larsen Road PSP Bridge
R30-CMW-RPT-GE560-0006: GDR Armadale Viaduct
R30-CMW-RPT-GE560-0007: GDR Armadale Precinct
R30-CMW-RPT-GE560-0008: GDR Byford Precinct
R30-CMW-RPT-GE560-0009: GDR – Flood and Hydrology

Assurance Claim

MetCONNX has undertaken sufficient design development to satisfy requirements of EM4P for PMF Gate 3 and is seeking approval from PTA to proceed to PMF Gate 4 – Detailed Design

Package Interfaces (Section 3.2)

- Numerous project Interfaces with GE-560 extending across whole site.
- Individual Interfaces completed in each report to reflect each structure /area of project

Risks & Uncertainties (Section 4.15)

- RD reports completed whilst additional Investigation is being completed. Designs to be updated to reflect information in IDD stage.
- Viaduct pile foundations to be updated upon completion of planned pile load testing
- Armadale PSP Bridge RD report on hold awaiting details of structural options
- Structural details to be finalised upon completion of additional Geotechnical Investigation. GDR will require updating if solutions/options change during IDD
- Design Groundwater Levels (GWL) to be finalised upon additional investigation and monitoring of shallow ground water levels. Prior investigations have focused upon deeper GWL. Investigations ongoing.

Figure 1 - Design Report Summary

Acknowledgment of Country

MetCONNX acknowledges the Whadjuk People and the Gnala Karla Booja People as the Traditional Custodians of the land and waters on which Byford Rail Extension Project is located.
We pay our respects to Elders, past, present and emerging, and thank them for their continuing connection to country, culture and community.

2. Project overview

2.1 METRONET Vision and Objectives

As one of the largest single investments in Perth's public transport, METRONET will transform the way the people of Perth commute and connect. It will create jobs and business opportunities and stimulate local communities and economic development to assist communities to thrive. The METRONET vision is for a well-connected Perth with more transport, housing and employment choices. In delivering METRONET, the WA Government has considered peoples' requirements for work, living and recreation within future urban centres with a train station at the heart.

The objectives are to:

- Support economic growth with better-connected businesses and greater access to jobs
- Deliver infrastructure that promotes easy and accessible travel and lifestyle options
- Create communities that have a sense of belonging and support Perth's growth and prosperity
- Plan for Perth's future growth by making the best use of our resources and funding
- Lead a cultural shift in the way government, private sector and industry work together to achieve integrated land use and transport solutions for the future of Perth.

2.2 Byford Rail Extension Overview

The Byford Rail Extension (BRE) Project has been identified as an essential component of the METRONET program. The Project will extend the electrified passenger rail service from Armadale to Byford, providing a strong transport connection between these two centres, supporting economic growth and providing greater access to jobs. The Project has been developed in line with policy objectives for highly integrated transport and land use planning.

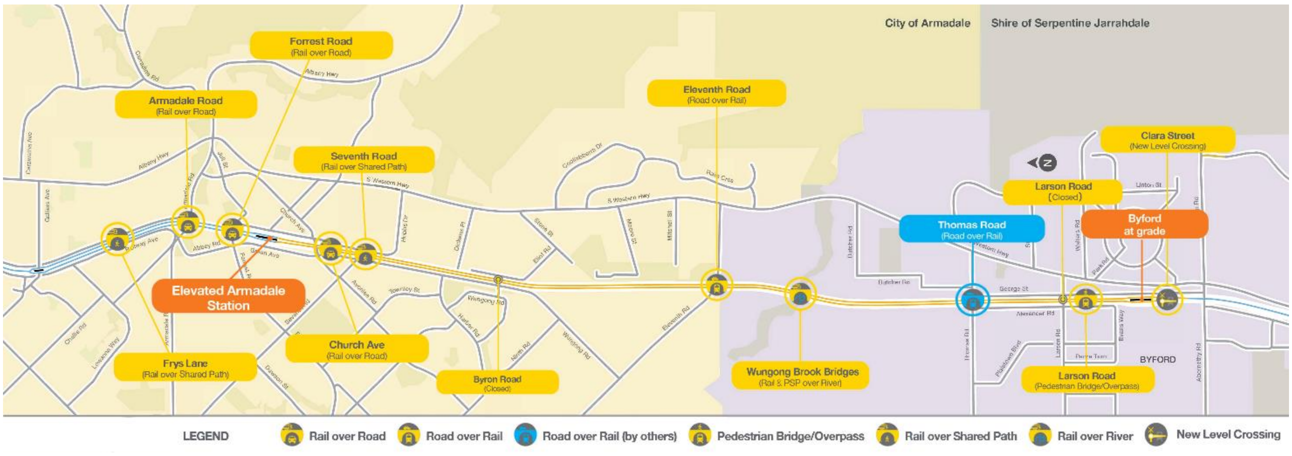


Figure 2: METRONET Byford Rail Extension Project

2.2.1 Project features

Transport infrastructure works for the BRE Project include:

- Demolition of existing station at Armadale and construction of a new elevated station
- Construction of a new Byford station at grade (Base Case)
- Construction of approximately 8km of dual track narrow gauge electrified passenger railway line extending from Armadale station to the newly created Byford station, with a dedicated platform for the Australind line
- Removal of level crossings between the Byford and Armadale stations
- Construction of PSPs and associated infrastructure (including 'rail over road' and 'road over rail' bridges and roads)
- Parking areas at Armadale and Byford stations
- Bus interchange at Armadale and Byford stations
- Upgrade of local roads surrounding both Armadale and Byford stations.

2.2.2 General scope of works

The Project's general scope of works includes designing, procuring, manufacturing, constructing, installing and commissioning all rail infrastructure and ancillary works to support an electrified operational passenger rail between Armadale and Byford Stations. Also, in the case of the Australind train service, tying into the non-electrified rail network south of Byford Station.

The Project activities include all site investigation, design, planning, scheduling, procurement, cost control, approvals, construction, OH&S management, environmental management, quality management, testing and commissioning, Entry Into Service (EIS), training and operational readiness required to tie the rail extension to Byford into the existing rail network including the associated road, utilities and other required works to interface with adjacent works and contracts. This will include bulk earthworks and retaining structures, grade separations, roads, and drainage, the demolition and removal and treatment of waste material and contaminated material resulting from construction of the Works, and temporary works constructed for the purpose of facilitating the Works.

The project scope also includes any new road works, modifications to existing roads and signalised intersections, utilities (diversion, protection, and new installation) and any other ancillary works to enable the BRE Project.

2.2.3 Future Proofing the works

As part of the Project, space must be allowed within the rail corridor for the option of a 4-track scenario for a potential high-speed regional service from Bunbury. The additional 2 tracks shall be constructed in the eastern half of the rail corridor, so that future infrastructure can be constructed without impacting on existing rail operations. The Project should also allow for the possibility of future extension of the electrified line south of Byford to Mundijong, and a future stabling yard south of Abernethy Road.

2.3 Alliance Vision and Delivery Approach

The BRE Project will be delivered under an alliance contract to support the management of project and stakeholder interfaces and to mitigate project risks. A collaborative alliance approach will see the Works carried out in a cooperative, coordinated and efficient manner, in compliance with the Alliance Principles.

MetCONNX understands that the successful delivery of the Project is critically linked to meeting the PTA's Key Project Objectives. These objectives have shaped our vision for the Project that is around delivering a high-quality product and creating exceptional value-for-money. We are committed to a no-blame culture and to the prompt and mutual resolution of any issues that may arise.

During the AD Stage, an interactive ALT Visioning Workshop was held with representatives from the PTA and MetCONNX to develop a suitable Alliance Vision for the Project, refer Figure 3.

> " **Collaborating** to **deliver excellence** in transport infrastructure with **certainty** which **connects and activates the community,** for current and future generations "

Figure 3: AD Stage Alliance Vision Development Outcomes (developed with the PTA)

To support the realisation of this vision, we will develop a robust and highly collaborative alliance culture in which everyone challenges 'business-as-usual' and pursues better outcomes in the design and construction of the Project. In line with this, during the AD Stage the MetCONNX team refined their priorities for the Project as being:

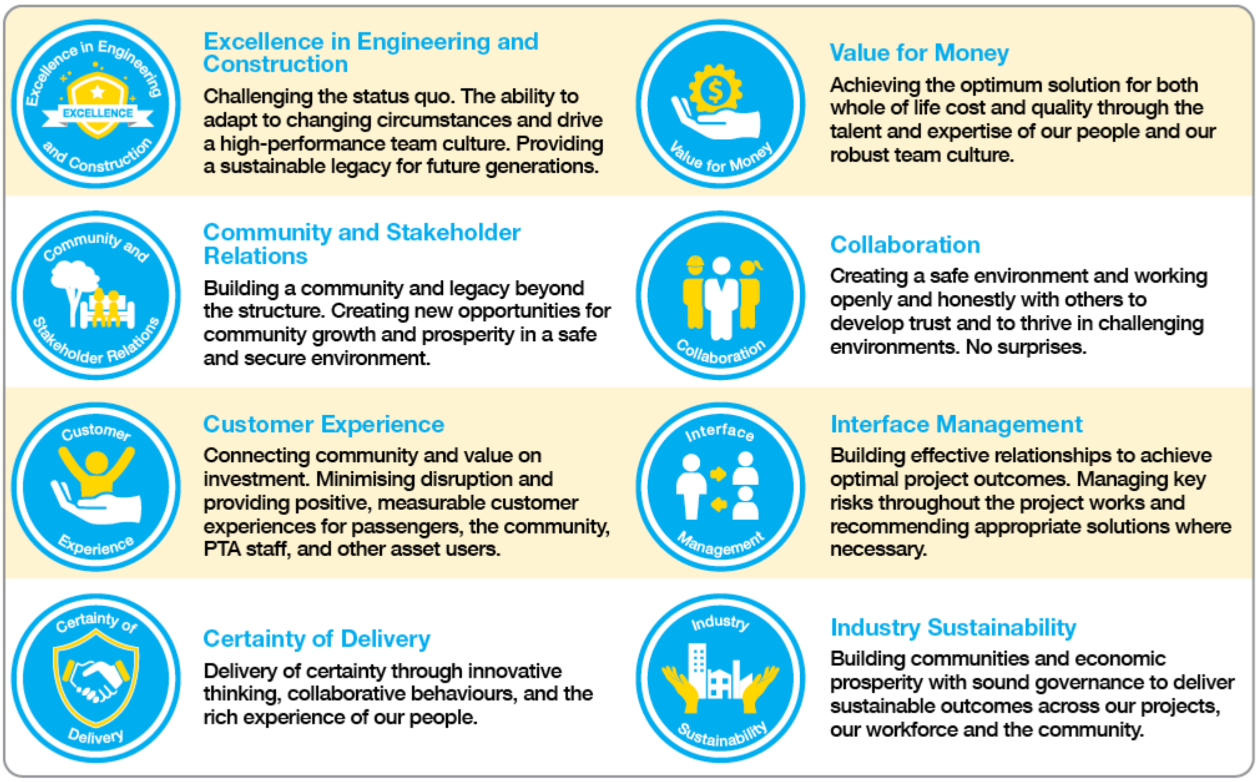


Figure 4: MetCONNX Priorities aligned with Key Project Objectives

2.4 Purpose of the Report

This Design Report presents the geotechnical design information for the Armadale Viaduct Design Package (Design Lot ST-170) for inclusion in Design Report R30-DEA-RPT-ST-170-00001. This report shall provide the geotechnical design's rationale and context of the foundation and retention design works for review by the PTA and stakeholders.

Table 1 - Project Interfaces

Design Package ID	Title	Description of Interface
CI-001	Temporary Facilities at Sherwood & Armadale including Temporary Carparks (Include lighting, security etc), bus infrastructure, staff facilities.	Provide geotechnical advice for temporary facilities
UT-040	Utilities (Optus, Telstra, NBN, Vocus, ATCO, WaterCorp, Western Power)	Earthworks and drainage/ culverts consider the location of utilities
CI-080	Temp MCR	No direct interface with this package
TR-100	Permanent Way - Alignment Design	Track alignment determines arrangement for formation, earthworks, and drainage.
SI-120	Signalling	The signalling equipment located in the corridor has been considered in terms of access provisions.
TL-130	Communications & Controls Sitewide	No direct interface with this package
OH-140	Overhead Wiring	Overhead Wiring structure locations are considered as part of the earthworks and formation

3. Design Description

3.1 Scope of this Design Package

This design report has been prepared to provide a documented record of the geotechnical design information for the design of the following referenced structures.

- Armadale Viaduct and associated structures (in particular, approach embankment retaining walls)
- Any other structures associated with the project are covered in separate submissions.

This design report provides the following information:

- Approach, methodology and assumptions made for the geotechnical design
- Geotechnical pile design information for the Armadale Viaduct
- Geotechnical design information for the proposed retaining walls for the approach embankments and other structures associated with the bridge such as shallow foundations, transitions slabs.

The structures covered in this report have been designed in accordance with the relevant sections of the SWTC, PTA Specifications and Australian Standards, except as noted through this report. The geotechnical design information has been developed in collaboration with the structural designers.

The design of the structures is contained in the main package design report.

3.2 Relationship with other Design Packages

This Design Report presents the geotechnical design information for the Armadale Viaduct Design Package (Design Lot ST-170) for inclusion in Design Report R30-DEA-RPT-ST-170-00001. See Table 1 and the main design package for project interfaces.

3.3 External Interfaces

N/A

3.4 Changes Since Previous Design Submission

3.4.1 Alliance Development (AD) Phase to Reference Design (RD) Phase

The overall Armadale Station precinct has had limited design changes between AD and RD phase with the Armadale Station viaduct/piles and new Armadale Station likely to be supported on shallow foundations still currently proposed.

3.5 Armadale Viaduct

The proposed Armadale viaduct is a multi-span railway bridge structure carrying the Armadale Up and Dn Main Lines from north of Armadale Road, through the proposed elevated Armadale station to south of Church Avenue. The span lengths vary between about 30 m and 40 m.

Each abutment and pier will be supported by large (1.8 m) diameter piles/columns extending up to the bridge beams with a transverse column spacing of about 10 m. The approximate extent of the viaduct is shown on the marked up geological section in Appendix C as the production of structural drawings was still in progress at the time of reporting.

3.6 Retaining Walls

Reinforced L-shaped concrete retaining walls up to about 6.5 m high are proposed to retain the approach embankments leading to the abutments. Design of other retaining walls along the alignment will be included in the retaining walls package.

4. Design Inputs

4.1 Project Design Requirements

Design and drawings for the viaduct structure and approach embankments were in progress and not available for this reference design stage Geotechnical Design submission. Indicative long sections and cross-sections through the viaduct structure were provided from the AD stage indicating approximate dimensions and these have been transferred onto the marked up section in Appendix C to indicate the extent of the proposed viaduct. Reference should be made to the main design package for the latest civil and structural drawings.

A full set of pile design actions will be developed by the structural engineer depending on the structural layout for the next design stage. In order to provide initial geotechnical design advice, the structural engineer has indicated the following:

- Piles to be reinforced concrete, 1.8 m diameter
- Approximate ULS Design Action, E_d = 15 MN
- Approximate SLS Design Action, E_{ds} = 12 MN

4.2 Design software used for this package

In-house design calculation spreadsheets have primarily been used for this RD stage package.

4.3 Applicable Codes and Standards

The applicable standards, codes and guidelines are in accordance with SWTC Appendix 3 and applicable codes and standards are summarised in Table 2.

Table 2 - Applicable Codes and Standards

Reference	Revision	Description/Title
AS5100	2017	Bridge Design Code
AS1170.0	2002	Structural design actions: General Principles
AS1170.4	2007	Structural design actions: Earthquake Actions in Australia
AS4678	2002	Earth retaining structures
AS2159	2009	Piling – Design and Installation
BRE-PTAWA-PM-RPT-00001	0	SWTC Book 1A: General Scope
BRE-PTAWA-PM-RPT-00002	0	SWTC Book 1B: Limit of Works
BRE-PTAWA-PM-RPT-00003	0	SWTC Book 2: Management Plan Requirements
BRE-PTAWA-PM-RPT-00004	0	SWTC Book 3A: Scope of Works
BRE-PTAWA-PM-RPT-00006	0	SWTC Book 3C: Elevated Option
BRE-PTAWA-PM-RPT-00007	0	SWTC Book 4 : Technical Criteria
BRE-PTAWA-PM-RPT-00007	0	SWTC Book 5: Appendices to the SWTC
8103-400-004	5	Working In and Around PTA Rail Reserve
8190-400-002	2.5	Narrow Gauge Main Line Track and Civil Infrastructure Code of Practice
8880-450-010	2	Specification Design Actions, Asset Design Life and Maintenance Free Period
8880-450-053	1	Specification Retaining Walls and Shallow Foundations
8880-450-059	1	Specification Buildings and Station Structures
8880-450-070	0	Specification Geotechnical Investigations
8880-450-074	1	Specification Earthworks Slope Stability Geotextiles and Erosion Protection
8880-450-077	1	Specification Deep Foundations

4.4 Reference Information

The project specific reference information and reports that have been used as inputs into the development of the design are included in Table 3.

Table 3 - Geotechnical and Hydrogeological Information

Document Reference	Description/Title	Revision
BRE-ADV-GE-RPT-00004	Geotechnical Investigation Factual Report , Advisian (18 Oct 2021)	-
BRE-ADV-GE-RPT-00005	Geotechnical Interpretative Report, Advisian (6 October 2021)	-
BRE-MNO-WSP-GE-RPT-0001	Geotechnical Factual and Interpretive Report, WSP	-
BRE-ADV-GE-RPT-00012	Monthly Groundwater Monitoring (February 2022), Advisian, 28 February 2022	-
311012-00745-GT-MEM-0011	Monthly Groundwater Monitoring (April 2022), Advisian, 10 May 2022	-

4.5 Design Criteria

The design criteria utilised in the development of this design package are outlined below. These design criteria include material properties, design loading and serviceability requirements.

In accordance with PTA Specification 8880-450-054-Rev1 (Specification Rail Bridges):

- Rail bridge foundations shall be designed in accordance with AS5100.3 and PTA Specification 8880-450-053 and 8880-450-077.
- Pile foundations shall be designed in accordance with AS 2159.

In accordance with PTA Specification 8880-450-053-Rev1 (Specification: Retaining Walls and Shallow Foundations):

- All retaining walls within the PTA rail reserve shall be Classification C in accordance with Table 1.1 of AS4678.
- The design groundwater levels shall not be lower than the 1% AEP groundwater levels.
- Maximum allowable settlement/heave and horizontal deflection of any type of foundation through the design life are summarised in Table 4 and Table 5.

Table 4 - Maximum Allowable Settlement/Heave

Foundation Type	Total Settlement/Heave		Differential Settlement/Heave	
	Short Term	Long Term	Short Term	Long Term
Shallow	20 mm	20 mm	1:1,000	1:1,000
Deep raft	20 mm	20 mm	1:1,000	1:1,000
Deep foundation element piles (DFEs)	15 mm	25 mm	1:1,000	1:1,000

Table 5 - Maximum Allowable Horizontal Deflection

Foundation Type	Horizontal Deflection		Horizontal Deflection	
	Short Term	Long Term	Short Term	Long Term
Laterally loaded DFEs	15 mm	25 mm	1:1000	1:1000
Gravity walls including cantilever reinforced concrete walls	15 mm	25 mm	1:1000	1:1000

Notes to Table 5:

- Settlement/heave/horizontal deflection are defined as the movement occurring from the time at which a foundation/retaining wall is cast and shall be measured at the structural surface of the foundation.
- The long term total allowable displacement magnitudes are inclusive of short-term displacement magnitudes.

4.6 Design Life

The design life requirements related to this design package are outlined in Table 6. These design life requirements are based on the minimum requirement specified in Clause 4.1 of the PTA Specification – Design Actions, Asset Design Life and Maintenance Free Period (8880-450-010). All works shall be designed and constructed to satisfy the required minimum design life.

Table 6 - Design Life

Item	Asset Element of the Works	Durability Design Life (Years)
1	Armadale Viaduct	100 years (1), 120 years (2)(3)

Notes to Table 6:

- (1) Design Life for the considerations of structural design actions on structures
- (2) Design Life for durability design and considerations on bridge structures
- (3) Secondary elements of bridges, such as bearings, deck joints, noise barriers etc., the service life shall satisfy the minimum design life requirements of AS5100 and not less than 50 years. Classification on bridge structures and secondary elements shall refer to AS5100.

4.7 Durability Requirements

Details of durability issues and risks, and measures to comply with the durability requirements will be outlined in the Durability package produced under separate cover.

4.8 Access and Maintenance – Structural Input

N/A

4.9 Constructability Requirements

See construction methodology section.

4.10 Environmental & Sustainability Design Criteria

Details of environmental & sustainability issues and risks, and measures to comply with the design criteria will be outlined in the Environmental & Sustainability package produced under separate cover.

4.11 Future Proofing

No input provided at Reference Design stage.

4.12 Value Engineering

No input provided at Reference Design stage.

4.13 Third Party Operational Stakeholders

N/A

4.14 Design Input from Stakeholders and Community Involvement Process

N/A

4.15 Design Assumptions, Dependencies, and Constraints (ADC's)

See Project Design Requirements section.

4.16 Requests for Information (RFI)

No Requests for Information have been submitted at Reference Design stage.

5. Design Outputs

5.1 Design Reviews and Ce Deliverables List

N/A

5.2 Specifications

See Geotechnical Design Advice and Calculations section and Table 2.

5.3 Standard Reference Drawings

No geotechnical standard reference drawings provided at Reference Design stage.

5.4 System Coordination Drawings and Models

N/A

5.5 Type Approvals

N/A

5.6 Summary of Subsurface Conditions

5.6.1 Available Geotechnical Investigation

The available geotechnical information is contained in the reports listed in Table 3. This information has been reviewed to develop geotechnical design profiles and parameters for the structures covered in this report.

The existing geotechnical investigation locations are shown in Appendix C and have been marked up to indicate the investigation locations considered and approximate areas classified in Section 5.6.4.

5.6.2 Supplementary Geotechnical Investigation

Additional geotechnical investigations are required to confirm the ground conditions, in particular the level and properties of the deeper geology, into which the piles may need to extend.

The proposed supplementary geotechnical investigations are presented in the Geotechnical Investigation and Testing Plan which is part of the Geotechnical Plan Ref. R30-MET-PLN-GE-000-

00001.The scope of the investigation has been designed in accordance with AS5100.3 and PTA Specifications 8880-450-070 and 8880-450-077.

A summary of the basis on which the investigation has been scoped as it relates to the structures in this package is included in Table 7. The approximate proposed geotechnical investigation locations for the proposed viaduct are shown on Figure 1A to 1D (from the Geotechnical Investigation and Testing Plan) in Appendix C.

Table 7 – Additional site investigation Requirements

Structure	PTA Requirements	Proposed Geotechnical Investigations
In-situ retaining walls	1 test location every 25 m along the alignment – locations to be confirmed once wall geometry is better understood.	CPTs/Boreholes to confirm relative densities, stiffness and strength of the underlying soils. Investigations approx. every 25 m to supplement nearby existing investigations and depending on consistency.
Bridge/Viaduct Piled Foundation	In accordance with AS5100.3:2017, 1 test location at each support where width <10 m, additional test location for each additional 10 m width. Greater of 5m into competent rock (moderately weathered or better), or 35 m below existing ground level.	Boreholes extended to about 10 m below the anticipated pile toe levels (based on current loading information) at pier locations (or as close as possible depending on access) where boreholes/CPTs are not present or to supplement CPTs that have terminated at depth above the anticipated pile toe levels.

Samples from boreholes will be selected for laboratory testing. The following laboratory tests are expected to be carried out:

- Classification tests (particle size distribution, Atterberg limit, moisture content)
- Unconfined compression test (UCS) and Point load Tests (PLT) on rock strength materials, where encountered

The investigation works are likely to be phased in order to provide timely information to the design and to align with access constraints of the project.

5.6.3 Geological Model Appreciation

The geological model has previously been presented by Advisian for PTA in the factual report (ref. BRE-ADV-GE-RPT-00004). A further discussion of the regional geology is included in the rail alignment package under separate cover, however a summary as it relates to this package is presented below.

Superficial Geology

The Armadale Sheet of the 1:50,000 Environment Series of maps shows the published surficial geology. It depicts most of the project area as being underlain by a unit denoted as Csg. This unit is described on the map as Gravelly Sandy Clay - variable with lenses of silt and gravel, quartz sand, subangular with aeolian rounded component; heavy mineral common; gravel rounded. This material is the result of colluvial/alluvial deposition. The colluvial materials were likely derived material from the erosion of the granite, gneiss and dolerite rocks and any surficial duricrust and soil development present above these rocks at and beyond the nearby Darling Scarp which is present about 1.2 km to the west of the site.

Duricrust development as ferricrete (laterite) is likely and has been noted in boreholes within the Colluvium. These duricrust layers are likely laterally discontinuous and of variable thickness and strength.

The Neerigen Brook is shown to cut east to west through the alignment at approximate Chainage 28630 and as such alluvial deposits are expected to be present at this location.

Yogunup Formation is anticipated and was encountered in geotechnical boreholes across most of the alignment beneath the Colluvium. Based on the available literature, the Yogunup Formation consists of up to 25 m of unconsolidated poorly sorted sand, gravel, pebbles, with minor clay in a belt up to 5 km from the Darling Scarp.

Deeper Solid Geology

GSWA Bulletin 41 published by the Geological Survey of Western Australia and titled Hydrogeology and groundwater resources of the Perth region, Western Australia, provides an insight into the deeper geology underlying the shallow superficial formations. This information on the deeper geology cannot be found on the published geological map. This bulletin has largely been compiled through interpretation of water borehole records and is a regional scale publication which approximates geological boundaries at depth.

To the south of Armadale Station, Bulletin 41 shows the sub-superficial geology along the project alignment comprises either the Pinjar Member of the Leederville Formation (Cretaceous Period) or the Cattamarra Coal Measures (Jurassic Period). Both units have similar lithologies. The Pinjar Member comprising sandstone, siltstone and shale and the Cattamarra also principally comprising sandstone, siltstone and shale. The Cattamarra Coal Measures also contains minor coal seams.

Advisian has interpreted the Cattamarra Coal Measures to be present below the whole alignment however at Armadale Station the deeper geology may comprise the Wanneroo Member of Leederville Formation described as interbedded sandstone and shale. The supplementary ground investigation will provide further information to be considered in future design stages.

In addition, there is some further uncertainty regarding the deeper geology, as the project alignment runs in north to south direction approximately parallel to the Darling Fault. Armadale Station appears to be located directly on the Fault based on the available literature and maps, although Advisian has interpreted the fault to be about 100 m to the west. Palynology assessment for age determination was carried out by Advisian on samples from BRE-BH03 at about CH 32800 (about 3 km south of the viaduct) which inferred that the deeper geology (below about 20 m depth) was of the Cattamarra Coal Measures (implying that the fault is to the east of the alignment). However, as the alignment is further east at the viaduct site, this may not be the case for this package.

The presence of the Darling Fault means that the underlying rock formations at depth could vary along the project alignment, and even along the viaduct. There is a risk that the faulted surfaces, if encountered, may comprise some significantly less competent materials and rock fragments in clay matrix. It is not known exactly at what depth the Darling Fault is encountered below the ground surface.

It is expected that the supplementary ground investigation will provide further information to be considered in future design stages.

5.6.4 Subsurface Conditions

The project specific geological units are summarised in Table 8 below together with some high-level comments on how they vary in thickness, constituents, and strength/stiffness along the viaduct alignment.

In order to provide geotechnical advice at this stage prior to the supplementary ground investigation, preliminary ground profiles have been developed and are included in Appendix E1.

Table 8 - Summary of Project Specific Geological Units

Unit	Description
Uncontrolled Fill (FILL)	Fill materials varied in thickness and composition along the length of the viaduct as follows from North to South: • Ch 28,200 to Ch 28,900 (Forrest Rd) - up to about 1 m thick comprising sandy gravel associated with eth existing rail alignment, • Ch 28,900 to Ch 29,600 (Church Avenue)/Armadale Station area – variable thickness up to about 4 m thick comprising sand and sandy gravel over probable construction waste layer (brick, ballast, slag) with minor organic content. Density varies form medium dense down to loose at depth. • Ch 29,600 to Ch 30,000 - up to about 1 m thick comprising sandy gravel associated with eth existing rail alignment.
Colluvium (COL)	Variable mixtures of Clay, sandy Clay and clayey Gravel, typically dense to very dense or stiff to hard locally weakly to well cemented (Ferricrete Duricrust – see comments below).
Yoganup Formation (YOG)	Varying mixtures of clayey Sand and Clay with sand locally, typically medium dense to very dense or very stiff to hard locally with well cemented zones (particularly in clay dominated zones - Ferricrete Duricrust – see comments below). Typically greater thicknesses of clay dominated materials are present North of Armadale Road (Ch 28,500), becoming sand dominated with rare clay bands/layers south of Ch 28900 (Forrest Rd).
Cattamarra Coal Measures (CCM)	See comments in text above this table. Unit not encountered/proved north of Ch 28,900 (Forrest Rd). Where encountered, variably dense to very dense (silty) Sand with weak cementing to hard Clay and weathered conglomerate from very low up to medium strength. Typically weathering reduces and strength increases with depth below top of layer.

5.6.5 Variably Cemented Materials

Duricrust or cemented material (Ferricrete) is intermittently present through the project alignment, of variable thickness and strength and has been classified by Advisian in the investigations carried out to date in accordance with AS1726-2017, as follows:

- Grade DIII - (Nodular/Fragmental) Less than 50% of the ground consists of gravel and cobble sized nodules (rounded or sub-rounded) or fragments (angular or subangular) of duricrust rock material and is described as a soil.
- Grade DII - (Vuggy or Patchy) Between 50% and 90% of the ground consists of duricrust rock material which forms a continuous framework and is described as a rock (Ferricrete).
- Grade DI - (Massive) More than 90% of the ground consists of duricrust rock material with forms a continuous framework and is described as a rock (Ferricrete).

Where present the duricrust may provide a good founding stratum or the capability to excavate with steep temporary batters, however the variable thickness and grade of cementing may form obstructions or zones of difficult excavation/piling, similar to the difficulties that have been experienced with advancing certain investigation methods (e.g. CPTs). In addition, although cemented, the duricrust layer is also likely to exhibit variable permeability depending on the grade of cementing.

Location specific assessments will need to be made where this layer is critical to foundation solutions or temporary works.

5.6.6 Design Groundwater Level

Available groundwater monitoring data from the latest Advisian monitoring report (ref 311012-00745-GT-MEM-0011- April 2022) in the vicinity of the viaduct is summarised below. Locations are shown in Figure 5:

- BRE-BH01 – screened in YOG – Max recorded GWL, 37.36 m AHD (18.7 m bgl) - Jan 22
- BRE-BH02 – screened in YOG - Max recorded GWL, 38.52 m AHD (15.8 m bgl) – Dec 21
- BRE-BH04 – screened in CCM - Max recorded GWL, 37.85 m AHD (18.4 m bgl) – Jan 22
- BRE-PBH01 – screened in YOG - Max recorded GWL, ~37.5* m AHD (17.7 m bgl) – Nov 21
- BRE-PBH02 – screened in YOG - Max recorded GWL, 36.65 m AHD (19.9 m bgl) – Dec 21
- BRE-PBH03 – screened in COL/Duricrust - Max recorded GWL, DRY
- BRE-ABH04 – screened in YOG/Duricrust - Max recorded GWL, DRY

Note: * Value interpreted from hydrograph between manual monitoring dates

In addition groundwater monitoring was carried out by WSP (ref BRE-WSP-GE-RPT-00002) at WSP–BH01 to WSP–BH04 (locations shown in Figure 5) in monitoring wells screened to 15 m depth (YOG). However, all four wells were dry during monitoring between January 2021 and March 2021 and no other data is available for these locations.

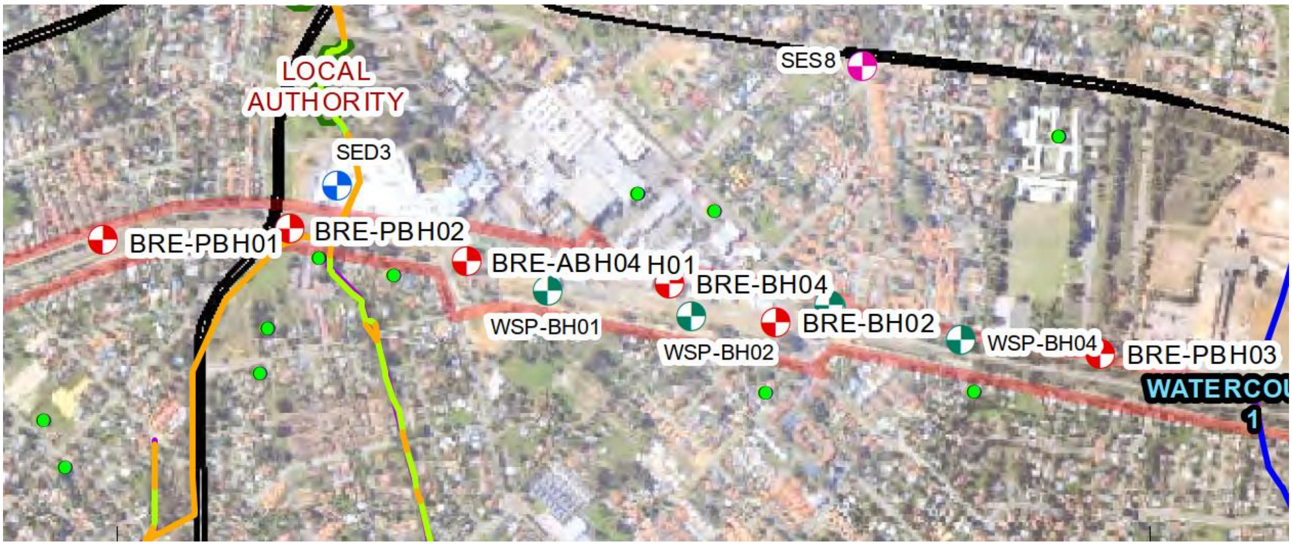


Figure 5: Available Groundwater monitoring data locations

Advisian has assessed the following design ground water levels (noting that they vary along the length of the viaduct):

- DGWL – 1% AEP varies between RL 42 m AHD (at the northern and southern end of the viaduct) and RL 37 m AHD (below Armadale Station)

- DGWL – 2% AEP varies between RL 42 m AHD (at the northern and southern end of the viaduct) and RL 37 m AHD (below Armadale Station)

The assessed 1% and 2% AEP levels are very similar.

The above DGWLs are for the “deeper” aquifer within the Yoganup Formation. There remains a risk that perched groundwater may occur in and on the shallow Fill and Colluvium geology which is typically fine grained.

A design groundwater study is being carried out for the alignment and will be reported under separate cover. At this Reference Design Stage, for preliminary geotechnical design a conservative general design groundwater level at 10 m below ground level (about RL 45 m AHD) has been assumed for design of piles and a shallow design groundwater level at 0.5 m bgl has been considered for the design of shallow foundations. Design groundwater levels will be confirmed in the next stage of design.

5.6.7 AS1170 Hazard Factor and Site Sub-Class

Based on the general geology beneath the site (i.e. typically dense to very dense or very stiff to hard soils overlying rock by about 25 to 35 m depth), the results of the investigation to date and the recommendations provided in AS1170.4-2007, a site subsoil class of Ce to Section 4.2 of AS1170.4 is recommended for seismic design purposes. This will be confirmed following the supplementary ground investigation.

The hazard factor (Z) for the site is shown on Figure 3.2(D) of AS1170.4 as 0.09.

The Spectral Shape Factor (Ch(T=0s)) for Ce sub-soil class is 1.3.

5.6.8 Liquefaction

Liquefaction during an earthquake is a process resulting in saturated soils exhibiting a drastic loss in strength and stiffness. Liquefaction is the result of a rapid pore water pressure increase in response to the cyclic earthquake shaking. Materials that are typically susceptible to liquefaction during an earthquake are usually geologically young granular materials with low fines content in a relatively loose condition below the water table.

The materials present at the site do not generally fall under this general description and based on a preliminary qualitative assessment the materials are generally not considered liquefiable.

The exception may be the shallow fill materials in the vicinity of Armadale Station site and a further assessment will be made of these materials following the supplementary ground investigation. It is noted however, that based on the investigation to date, loose zones appear to be discrete and discontinuous.

5.6.9 Soil and Groundwater Aggressivity

Based on our review of the soil chemical testing carried out at the site and broader results from the project, conditions are indicated to be non-aggressive to mild for pile design, in accordance with AS2159. On this basis, at this stage we recommend the following exposure classifications for reinforced concrete (in accordance with AS3600:2018 Table 4.8.1):

- Reinforced concrete piles – Category B1 (to be reviewed at next stage in particular if piles extend into CCM)
- Shallow reinforced concrete foundations – Category B1

The results of the aggressivity testing will be reviewed by the durability consultant to develop the project specific durability management plan.

5.7 Design Approach and Methodology

5.7.1 Integration with Structural Design

The calibration process between the geotechnical and structural models is an iterative process. In order to include soil-structure interaction effects in the structural models, soil spring stiffnesses, either non-linear or simplified bi-linear elasto-plastic springs, are developed for the design of the foundation elements as provided in this report. Further analysis such as 2D or 3D finite element analysis of selected elements may be carried out at the detailed design stage to confirm the correlation between the structural and geotechnical models.

5.7.2 Pile Design

5.7.2.1 Geotechnical Risks and Uncertainties

Large, 1.8 m diameter cast in situ bored piles are proposed to support the viaduct. Although fairly common in Australia, such large diameter piles have not commonly been used to date in the Perth metropolitan area (although it is noted that it is currently proposed to use this foundation solution for a very similar viaduct structure on another PTA project) and as such the following has been considered in the design:

- Constructability considerations: depending on bore support method (i.e. temporary support fluid or casing) there may be a stress relaxation/softening mechanism prior to pouring concrete which is likely time dependent (i.e. how long the bore remains open) and will affect the actual friction at the interface. There is also an increased risk of bore instability for larger diameter piles.
- Design considerations: based on above, selection of design parameters to consider the potential for scale effects between a large diameter bored piles and more common diameter bored piles (i.e. 750 mm to 1200 mm diameter).

5.7.2.2 Design and Construction Risk Mitigation Measures

In order to alleviate the design and construction risks, the following design and construction approach will be taken:

- Additional geotechnical investigation is to be carried out in accordance with Section 5.6.2
- Pre-production pile load testing of large diameter piles.
- Construction requirements from PTA Specification 8880-450-077 are adhered to (where relevant).
- Supervision of all piles by a geotechnical engineer and sign-off on hold points such as base cleaning, de-sanding, etc.
- Comprehensive record keeping and verification of records.
- Integrity testing of selected piles using a combination of thermal integrity profiling or cross-hole logging.
- Design for serviceability ensuring that the serviceability loads are mobilised primarily by shaft resistance and that end bearing requirement is limited.

- Adoption of prudent pile design parameters and axial geotechnical capacities calculated based on a ϕ_g selected in accordance with AS2159 where pre-production pile load testing is carried out on similar size piles in similar geology with increased construction supervision.

In particular it is expected that experience gained on the other PTA project will greatly reduce any potential uncertainty in the appropriate design and construction method.

5.7.2.3 Pile Load Testing Considerations

The pile load test requirements based on PTA Specification 8880-450-077 are summarised in Table 9.

Table 9 – Pile Load Test Requirements Based on PTA Specification 8880-450-077

Type of Tests	Testing regime*	No. of tests based on total number of production piles (56)
Load Tests		
Static (including, compression, lateral and tension)	Minimum 2 tests or 2% of the total number of piles	2
Dynamic	Minimum 5 tests or 5% of the total number of piles	5
Integrity Tests		
Proof coring	Minimum 2 tests or 0.5% of the total number of piles	2
Sonic logging	Minimum 2 tests or 1% of the total number of piles	2
Low Strain	Minimum 2 tests or 2% of the total number of piles	2

In general, the objectives of pile load testing may be viewed as follows:

- Confirm the suitability of the construction method to achieve the design requirements
- Confirm the suitability of the design parameters adopted to estimate pile load capacity

We consider that in order to achieve both the objectives above and for the load tests to be representative of the proposed piles, the pile load tests should be conducted on similar diameter piles to the production piles (either 1.5 m or 1.8 m diameter).

However, it is unlikely to be practical to test the proposed 1.8 m diameter piles to the required working or ultimate loads by high strain dynamic load tests (a hammer weight of about 1% to 2% of the test load will be required) or conventional static load test (large footprint due to spacing with reaction pile, test frame, etc.). The only method to test such large diameter piles will likely be using bi-directional load cells. This is currently the preferred option and a comparative design has been carried out with a geotechnical reduction factor assuming that two static load tests are carried out in accordance with AS2159, see Section 5.7.2.4 below.

The load testing strategy is currently being reviewed and will be discussed and agreed with PTA, but it is expected that it will not comply with the requirements listed in Table 9. A deviation will be prepared and submitted to PTA for consideration.

5.7.2.4 Design Axial Geotechnical Strength

Pile design is undertaken in accordance with Australian Standard AS 2159-2009.

Australian Standard AS 2159-2009 provides recommendations for reducing the design ultimate geotechnical capacity $R_{d,ug}$ to the design geotechnical strength $R_{d,g}$ by using a geotechnical strength reduction factor ϕ_g according to the following equation:

$$R_{d,g} = \phi_g R_{d,ug}$$

The design ultimate geotechnical capacity $R_{d,ug}$ is calculated as follows:

- In compression: $R_{d,ug} = Q_s + (f_b + p_0)A_b - W$
- In tension: $R_{d,ug} = 0.8Q_s + W$

Where:

- Q_s is the total shaft resistance (sum of the unit shaft resistance fs in each layer multiplied by the pile perimeter over the pile length
- f_b is the unit end bearing resistance
- A_b is the pile end bearing area
- W is the self weight of the pile
- p_0 is the soil total vertical stress at the base level of the pile

Refer to section 5.7.2.5 for ϕ_g adopted for the design.

5.7.2.5 Geotechnical Strength Reduction Factor, ϕg

AS 2159-2009 provides recommendations for the geotechnical strength reduction factor (ϕ_g) based on the following:

- Type of load testing (static, rapid, dynamic, and bi-directional load testing).
- Percentage of the total piles tested.
- Risk assessment based on several factors related to site, design and installation including complexity, amount and quality of geotechnical data, pile design and installation procedures, experience with similar foundations, level of construction control, and level of redundancy.

A basic Geotechnical Strength Reduction Factor, $\phi_g = 0.56$ has been adopted based on the design and construction risk mitigation measures listed in Section 5.7.2.2. See Appendix E2 for the assessment.

For comparative purposes a preliminary design has also been carried out for $\phi_g = 0.70$, assuming static load testing was carried out on 2 piles (about 1.5% of the total number of piles). See comments regarding potential pile load testing in Section 5.7.2.3 above.

5.7.2.6 Lateral Behaviour

Horizontal soil spring stiffness values have been provided to the structural designer for modelling of the lateral pile behaviour. The horizontal spring stiffness and limiting spring forces to be used in the structural model were assessed using non-linear soil pressure versus lateral displacement curves (also commonly termed “p-y” curves).

These parameters will be used by the bridge designer in a model where the bending response of the individual piles will be assessed.

5.7.2.7 Design for Serviceability

The piles are designed to comply with the design criteria summarised in Section 4.5.

In particular, to ensure serviceability requirements and limit geotechnical risks, the piles are designed such that the serviceability design load (permanent effects plus live load) is primarily mobilised by the unfactored pile shaft resistance and that the requirement for end bearing under serviceability load is limited.

The end bearing resistance is only considered in full for the ULS load combination to check the axial geotechnical strength against the ULS design actions in accordance with AS1259-2009.

5.7.3 Retaining Wall Design

The design/sizing of the reinforced concrete gravity retaining walls for the approach embankments, including structural design and the external stability check (sliding, overturning, bearing pressures) is completed by others based on the geotechnical information provided in this report.

This report provides a preliminary assessment of the retaining wall footings and will provide global stability verification at future stages once geometry is set, as well as other geotechnical input for the design of the retaining walls.

Reference must be made to the SWTC Book 4: Technical Criteria and to PTA specification 888—450-053, Retaining Walls and Shallow Foundations. In particular:

- Any substructure elements within the PTA reserve shall retain 2.5m clear depth from existing or future ground surface level (clear zone for PTA services and/or third party services or future development etc.).
- The foundation depth must be designed for provision of proposed and future services, such that services do not traverse under the foundations zone of influence.
- If embedded walls are considered (e.g. piled walls etc. – not currently considered) then the passive resistance 1 m below the design ground level must not be relied upon.

5.7.3.1 Design for Serviceability

The retaining wall footings/shallow foundations are designed to comply with the design criteria in Section 4.5.

5.7.3.2 Bearing Capacity

The bearing capacity of shallow footings is assessed in accordance with AS5100.3-2017. The footings shall be proportioned such that $R_{dg} = \phi_g \times R_{ug} \geq E_d$ where:

- R_{dg}: design geotechnical strength of the footing (or factored bearing capacity).
- R_{ug}: ultimate geotechnical strength of the footing using unfactored characteristic values of material parameters (ultimate/unfactored bearing capacity).
- ϕ_g: geotechnical strength reduction factor which was taken as 0.45 for shallow footings based on the current level of geotechnical investigation, ground conditions and footing preparation procedures carried out in accordance with the Project Specifications.
- E_d: factored structural design action effects (Ultimate Limit State, ULS).

R_{ug} is assessed using the Brinch-Hansen bearing capacity formulae.

The bearing capacity of the retaining wall footings must be checked considering moment induced load eccentricity using approaches recommended by Meyerhof or similar.

Influence of the groundwater level is allowed for by adjusting the unit weight of the soil above and below the base of the footing based on recommendations provided in the Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 2006) and summarised in Table 10.

Table 10– Groundwater Level and Soil Unit Weight for Bearing Capacity of Footings

Depth of groundwater below finished ground surface	Unit weight of soil below the base of the footing	Unit weight of soil above base of the footing
$0 \leq z < D$	γ'	$\gamma - \left(\frac{z}{D}\right)\gamma_w$
$D \leq z < D + B$	$\gamma' + \frac{z - D}{B}\gamma_w$	γ
$Z \geq D + B$	γ	γ

Notes:

D = depth below ground level to base of footing, B = footing width, γ = bulk unit weight, γ = effective bulk unit weight, □w = unit weight of water

5.7.3.3 Lateral Capacity

In accordance with AS5100.3-2017, footings subject to horizontal loads shall be proportioned such that the design action effect (S*) shall satisfy the following:

$$\phi_g(H_{ug} + E_{pr}) \geq E_d$$

With:

- ϕ_g: geotechnical strength reduction factor taken as 0.55 for shallow footings.
- H_{ug}: ultimate shear resistance at the base of the footing which in sand is taken as $H_{ug} = Vtan\delta$. The interface friction angle (δ) between concrete and soil varies depending on how the concrete was placed against the soil. If mass concrete is cast in situ on soil, then the interface friction angle may be taken as ϕ’. If the soil is placed against a preformed concrete surface or a precast concrete structure is placed on the soil, the interface friction angle is expected to be less than the drained friction angle of the soil. Typical values quoted in the literature suggest that δ’ = 0.6ϕ’ to 0.8ϕ’ for fully drained granular soils. For concrete cast directly against in situ sand or granular fill, δ’ = ϕ’.
- E_{pr}: passive resistance of the ground in front of the footing. Potential for future planned or unplanned excavation must be considered if the passive resistance is relied upon in the design.

5.7.3.4 Global Stability

The global stability verification is carried out using the commercially available software Slide (Rocscience) and the General Limit Equilibrium/Morgenstern-Price method using unfactored soil properties and loads. This approach is adopted because the factoring of the unit weight has two effects as follows:

- an increase in driving forces, which is the effect sought after by increasing the dead weight of fill
- an increase of the shear strength (as it is related to the vertical stress) and therefore of the resisting forces.

Based on the approach adopted for the global stability analysis, a minimum factor of safety (FS) of 1.35 has been adopted. This criterion is in accordance with PTA Specification 8880-450-074 recommended minimum FS for slope stability.

5.8 Geotechnical Design Advice and Calculations

5.8.1 Viaduct Foundation Design

5.8.1.1 Geotechnical Design Profiles and Parameters

Preliminary geotechnical design profiles have been developed based on the currently available geotechnical data. Adopted design profiles and parameters are presented in Appendix E1.

5.8.1.2 Pile Axial Capacity

Axial pile capacity has been assessed using the methods described in Section 5.7.2and is plotted against elevation on Figures E2-1 to E2-8 in Appendix E2.

5.8.1.3 Design Pile Toe Levels and Settlement Estimates

The proposed design pile toe levels based on the preliminary loading information and the design charts in Appendix E2 are summarised in Table 11.

Table 11– Preliminary Assessment of Pile Toe Levels (Approx. pile lengths in brackets)

Design Location (see Appendix E1)	Pile Toe Level RL m AHD (m bgl)	
	$\Phi_g = 0.7$#	$\Phi_g = 0.56$
North – Profile A	35.0 (18)*	28.5 (26.5)**
North – Profile B	27.5 (27.5)**	27.5 (27.5)**
Central	36.0 (19)*	25.5 (29.5)***
South	34.0 (21)*	24.0 (31)

Notes:
* Pile length based on Serviceability/Working Load criteria (see assumptions listed below)
** Minimum 2 m into Dense Sand of Cattamarra Coal measures
*** Minimum 2 m into Low to Medium Strength Cattamarra Coal measures
# Pile design assuming 2 No. (about 1.5%) static load tests – See Appendix E2.

The pile toes levels in Table 11 above are recommended based on the following criteria:

- $E_d < R_{dg}$, and
- $0.8 \times E_{ds}$ < Ultimate Shaft Resistance, F_s

Where E_d = Design Action Effect, E_{ds} = Design Serviceability Load, R_{dg} = Design Geotechnical Strength of pile.

5.8.1.4 Pile Settlement

A preliminary assessment of pile settlement has been made as summarised in Appendix E2.

Where E_{ds} exceeds the ultimate shaft resistance (F_s), the pile base is required to supplement working load resistance and pile settlement is likely to become critical. Where $E_{ds} < F_s$, pile settlements are not expected to exceed 15 mm. Where the base resistance comprises 20% of the unfactored load resistance, pile settlement is expected to be of the order 20 to 25 mm (to be confirmed based on actual ground conditions, and pile length etc).

5.8.1.5 Vertical and Horizontal Spring Stiffness

Horizontal spring stiffness are provided in Appendix E2 and will be used by the structural designer to model soil-pile interaction in the structural model. Vertical springs will be provided based on a detailed assessment of pile settlement once pile loads and pile lengths are confirmed at the next stage. The preliminary advice above and in Appendix E2 regarding pile settlement may be used at this stage for assessment of vertical pile behaviour.

5.8.2 Retaining Walls

L-shaped retaining walls are proposed to confine the approach embankments at the northern and southern end of the viaduct. Walls may be up to about 6.5 m high.

5.8.2.1 Earth Pressures

The retaining wall may be designed using the parameters presented in Table 12 below, which assumes a compacted well graded granular sand fill.

Table 12 – Cantilever Reinforced Concrete Retaining Structures – Earth Pressure Design Parameters for Compacted Granular Fill

Soil Unit	γ (kN/m3)	ϕ' (°)	E' (MPa)	K_0	Soil-Wall friction = $0.5\phi'$	
					K_a	K_p
Compacted Granular Fill	19	36	60	0.6	0.22	6.5

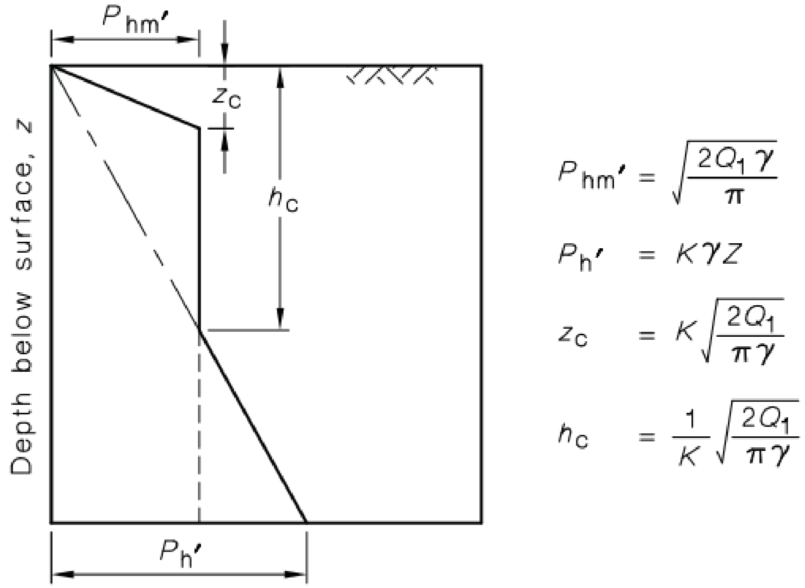
Notes:
γ : soil unit weight; ϕ': angle of internal soil friction; K_0: coefficient of earth pressure at rest, K_a: coefficient of active earth pressure, K_p: coefficient of passive earth pressure; E' – long term Young's modulus.
Values of K_0 are based on estimated initial conditions following compaction.

The above parameters are based on the condition of a horizontal ground surface behind the retaining structure. Applicable surcharge loads behind the wall must also be considered in the design.

Retaining structures should be designed in accordance with AS 4678-2002 "*Earth Retaining Structures*" or an alternate approved factor of safety approach (e.g. AS5100). A geotechnical reduction factor of 0.50 for stability calculations is recommended for simple methods of analysis if using AS5100.

In addition to the above loads, pressures due to compaction must be considered. Induced compaction pressures are dependent on the stiffness of the wall, as the deflection of the wall will act to dissipate the pressure on the back of the wall. Some general advice on assessing compaction pressures is provided below.

The calculation of earth pressure behind retaining structures can be idealised using Figure J5 in AS4678:2002, based on Ingold (1979), as shown on Figure 6 below.



LEGEND:

K	= earth pressure coefficient (see Note 2)
Q_1	= intensity of effective line load imposed by compaction plant (see Note 3)
z_c, h_c	= critical depths as shown
γ	= soil unit weight
P_{hm}'	= maximum horizontal earth pressure induced by compaction
P_h'	= horizontal earth pressure induced by overburden stress

Figure 6: Compaction-Related Earth Pressures (AS4678:2002 Fig J5, based on Ingold 1979)

For the use of the above equations, the Q_1 value should be calculated as follows, expressed in kN/m:

$$Q_1 = \frac{(Weight\ of\ Plant + Centrifugal\ Compaction\ Force)}{Smallest\ of\ Plate/Roller\ Plan\ Dimensions}$$

The above equations generally result in a load $P_{hm'}$ of between 20-30kPa for small to large plate compactors respectively. Where heavier vibrating rollers/compaction is proposed, roller loads between 50 kPa and 73.5 kPa may be assumed.

Compaction-induced horizontal pressures can be considered as an increase in the effective K_0 for a given section of wall. For the assessment of geotechnical ULS stability cases where the retaining wall under consideration fails via overturning, sliding or bearing capacity failure and the destabilising pressures would ordinarily reduce from K_0 to K_A as part of this assessment, compaction pressures need not be considered.

For the structural assessment of walls (e.g. shear/moment capacity), compaction-related pressures generally form a temporary load condition, which must be assessed within standard load combinations for temporary loads. Unless the walls are rigid, this temporary load should not normally be combined with other live or temporary loads (e.g. wind/surcharge or impact loads). Horizontal flexibility of at least 0.1% of the retained height (e.g. 1mm per 1m of retained height) is generally required to release compaction-induced pressures and classify a wall as non-rigid.

5.8.2.2 Bearing Capacity

The proposed retaining walls for the approach embankments are likely to be formed in Fill associated with the existing rail, or in the underlying Colluvium. Typically, the Fill comprises a medium dense sandy gravel.

On this basis a general preliminary design geotechnical strength ($\phi_g R_{ug}$) of 300 kPa may be used for strip footings greater than 2 m width, assuming:

- Normal site preparation procedures (as typically included in project specifications).
- A bulk unit weight of 18 kN/m3.
- An angle of internal friction of 35° for medium dense sand/sandy gravel founding materials.
- Geotechnical strength reduction factor (fg) of 0.45.
- Footings are not located on or adjacent to sloping ground (such footings will need to be assessed separately).
- Permanent embedment depth remains in place for the duration of the design life.
- Design Groundwater Level (DGWL) is 0.5 m below ground level (at footing level).

It is noted that further assessment of founding conditions will be carried out as part of the supplementary ground investigation.

Settlements will be assessed in the next design stage, noting that where strip footings are large (for up to 6.5 m high walls, footing widths of the order of 4 to 5 m may be required), settlements may exceed 20 mm during construction and/or long term.

5.8.2.3 Sliding

Sliding resistance on the base of the retaining wall will depend on how the retaining wall foundation is formed. If the foundation is cast in situ on the soil, then the interface friction angle may be taken as the peak friction angle of the soil, ϕ' (in this case a value of 35 degrees may be assumed). Where the retaining wall relies on some passive resistance to resist sliding the interface friction able should be limited to the critical state friction angle (30 degrees).

Where the retaining wall footing is formed by a precast element placed on the soil, the interface friction angle δ, should be reduced to a value of between 0.6ϕ' to 0.8ϕ' for fully drained granular soils.

5.8.2.4 Global Stability

The global stability of the retaining walls will be checked during future design stages once the retaining wall design has progressed.

5.8.3 Track Slab – Modulus of Subgrade Reaction

A vertical coefficient of subgrade reaction is required by the structural designer to design the transition slabs between the viaduct abutment and the approach embankment. At the bridge abutment location, the transition slab fill be founded on up to 8 m of compacted select fill. A coefficient of vertical subgrade reaction, relevant for unload-reload transient train loading (70 kPa over a 2.5 m width was assumed – load assumption to be confirmed), of 17.5 MPa/m is proposed. It is recommended that the sensitivity of the structural output is checked by assuming +/-30% of this value.

5.9 Schedules

No geotechnical schedules provided at Reference Design stage.

6. Design Reviews and Certification

6.1 Interdisciplinary Design Coordination (IDC) Review

IDC review has been completed and comments incorporated in this Reference Design submission.

6.2 IDC Certificate

See main design package for IDC certificate.

6.3 Design Checking and Verification

In accordance with internal procedures.

6.4 Independent Verification

To be carried out.

6.5 BCA

N/A

6.6 DDA

N/A

6.7 PTA Design Submission Reviews

To be carried out.

7. Safety Assurance

See main design package and SiD report.

8. Systems Engineering

See main design package.

9. Sustainability in Design

See main design package.

10. Human Factors

N/A

11. Reliability, Availability and Maintainability (RAM)

See main design package.

12. Construction Methodology

Further information on constructability issues as they relate to the geotechnical design will be provided in later design stages.

12.1 Construction Methods

Information will be provided in future stages.

12.2 Operational Staging

Where relevant information will be provided in future stages.

12.3 Works in Track Occupancies

N/A (included in main package where relevant)

13. Asset Operations Strategy

See main design package.

14. Non-Compliances

The following have been identified as potential non-compliances at the Reference Design Stage which may require further consultation with PTA:

- Pile load testing – See Section 5.7.2.3.
- Settlement of retaining walls – Although not assessed at this stage (geometry to be confirmed), due to the height of the embankment (about 6.5 m maximum) the settlement of the retaining walls may exceed the criteria (see Section 4.5) in the specification. A settlement assessment will be made in the next design stage for the key construction stages (i.e. end of construction, long term etc.). The structural engineer will assess the structural integrity of the wall for these settlements and if satisfactory (also assuming long term settlement is acceptable for the rail) will issue a deviation request.

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