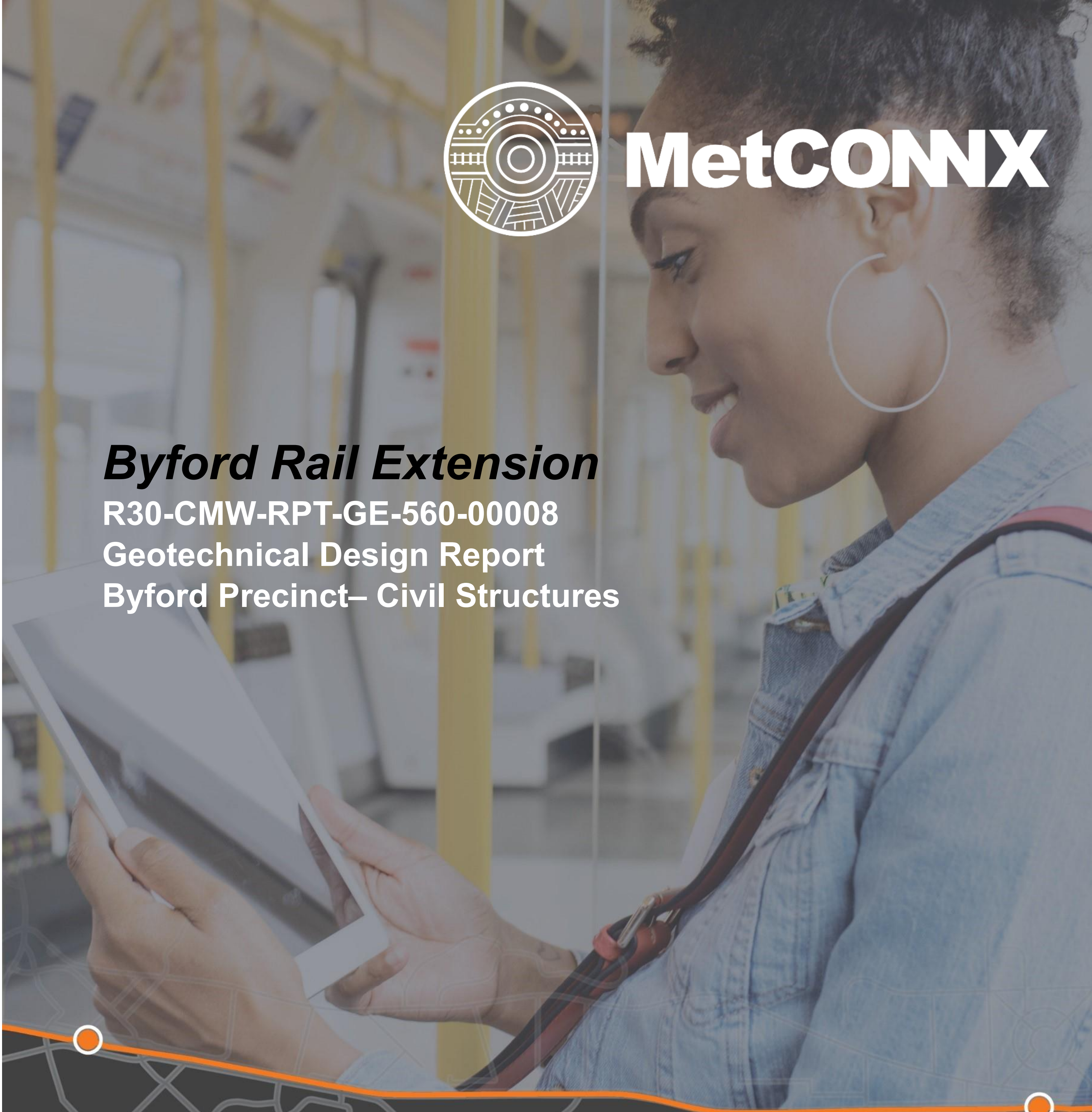
Appendix M: Geotechnical Report



Byford Rail Extension

R30-CMW-RPT-GE-560-00008
Geotechnical Design Report
Byford Precinct– Civil Structures

Connecting communities.
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METRONET

Byford Rail Extension

Document details	
Title	Geotechnical Design Report - Byford Precinct– Civil Structures
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-560-00008

Rev	Date	Revision Description	Prepared by	Reviewed by	Approved by
A.	13-Oct-2022	Issued for Reference Design	Tara Ranasinghe	Praba Pirapakaran	Alec Courts
B.02	10-May 2023	Issued for Interim Detail Design - IV	Kumara Wanayalage	Ondrej Synac	Craig Butterworth

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MetCONNX

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1. Executive Summary

This report provides the geotechnical design associated with the Byford Station Precinct.

Sections of the report common to all the geotechnical design input, including shallow footings for canopies and single-story buildings, earthworks will be progressed once the combined design is more mature. Final acceptance of this Geotechnical Design Report will only occur once all the design elements are reported on.

The ground conditions of the Byford Station Precinct are fairly consistent with Colluvium and Yoganup Formation overlying the Cattamarra Coal Measures.

The canopy structures of the station and bus interchange are supported on shallow footings. The precinct will be backfilled from up to 3m in most areas and the shallow footings will be found on the engineered fill. Allowable bearing capacity of the footings with nominal sizes are provided similar to the reference design stage in this report. Indicative soil springs are provided in this stage to assess the shallow footings by the structural engineer.

The information outlined in this Geotechnical Design Report is suitably developed to enable key details and drawings to be extracted and submitted for Integrated Digital Delivery (IDD)
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Acknowledgement of Country

MetCONNX acknowledges the Noongar people as the Traditional Custodians of the land and waters, upon which the Byford Rail Extension Project will be delivered. We pay our respects to the Noongar Elders past, present and emerging and respect 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 1: METRONET Byford Rail Extension Project

2.2.1 Project Features

Transport infrastructure works for the BRE Project include:

- Construction of a new Byford station at grade (Base Case)
- Demolition of existing station at Armadale and construction of a new elevated station
- Construction of approximately 8 km 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 2.

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

Figure 2: 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:

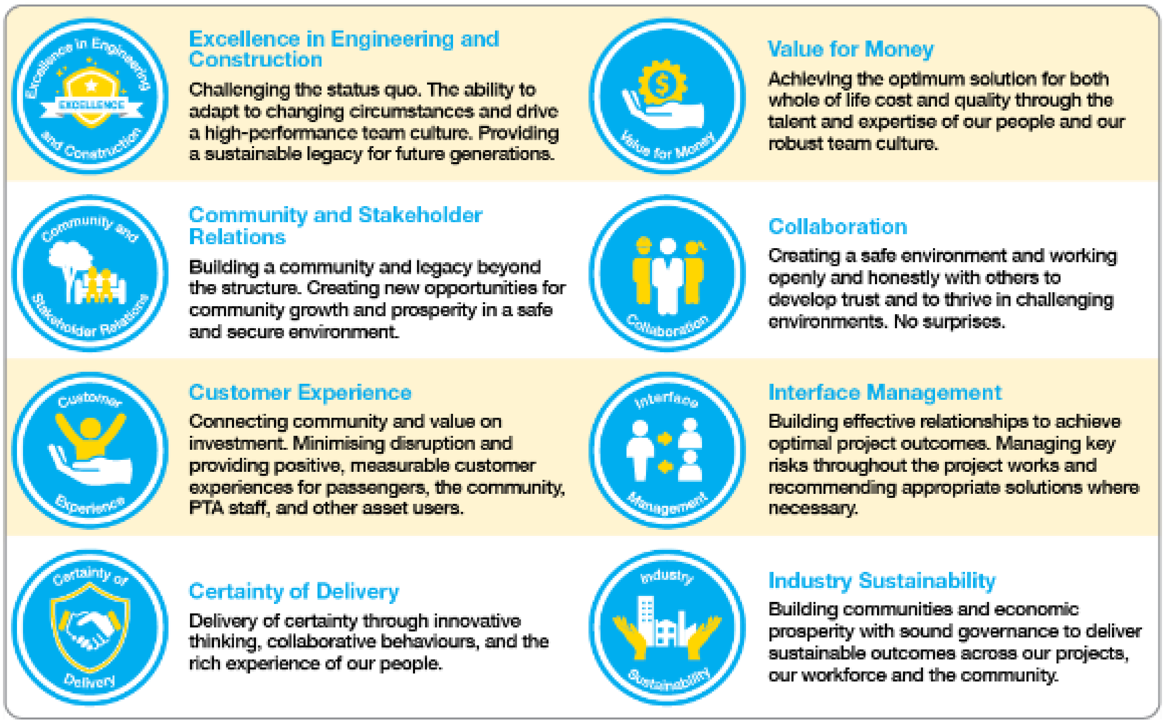


Figure 3: MetCONNX Priorities aligned with Key Project Objectives

2.4 Purpose of the Report

This Design Report presents the geotechnical design information for the Byford Station Precinct - Station Structures Design Package to support Design packages ST-335 and CI-300. 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
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
ST-335	Byford Station Structures	Geotechnical input to structures design
CI-300	Byford Civil (Earthworks, Drainage, Roads & Pavements)	Geotechnical input to civils design

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.

- Byford Station 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 shallow foundation design information for the Byford Station precinct
- Geotechnical design information for the civil design works around the Byford Station precinct
- Geotechnical design information for the proposed retaining walls around the Byford Station precinct

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 Byford Station - Station Structures Design Package to support Design packages ST-335 and CI-300.

3.3 External Interfaces

N/A

3.4 Changes Since Previous Design Submission

3.4.1 Reference Design (RD) Phase to Integrated Design Delivery Review Phase

Additional geotechnical investigation taken place after the RD stage by CMW. The ground model was revised with additional information in IDD stage. The overall Byford Station precinct has had limited design changes between RD and IDD phase with the proposed Byford Station to be supported on shallow foundations still currently proposed.

3.5 Byford Station Precinct Structure

The proposed Byford Station is a single-level structure with an island platform to access the up and down main railway lines and a dedicated platform for the Australind line. Also, the development comprised of a canopy structures for the Byford bus interchange, car park and local road upgrades. Byford station, bus interchange, car park and associated local roads upgrade include earthworks with majority of fill and some areas to cut.

Structural columns, canopies and stair bases are currently proposed to be supported on shallow foundations. The approximate extent of the station, bus interchange, car park and upgrading local roads is shown on the reference design civil and structural drawings. Indicative soil springs are provided to assess the shallow footings by the structural engineer.

3.6 Retaining Walls

If any retaining walls are proposed, the design will be included in a separate design package. General Geotechnical design information has been provided in the present report.

4. Design Inputs

4.1 Project Design Requirements

Design and drawings for the Byford Station Precinct structure were in progress and were supplied with limited information for this reference design stage Geotechnical Design submission. The reference design civil drawings that indicate the extent of the proposed Byford Station structure have been used to approximate PTA chainages and reference with the investigation locations completed to date. Reference should be made to the main design package for the latest civil and structural drawings. A full set of 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:

- Proposed Byford Station and Bus port structural columns, walls and stair bases to be supported on pad and strip footings

- The proposed Byford Station concourse slab is to be at RL 54.405 m AHD

4.2 Design Software used for this Package

In-house design calculation spreadsheets have been used for this 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
AS1170.0	2002	Structural design actions: General Principles
AS1170.4	2007	Structural design actions: Earthquake Actions in Australia
AS4678	2002	Earth retaining structures
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

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)	1
BRE-ADV-GE-RPT-00005	Geotechnical Interpretative Report, Advisian (6 October 2021)	0
BRE-MNO-WSP-GE-RPT-0001	Geotechnical Factual and Interpretive Report, WSP	A
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	-
R30-CMW-RPT-GE-560-00001	Geotechnical Investigation Factual Report	A
11-A-109-C10001 to 11-A-109-C10043	Byford Station Precinct Civil Works Reference Design Drawings	A

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 8880-450-059-Rev1 (Specification: Buildings and Station Structures) and 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 2% 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	Byford Station	100 years (1), 50 years (2), 120 years (3)

Notes to Table 6:

- (1) Design Life for the considerations of structural design actions on structures
- (2) Service life for secondary structural elements. Classification on primary and secondary structural elements shall refer to Table 8 in 8880-450-010.
- (3) Design life for durability design and considerations on primary structural elements. Classification on primary and secondary structural elements shall refer to Table 8 in 8880-450-010.

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 Integrated Design Delivery Review stage.

4.12 Value Engineering

No input provided at Integrated Design Delivery Review 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)

Refer to Project Design Requirements Section 4.1.

4.15.1 Design Assumptions

Refer to Project Design Requirements Section 4.1.

4.15.2 Design Dependencies

Refer to Project Design Requirements Section 4.1.

4.15.3 Design Constraints

Refer to Project Design Requirements Section 4.1.

4.16 Requests for Information (RFI)

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

RFIs from civil teams to be includedRFI

5. Design Outputs

5.1 Design Reviews and Ce Deliverables List

N/A

5.2 Specifications

See Geotechnical Design Advice and Calculations Section 5.8 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

All currently available geological/geotechnical information specifically related to the Byford Station Precinct, including the geotechnical information contained in the reports listed in Table 3 have now been reviewed in detail. These information sources are detailed in Table 7 below.

Table 7 – Sources of information relevant for the design.

Information	Source Document	Reference
Cone Penetration Test logs BRE-CPT85A, BRE-CPT86, and BRE-CPT88 Borehole Logs BRE-ABH11, BRE-PBH07 and associated in-situ and laboratory datasets	Byford Rail Extension Project, Geotechnical Investigation Factual Report Ref. BRE-ADV-GE-RPT-00004 dated 18 October 2021.	Advisian (2021)
Borehole log for BH06, BH07 and associated in-situ and laboratory datasets	Byford Rail Extension – Geotechnical Factual and Interpretive Report, ref. PS121075-GEO-REP-0001 Rev A, dated 1 February 2021	WSP (2021)
Borehole log for, BH81, BH82, and BH83. Borehole logs and Cone Penetration Test logs for BH+CPT81.	Geotechnical Investigation Factual Report, Ref. R30-CMW-RPT-GE-560-00001	CMW (2022)

The exploratory holes data specific to the Byford Station Precinct has been extracted from the source documents listed in

Table 7 above and has been summarised in Table 8 below.

Table 8 – Exploratory tests and holes used to inform the design.

Exploratory Hole No.	Type	GI/Date	Surface Elevation (mAHD)	Depth (mBGL)	Scheme Chainage (m Approx.)	Coordinate Reference (m Approx.)
BRE-CPT85A	Electric Friction Cone Penetrometer	Advisian 04/03/2021	51.60	3.4	36,149	67934E 234138N
BRE-CPT86	Electric Friction Cone Penetrometer	Advisian 04/03/2021	52.19	3.24	36,175	67965E 2341111N
BRE-CPT88	Electric Friction Cone Penetrometer	Advisian 04/03/2021	52.81	3.46	36,248	67960E 234036N
BRE-ABH11	Rotary Cored Borehole	Advisian 14/04/2021	55.00	15.0	36,304	68026E 233982N
BRE-PBH07	Rotary Cored Borehole	Advisian 12/04/2021	52.60	35.5	36,235	67964E 234050N
BH06	Rotary Cored Borehole	WSP 15/12/2020	51.64	15.45	36,126	67963E 234158N

Exploratory Hole No.	Type	GI/Date	Surface Elevation (mAHD)	Depth (mBGL)	Scheme Chainage (m Approx.)	Coordinate Reference (m Approx.)
BH07	Rotary Cored Borehole	WSP 16/12/2020	53.04	15.45	36,303	67957E 233982N
BH81	Rotary Cored Borehole	CMW 04/11/2022	51.44	10.00	36,040	67985E 334244N
BH82	Rotary Cored Borehole	CMW 03/11/2022	51.79	10.00	36,151	67986E 334132N
BH83	Rotary Cored Borehole	CMW 02/11/2022	52.63	10.00	36,253	67987E 334031N
BH+CPT81	Rotary Cored Borehole	CMW 07/11/2022	51.19	33.50	36,096	67986E 334188N

The exploratory hole locations presented in Table 8 above are shown in Appendix E Figure 1.

All available in-situ and laboratory testing data, which has been extracted from the exploratory holes listed in Table 8 above has been reviewed and interpreted in the following sections.

5.6.2 In Situ and Laboratory Testing

The in situ geotechnical testing datasets (CPT and SPT) relevant to the Byford Station Precinct are summarised in Table 9 below.

Table 9– In-situ tests used to inform the design.

Exploratory Hole No.	Cone Penetration Test (CPT)	Standard Penetration Test (SPT)	Comments
	No. Tests	No. Tests	
BRE-CPT85A	1	-	Performed on Colluvium CLAY
BRE-CPT86	1	-	Performed on Colluvium CLAY
BRE-CPT88	1	-	Performed on Colluvium CLAY
BRE-ABH11	1	-	Performed on Colluvium CLAY
BRE-PBH07	1	-	Performed on Colluvium CLAY
BH06	-	10	Performed on Colluvium CLAY, Yoganup CLAY and Yoganup SAND
BH07	-	17	Performed on Colluvium CLAY, Yoganup CLAY and Yoganup SAND
BH81	-	6	Performed on Colluvium CLAY and Yoganup
BH82	-	6	Performed on Colluvium CLAY and Yoganup
BH83	-	6	Performed on Colluvium CLAY and Yoganup

Exploratory Hole No.	Cone Penetration Test (CPT)	Standard Penetration Test (SPT)	Comments
	No. Tests	No. Tests	
BH+CPT81	-	21	Performed on Colluvium CLAY, Yoganup CLAY, Yoganup SAND and Cattamarra CLAY and Conglomerate

Geotechnical laboratory soil testing has been undertaken on samples recovered from some of the exploratory holes listed in Table 9 above.

The geotechnical laboratory soil testing datasets relevant to the Eleventh Road bridge are summarised in Table 13 below.

Table 10 - Laboratory tests used to inform the detailed design.

Exploratory Hole No.	Atterberg test	Moisture content	PSD	Sulfate	Comments
	No. Tests	No. Tests	No. Tests	No. Tests	
BH06	3	3	4		Performed on Colluvium CLAY, Yoganup CLAY, Yoganup SAND and Cattamarra CLAY
BH07	2	2	3		Performed on Colluvium CLAY, Yoganup CLAY, Yoganup SAND and Cattamarra CLAY
BH81	1	-	1		Performed on Yoganup SAND
BH+CPT81	5		5	4	Performed on Colluvium CLAY, Yoganup CLAY, Yoganup SAND and Cattamarra CLAY

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 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 4 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.

Bus Interchange area is 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.

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).

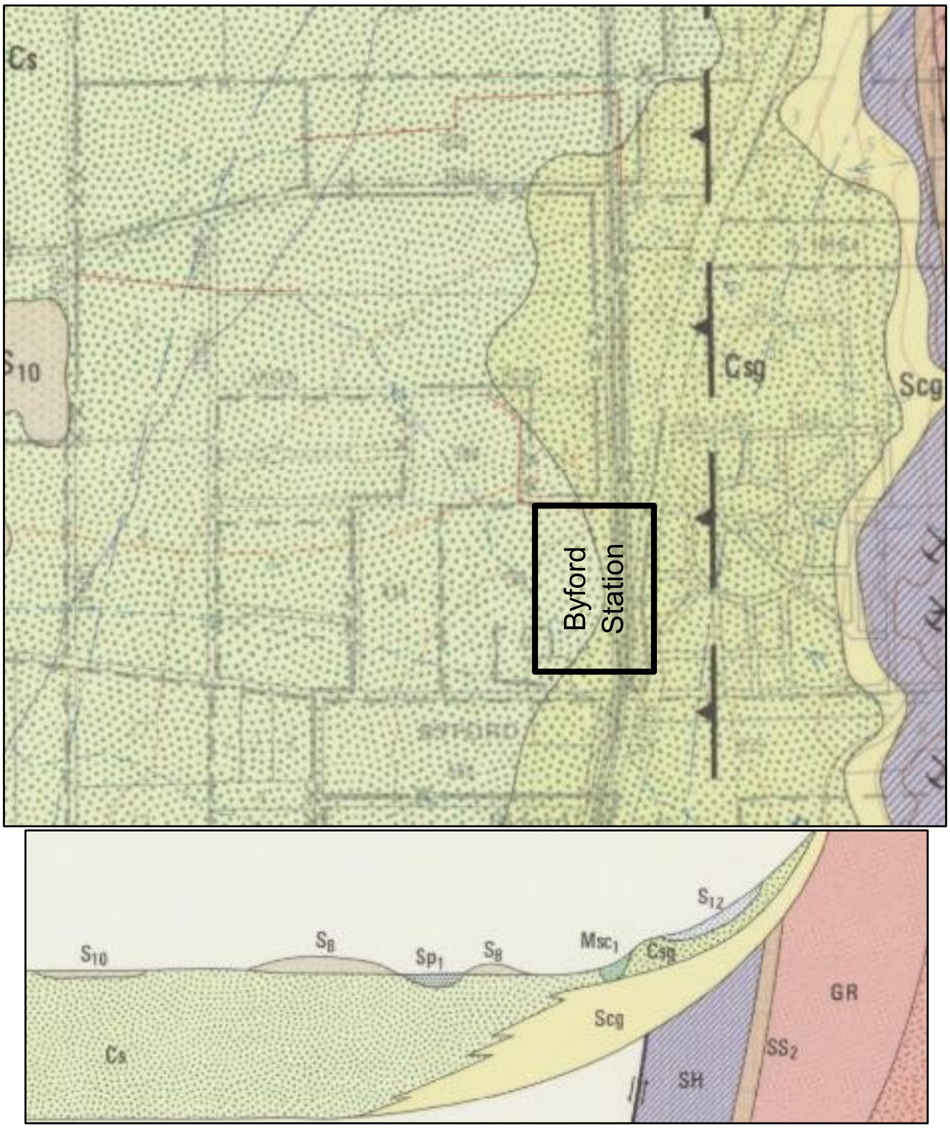


Figure 4 Geological Profile

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 flows approximately 3 km north of the Byford station precinct site while Beenyup Brook flows approximately 1km south of the site. 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 Byford station precinct 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.

5.6.4 Subsurface Conditions

The subsurface profile presented in Table 11 has been adopted as the ground model for detailed design geotechnical advice at Byford station.

The inferred generalised stratigraphy at the Byford Station precinct has been prepared based on the descriptions provided in Geotechnical Factual and Interpretive reports by Advisian, WSP and CMW and a review of the boreholes in proximity, i.e., WSP_BH06, WSP_BH07, and Advisian PBH07, ABH11 and CPTu84, 85, 86, 87, 88, 89 and 91 and CMW BH+CPT81, BH81, BH82 and BH83.

The ground conditions are interpreted to comprise a superficial layer of colluvial clay (Colluvium Sandy CLAY) overlying the Yoganup Formation. The Yoganup Formation includes a layers of Clayey SAND and CLAY overlying a layer of Clayey SAND. The Yoganup Formation is underlain by the Cattamarra Coal Measures.

Colluvium

The Colluvial soils present along the entire BRE alignment that comprises a mixture of sandy clays, clayey sands and clayey gravels.

In proximity of the Byford Station, the Colluvium comprises mainly stiff to very stiff Sandy CLAY. The average SPT N_{60} value of 20 has been recorded for Sandy CLAY layer. Clay is medium plasticity. The sand is fine to coarse grained, sub-angular to sub-rounded of quartz minor lithics. Gravel is fine to medium grained, angular to rounded. The clay has average plasticity index value (PI = 19.5%) which has been recorded in borehole WSP-BH06, WSP-BH07,BRE-ABH11 and CMW-BH+CPT81 between 1.5m and 3.5 mBGL. Occasional layers of fine to medium grained Clayey SAND have also been encountered.

The Colluvium layer appears to be persistent throughout the area of the Byford Station with the average total thickness of 3.5 m.

Yoganup Formation

The Yoganup Formation was encountered along the BRE alignment as a relatively uniform unit, which comprises layers of sandy CLAY and layers of SAND with CLAY and clayey SAND, with a subordinate gravel component.

Yoganup Formation – Clayey SAND

In proximity of the Byford station, the upper portion of the Yoganup Formation comprises mainly dense Clayey SAND. Majority of the SPT tests conducted in this layer were refused, apparently on gravels. The sand is fine to coarse grained, sub-angular to sub-rounded of quartz minor lithics. Gravel is fine to coarse grained, angular to sub-rounded. The clay has plasticity index value (PI = 30%), which has been recorded in borehole BH81.

The CLAY layer appears to be persistent throughout the station area and reduce the thickness in the southern side of the station. Ferricrete gravels presents in places of this unit throughout the station area. The layer has an average thickness of 4.5m.

Yoganup Formation - CLAY

The middle portion of the Yoganup Formation around Byford station area comprises mainly very stiff to hard CLAY. Clay is medium to high plasticity. The average SPT $(N_1)_{60}$ value of 30 to 40 has been estimated. The clay has a high average index value (PI = 32%), which has been recorded in boreholes BH+CPT81, WSP-BH06, WSP-BH07 and BRE-ABH11.

The clay layer appears underneath the colluvium layer to the south of the station area. Average thickness within the station area is 2.5m while the layer is around 8m thick towards south where overlying Clayey SAND is absent.

Yoganup Formation – Clayey SAND (lower)

The bottom aprt of the Yoganup Formation around Byford station area comprises mainly dense Clayey SAND. The sand is fine to coarse grained, sub-angular to sub-rounded of quartz minor lithics, clay is medium to high plasticity. The average SPT $(N_1)_{60}$ value ranges from 30 to 60 with some refusals. The layer is up to 7m thick which overlies the Cattamarra Coal Measures.

Cattamarra Coal Measures

The Cattamarra Coal Measures along the BRE alignment shows that the original sediments were deposited within highly variable energy conditions, from low-energy depositional sediments comprising sandy, organic-rich silt and clay within estuarine to lagoonal environments; moderate-energy depositional environments comprising dominantly sands/sandstone with subordinate gravels and preserved cross-bedding indicative of fluvial deposition; and high-energy depositional conditions comprising gravel, cobble and boulder sized conglomerate and Conglomeratic Sandstone.

Two boreholes (BRE-PBH07 and CMW-BH+CPT81) have penetrated this unit within the Byford station area. The extremely weathered upper part of the Cattamarra coal measures comprises mainly medium dense Clayey SAND associated with moderate-energy depositional environments. The average SPT N_{60} value of 15 has been estimated. The sand is fine to coarse grained, sub-angular to sub-rounded of quartz minor lithics. Clay is low to medium plasticity.

Table 11 – Byford Station - Generalised Stratigraphy

Unit Description	Approximate Elevation (mAHD)		Thickness (m)	Typical Geotechnical Description
	From	To		
Colluvium –Sandy CLAY	52.5	49.0		**Stiff to Very Stiff Sandy Clay**. Low to medium plasticity clay. Sand is fine to coarse grained, sub-angular to sub-rounded. Gravel is fine to medium grained, angular to sub-rounded.
Yoganup Formation - Clayey SAND	49.0	44.5		**Dense Clayey Sand**. Sand is fine to coarse grained, sub-angular to sub-rounded of quartz minor lithics. Gravel is fine to coarse grained, angular to sub-rounded. Clay is medium plasticity
Yoganup Formation - CLAY	44.5	42.0		**Very Stiff to Hard Clay**, medium to high plasticity clay.
Yoganup Formation - Clayey SAND	42.0	35.0		**Dense Clayey Sand**, Sand is fine to coarse grained, sub-angular to sub-rounded of quartz minor lithics, clay is medium to high plasticity.
Cattamarra Coal Measures - Clayey SAND	35.0	-		**Medium Dense Clayey Sand**, The sand is fine to coarse grained, sub-angular to sub-rounded of quartz minor lithics. Clay is low to medium plasticity

5.6.5 Geotechnical Parameter Assessment

The geotechnical parameter values derived for the units identified in the design ground model presented in section 5.6.4 of this report were selected based upon the available, location-specific log descriptions, in situ SPT results, laboratory testing datasets, and engineering judgement.

Figure 5 presents the SPT N values resulted in geotechnical investigations of the sub-surface units. The average SPT N value of each unit is plotted in red which was used to derive design parameters. Refused SPTs and SPT N values over 60 were plotted as SPT N 60. The top portion of Yoganup formation consist of ferricrete gravels that may have caused SPT tests to refuse in most places. The average SPT N 40 and 32 assumed for the Yoganup Formation Clayey SAND and CLAY units respectively.

Figure 6 plots the plasticity index against the liquid limit to categorise the plasticity of the clays. The plot indicate colluvium clays are low to medium plasticity while Yoganup clays are medium to high plasticity. Two tests done on Cattamarra coal measures show low plasticity clays.

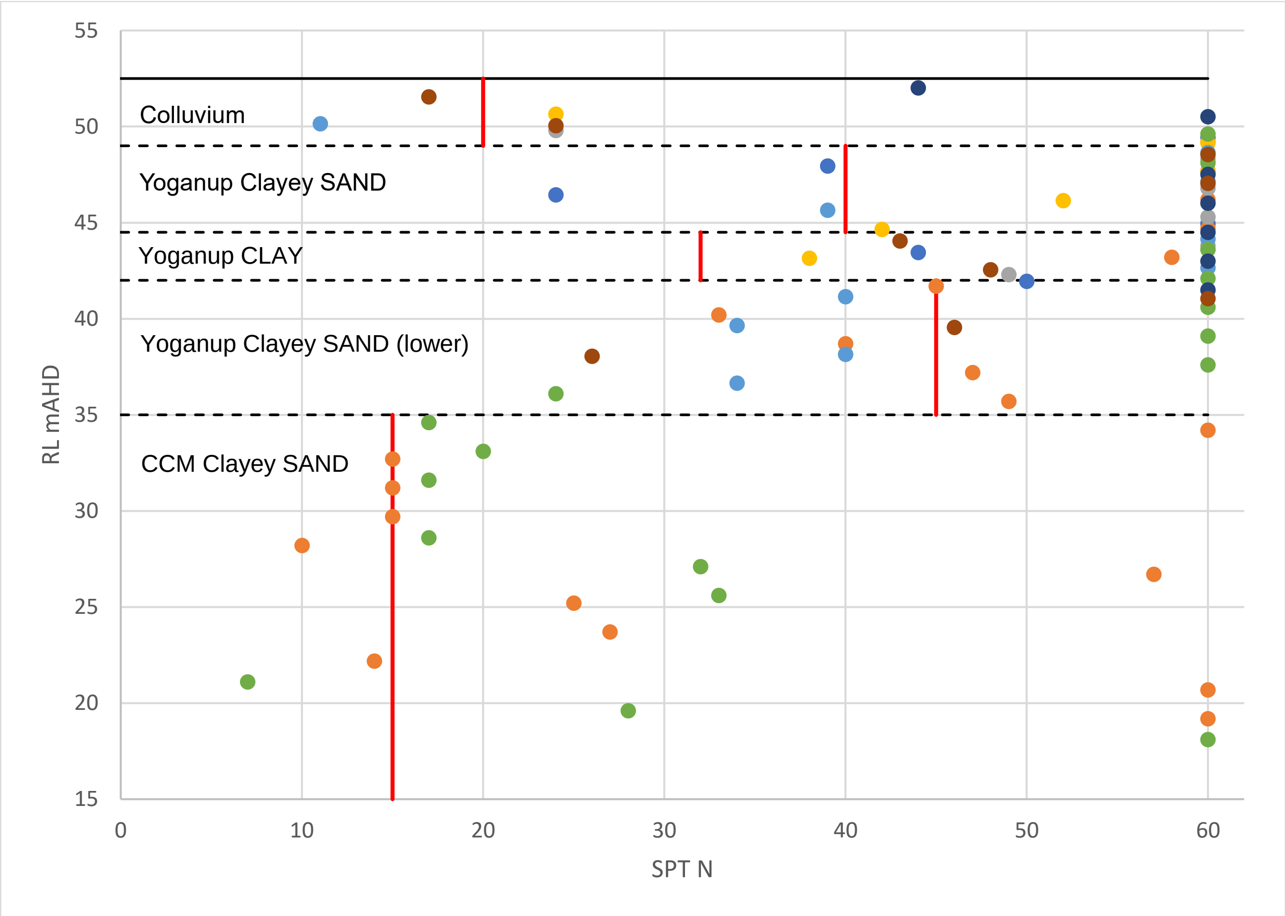


Figure 5 SPT N results of the sub-surface units

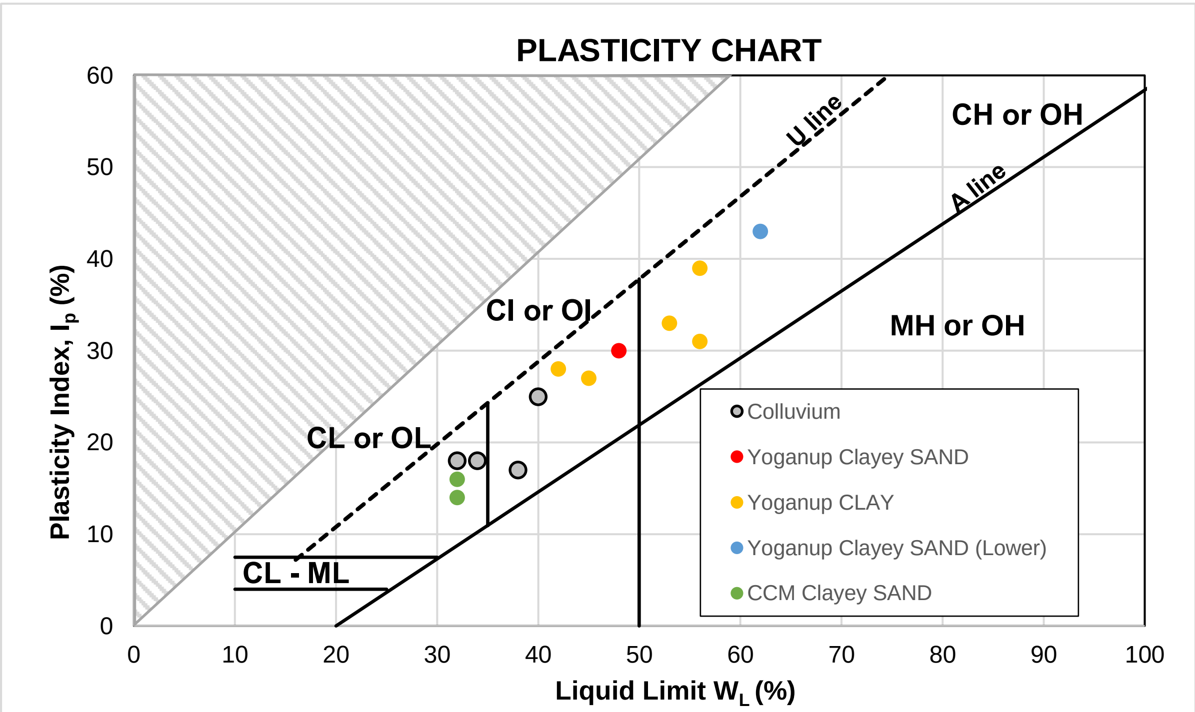


Figure 6 Plasticity chart for the sub-surface units

Undrained Shear Strength

Values of undrained shear strength (Cu or Su) for fine grained soils has been assessed based on the results of the in-situ SPT testing of the boreholes. Experience-based judgement should be used to assess Cu from insitu testing. The ranges and adopted undrained shear strengths are based on in situ testing for different soil consistency, according to AS1726-2017: Geotechnical site investigations. Following correlations of SPT test results were used for soil consistencies.

Table 12 – Soil Consistency Correlations for Fine Grained Soils

SPT-N	Consistency	Undrained Shear Strength (Cu)
0 – 2	Very Soft	0 – 12
2 – 4	Soft	12 – 25
4 – 8	Firm	25 – 50
8 – 16	Stiff	50 – 100
16 - 32	Very Stiff	100 – 200
>32	Hard	>200

Young's Modulus

The drained Young's modulus (vertical direction), E_v' has been assessed based on the following relationships based on Clayton C.R.I. (1995):

- For fine grained soils: E_v' = 300 x Cu kPa (Jardine et al, 1985)
- For coarse grained soils: E_v' = 2.0 x N_{60} MPa (Clayton C.R.I., 1995)

where: E_v' = Young's modulus (vertical direction)

C_u = undrained shear strength

N_{60} = corrected SPT N value as per standard requirements

The relationship between undrained Young's Modulus (Eu) and drained Young's Modulus can be estimated assuming the material is isotropic and related by E'/Eu = (1+v')/(1+ v_u) where v' and v_u are the drained and undrained Poisson's ratio, respectively. Therefore, E' = 0.8Eu for an assumed v' of 0.2 and an assumed v_u of 0.5.

Summary of Geotechnical Parameters Assessment

Geotechnical design parameters for the engineering geological units summarised in Table 11 are presented in Table 13. The published data and other project experiences have been used for the following parameters:

- Unit weight
- Effective cohesion, c'
- Effective Friction Angle, ϕ'

Table 13 – Byford Station – Preliminary Geotechnical Design Parameters

Unit	Ɣ (kN/m³)	c' (kPa)	φ' (°)	Su (kPa)	E' (MPa)	Eu (MPa)	v
Colluvium –Sandy CLAY	18	3	26	100	28	35	0.3
Yoganup Formation - Clayey SAND	18	3	36	-	80	100	0.3
Yoganup Formation - CLAY	18	10	28	150	50	62.5	0.3
Yoganup Formation - Clayey SAND	18	0	35	-	80	100	0.3
Cattamarra Coal Measures - Clayey SAND	18	0	31	-	30	37.5	0.3

5.6.6 Design Groundwater Level

As per the information provided in Golder Hydrogeological report, the estimated maximum GWL is 48.8 m AHD. However, the Design ground water level of 45 m AHD has been recommended by WSP_BRE Geotechnical Factual and Interpretive Report, 2021.

It should be noted that these levels do not account for perched groundwater, which is a risk on the shallow fill, Yoganup formation and Colluvium geology, which frequently host shallow clay layers, indurated horizons (coffee rock) and duricrust. WSP (2021) and Golder (2021) advised to expect perched groundwater particularly around Byford Station.

WSP (2021) also states that data loggers were installed in all monitoring wells, however the groundwater monitoring report shows that data loggers are only in the deeper wells (MW01 to MW07), but not the shallow dry wells (MW01a to MW07a). These latter wells were installed to check for perched water. Therefore, no logging/monitoring of perched water is currently being conducted.

A key issue in this regard, although not discussed in either WSP or Golder reports, is the comparatively high annual rainfall which exceeds 900 mm/annum. Combined with the permeable surface soils, significant and rapid water level response is possible in shallow perched aquifers above the clay layers or indurated horizons. However, no data is available to characterize this.

Other Geotech reports (not provided as reference information by PTA) in the vicinity also identify and document perched water risks, for example:

- Arup 2015 – Byford Sec College:
  - Investigation found perched water 0.8 mBGL to 1.2 mBGL.
- Structerre 2010 – Town Centre Geotech Report:
  - Recommended design groundwater level was 0.0 mBGL to 0.5 mBGL.

Based on the above information, as a potential solution to mitigate the risk of perched groundwater, a shallow groundwater level has been assumed in design structures. As such, a likely conservative shallow design groundwater level at 0.75 mBGL has been considered for the design of shallow foundations. Recent CMW boreholes drilled within the Byford station precinct did not encounter any groundwater. Design groundwater levels will be confirmed in the next stage of design with the results of currently on-going geotechnical investigation.

5.6.7 AS2870 Site Classification

Site classification used, primarily in residential development, for quantifying the anticipated ground movements that may occur on a site principally due to soil reactivity. The site has been classified in accordance with AS 2870-2011 “Residential Slabs and Footings” to give an indication of the potential performance of shallow footings.

The depth of suction change Hs has been taken as 1.8 m based on AS2870 (2011) and in accordance with local practice, with a design suction change at ground surface of pF 1.2 and crack depth of 0.9 m.

The soils encountered were generally Stiff to Very Stiff Sandy Clay and the average plasticity index from the Atterberg test results within the Hs depth is 18%. This results in instability index of 2.5 based on BS5930:1999.

On this basis, a characteristic surface movement of ys of 54mm is assessed. The shrink/swell potential is generally considered to be high, the equivalent of a Class H1 site as defined in AS2870 – 2011. This site classification provides a guide to the level of surface movement due to seasonal moisture changes that could expected on the site. Further test pit investigations are planned for this site to confirm this assessment.

Larger ys values may occur when the future moisture content change in the soil exceeds design moisture content changes as determined from AS2870. Such changes may occur, for example, adjacent to leaking water services or where the soils are desiccated by the roots of trees.

5.6.8 AS1170 Hazard Factor and Site Sub-Class

Based on the general geology beneath the site (i.e., typically medium dense to dense or very stiff to hard soils), 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.

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.

5.6.9 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 Byford Station site. It is noted however, that based on the investigation to date, loose zones appear to be discrete and discontinuous.

5.6.10 Soil and Groundwater Aggressivity

Soil and groundwater aggressivity testing have been carried out and commented on in the reports listed in Table 3.

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):

- 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 design process with the structural and civil designers will be an iterative process. At this stage general geotechnical design information has been provided for use in developing the initial designs. This advice and information will be refined upon receipt of the initial structural and civil designs (structural loads/layouts and civil layouts and levels etc.).

5.7.2 Shallow Foundation Design

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 0.4 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 Terzaghi bearing capacity formulae.

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 14.

Table 14 - 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

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
Z ≥ 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, γ_w *= unit weight of water*

5.7.2.1 Design for Serviceability

Shallow foundations are designed to comply with the design criteria in Section 4.5.

5.8 Geotechnical Design Advice and Calculations

5.8.1 Shallow Foundations – Allowable Bearing Pressures

Preliminary geotechnical design profiles have been developed based on the currently available geotechnical data. Adopted design profiles and parameters are presented in Section 5.6.4.

It is assumed that the top 1.0 m below the base of proposed strip and pad foundations will comprise Engineered fill or compacted in-situ material (and is not unsuitable fill material as summarised in Section 2.4.3 of PTA Specification 8880-450-074) and that the excavation base is compacted to 96% MMDD in accordance with PTA Specification 8880-450-074. The existing fill should be removed and replaced with Engineered fill.

Table 10 below summarises the assumed parameters for imported Engineered or compacted in situ fill material on which the following assessment of allowable bearing pressure has been made (see tables).

The Project Structural Engineer has advised at the Reference Design stage that the proposed Byford Station concourse slab is to be at RL 54.905 m AHD and has been used as a reference design ground level herein.

Table 15 - Preliminary Geotechnical Design Parameters for Imported/Compacted in-situ Fill

Unit	ϒ (kN/m^3)	c' (kPa)	φ' (°)	Su (kPa)	E' (MPa)
Imported and Engineered FILL or Compacted in situ Material	18	0	34	-	45

Based on the preliminary ground model summarised in Table 11 and the Byford Station concourse slab at RL 54.905 m AHD, it is assumed that shallow foundations up to 1.0m depth will be founded within Engineered Fill material or compacted in-situ material (Upper Colluvium – natural Sandy Clay) with a minimum drained Young's Modulus of 45 MPa. Geotechnical design parameters assumed are shown in Table 13 and Table 10.

The design of available foundation bearing pressures for strip and pad footings at the Byford Station precinct has been carried out using the Terzaghi bearing capacity equation. Subject to completing the earthworks and foundation preparation recommendations provided herein, pad and strip footings founded within medium dense to dense sand may be designed based on the maximum allowable bearing pressures provided in Table 11. These values are based on a on a geotechnical strength reduction factor of 0.4 as specified in AS 5100.3: 2017 (equivalent factor of safety = 2.5).

It should be noted that the allowable bearing pressures assume isolated vertical, non-eccentric loads. Dewatering requirements must be considered to complete foundation excavation and to achieve sufficient subgrade compaction depending on the perched groundwater level in relation to the proposed founding level.

Table 16 - Summary of Shallow Footing Design Bearing Pressure for Byford Station Precinct

Embedment depth (m)	Footing Width (m)	Footing Length (m)	Allowable Bearing Pressure (kPa)*	Settlement (mm)**
0.5	0.5 strip		190	5 to 10
	1.0 strip		230	10 to 15
	1.0	1.0	210	5 to 10
	2.0	2.0	260	10 to 15
1.0	0.5 strip		260	5 to 10
	1.0 strip		290	15 to 20
	1.0	1.0	270	5 to 10
	2.0	2.0	320	15 to 20

*Based on Terzaghi Method.

**Long term based on Terzaghi Method.

Note: 1. Maximum allowable settlement/heave for shallow foundations is 20 mm for both short term and long term (long term allowable is inclusive of short-term displacement magnitudes), as stated in PTA Specification 8880-450-053. Differential settlement must not be more than 1:1000 for both short and long term, as stated in PTA Specification 8880-450-053.

2. Additional plate load test might be required to confirm the Young's modulus of the engineered fill.

5.8.2 Shallow Foundations – Soil Springs

It is understood that the shallow footings will be modelled as a series of springs within the structural model. Spring stiffness is not a unique soil parameter but rather is a strain dependent, soil structure interaction parameter which depends on the magnitude of load and the size and stiffness of the loaded area.

Based on the bearing capacity calculations and estimated settlements, subgrade modulus to be adopted for springs beneath the shallow footings are given in Table 17.

Table 17 – Indicative subgrade modulus and spring stiffness for shallow footings

Footing Dimensions W(m) x L(m) x D(m)	Applied Pressure (kPa)	Settlement (mm)	Subgrade Modulus (kPa/m)	Spring Stiffness (kN/m)*
0.5 strip x 0.5	190	10	19,000	9,500
1.0 strip x 0.5	230	15	15,333	15,333
1.0 x 1.0 x 0.5	210	10	21,000	21,000
2.0 X 2.0 X 0.5	260	15	17,333	69,333
0.5 strip x 1.0	260	10	26,000	13,000
1.0 strip x 1.0	290	20	14,500	14,500
1.0 x 1.0 x 1.0	270	10	27,000	27,000
2.0 X 2.0 X 1.0	320	20	16,000	64,000

Note: * for strip footings, 1m length considered for spring stiffness

We recommend that the sensitivity of the superstructure be assessed based on 50% and 200% of the spring/modulus values provided.

It is expected that the spring stiffness provided in this stage will be used iteratively for the column load calculations by the structural engineer.

5.8.3 Retaining Walls

Current Reference Design does not include any major retaining walls at Byford Station Precinct. A general retaining wall design advice is provided herein for reference.

5.8.3.1 Earth Pressures

The retaining wall may be designed using the parameters presented in Table 18 below, which assumes a compacted well graded granular sand fill at foundation level.

Table 18 - Retaining Walls – Earth Pressure Design Parameters for Compacted Granular Fill

Soil Unit	g (kN/m³)	ϕ' (°)	E' (MPa)	K_0	No Wall friction		Soil-Wall friction = 0.5 ϕ'	
					K_a	K_p	K_a	K_p
Compacted Granular Fill	18	34	45	0.44	0.28	3.5	0.25	5.0
Colluvium	18	26	28	0.56	0.39	2.6	0.35	3.4

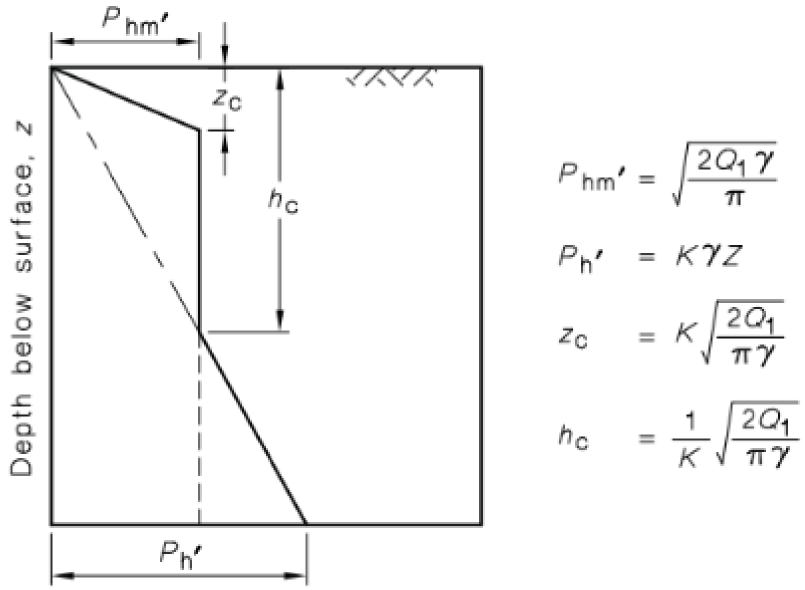
Notes:
γ: soil unit weight; ϕ': angle of internal soil friction; K_0: coefficient of earth pressure at rest, K_a: coefficient of active earth pressure, K_p: coefficient of passive earth pressure, E': long term Young's modulus.
Values of K_0 are based on estimated initial conditions following compaction.

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 overturning and sliding calculations, and 0.40 for bearing assessment is recommended based on the requirements of 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 **7** 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 7: 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 20-30 kPa 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.

5.8.3.2 Bearing Capacity

Refer to Section 5.8.1 for allowable bearing pressures, assuming that footings are not located on or adjacent to sloping ground (such footings will need to be assessed separately) and the permanent embedment depth remains in place for the duration of the design life.

5.8.3.3 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 $0.6\phi'$ to $0.8\phi'$ for fully drained granular soils.

5.8.3.4 Global Stability

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

5.8.4 Earthworks

Detailed earthworks specification will be updated following the proposed test pit results.

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 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 packages and SiD report.

8. Systems Engineering

See main design packages.

9. Sustainability in Design

See main design packages.

10. Human Factors

N/A

11. Reliability, Availability and Maintainability (RAM)

See main design packages.

12. Construction Methodology

12.1 Construction Methods

When constructing the proposed shallow foundations for the new Byford Station structure, temporary localised dewatering during excavation will be considered to at least 0.5 m depth from the underside of the shallow foundation blinding layer. Dewatering assessments will be completed under separate cover at the next design stage.

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 packages.

14. Non-Compliances

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

- No geotechnical non-compliances have been identified at this stage.

Appendix A: Deliverables List

Refer to Appendix A

Appendix B: Specifications (Not is Use)

Appendix C: Drawings (Not is Use)

Appendix D: Engineering Change Approvals (Not is Use)

Appendix E: Calculations

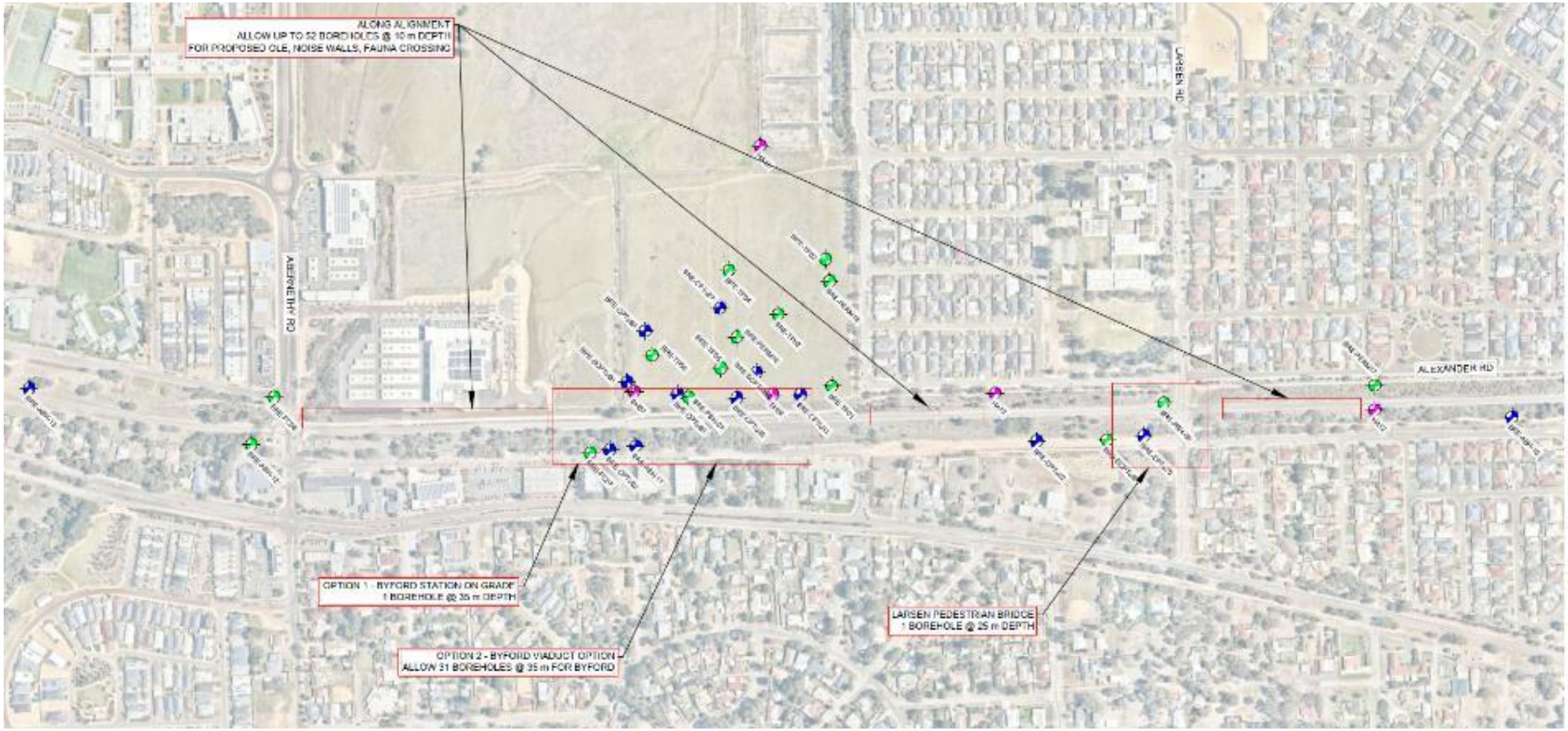


Figure A8: Existing Ground Investigation Locations

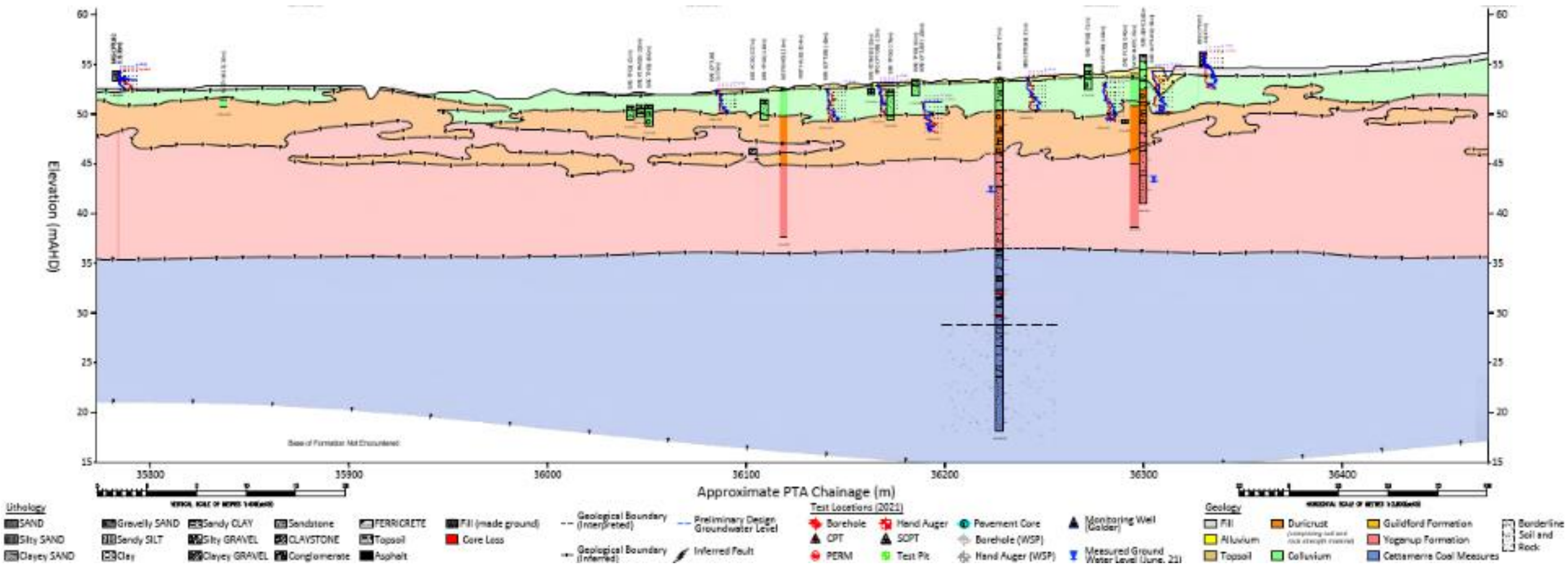


Figure A9: Geological Long Section

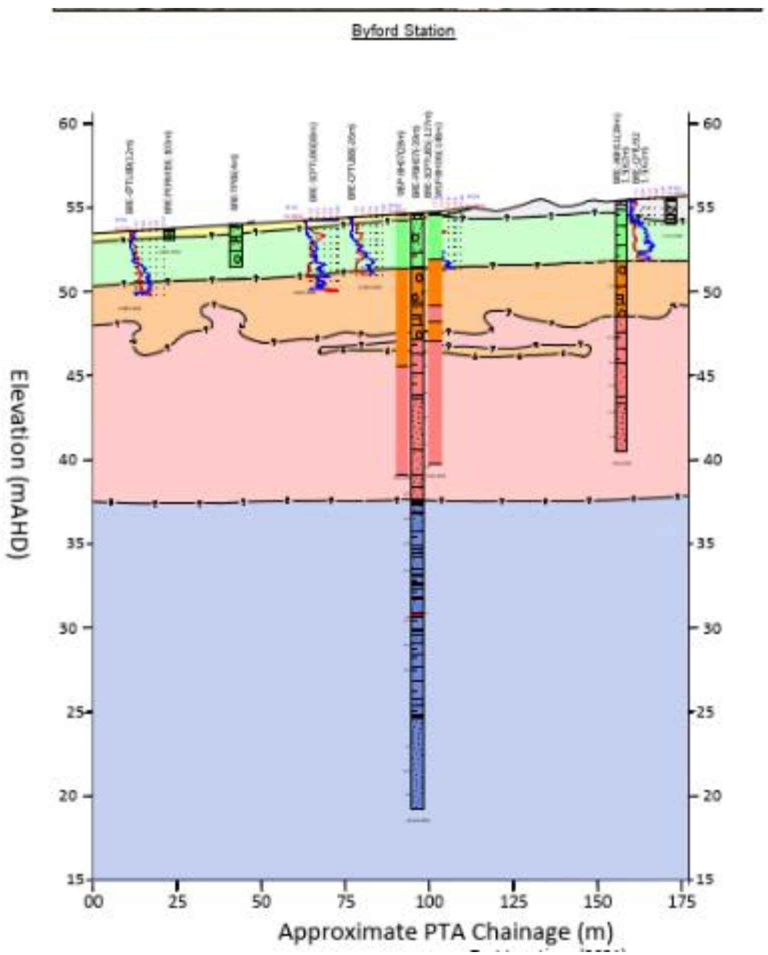


Figure A10: Geological Cross Section

Appendix F: Schedules (Not is Use)

Appendix G: IDC Certificates

Refer to Appendix G

Appendix H: Independent Verification Certificates (Not is Use)

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

Appendix J: Third Party Approvals (Not is Use)

Appendix K: RFIs (Not is Use)

Appendix L: Project Interfaces (Not is Use)

Appendix M: Departures (Not is Use)

Appendix N: Deviations (Not is Use)

Appendix O: RATM Extract (Not is Use)

Appendix P: Project Hazard Log (Not is Use)

Appendix Q: Safety in Design (Not is Use)

Appendix R: Human Factors (Not is Use)

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

Appendix T: Durability Assessment (Not is Use)

Appendix U: Sustainability (Not is Use)

Appendix V: ITP Strategy (Not is Use)

Appendix W: BCA Certificates (Not is Use)

Appendix X: DDA Certification (Not is Use)

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