Appendix G: Geotechnical

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MetCONNX Byford Rail Extension

Byford Rail Extension

R30-CMW-RPT-GE-560-00004

Geotechnical Design Report – Eleventh Road Bridge

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METRONET

Byford Rail Extension

Document details	
Title	Eleventh Road Bridge Geotechnical Design Report
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-560-00004

Rev	Date	Revision Description	Prepared by	Reviewed by	Approved by
A.	18-Oct-2022	Issued for Reference Design	Lipjin Tan	Ondrej Synac	Alyssa Edwards

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Glossary

Acronym Term	Description
AHD	Australian Height Datum
AEP	Annual Exceedance Probability
ARI	Annual Recurrence Interval
AS	Australian Standard
ASS	Acid Sulfate Soil
AASS	Actual Acid Sulfate Soil
PASS	Potential Acid Sulfate Soil
BGL	Below Ground Level
BH	Borehole
CPT	Cone Penetration Test
CPTu	Cone Penetration Test with pore pressure measurement (piezocone)
CH	Chainage
DN	Down Line
EIS	Entry Info Service
FS	Factor of safety
IV	Independent Verifier/Verification
MEL	Morley-Ellenbrook Line
MGA	Map Grid of Australia
NA	Not Applicable
PGA	Earthquake Peak Ground Acceleration
PSP	Principal Share Path
PTA	Public Transport Authority
RL	Reduced Level
SCPT	Seismic Cone Penetration Test
SLS	Serviceability Limit State
SWTC	Scope of Works and Technical Criteria
ULS	Ultimate Limit State
UP	Up Line

Term	Description
DGWL	Design groundwater level used for ultimate limit state design for foundations and retaining walls.
C_c	Compression index (consolidation parameter)
C_r	Re-compression (or swelling) index (consolidation parameter)
C_v	Coefficient of vertical consolidation (consolidation parameter)
C_α	Secondary compression index
c'	Drained cohesion of soil
E'	Drained Elastic Modulus
E_d	Design Action Effects
E_{ds}	Design Serviceability Action Effects
F_{nf}	Negative Skin Friction of a pile
K_0	Coefficient of at rest earth pressure
K_a	Coefficient of active earth pressure
K_p	Coefficient of passive earth pressure
OCR	Over consolidation ratio
Q_b	Pile end bearing resistance
Q_s	Pile shaft resistance
$R_{d,g}$	Design geotechnical strength
$R_{du,g}$	Design ultimate geotechnical strength
s_u	Undrained shear strength
UCS	Unconfined compressive strength
δ	Soil/structure interface friction angle
ϕ'	Drained friction angle of soil
ϕ_g	geotechnical strength reduction factor
γ	Unit weight
γ'	Effective (buoyant) unit weight
ν	Poisson's ratio
ρ	Density
σ_v	Total vertical stress
σ'_v	Effective vertical stress

Unit	Description
°	Degree
kN/m³	Kilonewtons per cubic metre
kPa	Kilopascal
m	Metre
mm	Millimetre
kN	Kilonewton
MN	Meganewton
MPa	Megapascal
t/m³	Tonnes per cubic metre

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 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

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

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.

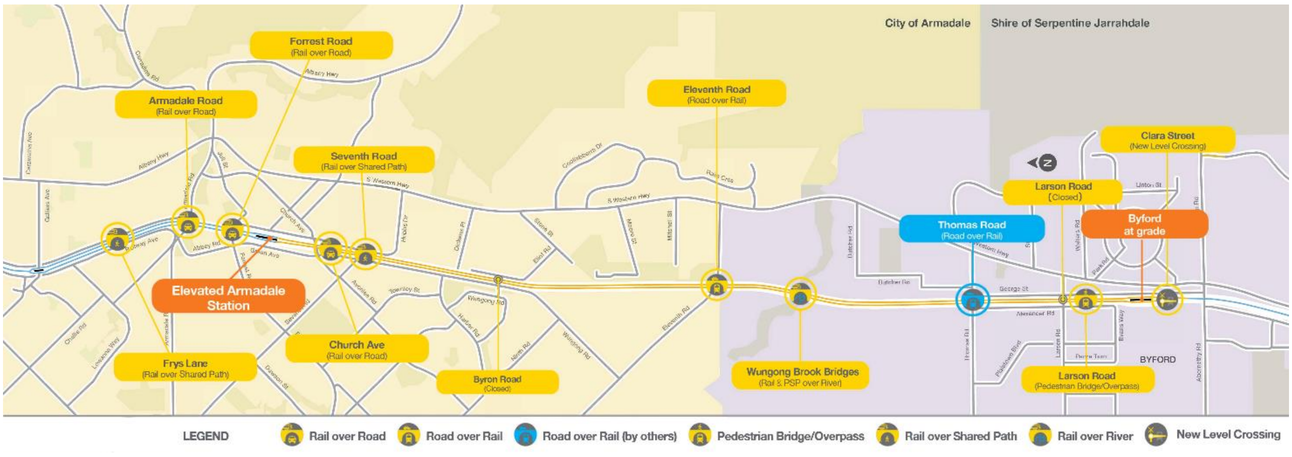


Figure 2: METRONET Byford Rail Extension Project

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:

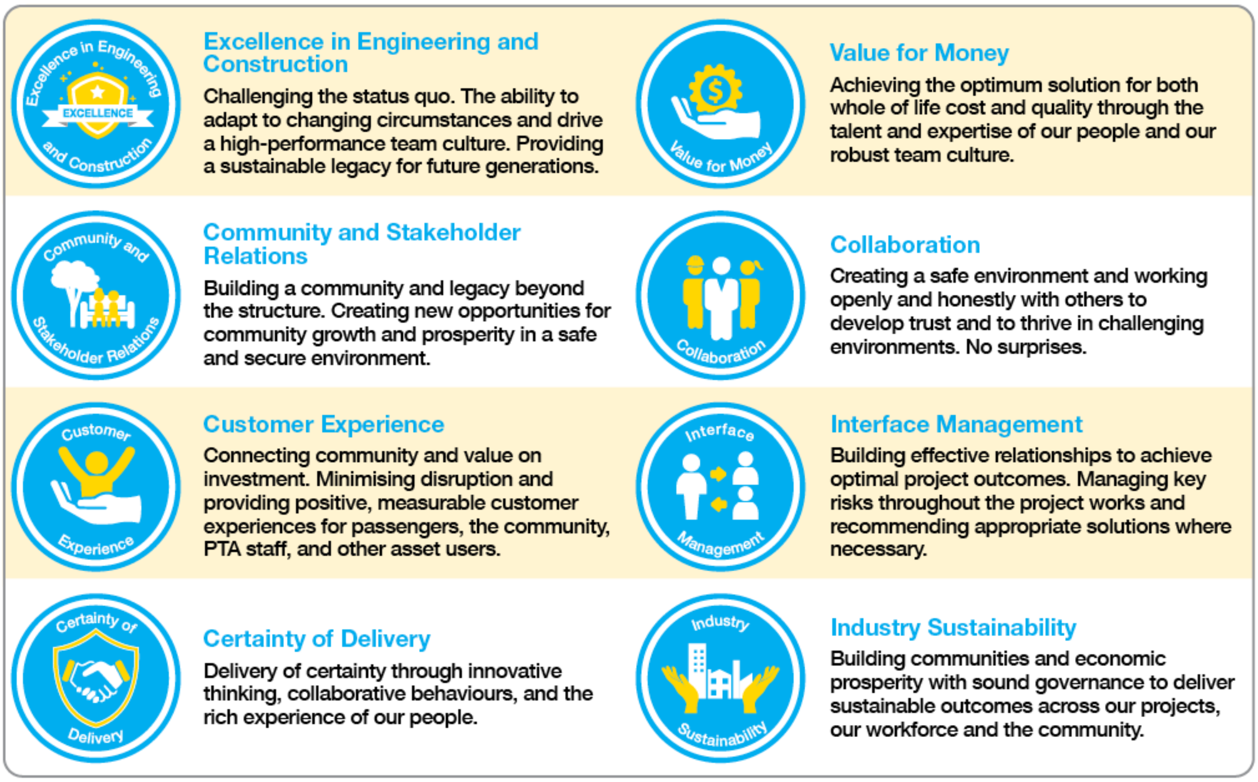


Figure 4: MetCONNX Priorities aligned with Key Project Objectives

2.4 Purpose of the Report

This Design Report presents the design proposals for the geotechnical design information for the RD010: Eleventh Road Bridge Design Package for inclusion in Design Report R30-DEA-RPT-ST-440-00001. This report shall provide the 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-405	Eleventh Road Civil	Provide geotechnical advice for 11^{th} Road bridge
ST-440	Eleventh Road Bridge	Provide geotechnical advice for 11^{th} Road bridge foundations

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.

- Eleventh Road Bridge and associated structures

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 Eleventh Road Bridge
- Geotechnical design information for the proposed retaining walls and other structures associated with the bridge such as MSE Walls, shallow foundations, transitions slabs and deflection walls.

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 design report.

3.2 Relationship with other Design Packages

This Design Report presents the design proposals for the geotechnical design information for the RD010: Eleventh Road Bridge Design Package for inclusion in Design Report R30-DEA-RPT-ST-440-00001.

3.3 External Interfaces

The relationship and/or reliance of this design package on external interfaces and details of integration strategies are outlined in the Table 2 below.

Table 2 – External Interfaces

Item	External Party	Interface Elements	Integration Strategy
N/A	There are no external interfaces in this design package.		

3.4 Changes Since Previous Design Submission

N/A – first submission.

3.5 Bridge

The proposed road bridge has been planned to be constructed in the Eleventh Road and Armadale rail line intersection as shown in Figure 4. The bridge comprises two abutments with the proposed MSE wall and abutments supported by 18 piles of 900mm diameter.

Figure 5: Proposed location of Eleventh Road Bridge

3.6 MSE Retaining Wall

The MSE wall detail will be assessed following the planned additional site investigation.

4. Design Inputs

4.1 Project Design Requirements

The design inputs summarised in Table 3 and Table 4 were relied upon for the design.

Table 3 – Summary of Project Related Design Input

Reference	Revision	Title
Design & Analysis		Pier loading information (email dated 02/05/2022)
BRE-CMW-TAN-170-GE-0001		Tender Advice Notification
BRE-ADV-GE-RPT-00004		Geotechnical Investigation Factual Report

Table 4 – Preliminary Pile Design Actions

	Axial SLS Design Action, E_{ds} (MN)		Lateral SLS Design Action, E_{ds} (MN)	Axial ULS Design Action, E_d (MN)			Lateral ULS Design Action, E_d (MN)
	Permanent Effects	Transient	Transient	Permanent Effect	Transient	Seismic	Transient
Case 1	26.25	-	-	35		-	-

4.2 Design software used for this package

The following design software has been used in preparation of this design report.

- RSPile
- Slide2

4.2.1 RSPile

- RSPile is a general pile analysis software for: Axial Load Capacity of Driven Piles
- Analysis of Piles Under Lateral Loading
- Analysis of Pile Groups under Lateral and Axial Loading
- Bearing Capacity of Driven Piles
- Bearing Capacity of Bored Piles

The software can be used for analysing driven pile installation, axially loaded piles and laterally loaded piles. It can compute the axial capacity for driven piles as well as the pile internal forces and displacements under various loads and soil displacements. RSPile can also compute pile resistance forces for use in Slide2 for enhanced slope stability analysis

4.2.2 Slide2

Slide2 is a 2D limit equilibrium slope stability program for evaluating the safety factor or probability of failure, of circular or non-circular failure surfaces in soil or rock slopes. Slide2 analyses the stability of slip surfaces using vertical slice or non-vertical slice limit equilibrium methods. Slide2 also includes finite element groundwater seepage analysis built right into the program, for both steady state and transient conditions.

4.3 Applicable Codes and Standards

The applicable standards, codes and guidelines are in accordance with SWTC (Table 5).

Table 5 – 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
8103-400-004.5.0.IFU	5	Working In and Around PTA Rail Reserve
8190-400-002.2.5.IFI	2.5	Narrow Gauge Main Line Track and Civil Infrastructure Code of Practice
8880-450-010.2.0.IFU	2	Specification Design Actions, Asset Design Life and Maintenance Free Period
8880-450-053.1.0.IFU	1	Specification Retaining Walls and Shallow Foundations
8880-450-054.1.0.IFU	1	Specification Rail Bridges
8880-450-070.0.IFU	0	Specification Geotechnical Investigations
8880-450-074.1.0.IFU	1	Specification Earthworks Slope Stability Geotextiles and Erosion Protection
8880-450-077.1.0.IFU	1	Spec Deep Foundations

4.4 Reference Information

The project specific reference information and reports that have been used as inputs into the development of the detailed design are included in the Table 6 below.

Table 6 – Geotechnical and Hydrogeological Information

Document Reference	Description/Title	Revision
BRE-CMW-AN-170-GEO-0001	T006 – Geotechnical Site Wide	0
BRE-MNO-WSP-GE-RPT-0001	Geotechnical Factual and Interpretive Report - WSP	-

Document Reference	Description/Title	Revision
BRE-ADV-GE-RPT-00004	Geotechnical Investigation Factual Report	-
BRE-ADV-GE-RPT-00005	Geotechnical Interpretative Report	-

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
- Pile construction using continuous flight auger techniques is not permitted
- Maximum allowable settlement/heave and horizontal deflection of any type of foundation through the design life are summarised in Table 7 and Table 8.

Table 7 – 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	1:1000	1:1000
Deep raft	20 mm	20 mm	1:1000	1:1000
Deep foundation element piles (DFEs)	15 mm	25 mm	1:1000	1:1000

Table 8 – 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 7 and Table 8:

- 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 the Table below. 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 9 – Design Life

Item	Asset Element of the Works	Durability Design Life (Years)
1	11th Road Bridge	120 years

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

N/A

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.

4.15.1 Design Assumptions

See Project Design Requirements section.

4.15.2 Design Dependencies

See Project Design Requirements section.

4.15.3 Design Constraints

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 5.

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 6. 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 on Appendix E Figure A1.

5.6.2 Additional Geotechnical Investigation

Additional geotechnical investigations are required to confirm the ground conditions, in particular the level and properties of the underlying soil layers.

The proposed additional geotechnical investigation is in accordance with AS5100.3 and PTA Specifications 8880-450-070 and 8880-450-077.

The proposed additional geotechnical investigations are presented in the Geotechnical Investigation and Testing Plan which is part of the Geotechnical Plan. A summary of the basis on which the investigation has been scoped is included in Table 10 – Additional Site Investigation Requirements. The existing geotechnical investigation locations are shown on Appendix E Figure A2 and A3.

Table 10 – 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.	2 Boreholes or CPTs to confirm relative densities, stiffness and strength of the underlying soils. Investigations approx. every 25 m to supplement nearby existing

Structure	PTA Requirements	Proposed Geotechnical Investigations
		investigations and depending on consistency.
Bridge Piled Foundation	In accordance with AS5100.3:2017, 1 test location per 10 m length of abutment	6 Boreholes or CPTs to 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 are not present or to supplement CPTs that have terminated at depth above the anticipated pile toe levels.
Deflection Walls	1 test location per 10 m length of wall – locations to be confirmed once deflection wall geometry better understood.	NA

Samples from boreholes will be selected for laboratory testing. The following laboratory test 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

5.6.3 Geological Model Appreciation

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 noted in boreholes within the Colluvium. These duricrust layers are likely laterally discontinuous and will likely be of variable thickness and strength.

Wungong Brook Bridge area on the alignment as well as Armadale Station area are shown to be underlain with Guildford Formation (Cs) described as Sandy clay – fine to coarse grained sub-angular to subrounded sand, clay of moderate plasticity gravel and silt layers near scarp.

Locally, presence of Holocene Alluvium (Msc1) described as Clayey Sandy Silt is also possible in Wungong Brook Bridge area. It is likely that Colluvium (Csg) will be encountered at relatively shallow depth beneath Guildford Formation (Cs) and Alluvium (Msc1) as depicted on schematic cross-sections shown on the map.

Yogunup Formation was encountered in geotechnical boreholes and is anticipated across most of the alignment beneath the Colluvium and Guildford Formation. Based on the available literature, the Yogunup Formation consists of up to 10m unconsolidated poorly sorted sand, gravel, pebbles, with minor clay in a belt up to 5 km from the Darling Scarp.

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 to 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.

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. At Byford the Darling Fault is located approximately 500m east to the proposed Station Location. The Darling Fault separates the Perth Basin (predominantly alluvial and eolian sediments).

Quaternary age over sedimentary rocks of Permian [280my] to Cretaceous [65my] age) from the crystalline rocks of the Yilgarn Craton (granitic and gneissic rocks [2500my] intruded by Dolerite dykes).

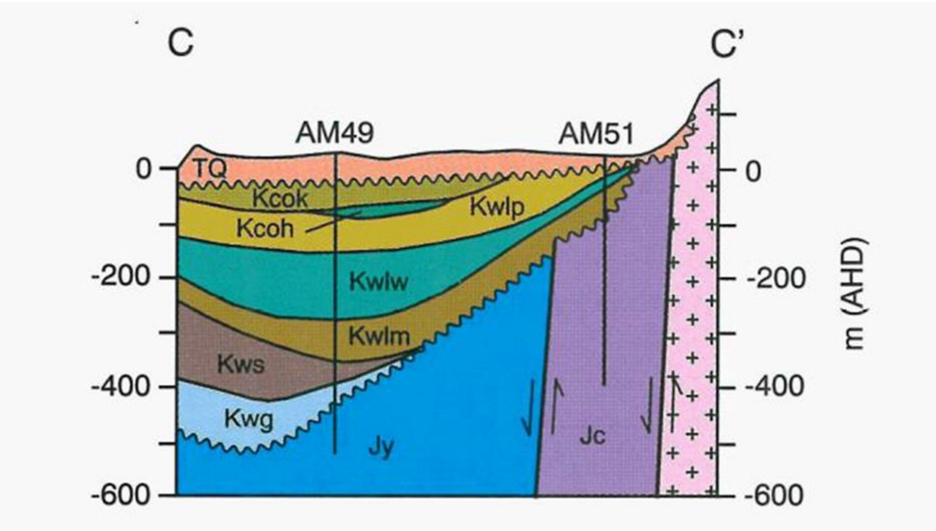


Figure 6: Geological profile

In the early Cretaceous block faulting occurred pushing the older Cattamarra Coal Measures (Jurassic age) west of Darling Scarp upwards. As a result, the Jurassic strata close to the Darling scarp are now at shallow depths <30 m, whilst they are many hundreds of metres below ground level west of the uplifted faulted blocks. It is emphasised that the Darling Fault has been largely inactive since the early Cretaceous circa 50 My.

The presence of Darling Fault means that the underlying rock formations at depth could be quite different at different locations along the project alignment. 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.

One deep borehole (Advisian BRE-BH04) in Armadale area encountered Conglomerate of Cardup Group, which comprise sedimentary rocks located in a 1km zone wide at the base of the Darling Scarp and typically east of the Darling Fault. According to the literature the sediments are weakly metamorphosed and dip steeply or moderately westwards.

It is important to note that some significant geomorphological features associated with various current and historical creeks are present which could result in presence of paleochannels.

Wungong Brook emerges from the Darling Range approximately 1.5kms southeast of the proposed Wungong bridge site before flowing in a north westerly direction towards the proposed bridge. The degree of incision through the granitic rock of the escarpment is considerable whilst the present day stream appears fairly minor. Based on the size of the incision valley it is anticipated that far larger flows existed in the geological past. It is possible that associated deep paleochannel (likely filled with more gravelly sandy material described on geology maps as colluvial debris flows and wash Scg) could be present at the Wungong bridge site below the upper layer of colluvium (Csg) and Guildford Formation (Cs) which might have subsequently covered the site. Such paleochannel, if found, could be a significant drainage feature with potential sub-artisian groundwater conditions.

Another such significant feature is located just north of Armadale Station. It is possible that other smaller paleochannels can be encountered crossing the project alignment in approximately east to west direction.

Available Geotechnical Investigation Data

The site was previously investigated by WSP for the proposed rail extension between Armadale Station to the proposed Byford Station. The investigation comprised of 14 hand augers up to 2m in depth along the rail tracks and 7 boreholes up to 15m located in Armadale Station, Church Avenue, Eleventh Road, and Byford Station. This information alone is insufficient to complete the required scope provided in the SWTC even at tender stage design.

It is understood that Advisian undertaken a geotechnical investigation and based on their revised scope (revised proposed investigation plans are dated 30/04/2021) this is to include 50 CPT tests, 7 hand augers, 19 permeability tests, 6 test pits, and 17 boreholes.

Additional geotechnical investigation is being proposed to supplement the current information and to inform the future detailed design.

Table 11 – Summary of Project Specific Geological Units

Unit	Description
Uncontrolled 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.
Alluvium (ALL)	Variable mixture of very soft to stiff Sandy SILT and Sandy CLAY, organic-rich, medium to high plasticity

Unit	Description
Colluvium (COL)	Variable mixtures of Clay, sandy Clay and clayey Gravel, typically dense to very dense or stiff to hard
Duricrust (DUR)	Variable mixture of Medium Dense to Dense Clayey SAND / Gravelly SAND, weakly to well iron-cemented (in part)
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 encountered
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.
Guildford Formation (GF)	Very stiff to hard Sandy CLAY with gravel and cobbles, medium plasticity

5.6.4 Subsurface Conditions

The inferred subsurface profile based on the available geotechnical investigation is shown in Table 11.

A summary of subsurface conditions at the location of 11th Road bridge is presented in Table 12.

Table 12 – General Design Section for Eleventh Road bridge

Unit	Typical Geotechnical Description
Ground Surface	Comprises an Uncontrolled Fill profile associated with the service corridor (up to 1.5m thick)
COL	Stiff CLAY and Gravelly CLAY very weakly cemented (in part), medium to high plasticity
YOG	Mixtures of poorly graded SAND with clay and Sandy CLAY, dense to very dense/very stiff to hard

For detailed geotechnical design profile and design parameters will be provided in the final report.

5.6.5 Variably Cemented Materials

Duricrust or cemented material (Ferricrete) is intermittently present through the project alignment, of variable thickness and strength.

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.

5.6.6 Groundwater

Details of the groundwater level at Eleventh Road bridge location based on the Golder Hydrogeological report and the WSP Geotechnical report is summarised in the table below.

Table 13 – Design Groundwater level

Structure	Approx. Existing Surface Level (RL mAHD)	Estimated Maximum GWL (from Golder) (RL mAHD)*
11th Road Bridge	46.8	42.7

WSP 2021 (BRE Geotechnical Factual and Interpretive Report) recommended design levels:

38 mAHD between Armadale and the Eleventh Road level crossing

However, it should be noted that these levels do not account for perched groundwater, which is a risk on the Guildford Formation and Colluvium geology, which frequently hosts shallow clay layers, indurated horizons (coffee rock) and duricrust. WSP (2021) and Golder (2021) advised to expect perched groundwater.

5.6.7 AS1170 Hazard Factor and Site Sub-Class

Based on the general geology beneath the site, the results of our investigation 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.

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 (i.e. sands from the Yoganup and Guildford Formation) do not generally fall under this general description. Silty or clayey materials are typically not expected to be susceptible to liquefaction although they may be prone to strength loss (cyclic softening) during an earthquake.

5.6.9 Soil and Groundwater Aggressivity

Soil and groundwater aggressivity testing has been carried out and commented on in the reports listed in Table 6.

Based on our review of the soil chemical testing carried out at the site and broader results from the project, we recommend the following exposure classifications for reinforced concrete (in accordance with AS3600:2018 Table 4.8.1):

Table 14 – Summary of Exposure Classification

Australian Standard Exposure Classification	AS 2159-2009		AS5100-2017
	Steel	Concrete	
Uncontrolled Fill (Only one sample tested)	Non-Aggressive	Mild	B1
Alluvium (Only two samples tested)	Non-Aggressive	Non-Aggressive to Mild	A (least severe exposure classification for concrete in sulfate, acidic and saline soils) to B1
Duricrust	Non-Aggressive	Non-Aggressive to Mild	A (least severe exposure classification for concrete in sulfate, acidic and saline soils) to B1
Colluvium	Non-Aggressive	Non-Aggressive	A (least severe exposure classification for concrete in sulfate, acidic and saline soils)
Guildford Formation (Only two samples tested)	Non-Aggressive	Mild	B1
Yoganup Formation	Non-Aggressive to Mild depending on Soil Condition	Non-Aggressive	A (least severe exposure classification for concrete in sulfate, acidic and saline soils) to B1 depending on Soil Condition
Cattamarra Coal Measures	Predominantly Non-Aggressive to Mild, two (2) samples Severe due to low pH values	Predominantly Non-Aggressive, two (2) samples Mild due to low pH values	A (least severe exposure classification for concrete in sulfate, acidic and saline soils) to B1 depending on Soil Condition. Two (2) samples C1 due to low pH values

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 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 5.7.2.2 for ϕ_g adopted for the design.

5.7.2.2 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.

Geotechnical Strength Reduction Factor **ϕ_g = 0.68** has been adopted based on the adhesion to the design and construction risk mitigation measures in Appendix E.

5.7.2.3 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.

Detailed geotechnical analysis including 3D finite element analysis for selected critical piers and lateral loads may be carried out at future design stages for comparison against or refinement of the structural model.

5.7.2.4 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 entirely mobilised by the unfactored pile shaft resistance.

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

5.7.2.5 Pile Load Testing Considerations

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

Table 15 – 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.

In our preliminary design we have assumed at min 5%of piles will be tested. However, we have provided the axial load calculations in Appendix E with no testing considered.

5.7.3 Retaining walls

The design of MSE walls is based on AS4678. The following ultimate limit states (ULS) and serviceability limit states (SLS) are considered in the design.

ULS:

- U1 : Sliding at the base of the retaining structure
- U2 : Overturning of the structure
- U3: Rupture of components and connections
- U4: Pull-out of reinforcing
- U5 : Global failure mechanisms
- U6 : Bearing failure

SLS:

- S1 : Rotation of the structure
- S2 : Translation or bulging of the retaining wall
- S3 : Settlement of the structure

The specialist MSE wall subcontractor is responsible for the internal stability of the wall and therefore U3, U4, S1 and S2 are outside the scope of this report.

5.7.3.1 Sliding, Overturning and Bearing

An in-house spreadsheet is used to calculate the factor of safety against failure under sliding (U1), overturning (U2) and bearing (U6). The load and material factors used are summarised in Table 18. Clause 5.7 and Table J1 state the required load cases to be checked with include strength (A), stability (B) and serviceability (C).

The Earthquake design category of the walls is Cer (Table I3 of AS4678) to design for static loads with a dead load factor of 1.5 (in lieu of 1.25), is used to meet the earthquake design requirements.

Table 16 – Summary of Load and Material Factors for MSE wall design

Item	Value Adopted in the Design		
	Load Case		
	A	B	C
Dead load (G_1) of structure	1.25	0.8	1
Dead load (G_2) of fill behind structure	1.25	1.25	1
Dead load (G_3) of fill on structure	1.25	0.8	1
Dead load of fill (G_4) in front of structure	0.8	0.8	1
Item	A	B	C
Traffic load (Q_1) or other live load on structure	1.5	0	0.7, 0.4, 1.0
Traffic load (Q_2) or other live load behind structure	1.5	1.5	0.7, 0.4, 1.0
Sand above base of MSE wall (φ_{uc}) – strength	0.90 (Table 5.1(A) AS4678)		
Sand below base of MSE wall (φ_{uc}) – strength	0.85 (Table 5.1(A) AS4678)		

5.7.4 Shallow Foundations (including Gravity Retaining Walls)

5.7.4.1 Design for Serviceability

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

5.7.4.2 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 17.

Table 17 – Groundwater Level and Soil Unit Weight for Baring 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

5.7.4.3 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:

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

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 $H_{ug} = V tan\delta$. 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.

5.7.4.4 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 Geotechnical Design Profiles and Parameters

Adopted design profiles are presented in Figure A2 and A3 of Appendix E.

5.8.2 Pile Axial Capacity

Axial pile capacity has been assessed using the methods described in Section 5.7.2.1 and is plotted against elevation on Figures C6 to C7 in Appendix E.

5.8.3 Design Pile Toe Levels and Settlement Estimates

The proposed design pile toe levels based on the preliminary loading information, and estimated settlement are summarised in Appendix E.

Table 18 – Proposed Design Pile Toe Levels

Pier	E_d (MN) PE+LL	E_{ds} (MN) PE+LL	Assumed Design Pile Cut-off Level RL (m AHD)	Proposed Design Pile Toe Level RL (m AHD)	Estimated Ultimate Shaft Resistance Q_{su} for proposed toe level (MN)	Percentage of Ultimate Shaft Resistance mobilised under Serviceability load (E_{ds}/Q_{su})	Estimated Settlement (mm)
	35	1.5	45	29	2.4	75%	2.5

5.8.4 Vertical and Horizontal Spring Stiffness

Vertical and horizontal spring stiffness are provided in Appendix E and will be used by the structural designer to model soil-pile interaction in the structural model.

5.8.5 MSE /Retaining Walls

The MSE Wall details are not available at this stage.

5.8.6 Transition Slabs

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 6 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), 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.

The slab details are not available at this stage.

5.8.7 Shallow Foundations

N/A

5.8.8 Earthworks

The detailed earthwork requirements are not available at this stage.

5.9 Calculations

All calculations are provided in Appendix E.

5.10 Schedules

No geotechnical schedules provided at Reference Design stage.

6. Design Reviews and Certification

6.1 Interdisciplinary Design Coordination (IDC) Review

IDC review will be completed and comments to be addressed after this first 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

12.1 Construction Methods

This will be provided in next stage of design.

12.2 Operational Staging

This will be provided in next stage of design.

12.3 Works in Track Occupancies

This will be provided in next stage of design.

13. Asset Operations Strategy

See main design package.

14. Non Compliances

To be confirmed following the review of the proposed additional site investigation.

Appendix A: Drawing and Model List (Not in Use)

Appendix B: Specifications (Not in Use)

Appendix C: Drawings (Not in Use)

Appendix D: Engineering Change Approvals (Not in Use)

Appendix E: Calculations

Refer to Appendix E

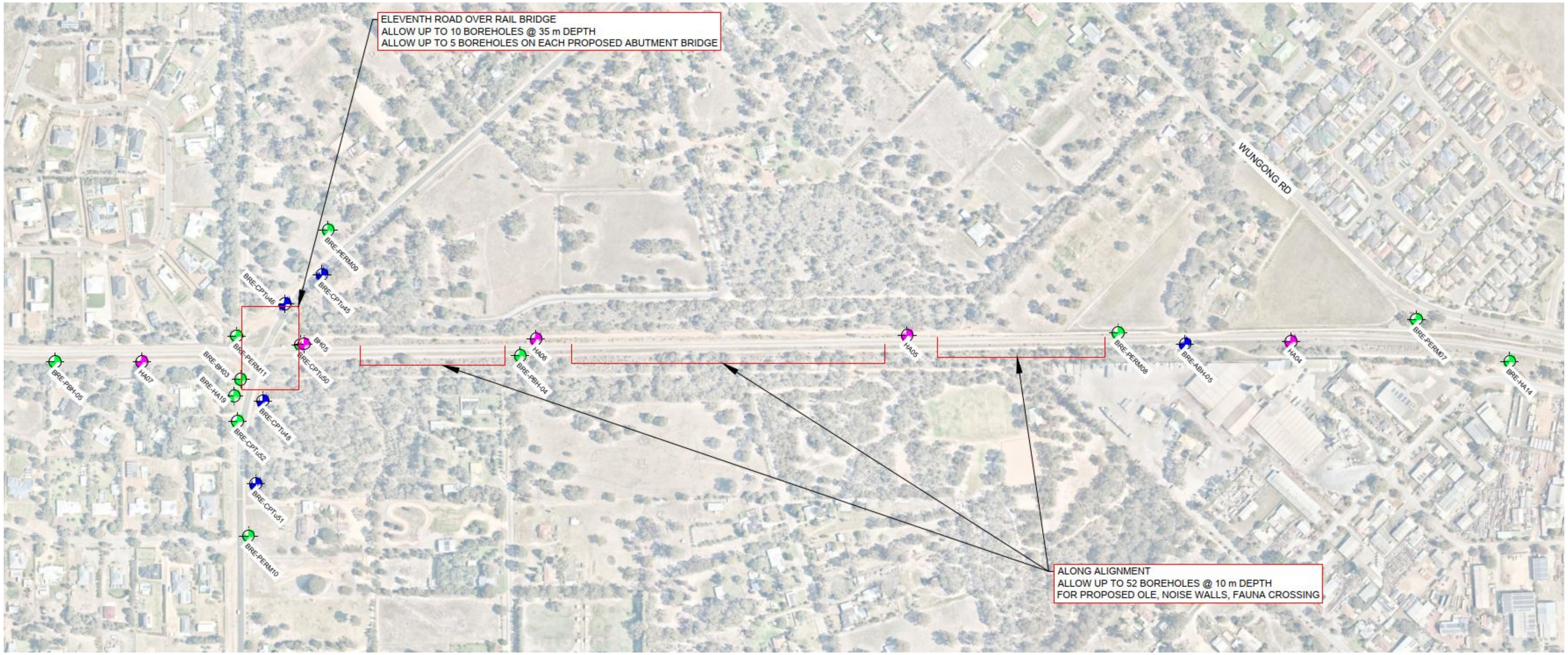


Figure A 1: The existing geotechnical investigation locations

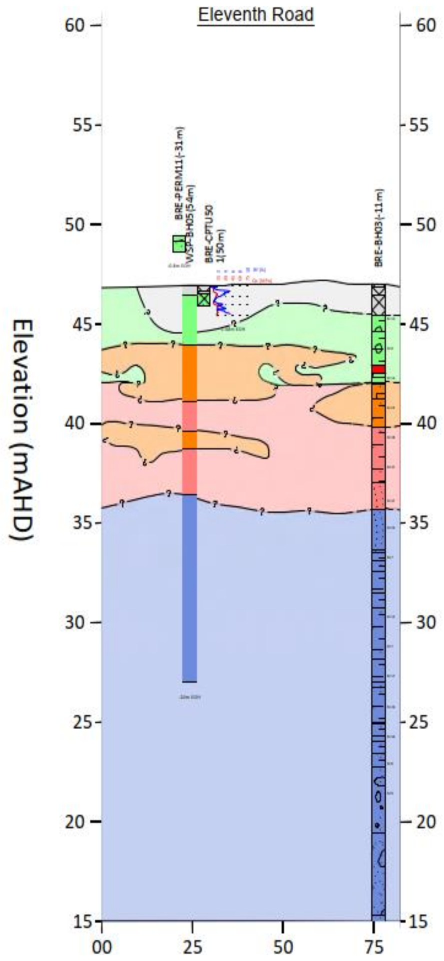


Figure A 2: Engineering geological cross section

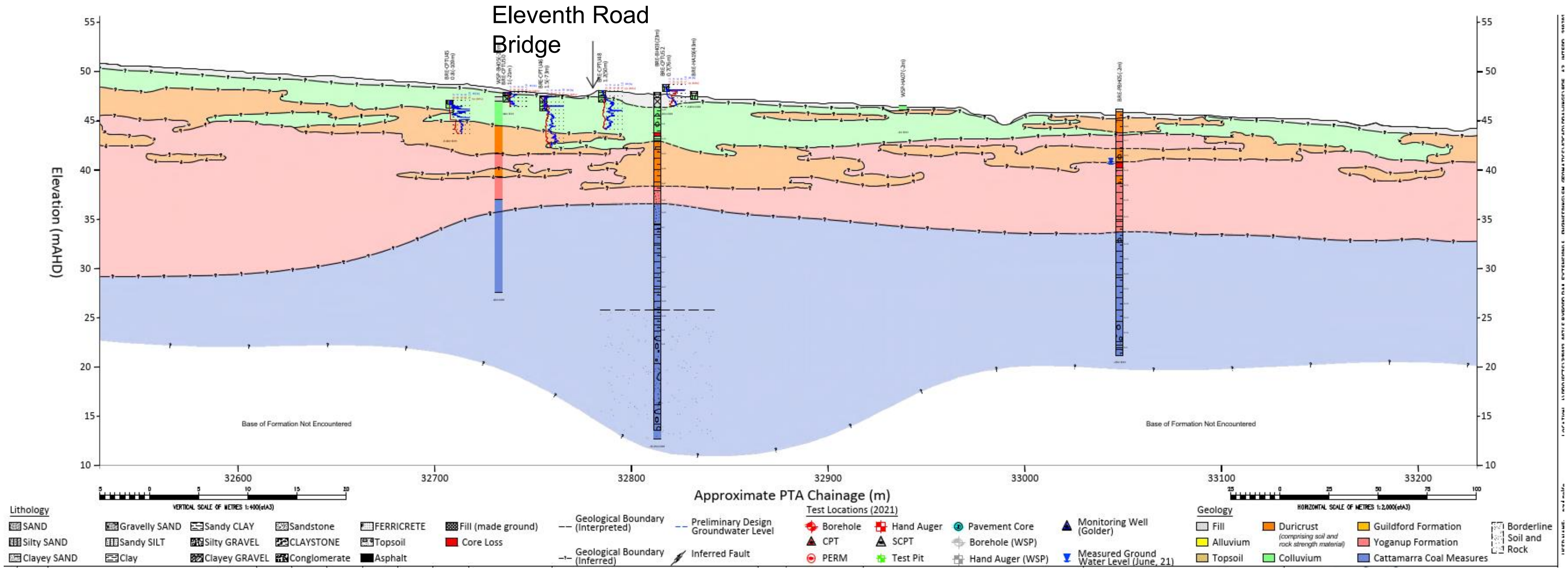


Figure A 3: Engineering geological long section

Term/Acronym	Definition
ϕ'	effective peak friction angle of soil
ϕ_g	Geotechnical strength reduction factor
γ	Bulk unit weight
ν	Poisson's ratio
API	American Petroleum Institute
f_s	Ultimate Unit friction resistance along the pile shaft
f_b	Ultimate Unit end bearing resistance of pile
k_h	Modulus of subgrade reaction of p-y curves
k_{hi}	Initial modulus of subgrade reaction of API Sand p-y curves

Pile Type

An abutment consistent with 18 No of 900 mm diameter piles as on drawings (Byford rail extension – Eleventh Road structures-Bridge Plan, elevation, and Typical sections) has been considered.

Loading

In the absence of information provided, we assumed a total vertical abutment load of

- ULS - 35 MN
- SLS - 26.25 MN

Geotechnical Design Profile and Parameters

The pile design parameters (ultimate unit end bearing f_b, ultimate unit skin friction fs) were developed based on the results of the in-situ tests.
The adopted geotechnical parameters are summarised in Table C1.

RL base of unit (m AHD)	Description	Bored Piles		γ	ϕ'	S_u	Soil Model for Horizontal Spring
		f_s (kPa)	f_b (MPa)	(kN/m^3)		(kPa)	
46.5	Fill	0	-	18	32	NA	API Sand
45	Clayey Sand	50	3	18	35	NA	API Sand
39	Sandy Clay	55	1.35	18	NA	150	API Clay
34.5	Clayey Sand	40	3	19	36	NA	API Sand

Table C 1: Geotechnical Design Profile and Parameters for Bored Piles

Geotechnical Pile Design

Axial Pile Capacity

Plots of axial pile capacities versus pile toe levels for the proposed pile type are provided in Figure C7 assuming 5% dynamic testing. Figure C8 provides pile capacities with no testing conducted.

Axial Spring Stiffness

The load-displacement curves are non-linear, and we have assumed a nominal working load under SLS to derive the approximate spring values.

At SLS, most of the vertical would be taken by the pile shaft resistance. Note that strain compatibility must be checked when modelling in ULS where the movements could be larger than the values that have been assumed.

The axial load vs axial displacement (obtained from RSPile models) curves shown in Figure C 1can be used to estimate the vertical spring based on the loads obtained in the structural model. Table C 2 summarises the vertical spring stiffness based on the assumed loads for the abutment.

Description	Kv SLS (MN/m)	Kv ULS (static) (MN/m)	Kv ULS (seismic) (MN/m)
Front Piles	13100	12900	Loads not provided
Back Piles	19400	17400	Loads not provided

Table C 2: Vertical Spring stiffness

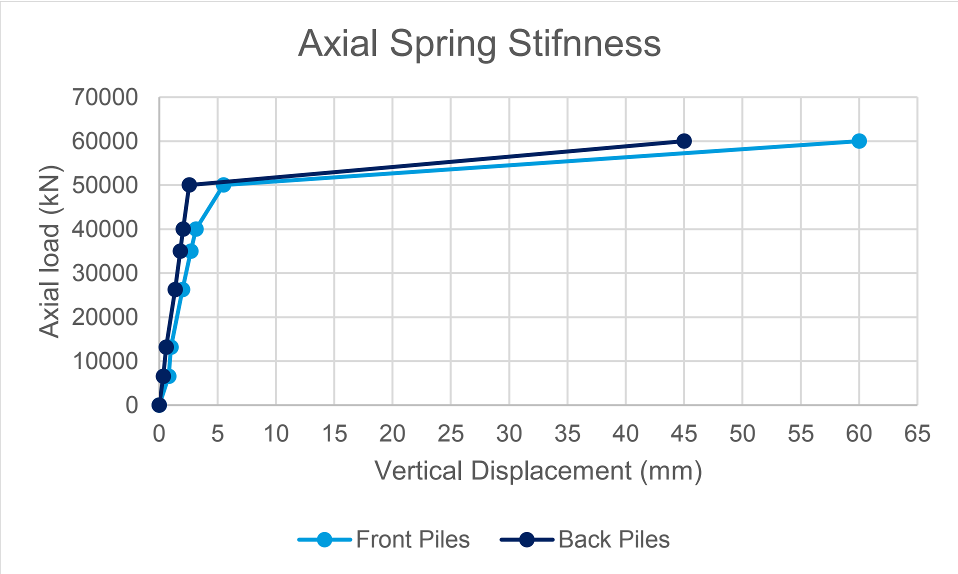


Figure C 1:Axial load versus settlement for the front and back piles

Horizontal Spring Stiffness

Soil horizontal spring stiffness and limiting spring forces as a function of depth provided Table C 4 and Table C 5will be used by the structural engineer in a model where the bending response of the individual piles within the pile group will be assessed.

The lateral displacement required to mobilise the soil capacity (i.e., limiting spring pressure of force) is generally relatively small (a few millimetres) at shallow depth and therefore, it is important that the limiting spring force is considered in the structural model. Failure to model the limiting spring capacity will result in an overestimate of the soil capacity and underestimate of the pile displacement/deformation and resulting bending moment/shear force in the pile.

The spring stiffness and limiting spring forces are provided in Table C 4 and Table C 5 for a single pile, the centre to centre pile spacing and the direction of the loading. With the p-y methodology, a p-multiplier is typically used to estimate the lateral response over the full depth of the individual piles in the pile group up to a limiting spring force.
In order to account for shadowing/group effects, group factors after Reese (Ref. 2), provided in Table C 3were applied to the spring stiffness and the limiting force provided in Table C 4 and Table C 5

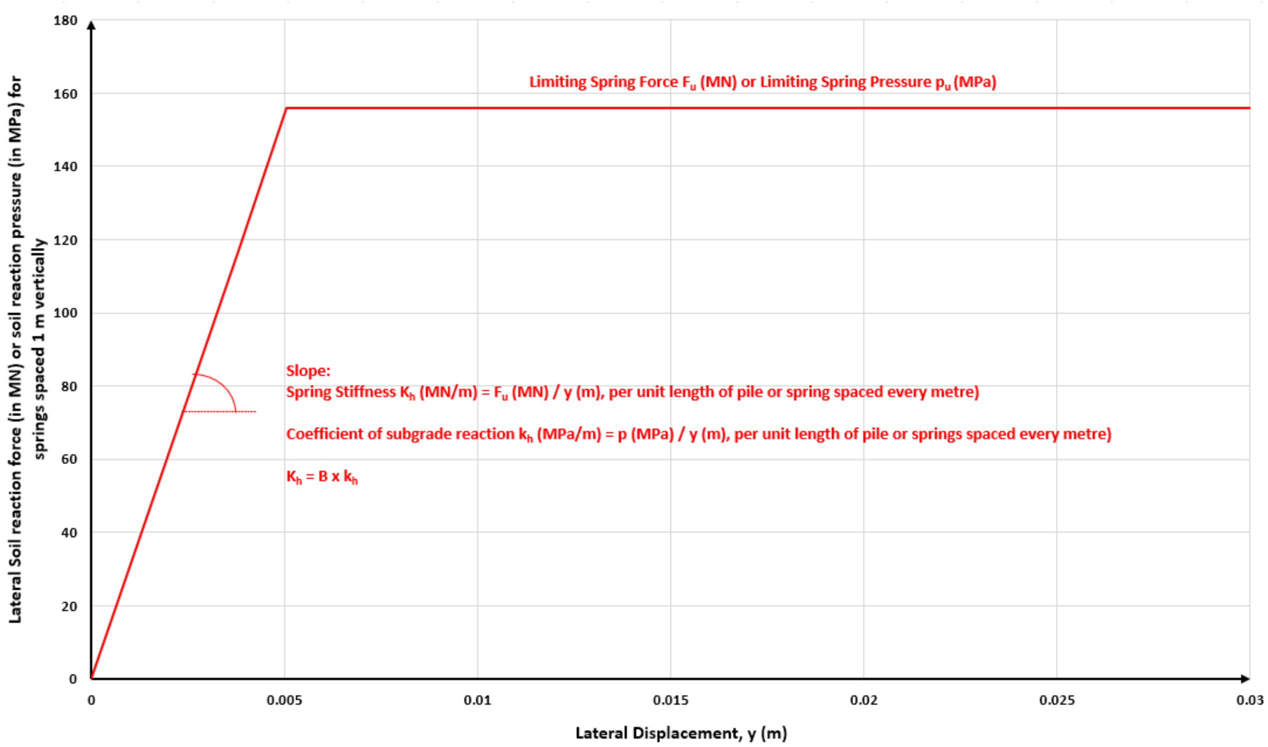
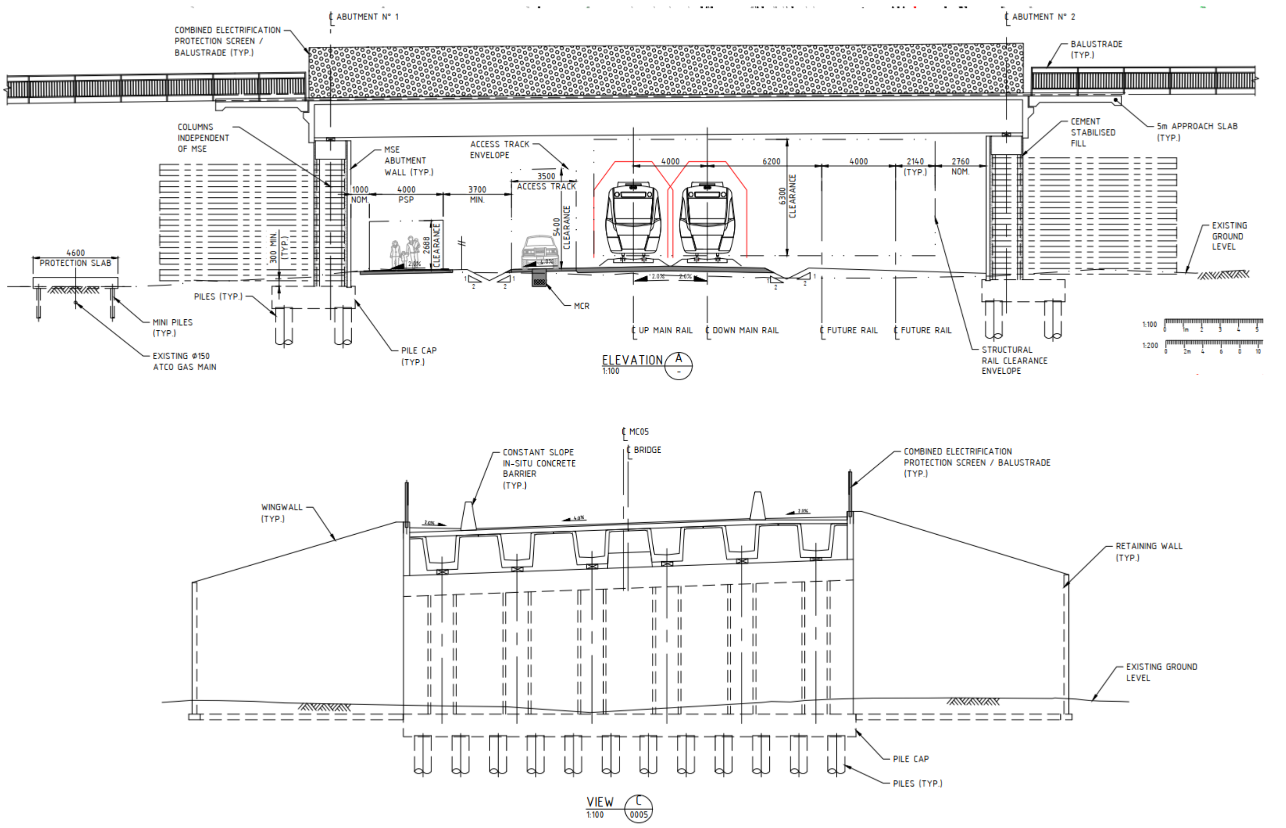


Figure C 2:Horizontal Soil Springs for Pile Lateral Analysis

Pile Group Configuration	Pile Centre to Centre Spacing
	Abutment (two rows, 900mm piles)
Transversal load	"front" piles, p=0.65 "back" piles, p=0.49

Table C 3:Pile Group Factor Applicable for Abutment Pile Layout (refer to Figure C 4)

The proposed pile arrangement is shown in Figure C3 and Figure C4.



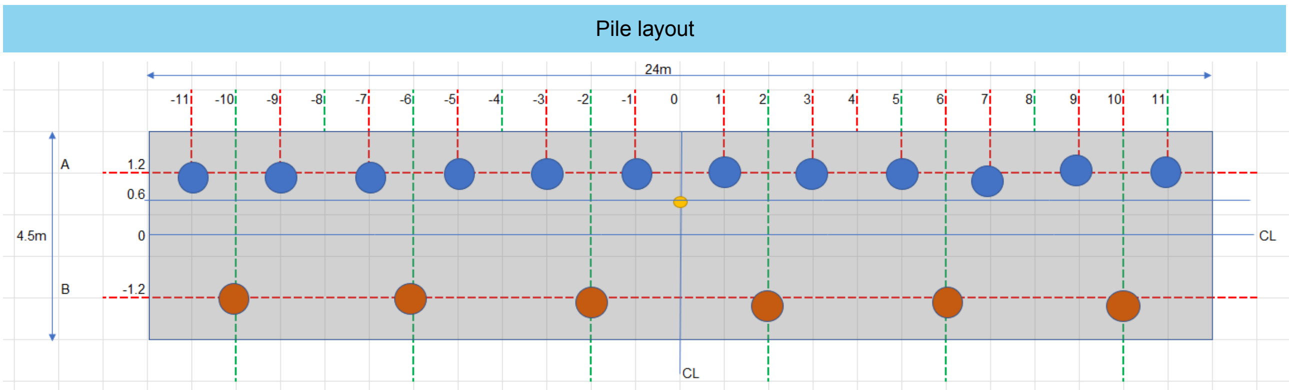


Figure C 3:Proposed Pile Arrangement for Eleventh Road Bridge

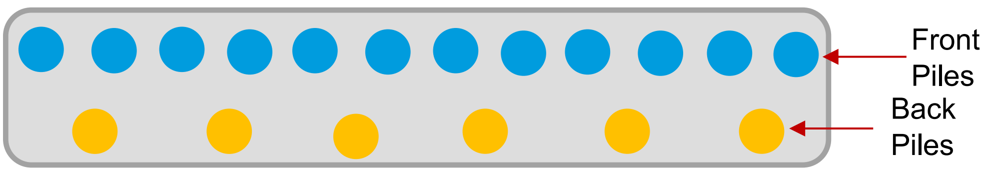


Figure C 4:Pile notation

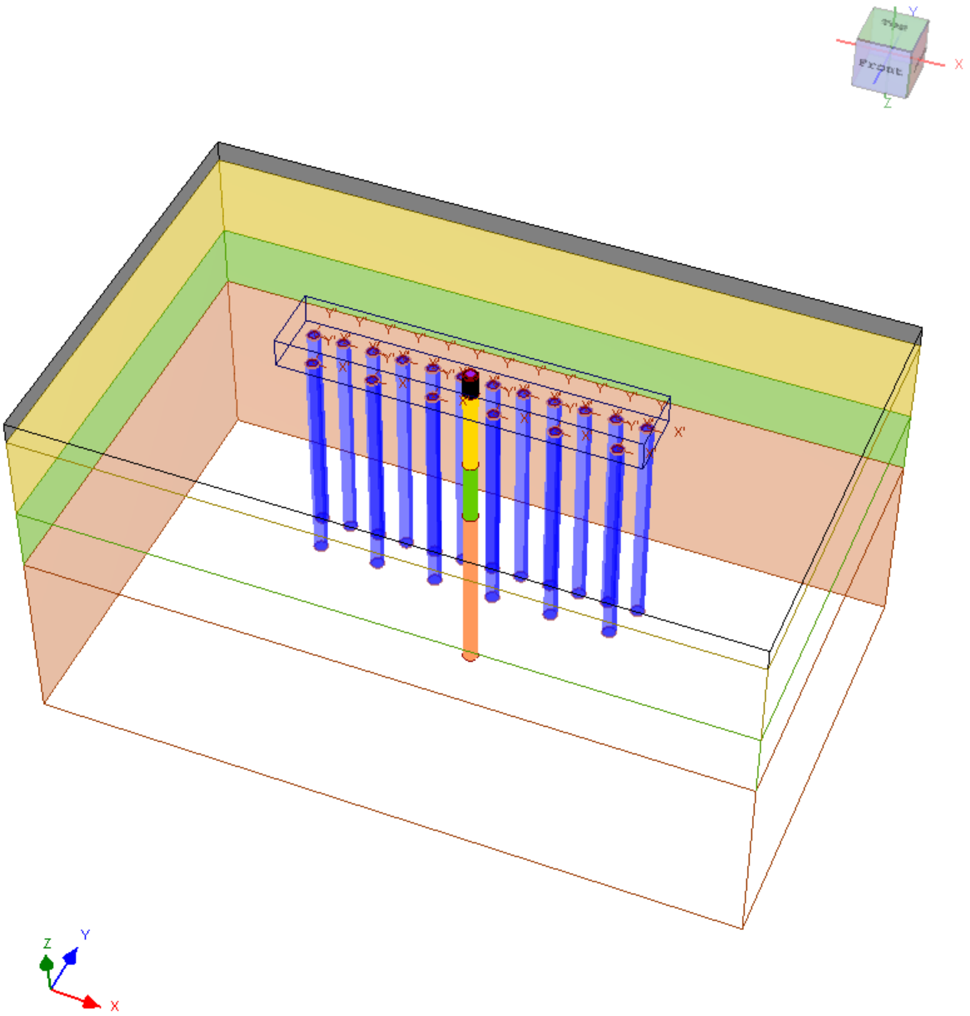


Figure C 5:RS Pile model for the pile group

Table C 4:Soil Horizontal Spring Stiffness and Limiting Spring Force for Spring Spaced 1 m Vertically (Front Piles)

Depth below pile cap (m)	**RL (m AHD)**	**Horizontal Spring Stiffness K_H (MN/m)** For springs spaced 1 m vertically	**Limiting Spring Force F_u (MN)** For springs spaced 1 m vertically
0	46	0	0
1	45	34	0.15
2	44	176	0.63
3	43	176	0.79
4	42	176	0.79
5	41	176	0.79
6	40	176	0.79
7	39	176	0.79
8	38	194	1.68
9	37	219	2.04
10	36	243	2.44
11	35	267	2.87

Table C 5:Soil Horizontal Spring Stiffness and Limiting Spring Force for Spring Spaced 1 m Vertically (Back Piles)

Depth below pile cap (m)	**RL (m AHD)**	**Horizontal Spring Stiffness K_H (MN/m)** For springs spaced 1 m vertically	**Limiting Spring Force F_u (MN)** For springs spaced 1 m vertically
0	46	0	0
1	45	34	0.1
2	44	122	0.44
3	43	122	0.55
4	42	122	0.55
5	41	122	0.55
6	40	122	0.55
7	39	122	0.55
8	38	194	1.16
9	37	219	1.41
10	36	243	1.69
11	35	267	1.98

Notes: group factor (p-multiplier) to be applied to the spring stiffness and limiting spring force (refer Table C 3:Pile Group Factor Applicable for Abutment Pile Layout (refer to Figure C 4)

The horizontal spring stiffness may be taken as two times the value in the table for transient load such as earthquake load.

The coefficient of subgrade reaction (MPa/m) = K_H/B with B the pile diameter width. Soil limiting spring pressure = F_u/B.

References

1) K. Flemming, A. Weltman, M. Randolph, K. Elson (2009), Piling Engineering, 3rd Edition

2) L.C. Reese, W.F. Van Impe (2011), Single Piles and Pile Groups Under Lateral Loads, 2nd Edition

CMW Geosciences Pty Ltd, Chapman Morton Woodward	CLIENT:	DESIGNER:	CJ
	PROJECT: PROJECT NAME	CHECKED:	OS
	PROJECT ADDRESS	REVISION:	0
	TITLE: GEOTECHNCALSTRENGTH REDUCTION FACTOR (φg)	DATE:	28/04/2022
	CALCULATION (AS 2159: 2009)	PROJECT:	MEL2022-0068

Step 1:

Redundancy System:	**High**	Note: Low = isolated heavily loaded piles, piles set out with large spacings, High = large pile groups under large pile caps, piled rafts and piled groups with more than 4 piles

Step 2: Risk Assessment

Risk Factor	Weighting Factor	Weighting Factors and Individual Risk Ratings for Risk Factors — Typical description of risk circumstances for individual risk rating (IRR): 1 (Very low risk)	3 (Moderate)	5 (Very High Risk)	Assigned Risk Rating
Site					
Geological complexity of site	2	Horizontal strata, well-defined soil and rock characteristics	Some variability over site, but without abrupt changes in rock stratigraphy	Highly variable profile or presence of karstic features or steeply dipping rock levels or faults present on site, or combinations of these	**3**
Extent of ground investigation	2	Extensive drilling investigation covering whole site to an adequate depth	Some boreholes extending at least 5 pile diameters below the base of the proposed pile foundation level	Very limited investigations with few shallow boreholes	**5**
Amount and quality of geotechnical data	2	Detailed information on strength compressibility of the main strata	CPT probes over full depth of proposed piles or boreholes confirming rock as proposed founding level for piles	Limited amount of simple in situ testing (e.g. SPT) or index tests only	**5**
Design					
Experience with similar foundations in similar geological conditions	1	Extensive	Limited	None	**1**
Method of assessment of geotechnical paramaters for design	2	Based on appropriate laboratory or in situ tests or relevant existing pile load test data	Based on site-specific correlations or on conventional laboratory or in situ testing	Based on non-site specific correlations with (for example) SPT data	**3**
Design methods adopted	1	Well-established and soundly ased method or methods	Simplified methods with well-established basis	Simple empirical methods or sophisticated methods that are not well established	**3**
Method of utilizing results of in site test data and installation data	2	Design values based on minimum measured values on piles loaded to failure	Design methods based on average values	Design values based on maximum measured values on test piles loaded up only to working load, or indirect measurements used during installation, and not calibrated to static loading tests	**5**
Installation					
Level of construction control	2	Deatiled with professional geotechnical supervision, construction processes that are well established and relatively straightforward	Limited degree of professional geotechnical involvement in supervision, conventional construction procedures	Very limited or no involvement by designer, construction processes that are not well established or complex	**2**
Level of performance monitoring of the supported structure during and after construction	0.5	Detailed measurements of movements and pile loads	Correlation of installed parameters with on-site static load test carried out in accordance with AS 2159-2009	No Monitoring	**5**

Average Risk Rating = $\sum (w_i \cdot IRR_i) / \sum w_i$ =	**3.62**
Overall risk category	**Moderate to High**
Basic Geotechnical Strength Reduction Factor, φ_{gb}	**0.53**

Testing shaft in testing, geotecl strengt service testing.

Step 3: Derivation of Test Benefit factor

In accordance with Clause 8.4.2 b), no testing is required if the geotechnical strength reduction factor is 0.4 or less. — **∴ Testing Required**

Serviceability testing is only required where the ARR is greater than 2.5. — **∴ Serv. Testing Required**

Minimum Percentage of piles requiring serviceability limit testing, if required — 3 %

Specify the Pile Testing Type:	4
1	Static Load Testing
2	Rapid Load Testing
3	Dynamic Load Testing of Preformed Piles
4	Dynamic Load Testing on Piles other than preformed
5	Bi-Directional Load Testing
6	No Testing

Dynamic or Bi-Directional

Intrinsic test factor, φ_{tf}	φ_{tf} =	**0.750**
Percentage of piles to be tested (minimum as defined above)		5 %
Testing benefit factor	k	**0.681**

$\varphi_g = \varphi_{gb} + (\varphi_{tf} - \varphi_{gb})K \geq \varphi_{gb}$	**Geotechnical Strength Reduction Factor, φg =**	**0.68**

Figure C 6:Geotechnical redundancy factor calculations for 5% Testing

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Figures

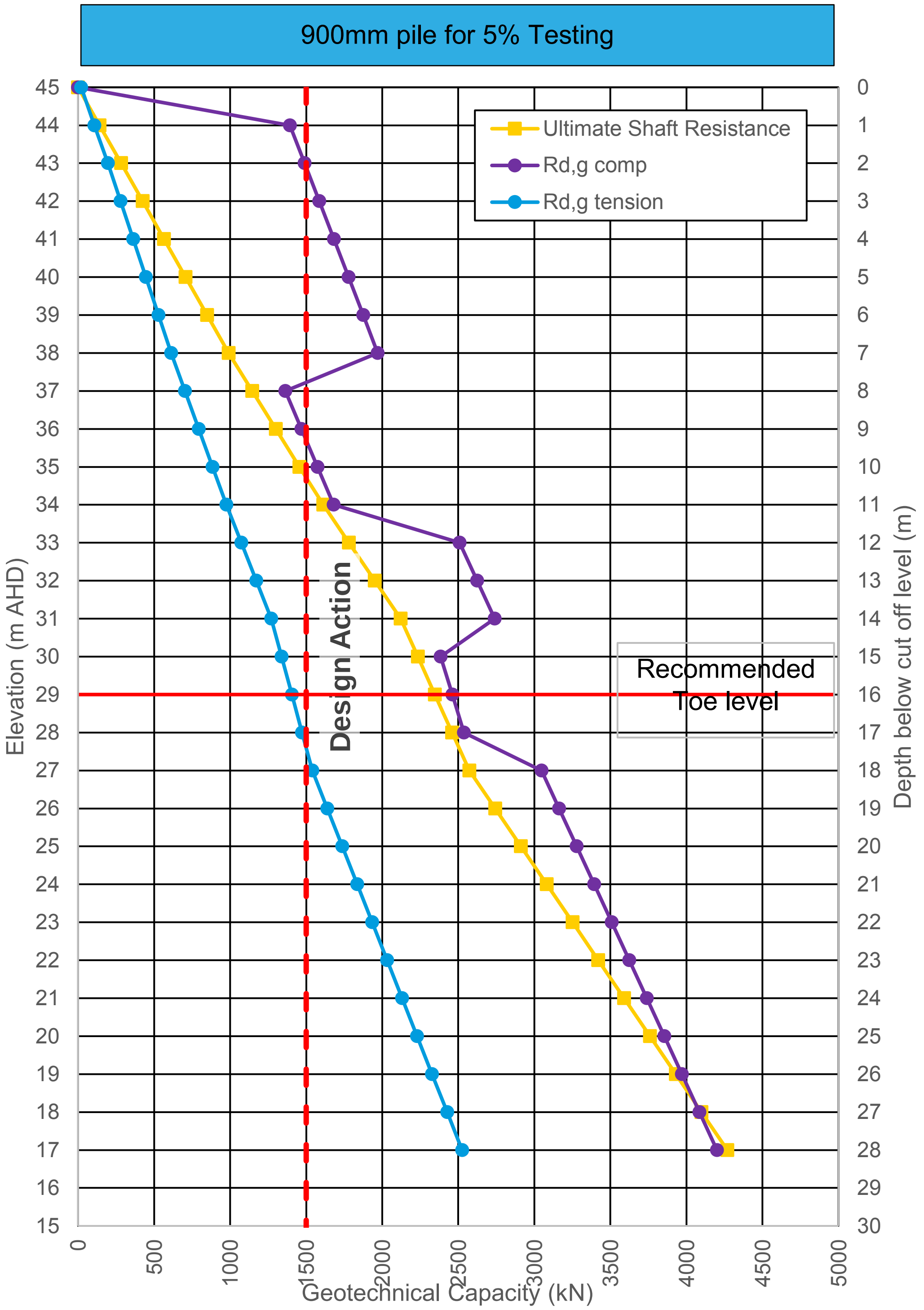

Figure C 7:Axial Pile Capacity for 5% testing (Ø_g=0.68)

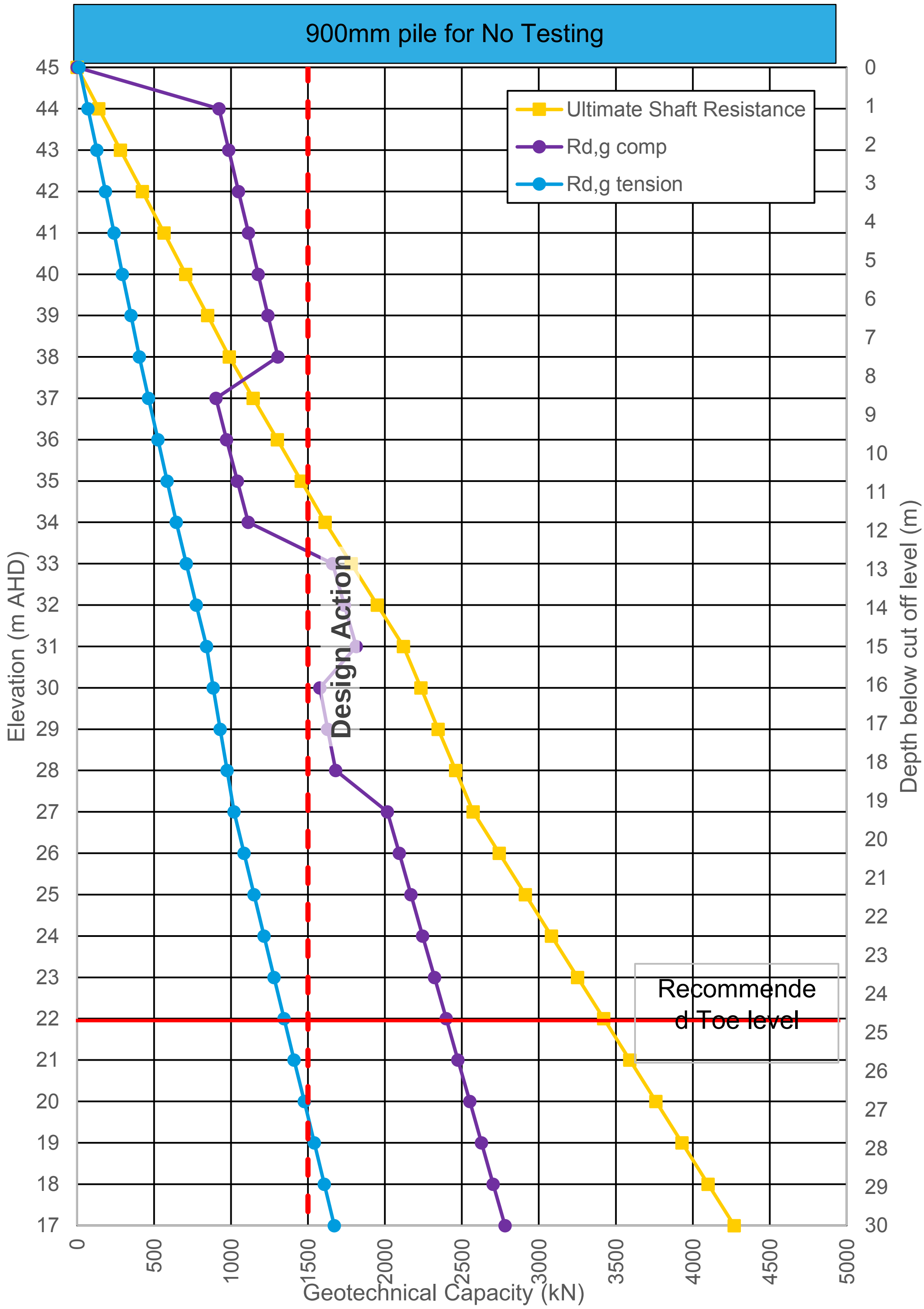

Figure C 8:Axial Pile Capacity for No testing (Ø_g=0.45)

Appendix F: Schedules (Not in Use)

Appendix G: IDC Certificates

Refer to Appendix G

Appendix H: Independent Verification Certificates (Not in Use)

Appendix I: PTA Comments Review Register (Not in Use)

Appendix J: Third Party Approvals (Not in Use)

Appendix K: RFIs (Not in Use)

Appendix L: Project Interfaces (Not in Use)

Appendix M: Departures (Not in Use)

Appendix N: Deviations (Not in Use)

Appendix O: RATM Extract

Refer to Appendix O

Appendix P: Project Hazard Log (Not in Use)

Appendix Q: Safety in Design (Not in Use)

Appendix R: Human Factors (Not in Use)

Appendix S: Reliability, Availability, Maintainability (Not in Use)

Appendix T: Durability Assessment (Not in Use)

Appendix U: Sustainability (Not in Use)

Appendix V: ITP Strategy (Not in Use)

Appendix W: Subsystem Allocation (Not in Use)

Appendix X: BCA Certificates (Not in Use)

Appendix Y: DDA Certification (Not in Use)

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