

Retention Wall Design Report for Sydney Trains

21 Atkinson St Liverpool

Client: Binah By Design

Job Number: 160507

DOCUMENT REGISTER

PREPARED BY	APPROVED BY	STATUS	ISSUE	DATE
N.V.	F.C.	For Approval	A	17/08/2015

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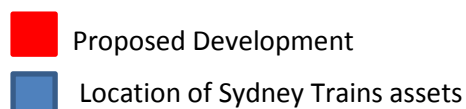
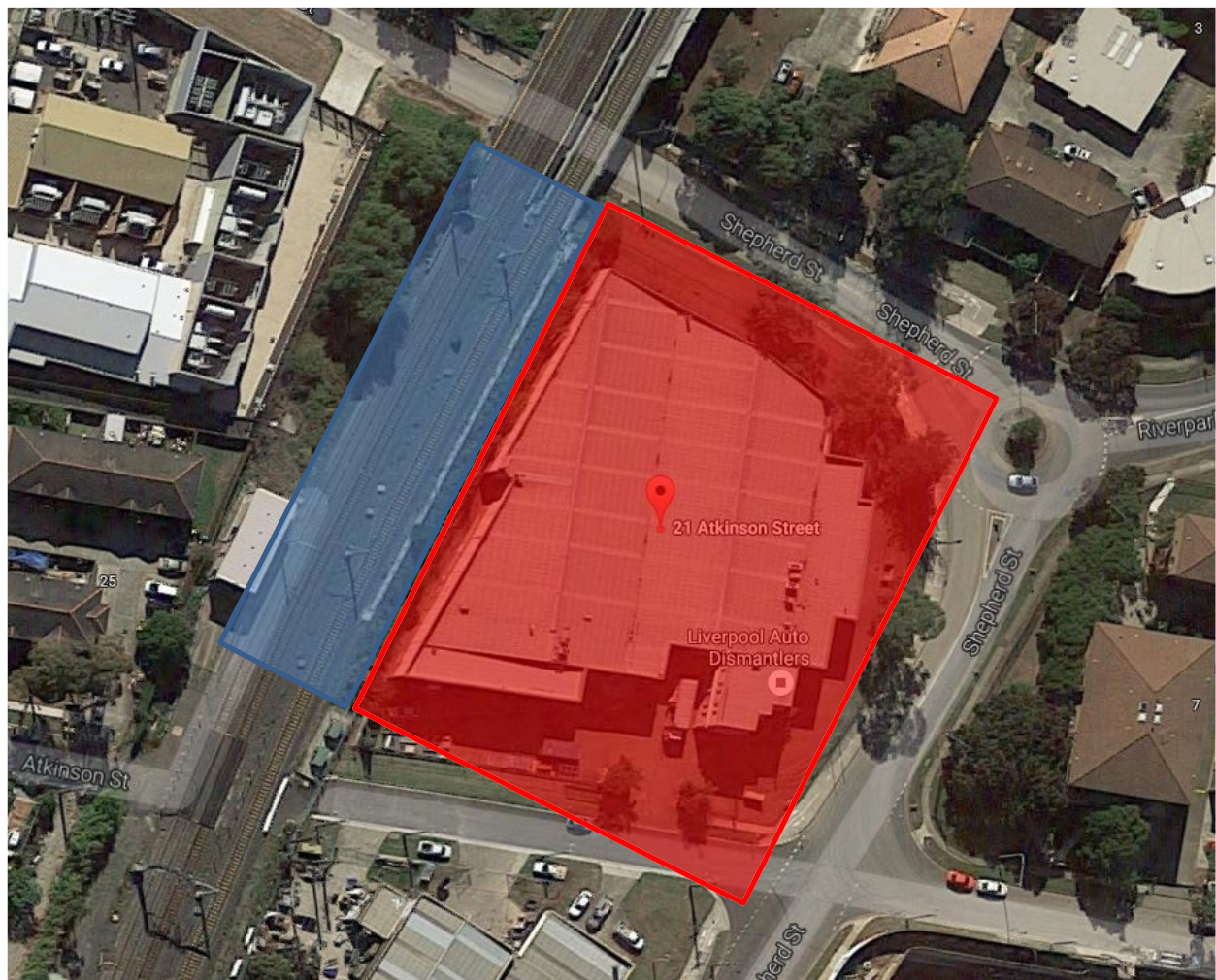
1.0 INTRODUCTION

1.1 Objective

Xavier Knight Consulting Engineers Pty Ltd is the structural engineers for the development located at 21 Atkinson St Liverpool.

The objective of this report is to provide a specialist engineering assessment of the foundation piling and retention wall design for the approval of Sydney Trains.

1.2 Project Location (Aerial)



1.3 Project Description

The development is a residential flat building consisting of approximately 150 apartments over part 7 and 9 stories.

The building is a column framed building with insitu concrete slabs. Lift and stair core walls laterally stabilize the building.

Minor excavation to the north of the site is required to construct the substation and bin room. This represents only 5% of the site.

The excavated level is RL 8.37.

The required retention wall has a maximum retaining height of 1.5 m and safely supports the proposed excavation in the temporary and permanent scenario, whilst ensuring the surrounding buildings, infrastructure and assets remain serviceable and adequate to for fill their intended purpose. The location of the shoring wall is shown on structural drawings in Appendix 8.1.

The remaining site must be raised by filling methods to achieve the platform height of RL10.10.

The filling is only used as a platform to construct a suspended ground floor slab. The filling is not required to be compacted with intensity and poses negligible risk to the rail corridor.

The building is founded on foundation piles.

The site topography is described as follows;

Regionally, the topography falls towards the north-northeast at an angle of about 2-3°. The site levels slopes down from RL of 13.0 m AHD at the south-western corner of the site to RL 8.4 m AHD at the northern corner.

The proximity of Sydney Trains assets are as follows;

- Centreline of the nearest rail line to the property boundary is between 9-12 m.
- Centreline of the nearest rail line to buildings is 11-14 m.
- Centreline of the nearest high voltage transmission line to the building is over 8m from its sag position.

The proximity of Sydney Trains assets is shown in structural drawings in Appendix 8.1.

1.4 Supplementary Documentation

The following documents are submitted as an Appendix of this report.

- Structural Drawings prepared by Xavier Knight Consulting Engineers Pty Ltd.
- Design Calculations prepared by Xavier Knight Consulting Engineers Pty Ltd.
- Geotechnical Report by Environmental Investigations Australia Pty Ltd.
- Survey prepared by Lawrence Group.
- Risk Assessment Report prepared by Minciv Management Services Pty Ltd
- Structural Design Certificate prepared by Xavier Knight Consulting Engineers Pty Ltd.

2.0 RETENTION WALL AND PILED FOUNDATION METHODOLOGY

2.1 Retention Wall Solution

The retention wall solution for the projects comprises of the following;

- 450dia reinforced concrete soldier piles spaced at 1000mm centres.
- Shoring piles are adequately embedded below the bulk excavation level
- Shoring piles are tied together at their head by a continuous reinforced concrete capping beam. The capping beam allows lateral load transfer and vertical load transfer between one or more piles
- The shoring piles are tied together along their length with a shotcrete wall to adequately support the material between them
- The retention wall is designed a permeable wall

2.2 Construction Methodology (Retention Wall)

The following construction sequence is to be adopted for the proposed cantilevered retention wall;

1. Install the piles and construct capping beam
2. Once piles have achieved 28 day strength, proceed with excavation to the bulk excavation level and install shotcrete as required
3. Carry out detailed excavation.
4. Construct permanent structures.

Special note:

- Excavation must be inspected by the geotechnical engineer every 1500mm of excavated depth. Allow for soil stabilization measures as required by the geotechnical engineer before proceeding with the excavation.
- Do not over excavate the site below the nominated bulk excavation levels.
- Surcharge including plant, machinery and stockpiles must be positioned 1000mm clear of the retention wall.

2.3 Piled Foundation Solution

The piled foundation solution for the projects comprises of the following;

- 600dia reinforced concrete isolated piles under the columns from Ground Floor to Level 3 socketed 7.0 m in the rock layer Unit 4b.
- 750dia reinforced concrete isolated piles under the columns from Ground Floor to Roof socketed 8.0 m in the rock layer Unit 4b.
- 600dia reinforced concrete piles under the lift and stair shafts socketed 7.0 in the soil layer Unit 4b.

For the subsoil profile, refer to Geotechnical Report in Appendix 8.2.

2.4 Construction Methodology (Piled Foundation)

In reference to Clause 4.6 of the Geotechnical Report in Appendix 8.2, two options are recommended for the proposed piled foundation;

- Bored piers with the use of temporary/sacrificial liners together with tremmie methods.
- Contiguous flight auger (CFA) piles.

The piling contractor is advised to adhere to the construction methodologies of the geotechnical investigation report.

3.0 IMPACTS ASSESSMENT

3.1 Proximity of Sydney Trains Assets

The Sydney Trains rail corridor within proximity of the proposed development site has been surveyed by Lawrence Group Dated 16/05/2016. Refer to Appendix 8.4.

The structural engineering plans prepared by Xavier Knight Consulting Engineers Pty Ltd demonstrate the geometric proximity of the retention wall and building foundations with respect to the Sydney Trains. Refer to Appendix 8.1.

All structures and supports are wholly within the development site boundaries.

By representing a 45 degree zone of influence line (cohesive soils) projecting from the bulk excavation level of the basement towards the assets in question it is identified that the closest Sydney Trains rail-track is not within the zone of influence of the excavated basement.

3.2 Derailment and Collision Assessment

A Sydney Trains Risk Assessment report has been carried out by Minciv Management Services Pty Ltd. The risk assessment finds that the likelihood of a derailed train travelling and displacing laterally an adequate distance down the embankment to strike and collapse a structural column resulting in fatality and major injuries is less than once in 1000years. The consequence is a risk level of C. in accordance with AS5100 the rail collision loading requirements for the support structures of the buildings adjacent the rail line is Nil.

3.3 Impacts of Proposed Works

3.3.1 Earthworks Equipment

Refer to geotechnical report no E23001 GA dated 17th June 2016 by Environmental Investigations Australia Pty Ltd clause 4.4 to identify acceptable earth works methods and equipment required to carry out the piling and excavation work.

Since the development site is lower than the Sydney trains corridor, there is no risk of heavy plant or machinery trafficking and surcharging on to the rail corridor of Sydney trains assets.

3.3.2 Vibration Effects

Refer to geotechnical report no E23001 GA dated 17th June 2016 by Environmental Investigations Australia Pty Ltd clause 4.4 to identify the construction and excavation procedure required to limit the vibration effects to the adjoining Sydney Trains assets.

The contractor is to ensure conformance with the geotechnical recommendations to limit vibratory effects.

As discussed, there is a minor excavation with maximum depth of 1.5 m below the natural ground level into clay/silt materials. Therefore, the excavation will be carried out by regular earth moving equipments with very low risk of vibration to the adjacent Sydney Trains assets.

The remains of the site are to be raised with engineered fill to the platform level above the natural ground level. The engineered fill is to be laid and compacted in accordance with clause 4.4.3.1 of the Geotechnical Report.

3.3.3 Lateral Deflection

Permissible deflections should be determined by Sydney Trains on a case by case basis taking into account the sensitivity of the their assets to movements, the proximity of the structure to such assets and the ground movements that will occur within Sydney Trains property.

Xavier Knight Consulting Engineers have adopted the following serviceability criteria applicable to any one direction unless instructed otherwise;

The lesser of;

- $L/200$ where L is the height of the supported excavation
- 10mm max

The estimated lateral deflection of the wall is up to 8 mm. Refer to Appendix 8.2 for specific deflection predictions for each design model.

A finite element analysis which assesses the different stages of loading/unloading of the site and its effect on the rock mass surrounding the rail corridor may not be required since the excavation of the site only effects less than 5% of the site and is very minor in depth from the natural ground level. Furthermore, the rail-track has been demonstrated to be outside the zone of influence line projecting from the bulk excavation level towards the assets in question.

3.3.4 Ground water and draw down effects

The apparent ground water level for the proposed basement excavation is RL 6.8m AHD as stated in the geotechnical engineering report.

In regards to the retention wall, the ground water is below the bulk excavation level and there will be negligible draw down effects to the surrounding buildings, infrastructure and assets.

Refer to Geotechnical Report no. E23001 GA dated 17th June 2016 by Environmental Investigations Australia Pty Ltd clauses 3.2 & 4.6 to identify the requirements for the installation of the piled retention wall and foundations.

3.3.5 Proximity of Foundations

The buildings foundations do not surcharge the rail corridor or Sydney Trains infrastructure as demonstrated on the structural drawings, Appendix A.

3.3.6 Lateral supports

The retention wall is a permanent cantilevered wall and does not need temporary or permanent lateral supports.

4.0 DESIGN CALCULATIONS

4.1 Design Methodology (Retention Wall)

As discussed in clause 2.1 of this report, the proposed retention system consists of the following;

- 450dia reinforced concrete soldier piles spaced at 1000mm centres.
- Shoring piles are adequately embedded below the bulk excavation level
- Shoring piles are tied together at their head by a continuous reinforced concrete capping beam. The capping beam allows lateral load transfer and vertical load transfer between one or more piles

The retention wall has been designed and analysis using “WALLAP” retaining wall analysis computer program (version 6) by D.L Borin. This is a widely recognized tool, generally considered suitable for this application.

The design analysis has been divided into individual design model reflecting different existing ground levels, geotechnical soil profiles, surcharges, excavation levels and permanent support levels. The location of the design models are shown in the below figure.

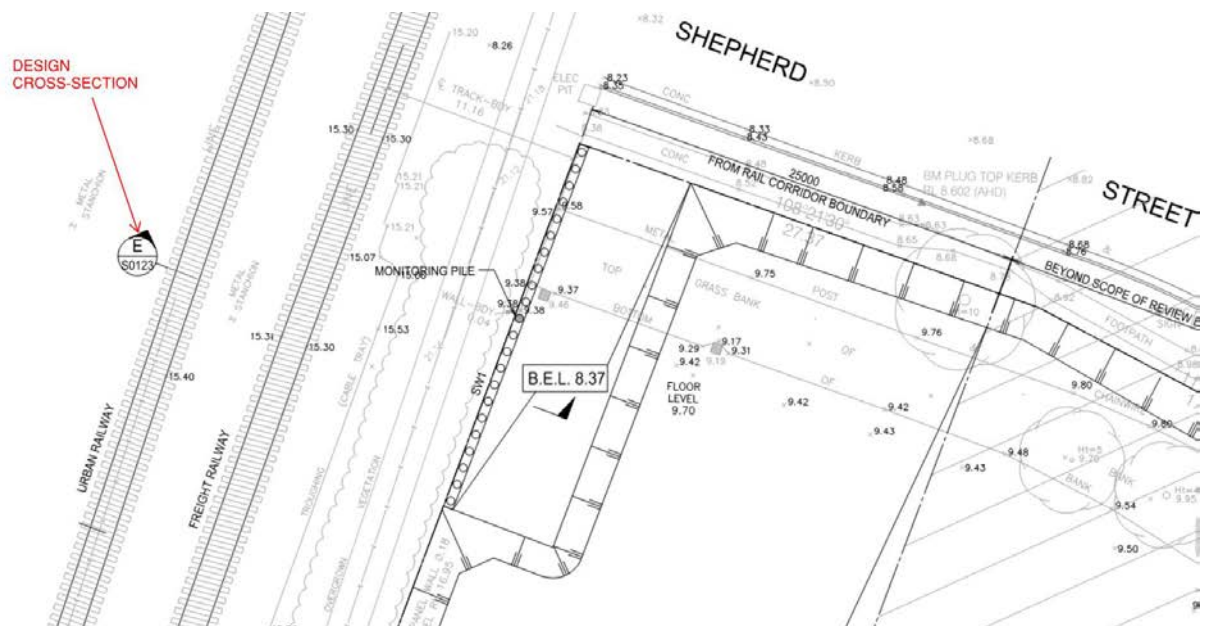


Figure 4.1 Plan layout of design model

The design satisfies the requirements of Sydney Trains letter named "STATE ENVIRONMENTAL PLANNING POLICY (INFRASTRUCTURE) 2007, DEVELOPMENT APPLICATION- DA - 84/2016 - 21 Atkinson Street, Liverpool dated 23rd March 2016.

4.2 Design Cross-section (Retention Wall)

Appendix 8.2 shows the design cross section used in the analysis of model and identifies;

- The proximity of the Sydney Trains assets to the proposed retention wall and basement excavation.
- The proximity of the Sydney Trains assets to the proposed development site boundary
- The zone of influence of the basement excavation
- The soil/rock profiles and geotechnical design parameters.
- Ground water level.
- Geometry of the retention wall, top of cap level, toe levels, temporary lateral support levels
- Stiffness of the retention wall
- Loading criteria

4.3 Design Output (Retention Wall)

Appendix 8.2 provides evidence of the WALLAP design output and includes;

- Input design parameters
- The factor of safety using the Strength Factor method. All walls have been designed to a minimum factor of safety against overturning of 1.5
- Deflection of wall
- Design actions for moment and shear applied to the wall
- The design load applied to the temporary supports

4.4 Structural Design

Appendix 8.2 Provides design calculation for the following;

- Design life of the retention wall (100 years)
- Assessment of durability
- Design of each pile within the retention wall
- Design of the capping beam
- Design of the shotcrete wall between the piles
- Design of the piled foundations including the pile caps

4.5 Geotechnical Design for Axial Load

Appendix 8.2 provides design calculation for checking the geotechnical design capacity of the piles within the retention wall as well as the piled foundation.

5.0 INSTRUMENTATION AND MONITORING

Refer to Xavier Knight Consulting Engineer drawing no. S0101 project no 160507 for the proposed instrumentation and monitoring plan.

The retention wall must be monitored as the excavation progresses and regularly for the duration of the works, to ensure deflections are in accordance with the design estimates. Baseline readings should be taken prior to commencement of excavation and weekly thereafter and at the following stages;

- After reaching bulk excavation level
- One month after the completion of the permanent retaining structure.

Monitoring of the wall deflections should comprise of the following;

Survey targets attached to the wall.

The survey targets should be fixed to the top of the wall and just below every anchor level down the wall with the last survey point at the bulk excavation level.

The following retention wall-monitoring schedule is proposed;

Alert:	Deflection less than 8 mm Excavation may proceed as scheduled.
Action:	Deflections between 8 mm and 12 mm Notify Xavier Knight Consulting Engineers and all stakeholders. Increase frequency of monitoring schedule. Proceed on advice from Xavier Knight.
Alarm:	Deflections greater than 15 mm All work is to be terminated and notify Xavier Knight Consulting Engineers and all stakeholders. Await advice from Xavier Knight to recommencing works.

6.0 WALL PERMEABILITY

A reinforced soldier piled wall is not watertight.

Shotcrete wall should be constructed in between the gaps of the piles immediately after every 1500mm of excavation depth and in accordance with the geotechnical engineer's requirements.

Appropriate drainage measures must be installed behind the wall to ensure there is no build-up of hydrostatic pressure behind the wall and that seepage behind the wall does not remove any soil behind the wall.

7.0 CONCLUSION

The retention wall design has been prepared based on the contractor implementing a high level of quality control and engineering supervision. The contractor shall ensure the proposed retention system is appropriate for the site conditions and that the proposed plant and tooling is capable of installing the piles in accordance with the requirements of this report.

8.0 APPENDIX

8.1 Structural Drawings

8.2 Design Calculations

8.3 Geotechnical Report

8.4 Survey

8.5 Risk Assessment Report

8.6 Structural Design Certificate

XAVIER KNIGHT CONSULTING ENGINEERS PTY LTD



FERIS CHEHADE BE(Hons) MIEAust CPEng NPER

Director

8.1. STRUCTURAL DRAWINGS

Document Issue Register and Transmittal

Structural Drawings

PAGE	1	OF	2
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JOB No.	160507
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Residential Development
No. 21 Atkinson St, Liverpool

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I INFORMATION Co CO-ORDINATION T TENDER

S COSTING **CC** CONSTRUCTION CERTIFICATE **B** AS BUILT DRAWINGS

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SENT BY :	Kris Kaminski
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SCALE 1:200

- SHORING NOTES:

1. NUMBER & SPACING OF SHORING PILES IS INDICATIVE ONLY. REFER TO DETAILS FOR MAXIMUM SPACING.
2. ALL RL's SHOWN ARE INDICATIVE ONLY. REFER TO ARCHITECTS DRAWINGS FOR ALL SET-OUT AND LEVELS.
3. ROADS AND MARITIME SERVICES TO BE NOTIFIED 7 DAYS PRIOR TO EXCAVATING BELOW ROAD LEVEL.
4. REFER TO FOOTING PLAN AND GROUND FLOOR PLAN FOR DETAILED EXCAVATIONS.

1. DEPTH OF SERVICES ARE TO BE CONFIRMED PRIOR TO INSTALLING ANCHORS.

1. TO BE IN ACCORDANCE WITH GEOTECHNICAL ADVICE BY EI AUSTRALIA.

THE FOLLOWING IS AN EXTRACT FROM THE PRACTICE NOTE NO. 10 BY THE ASSOCIATION OF CONSULTING STRUCTURAL ENGINEERS (ACSE) OF N.S.W.:

"The building owner should be advised that in carrying out shoring works, it is virtually impossible to entirely prevent any movement of the excavation, and that excavation movement may lead to damage to adjoining properties. It is recommended that consulting engineers, with the appropriate advice from the geotechnical consultant, advise owners of the possible risks associated with the works and the potential to damage to adjoining properties, which may subsequently need repairs (e.g., the damage may be in the form of cracks which would need to be repaired at the developer's expense). This course of action is not intended to be or appear to be alarmist, but simply a proper discharge of the engineers' responsibility. The building owner should be advised to allow a sum of money as a contingency for any repairs to adjoining buildings."

1. DENOTES MONITORING LOCATION INSTALL SURVEY POINT ON CAPPING BEAM ABOVE PILES AND AT EVERY ANCHOR LEVEL DOWN THE WALL.
2. MONITOR WALL MOVEMENT AND REPORT DEFLECTIONS TO XAVIER KNIGHT IMMEDIATELY AT THE FOLLOWING STAGES:
 - a) DETERMINE BASE LINE READINGS PRIOR TO EXCAVATION.
 - b) AFTER EXCAVATION TO FIRST ROW ANCHORS, BUT PRIOR TO INSTALLATION OF ANCHORS.
 - c) AFTER EXCAVATION TO SECOND ROW ANCHORS, BUT PRIOR TO INSTALLATION OF ANCHORS.
 - d) AFTER EXCAVATION REACHES BULK EXCAVATION LEVEL.
 - e) AFTER DESTRESSING OF ANCHORS.

SHORING WALL MONITORING SCHEDULE			
ALERT	8	≤	mm
ACTION	12	≤	mm
ALARM	15	>	mm

1. EXCAVATION MAY CONTINUE AS SCHEDULED FOR DISPLACEMENTS LESS THAN ALERT STAGE.
2. SHOULD ACTION STAGE BE REACHED XAVIER KNIGHT MUST BE NOTIFIED AND MONITORING PROCEDURE REVIEWED
3. SHOULD ALARM STAGE BE REACHED THE EXCAVATION WORKS IS TO BE TERMINATED AND XAVIER KNIGHT MUST BE CONSULTED IMMEDIATELY.



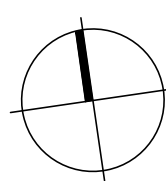
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North



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RESIDENTIAL DEVELOPMENT
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LIVERPOOL

Sheet Subject

BUILDING SHORING AND EARTHWORKS PLAN

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Job No
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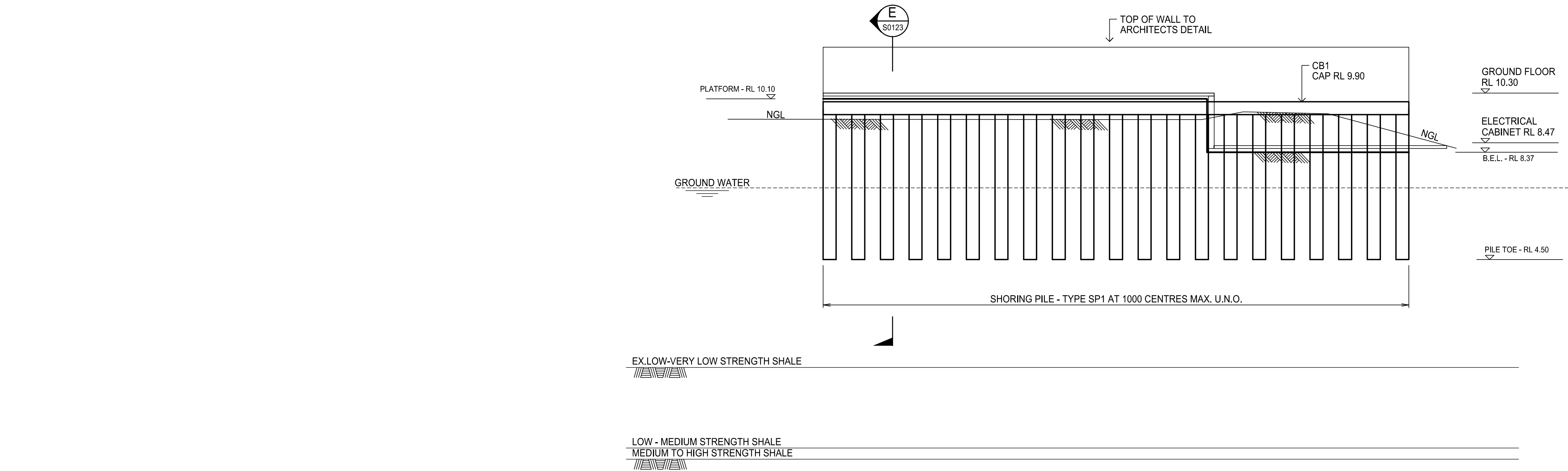
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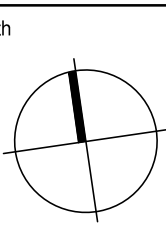
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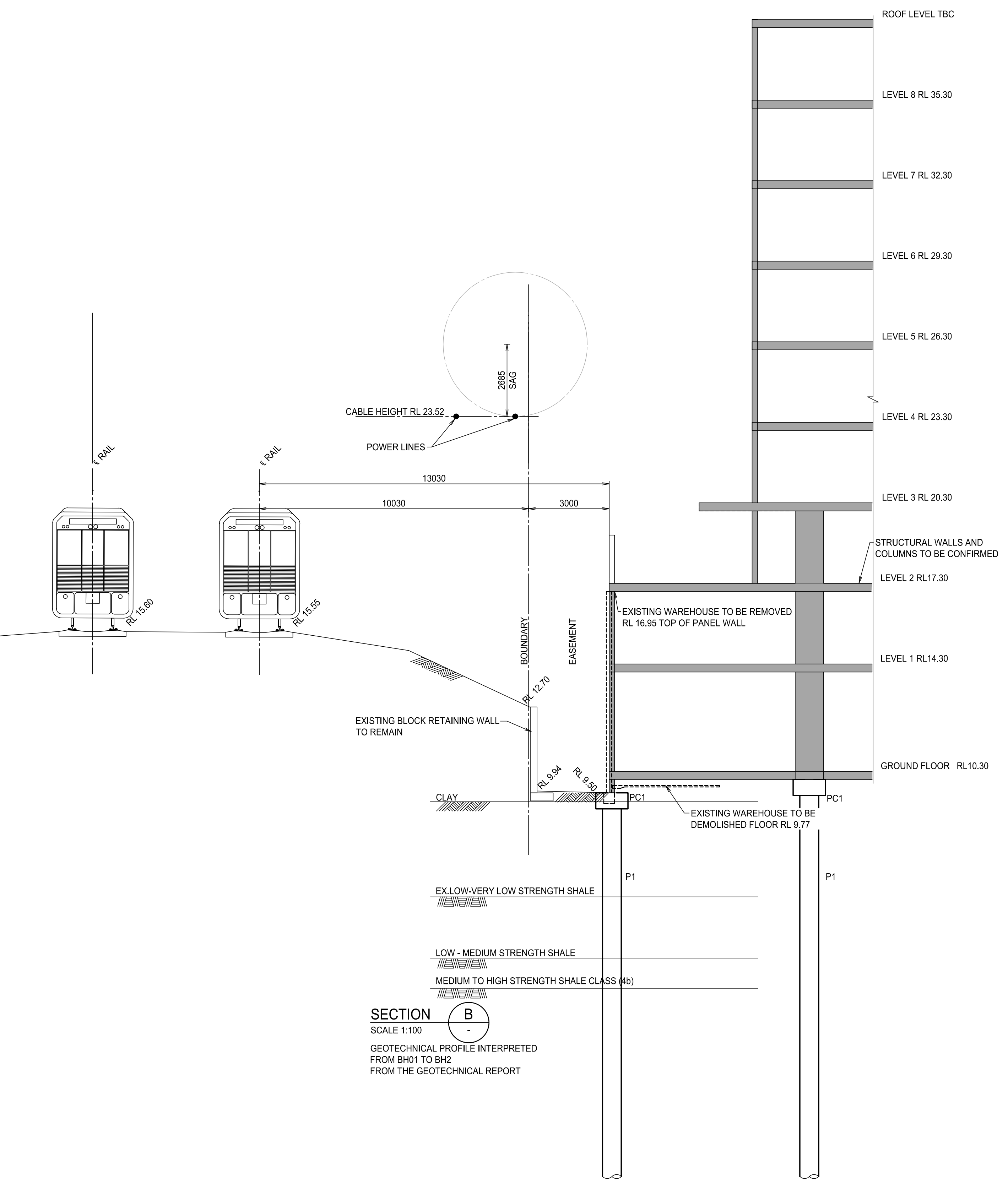
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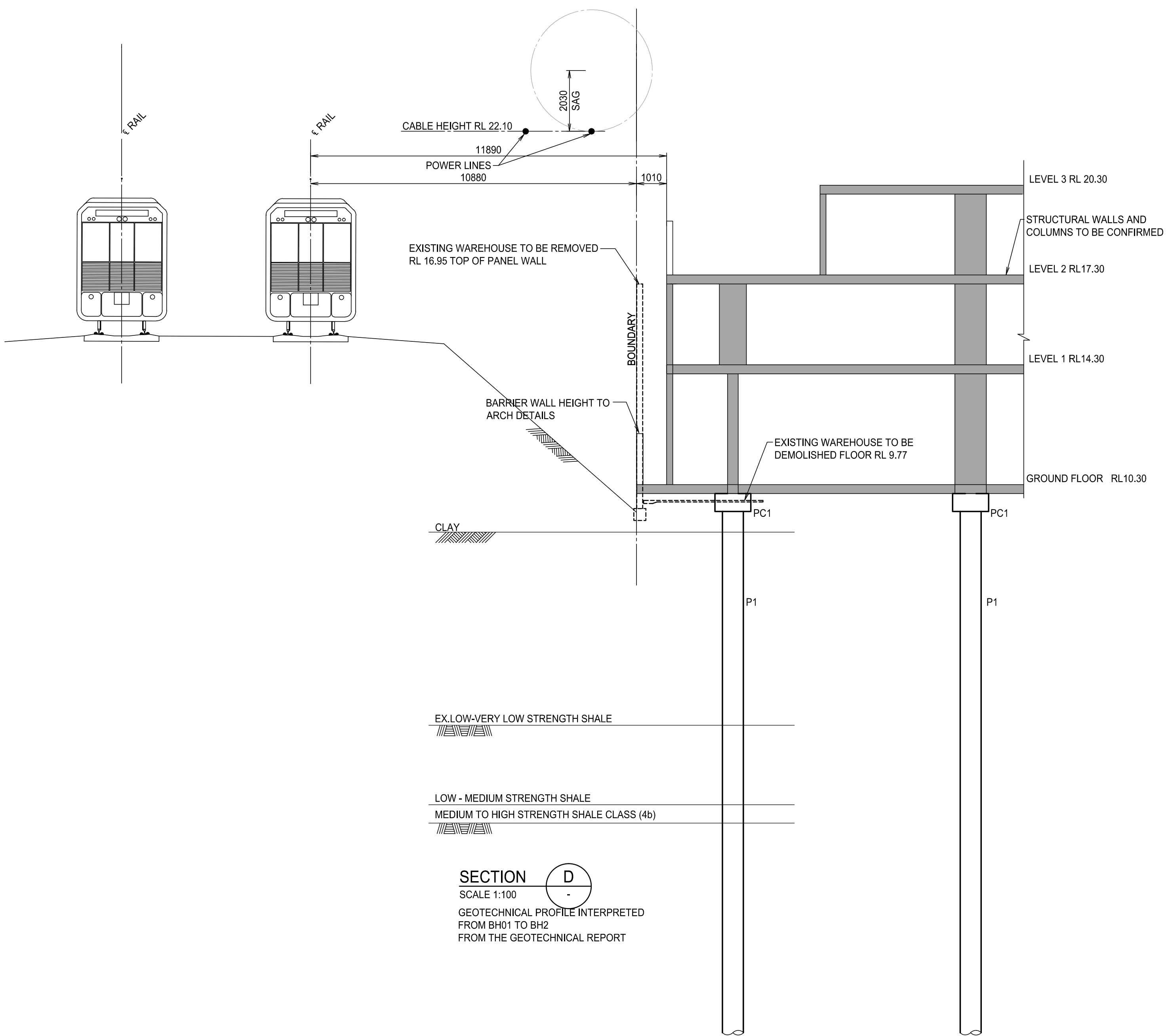


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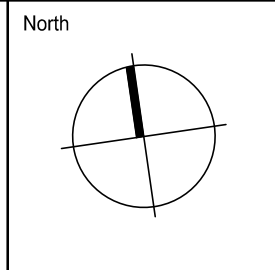
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Sheet Subject
RAIL CORRIDOR SECTIONS

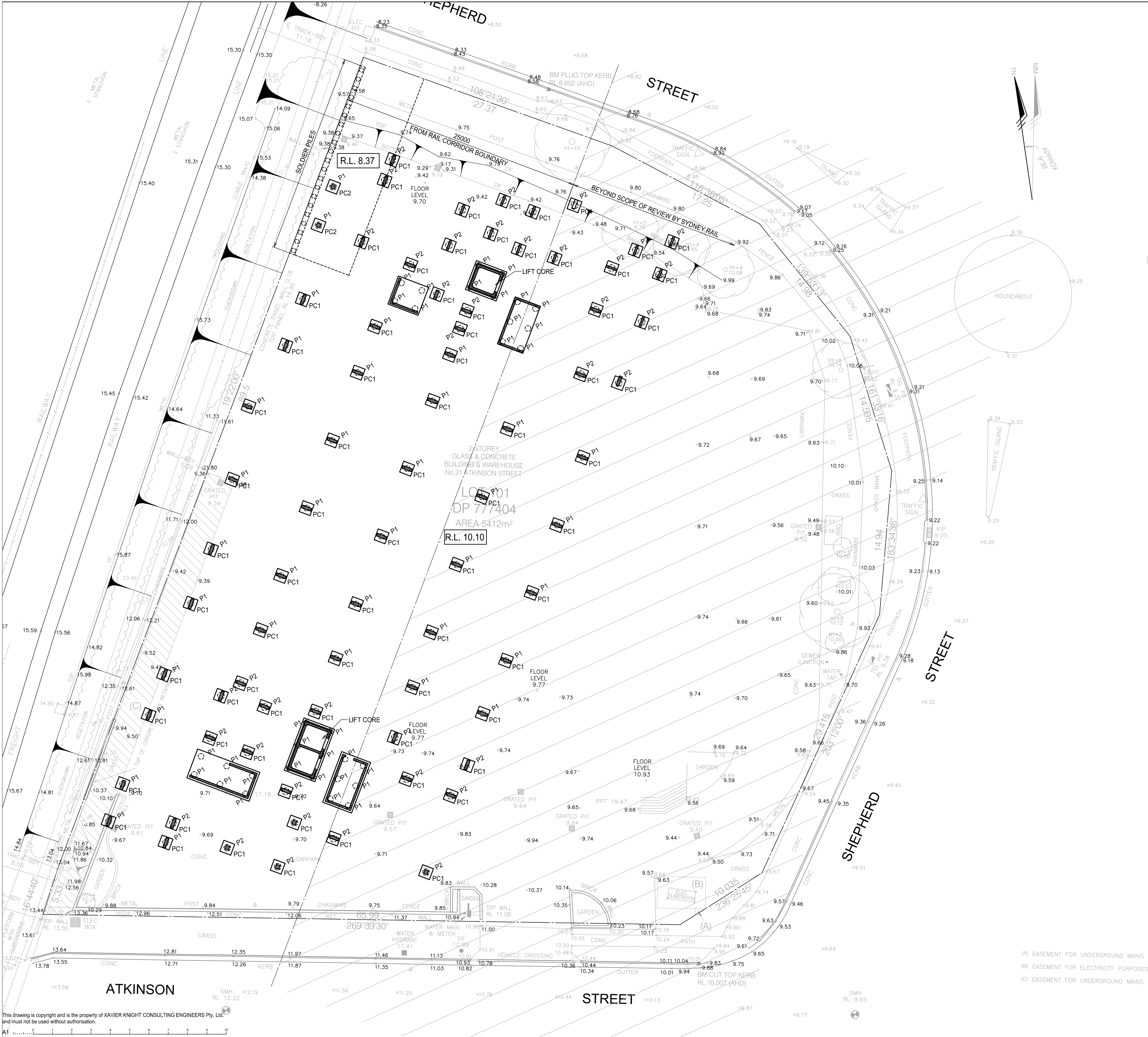
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Job No 160507	Drawing No S0123	Revision 1



SCALE 1:20

SCALE 1:20

SCALE 1:50



FOOTING PLAN

SCALE 1:200

- 1. [R.L. 10.10] DENOTES BUILDING PLATFORM LEVEL.
- 2. TOP OF FOOTING LEVELS TO BE 250mm BELOW FINISHED FLOOR LEVEL U.N.O. ALLOW FOR ADDITIONAL CLEARANCE TO TOP OF FOOTINGS FOR HYDRAULIC SERVICES AS REQUIRED.
- 3. ALL PILES TO BE LOCATED CENTRALLY UNDER WALLS AND COLUMNS.
- 4. FOOTING TO BE SOCKETED INTO UNIT(4b) AS PER SCHEDULE BELOW WITH AN ALLOWABLE BEARING PRESSURE OF 3500 kPa.
- 5. GEOTECHNICAL ENGINEER TO VERIFY AND APPROVE FOUNDATION MATERIAL PRIOR TO CASTING FOOTINGS.
- 6. BORE HOLE INFORMATION SHOWN ON DRAWINGS ARE INDICATIVELY ONLY. IT IS THE CONTRACTORS RESPONSIBILITY TO VERIFY THEM AGAINST THE GEOTECHNICAL REPORT AND ACTUAL SITE CONDITIONS

PILE SCHEDULE

MARK	DIAMETER	REINFORCEMENT	TIES	MIN. SOCKET INTO ROCK (m)
P1	Ø600	5N28	N10-300 SPIRAL	7000 INTO CLASS (4b)
P2	Ø700	6N28	N10-300 SPIRAL	8000 INTO CLASS (4b)

- NOTES:
- 1. MINIMUM OF 5% OF FOUNDATION PILES ARE TO BE INTEGRITY TESTED IN ACCORDANCE WITH AS 2159 PILING CODE.
 - 2. CONSTRUCTION OF THE FOUNDATION PILES MAY INTERSECT THE GROUND WATER TABLE. IT IS THE RESPONSIBILITY OF THE PILING CONTRACTOR TO ENSURE THE PILE EXCAVATION IS STABLE, DRY AND CLEAN PRIOR TO CASTING CONCRETE.
 - 3. ALL PILES ARE TO BE FOUNDED WITHIN UNIT (4b) WITH AN ALLOWABLE BEARING PRESSURE OF 3500 kPa AND ALLOWABLE SHAFT FRICTION IN COMPRESSION OF 350 kPa.

PILE CAP SCHEDULE

MARK	DIMENSIONS			REINFORCEMENT	
	LENGTH	WIDTH	DEPTH	BOTTOM	TOP
PC1	1200	1200	600	6N20 E.W.	6N20 E.W.
PC2	900	900	600	4N20 E.W.	4N20 E.W.

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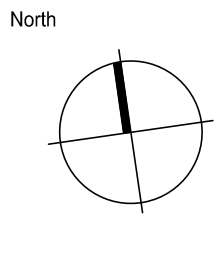
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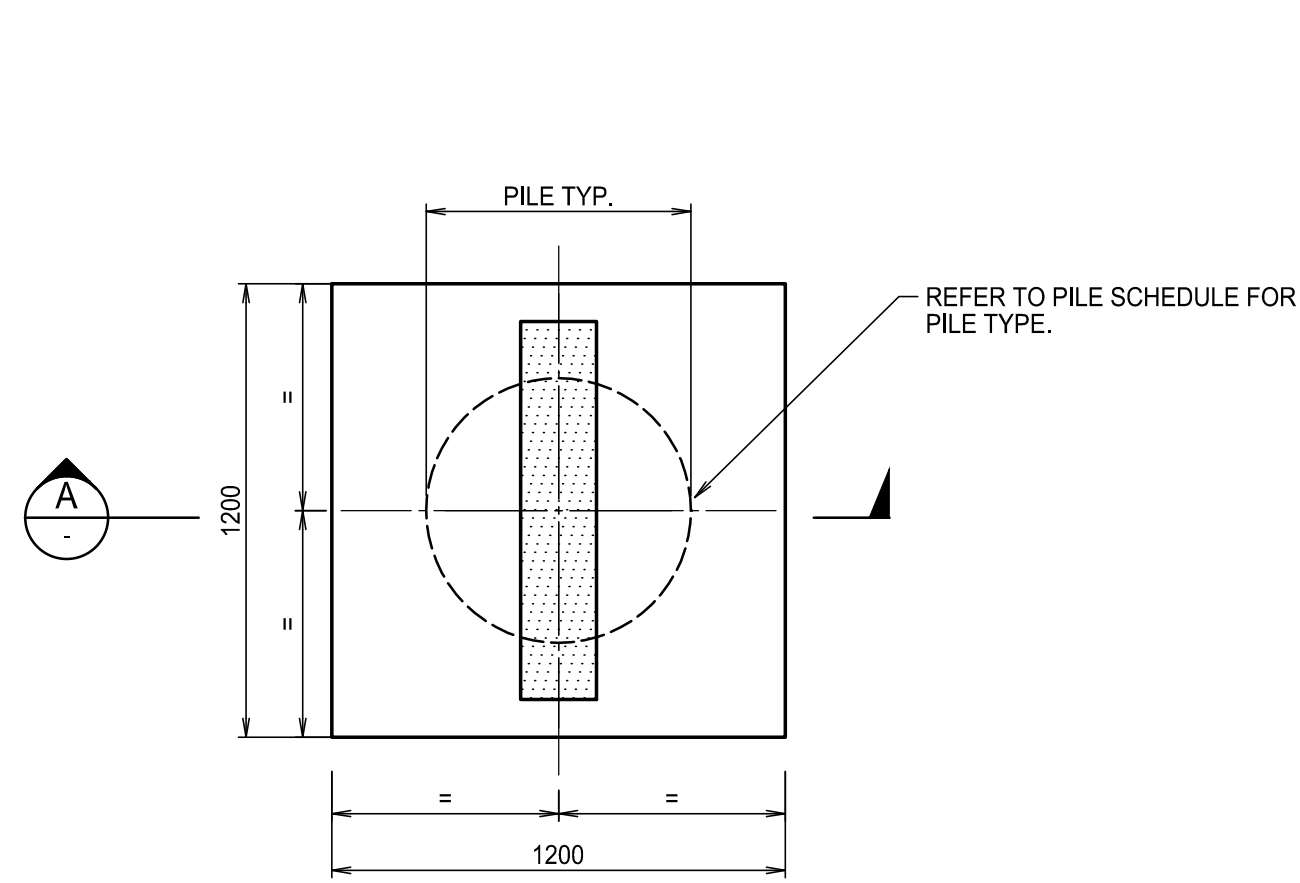
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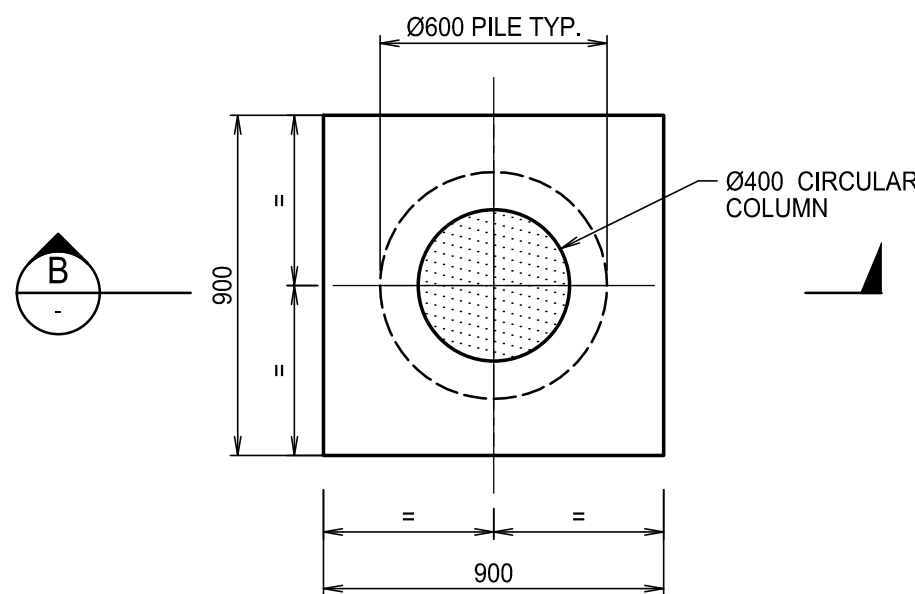
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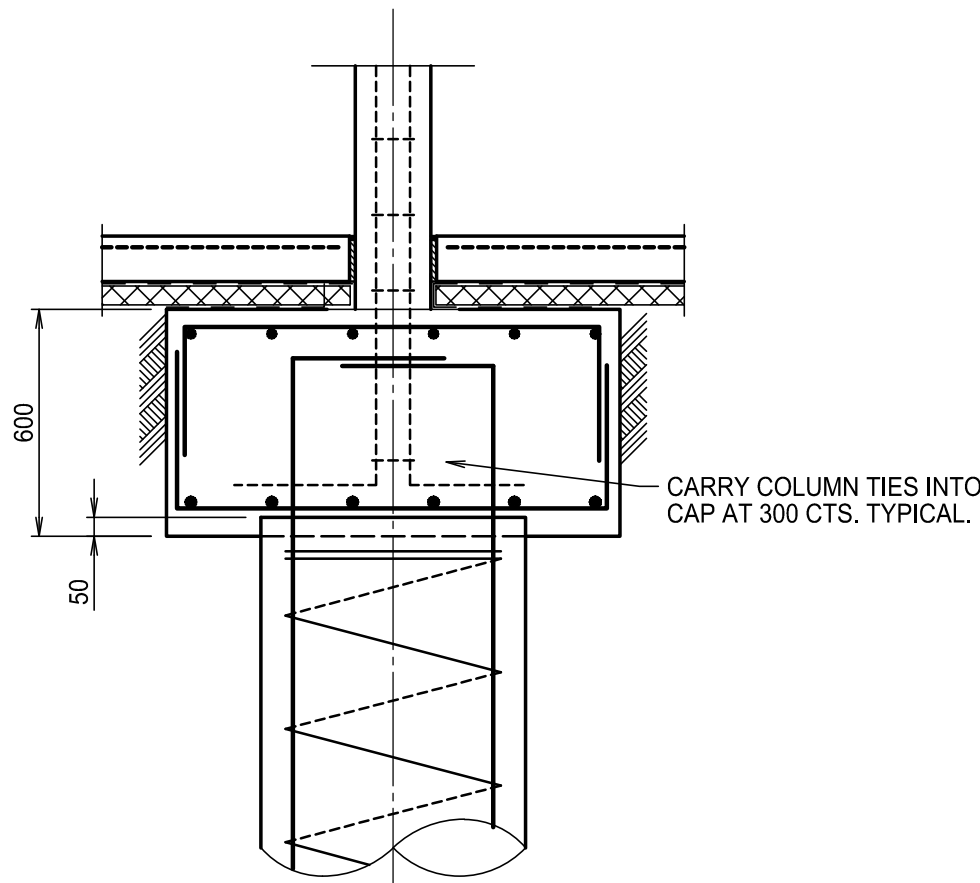
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Job No	Drawing No	Revision
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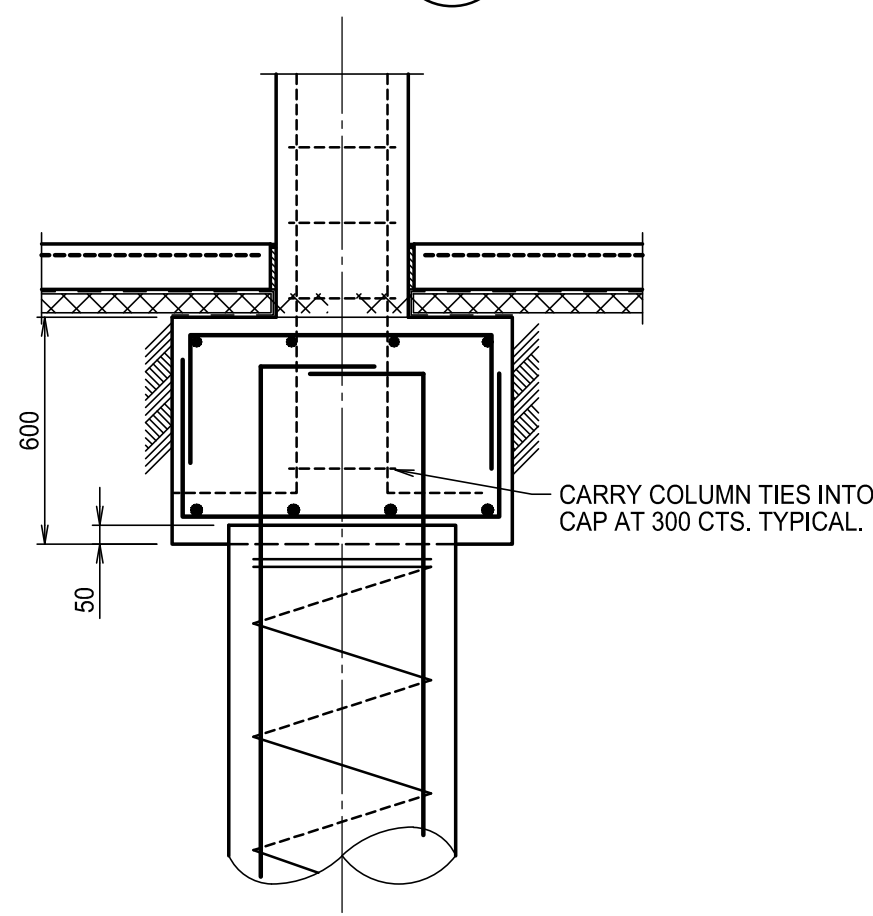
TYPICAL PILE CAP PLAN
PC1



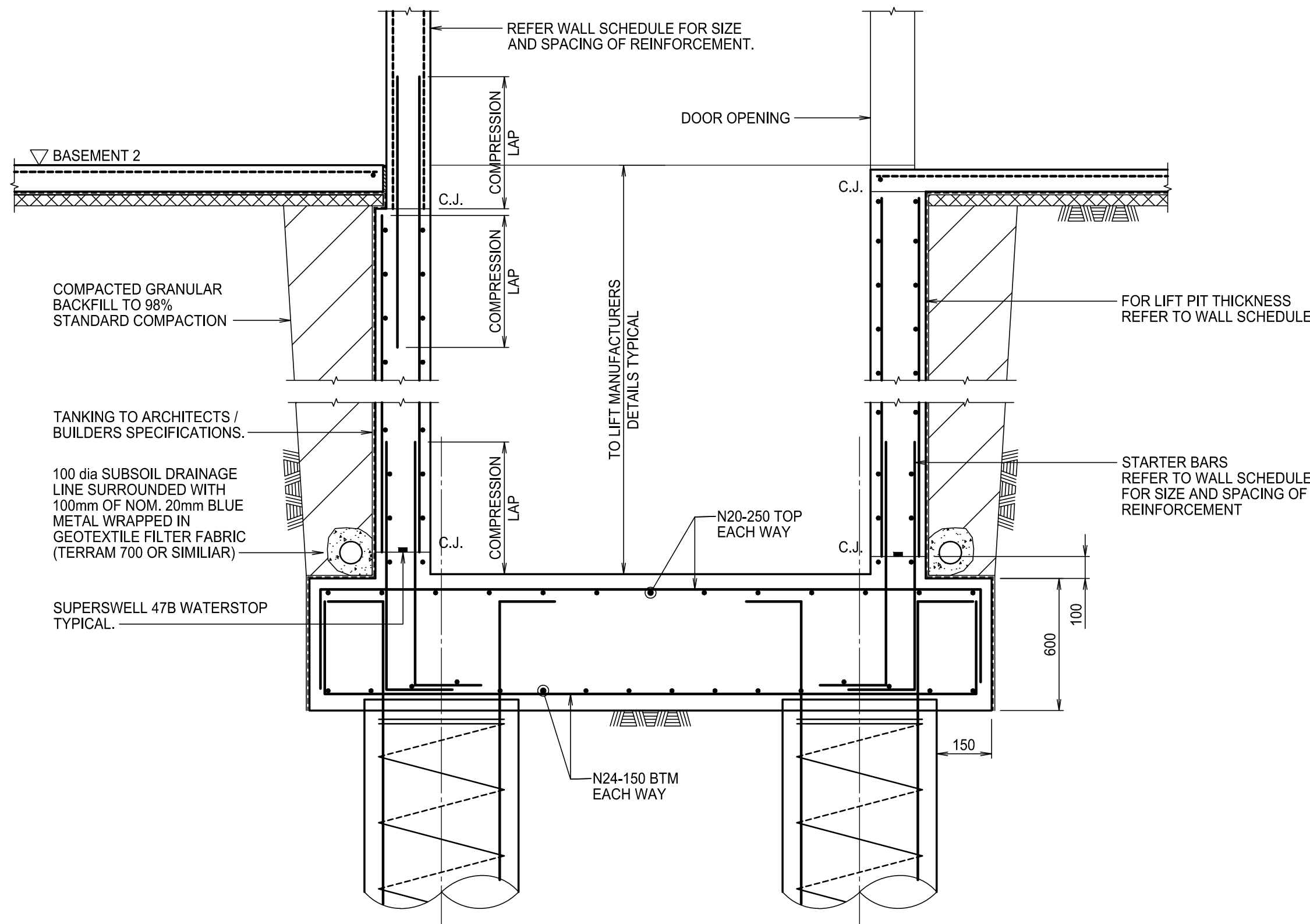
TYPICAL PILE CAP PLAN
PC2



SECTION A
SCALE 1:20



SECTION B
SCALE 1:20



TYPICAL LIFT PIT DETAIL
(PIT ON ROCK)
SCALE 1:20

PROVIDE XYPEX ADDITIVE CONCRETE
TO LIFT PIT FOOTINGS AND WALLS.

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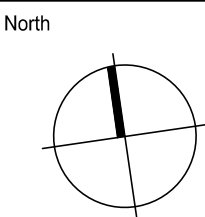
A1 0 1 2 3 4 5 6 7 8 9 10

				Architect			
1	FOR SYDNEY TRAIN APPROVAL	N.V.	K.K.	16.08.16			
Rev	Description	Eng	Draft	Date	Rev	Description	Eng

Structural and Civil Engineer

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Project
RESIDENTIAL DEVELOPMENT
No.21 ATKINSON ST,
LIVERPOOL

Sheet Subject
FOOTING DETAILS

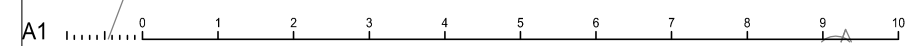
Scale at A1 1:20	Drawn KK	Approved -
Job No 160507	Drawing No S0211	Revision 1

GROUND FLOOR CONCRETE OUTLINE PLAN - 200 SLAB U.N.O.
SCALE 1:200

1. CONCRETE OUTLINES, LEVELS, SETDOWNS, REBATES, FALLS & PENETRATIONS TO ARCHITECTS DETAILS.
2. CHAMFER, FILLET AND DRIP GROOVES TO ARCHITECT DETAILS AND TO BUILDER REQUIREMENTS.
3. REFER TO HYDRAULIC ENGINEER'S DRAWINGS FOR SUMPS, PITS AND GRATED DRAINS.
4. ALL EXTERNAL & WET AREAS TO BE WATERPROOFED & INSULATED IN ACCORDANCE WITH THE ARCHITECTURAL SPECIFICATIONS
5. ALL FALLS IN EXTERNAL & WET AREAS TO BE MADE IN THE TOP OF STRUCTURAL SLAB TO HYDRAULIC OUTLETS. SOFFIT LAID FLAT ENSURE MINIMUM CONCRETE THICKNESS ARE MAINTAINED.
6. THE SOFFIT AND TOP OF STRUCTURAL SLABS ARE LAID TO FALLS. MAINTAIN MIN CONCRETE THICKNESS.

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Architect

Structural and Civil Engineer

Structural and Civil Engineer

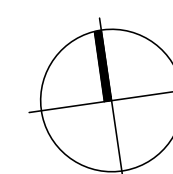
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	North
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Project
RESIDENTIAL DEVELOPMENT
No.21 ATKINSON ST,
LIVERPOOL

Sheet Subject

GROUND FLOOR
CONCRETE OUTLINE PLAN

Scale at A1

Scale at A1	Drawn KK	Approved -
Job No 160507	Drawing No S0901	Revision 1

Drawn _____ Approved _____



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Rev	Description	Eng	Draft	Date	Rev	Description	Eng	Draft	Date
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Structural and Civil Engineer

North



Sheet Subject

Scale at A1

420

Drawn

KV

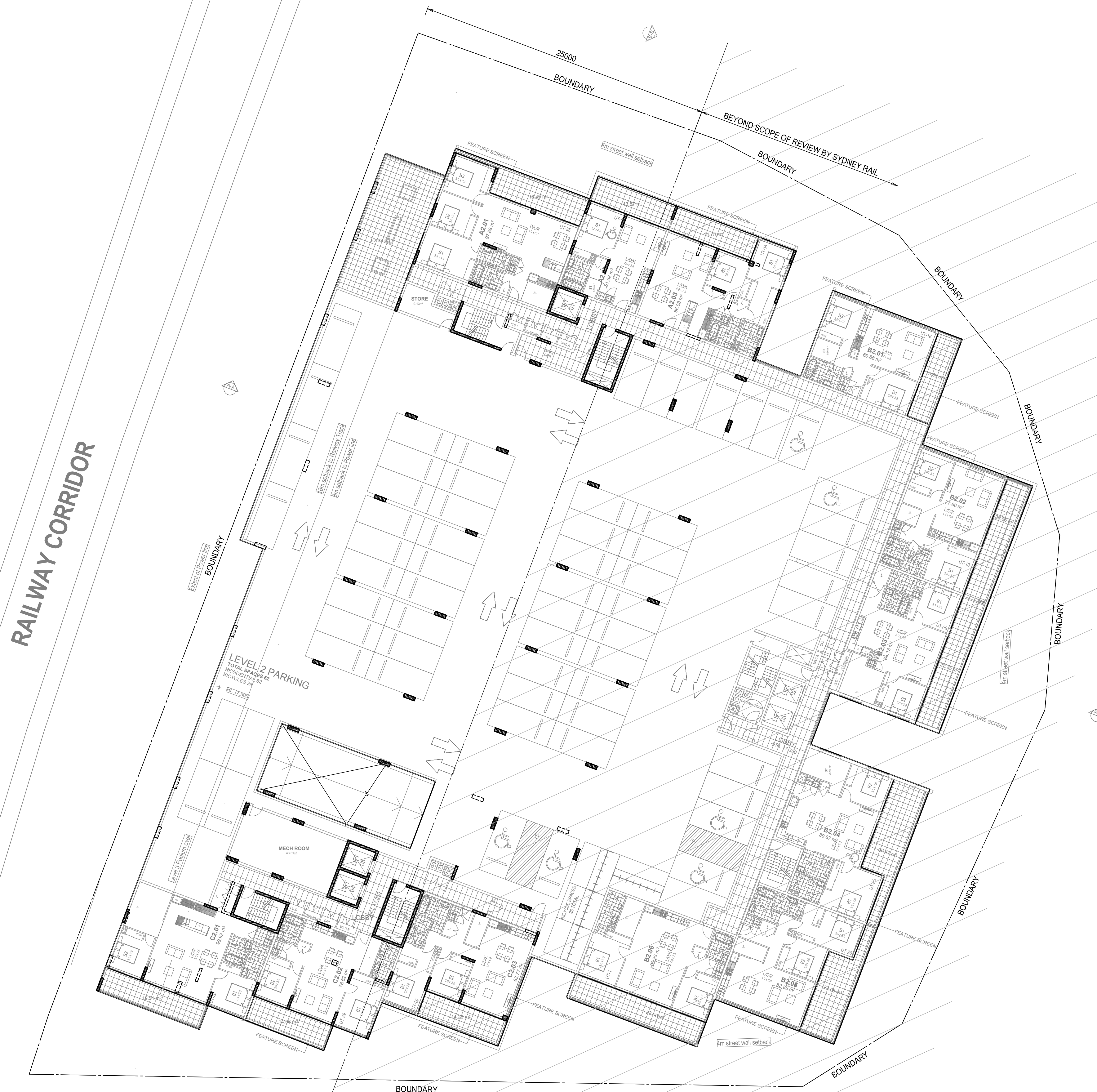
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Revision
1

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SCALE 1:200

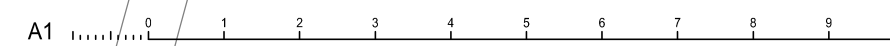
1. CONCRETE OUTLINES, LEVELS, SETDOWNS, REBATES, FALLS & PENETRATIONS TO ARCHITECTS DETAILS
2. CHAMFER, FILLET AND DRIP GROOVES TO ARCHITECT DETAILS AND TO BUILDER REQUIREMENTS.
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6. THE SOFFIT AND TOP OF STRUCTURAL SLABS ARE LAID TO FALLS. MAINTAIN MIN CONCRETE THICKNESS.



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Architect

Structural and Civil Engineering

Structural and Civil Engineer

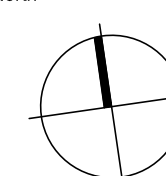
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RESIDENTIAL DEVELOPMENT
No.21 ATKINSON ST,
LIVERPOOL

Sheet Subject

SECOND FLOOR CONCRETE OUTLINE PLAN

	Scale at A1
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1:200

Job No
160507

Drawn

KK

Drawing No
S2001

Approved

Revision
1



SCALE 1:200

1. CONCRETE OUTLINES, LEVELS, SETDOWNS, REBATES, FALLS & PENETRATIONS TO ARCHITECTS DETAILS.
2. CHAMFER, FILLET AND DRIP GROOVES TO ARCHITECT DETAILS AND TO BUILDER REQUIREMENTS.
3. REFER TO HYDRAULIC ENGINEER'S DRAWINGS FOR SUMPS, PITS AND GRATED DRAINS.
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6. THE SOFFIT AND TOP OF STRUCTURAL SLABS ARE LAID TO FALLS. MAINTAIN MIN CONCRETE THICKNESS.

NOT REQUIRED FOR ASSESSMENT

A1

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Structural and Civil Engineer

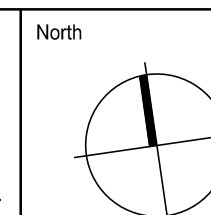
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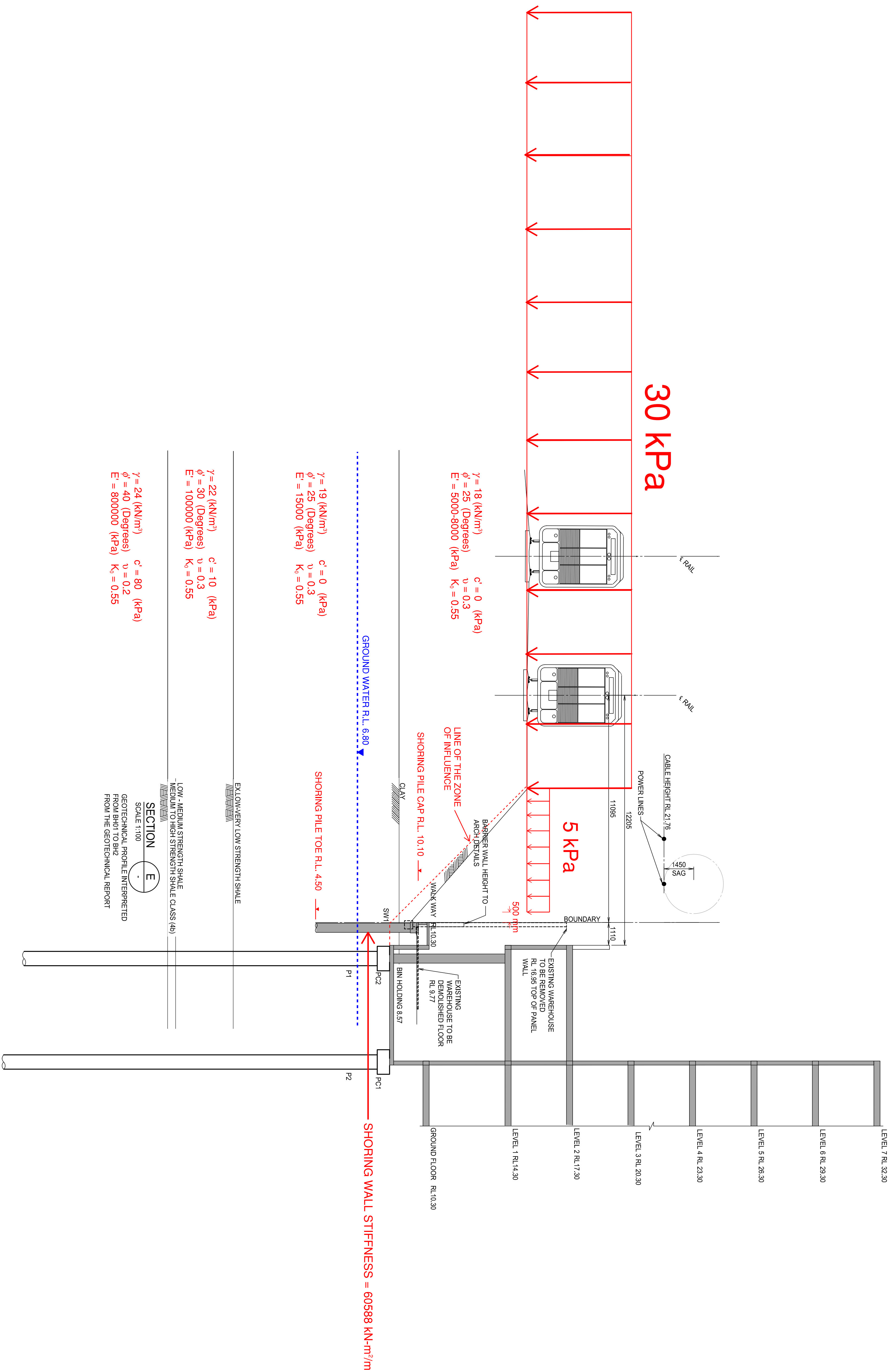


Project
RESIDENTIAL DEVELOPMENT
No.21 ATKINSON ST,
LIVERPOOL

THIRD FLOOR
CONCRETE OUTLINE PLAN

Scale at A1	Drawn	Approved
1:200	KK	-
Job No	Drawing No	Revision
160507	S3001	1

8.2. DESIGN CALCULATIONS



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A1 1 2 3 4 5 6 7 8 9									
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North									
Project									
RESIDENTIAL DEVELOPMENT NO.21 ATKINSON ST, LIVERPOOL									
Sheet Subject									
RAIL CORRIDOR SECTIONS									
Scale at A1									
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160507									
Drawing No									
S0123									
Revision									
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1 FOR STONEY TRAIN APPROVAL									
Rev Description Eng Draft Date Rev Description Eng Draft Date									

FERIS CHEHADE	Sheet No.
Program: WALLAP Version 6.05 Revision A44.B58.R48	Job No. 160507
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Data filename/Run ID: ShoringWall-450Dia.@1000mm-SectionE-Railway	Date:16-08-2016
21 Atkinson St Liverpool	Checked :
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Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types			
		Active side		Passive side	
1	10.10	1	Unit 1	1	Unit 1
2	7.30	2	Unit 3	2	Unit 3
3	0.80	4	Unit 4a	4	Unit 4a
4	-2.00	5	Unit 4b	5	Unit 4b

SOIL PROPERTIES

-- Soil type --	Bulk density	Young's Modulus	At rest coeff.	Consol state.	Active limit	Passive limit	Cohesion
No. Description (Datum elev.)	kN/m3	Eh,kN/m2 (dEh/dy)	Ko (dKo/dy)	NC/OC (Nu) (Kac)	Ka (Kpc)	Kp (dc/dy)	kN/m2
1 Unit 1	18.00	8000	0.550	NC (0.300)	0.406 (1.274)	2.464 (3.139)	0.0d
2 Unit 3	19.00	20000	0.550	OC (0.300)	0.406 (1.274)	2.464 (3.139)	0.0d
3 Not defined							
4 Unit 4a	22.00	100000	0.550	OC (0.300)	0.333 (1.155)	3.000 (3.464)	10.00d
5 Unit 4b	24.00	800000	0.550	OC (0.200)	0.217 (0.933)	4.599 (4.289)	80.00d

Additional soil parameters associated with Ka and Kp

--- parameters for Ka ---				--- parameters for Kp ---			
Soil	Wall	Back-		Soil	Wall	Back-	
friction	adhesion	fill		friction	adhesion	fill	
angle	coeff.	angle		angle	coeff.	angle	
1 Unit 1	25.00	0.000	0.00	25.00	0.000	0.00	
2 Unit 3	25.00	0.000	0.00	25.00	0.000	0.00	
3 Not defined							
4 Unit 4a	30.00	0.000	0.00	30.00	0.000	0.00	
5 Unit 4b	40.00	0.000	0.00	40.00	0.000	0.00	

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	6.80	6.80

Automatic water pressure balancing at toe of wall : No

WALL PROPERTIES

Type of structure = Soldier Pile Wall
Soldier Pile width = 0.45 m
Soldier Pile spacing = 1.00 m
Passive mobilisation factor = 3.00 m
Elevation of toe of wall = 4.50
Maximum finite element length = 0.30 m
Youngs modulus of wall E = 3.0100E+07 kN/m2
Moment of inertia of wall I = 2.0129E-03 m4/m run
E.I = 60588 kN.m2/m run
Yield Moment of wall = Not defined

SURCHARGE LOADS

Surch -arge no.	Elev.	Distance	Length	Width	Surcharge		Equiv. soil type	Partial factor/ Category
		from wall	parallel to wall	perpend. to wall	----- Near edge	kN/m2 Far edge		
1	9.60	0.50(A)	20.00	6.00	5.00	=	N/A	N/A
2	9.60	6.50(A)	20.00	50.00	30.00	=	N/A	N/A

Note: A = Active side, P = Passive side

CONSTRUCTION STAGES

Construction stage no.	Stage description
1	Excavate to elevation 9.60 on ACTIVE side Toe of berm at elevation 15.31 Width of top of berm = 0.50 Width of toe of berm = 6.50
2	Apply surcharge no.1 at elevation 9.60
3	Apply surcharge no.2 at elevation 9.60
4	Change EI of wall to 36353 kN.m2/m run Yield moment not defined Allow wall to relax with new modulus value
5	Excavate to elevation 8.37 on PASSIVE side
6	Change EI of wall to 15147 kN.m2/m run Yield moment not defined Allow wall to relax with new modulus value

FACTORS OF SAFETY and ANALYSIS OPTIONS

Stability analysis:

Method of analysis - Strength Factor method
Factor on soil strength for calculating wall depth = 1.50

Parameters for undrained strata:

Minimum equivalent fluid density = 5.00 kN/m3
Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:

Method - Subgrade reaction model using Influence Coefficients
Open Tension Crack analysis? - No
Non-linear Modulus Parameter (L) = 0 m

Boundary conditions:

Length of wall (normal to plane of analysis) = 1000.00 m

Width of excavation on active side of wall = 50.00 m

Width of excavation on passive side of wall = 50.00 m

Distance to rigid boundary on active side = 20.00 m

Distance to rigid boundary on passive side = 20.00 m

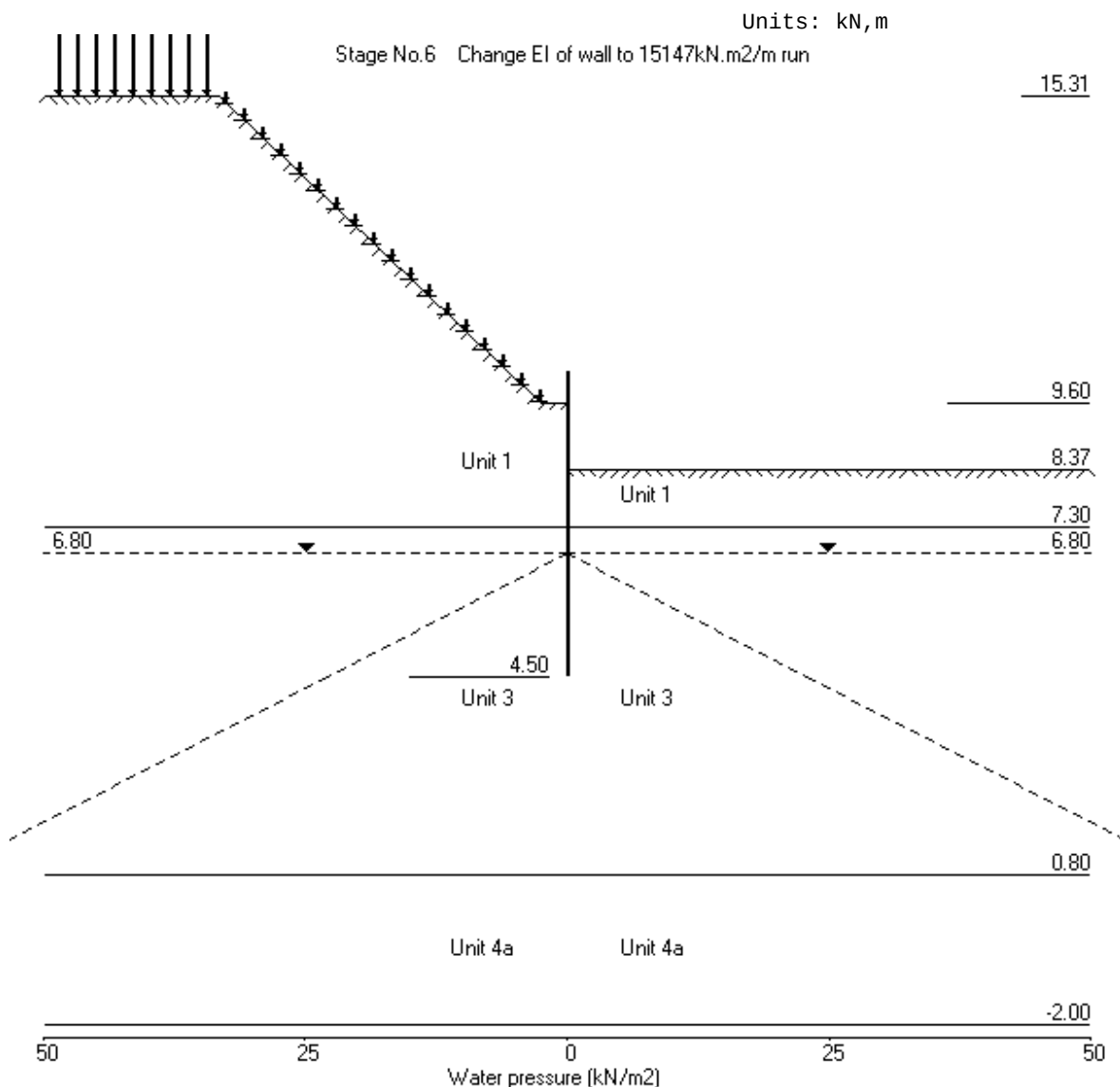
OUTPUT OPTIONS

Stage no.	----- Stage description	----- Displacement Bending mom. Shear force	----- Output options Active, Passive pressures	----- Graph. output
1	Excav. to elev. 9.60 on ACTIVE side	No	No	No
2	Apply surcharge no.1 at elev. 9.60	No	No	No
3	Apply surcharge no.2 at elev. 9.60	No	No	No
4	Change EI of wall to 36353kN.m2/m run	No	No	No
5	Excav. to elev. 8.37 on PASSIVE side	Yes	Yes	Yes
6	Change EI of wall to 15147kN.m2/m run	Yes	Yes	Yes
*	Summary output	Yes	-	Yes

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 21 Atkinson St Liverpool
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Sheet No.
 Job No. 160507
 Made by : N.V.
 Date:16-08-2016
 Checked :



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Program: WALLAP Version 6.05 Revision A44.B58.R48	Job No. 160507
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Data filename/Run ID: ShoringWall-450Dia.@1000mm-SectionE-Railway	Date:16-08-2016
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Units: kN,m

Summary of results

STABILITY ANALYSIS of Soldier Pile Wall according to Strength Factor method

Factor of safety on soil strength

				FoS for toe elev. = 4.50		Toe elev. for FoS = 1.500	
				-----		-----	
Stage	--- G.L. ---		Strut	Factor	Moment	Toe	Wall
No.	Act.	Pass.	Elev.	of Safety	equilib. at elev.	elev.	Penetr -ation
1	9.60	10.10		Wall tending to	move from	right to	left
2	9.60	10.10		Wall tending to	move from	right to	left
3	9.60	10.10		Wall tending to	move from	right to	left
4	9.60	10.10		Wall tending to	move from	right to	left
5	9.60	8.37	Cant.	1.761	4.89	5.29	3.08
6	9.60	8.37	Cant.	1.761	4.89	5.29	3.08

Note: To obtain a Factor of Safety for the case of wall failing from right to left you should reverse the data (Ctrl+K) and re-analyse.

Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall

Analysis options

Soldier Pile width = 0.45m; spacing = 1.00m
 Passive mobilisation factor = 3.000
 Length of wall perpendicular to section = 1000.00m
 Subgrade reaction model - Boussinesq Influence coefficients
 Soil deformations are elastic until the active or passive limit is reached

Rigid boundaries: Active side 20.00 from wall
 Passive side 20.00 from wall

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement		Bending moment		Shear force	
		maximum	minimum	maximum	minimum	maximum	minimum
		m	m	kN.m/m	kN.m/m	kN/m	kN/m
1	10.10	0.008	-0.002	0.0	0.0	0.0	0.0
2	9.85	0.008	-0.002	0.0	-0.0	0.0	-0.3
3	9.60	0.007	-0.002	0.0	-0.2	0.0	-1.0
4	9.30	0.007	-0.001	0.0	-0.6	0.3	-1.6
5	9.00	0.006	-0.001	0.2	-1.1	1.4	-1.5
6	8.70	0.005	-0.001	0.9	-1.5	3.3	-1.2
7	8.54	0.005	-0.001	1.5	-1.7	4.6	-1.1
8	8.37	0.005	-0.001	2.4	-1.9	6.2	-1.0
9	8.09	0.004	-0.001	4.4	-2.2	7.6	-0.9
10	7.80	0.004	-0.001	6.7	-2.4	7.0	-0.8
11	7.55	0.003	-0.001	8.3	-2.6	6.2	-0.9
12	7.30	0.003	-0.001	9.8	-2.7	5.6	-1.1
13	7.05	0.002	-0.001	10.7	-2.8	2.4	-0.3
14	6.80	0.002	-0.001	11.0	-2.7	0.6	-1.4
15	6.55	0.002	-0.001	10.5	-2.5	1.0	-3.6
16	6.30	0.002	-0.001	9.6	-2.2	1.4	-5.2
17	6.00	0.001	-0.000	7.8	-1.8	1.6	-6.5
18	5.70	0.001	-0.000	5.8	-1.3	1.6	-7.0
19	5.40	0.001	-0.000	3.7	-0.8	1.5	-6.6
20	5.10	0.001	-0.000	1.9	-0.4	1.1	-5.3
21	4.80	0.000	-0.000	0.5	-0.1	0.7	-3.1
22	4.50	0.000	-0.000	0.0	-0.0	0.0	-0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment				Shear force			
	maximum	elev.	minimum	elev.	maximum	elev.	minimum	elev.
	kN.m/m		kN.m/m		kN/m		kN/m	
1	0.0	10.10	-2.5	7.05	1.5	5.70	-1.3	9.30
2	0.0	10.10	-2.8	7.05	1.6	5.70	-1.6	9.30
3	0.0	10.10	-2.7	7.05	1.5	5.70	-1.6	9.30
4	0.0	10.10	-2.4	7.05	1.3	6.00	-1.4	9.30
5	11.0	6.80	-0.0	9.30	7.6	8.09	-7.0	5.70
6	10.5	6.80	-0.0	9.30	7.6	8.09	-6.4	5.70

- MAXIMUM DEFLECTION (D) = 8 mm $\leq \frac{L}{200} = \frac{1730}{200} = 8.65$ mm O.K.
- MAXIMUM BENDING MOMENT, M*=11.0×1.0×1.5=16.5 (kN-m)
- MAXIMUM SHEAR FORCE, V*=7.6×1.0×1.5=11.4 (kN)

Run ID. ShoringWall-450Dia.@1000mm-SectionE-Railway
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Summary of results (continued)

Maximum and minimum displacement at each stage

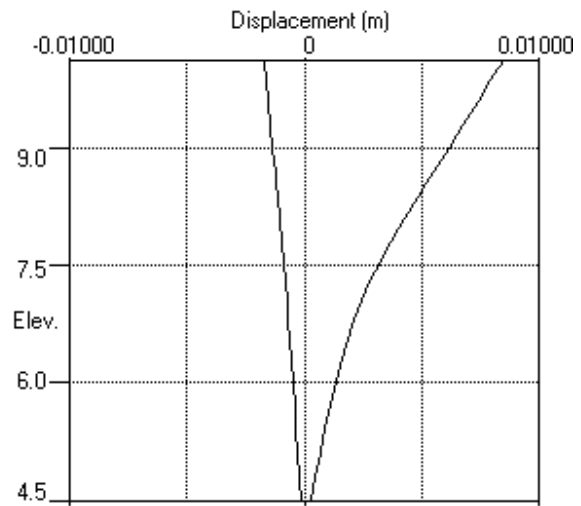
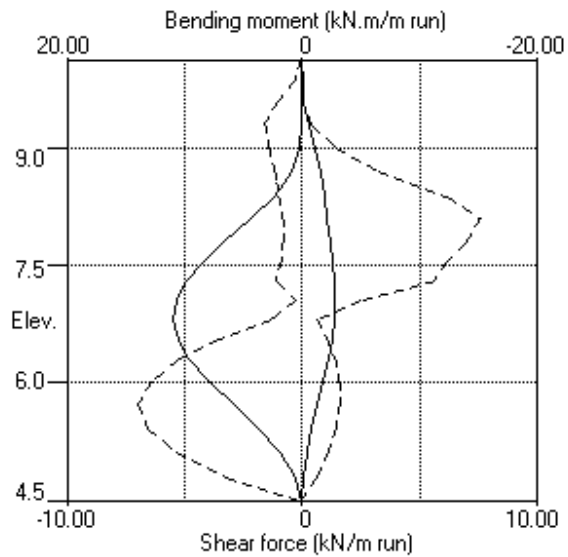
Stage no.	Displacement maximum	Displacement elev.	Displacement minimum	Displacement elev.	Stage description
	m		m		
1	0.000	10.10	-0.002	10.10	Excav. to elev. 9.60 on ACTIVE side
2	0.000	4.50	-0.001	10.10	Apply surcharge no.1 at elev. 9.60
3	0.000	4.50	-0.001	10.10	Apply surcharge no.2 at elev. 9.60
4	0.000	4.50	-0.001	10.10	Change EI of wall to 36353kN.m2/m run
5	0.007	10.10	0.000	10.10	Excav. to elev. 8.37 on PASSIVE side
6	0.008	10.10	0.000	10.10	Change EI of wall to 15147kN.m2/m run

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| Job No. 160507
| Made by : N.V.
| Date:16-08-2016
Checked :

Units: kN,m

Bending moment, shear force, displacement envelopes



Concrete Column V4.02

Xavier Knight Consulting Engineers Pty Ltd

Column: (Piles-450@1000mm-32MPa-4N20-N10-200) 450mm diameter, $f'_c=32\text{MPa}$
Reinf't: 4-N24, 75mm cover, R10 ties at 360mm cts
Capacity: $\phi N_u = 2638\text{kN}$ at min. ecc., $\phi M_{uy} = 59\text{kNm}$, $\phi M_{ux} = 0\text{kNm}$

Geometry

Concrete strength (f'_c) = 32 MPa
Diameter (cX) = 450 mm
Circle (cY > 0 for Rect.) = 0 mm

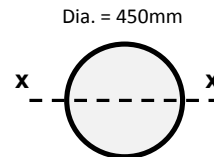
Cover to ties (cover) = 75 mm
Bar size = 24 mm
No. bars = 4 Warning

Steel yield strength (f_{sy}) = 500 MPa (Class N)
Axis distance (as) = 97.0 mm CI 5.2.2
Total number of bars = 4
Bar area (A_b) = 452 mm²
Total steel area (A_{st}) = 1809 mm²
Gross column area (A_g) = 159043 mm²
Reinforcement ratio (ρ) = 1.14 %

Formwork = S (S)standard,(R)igid
Exposure classification = (B2)* Tab 4.10.3.2 [*CI 4.3.2]

Min. ecc. ($0.05 \cdot c_X$) = 23 mm CI 10.1.2
Min. ecc. ($0.05 \cdot c_Y$) = 0 mm CI 10.1.2

Tie diameter = 10 mm
Min tie = 10 mm CI 10.7.4.3(a)
Max tie spacing = 360 mm CI 10.7.4.3(b)(i)
Clear gap (x) = 177 mm
Spacing (x) = 201 mm



α_1 = 0.850 ($0.72 \leq \alpha_1 \leq 0.85$) CI 10.6.2.2
 α_2 = 0.850 ($0.67 \leq \alpha_2 \leq 0.85$) Eq 10.6.2.5(1)
 γ = 0.826 ($0.67 \leq \gamma \leq 0.85$) Eq 10.6.2.5(2)
 $E_c = \rho^{1.5} \cdot 0.043 \cdot f_{cmi} = 28600 \text{ MPa} \pm 20\% \text{ CI 3.1.2}$

FRP = Refer CI 5.6

Capacity

About Y-Y (Axis rotated for circular bar setout)

	ϕN_{uy}	e.min	Decomp ku at bottom	Limit ku at dmaxy	Balanced ku=kuby	Ductile	ϕM_{uy}	ϕM_{udy}	
ϕN_{uy}	3109	2638	2589	1731	489	79	0	0	kN
ϕM_{uy}	0	59.3	65.9	134.9	129.2	110.5	104.7	124.1	kNm
Ecc.	0	22	25	78	265	1401	∞		mm
ku	∞	1.456	1.426	1.000	0.545	0.360	0.337	0.352	mm
							d.effy =	256.9	mm

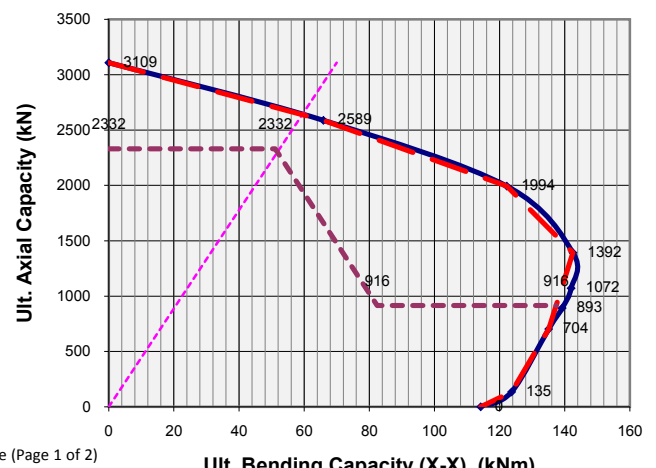
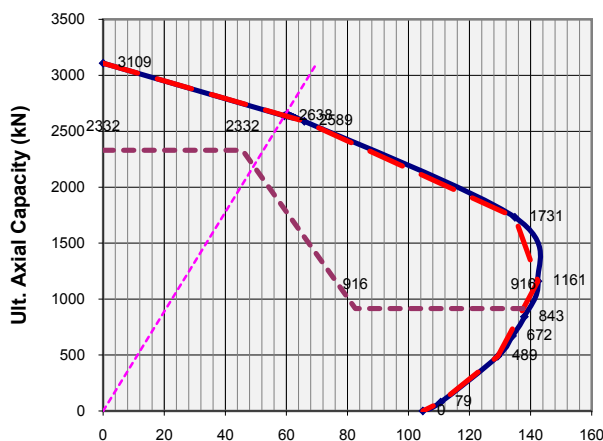
About X-X

	ϕN_{ux}	e.min	Decomp ku at bottom	Limit ku at dmaxx	Balanced ku=kuby	Ductile	ϕM_{ux}	ϕM_{udx}	
ϕN_{ux}	3109	3109	2589	1994	704	135	0	0	kN
ϕM_{ux}	0	0.0	65.9	122.2	135.1	123.7	114.2	124.1	kNm
Ecc.	0	0	25	61	192	919	∞		mm
ku	∞	7.100	1.275	1.000	0.545	0.360	0.318	0.352	mm
							d.effx =	256.9	mm

$$\phi N_u = \phi(0.6) \cdot (\alpha_1 \cdot f'_c \cdot (A_g - A_{st}) + f_{sy} \cdot A_{st}) = 3109 \text{ kN}$$

$$\phi N_{ut} = 724 \text{ kN (If all bars are anchored - Class N)}$$

Graphs



Concrete Column V4.02

Xavier Knight Consulting Engineers Pty Ltd

Capacity:	(Piles-450@1000mm-32MPa-4N20-N10-200) 450mm Diameter, $f'_c=32\text{MPa}$	
Reinf't:	4-N24, 75mm cover, R10 ties at 360mm cts	
Y-Axis:	$L_y = 5600\text{mm}$, key = 1.00, $N^* = 10\text{kN}$, $M_y^* = 0\text{kNm}$ < $\phi M_{uy} = 105\text{kNm}$	OK (0.00)
X-Axis:	$L_x = 5600\text{mm}$, key = 1.00, $N^* = 10\text{kN}$, $M_x^* = 17\text{kNm}$ < $\phi M_{ux} = 115\text{kNm}$	OK (0.14)
Biaxial:	Required ($c_x/c_y \leq 3$ but outside shaded limits)	OK (0.15)

Loading

Core confinement not required - Cl 10.7.3.1

For a short column:

Dead load ($N_{dl}=G$) =	6 kN	Max L_{ey} =	2813 mm (Braced)
Live load ($N_{ll}=Q$) =	2 kN	Max L_{ex} =	2813 mm (Braced)
$N^* = 1.2*G + 1.5*Q$ =	10 kN	Min Asc = $0.15*N^*/f_{sy}$ =	3 mm ² = 0.00% Cl 10.7.1
$G/(G+Q)$ =	0.75	$N^*/\phi N_{uo}$ =	0.003

About Y-Y (Strong axis) - Slender column - OK (0.00)

Core confinement not required - Cl 10.7.3.1

Ly =	5600 mm	ry =	113 mm	For a short column	
key =	1.00 Cl 10.5.3	Ley/ry =	49.8	Max ley/ry =	25.0 Cl 10.3.1
Ley =	5600 mm			$\alpha_c = \sqrt{1/(3.5*N*/\phi Nuo)}$ =	9.33
		Min. My* =	0.2 kNm	β_{dy} =	0.75 Cl 10.4.3
M.top* =	kNm	My* =	0.2 kNm	doy =	316 mm
M.bot* =	kNm	Ecc.y =	23 mm	ϕM_{uby} =	129 kNm
Type =	B (B)raccd, (U)nbraced			Buckling (Ncy) =	1335 kN
Transv. Load =	N (Y)es,(N)o	δ_{by} =	1.01 Eq 10.4.2		
X =	450 mm	δ_{sy} =	1.00 Cl 10.4.3	M1*/M2* =	-1.00 Single c.
Min. Ecc. =	22.5 mm	Design My* =	0 kNm	kmy =	1.00
		ϕM_{uy} =	105 kNm		

About X-X (Weak axis) - Slender column - OK (0.14)

Core confinement not required - Cl 10.7.3.1

Lx =	5600 mm	rx =	113 mm	For a short column	
kex =	1.00 Cl 10.5.3	Lex/rx =	49.8	Max lex/rx =	25.0 Cl 10.3.1
Lex =	5600 mm			$\alpha_c = \sqrt{1/(3.5*N*/\phi Nuo)}$ =	9.33
		Min. Mx* =	0.2 kNm	βdx =	0.75 Cl 10.4.3
M.top* =	16.5 kNm	Mx* =	16.5 kNm	dox =	353 mm
M.bot* =	16.5 kNm	Ecc.x =	1618 mm	$\phi Mubx$ =	135 kNm
Type =	B (B)raccd/(U)nbraced			Buckling (Ncx) =	1560 kN
Transv. Load =	N (Y)es,(N)o	δbx =	1.01 Eq 10.4.2		
Y =	450 mm	δsx =	1.00 Cl 10.4.3	M1*/M2* =	-1.00 Single c.
Min. Ecc. =	22.5 mm	Design Mx* =	17 kNm	kmx =	1.00
		ϕMux =	115 kNm		

Biaxial Check - Cl 10.6.3 - Required ($c_x/c_y \leq 3$ but outside shaded limits)

c_x/c_y =	1.00	
$\alpha_n = 1 \leq 0.7 + 1.7*N^*/\phi N_{uo} \leq 2$ =	1.00	
Biaxial ratio = $(M_x^*/\phi M_{ux})^n + (M_y^*/\phi M_{uy})^n$ =	0.15 Eq 10.6.4	OK (0.15)

Axial Shortening

Column Length =	5600 mm	
Load type =	N (N)ormal, (S)torage	
Short term LL factor (Ψ_s) =	0.7 (Normal load use)	AS/NZS 1170.0 - Table 4.1
Long term LL factor (Ψ_l) =	0.4	
Transformed area ($A_t = A_g + A_{st}*(E_s/E_c - 1)$) =	169885 mm ²	
Short term shortening ($\delta_s = (G + \Psi_s*Q)*L/(E_c*A_t)$) =	0.0 mm	
Long term shortening ($\delta_l = (G + \Psi_l*Q)*L/(E_c*A_t)$) =	0.0 mm	(Excluding creep and shrinkage)

Concrete Column V4.02

Xavier Knight Consulting Engineers Pty Ltd

Column: (Piles-450@1000mm-32MPa-4N20-N10-200) 450mm diameter, $f'_c=32\text{MPa}$

Reinf't: 4-N24, 60mm cover, R10 ties at 200mm cts

Strength: $V^* = 12.0\text{kN} < \phi V_{ux} = 232.6\text{kN}$, $\phi V_{usx} = 155.3\text{kN}$
 $\phi V_{ucx} = 77.3\text{kN}$, $\phi V_{ux.min} = 138.9\text{kN}$, $\phi V_{ux.max} = 582.7\text{kN}$
 $V^* = 12.0\text{kN} < \phi V_{uy} = 268.0\text{kN}$, $\phi V_{usy} = 175.2\text{kN}$
 $\phi V_{ucy} = 92.7\text{kN}$, $\phi V_{uy.min} = 154.4\text{kN}$, $\phi V_{uy.max} = 657.5\text{kN}$

OK (0.05)
Ligs not req'd (0.31)
OK (0.04)
Ligs not req'd (0.26)

Geometry

	About Y-Y		About X-X	
Diameter (cX) =	450 mm	Diameter (cY) =	450 mm	
doy =	326 mm	dox =	368 mm	
Effective flange (bv _x) =	399 mm	Effective flange (bv _y) =	399 mm	
Tensile steel at yield (Ast _x) =	904 mm ²	Ast _y =	1356 mm ²	
Min(0.75*cX & 500mm) =	337.5 mm	Min(0.75*cY & 500mm) =	337.5 mm	Cl 8.2.12.2

Loadings

Design shear (V^*) = 12 kN
 Compression (N^*) = kN (-ve = tension)

Reinforcement

Description =	2 legs N10-200 cts	Tie spacing (Col.) =	360 mm
Lig size =	10 mm		
Spacing (s) =	200 mm		
Legs crossing shear plane =	2		
Area of shear steel (As _v) =	157 mm ²	Ag =	159043 mm ²
Steel yield strength (f _{sy} .f) =	500 MPa		

Shear strength excluding reinforcement - Cl 8.2.7.1

Concrete strength (f'_c) =	32 MPa		
$\beta_1 = 1.1 * (1.6 - d_o/1000) \geq 1.1 =$	1.401	$\beta_1 =$	1.355
Comp./Tens. effects (β_2) =	1.000	$\beta_2 =$	1.000
$\beta_3 =$	1.000	$\beta_3 =$	1.000 Taken as constant
$f_{cv} = f'_c{}^{1/3} \leq 4\text{MPa} =$	3.175 MPa		
$V_{ucx} = \beta_1 * \beta_2 * \beta_3 * b_v * d_o * f_{cv} * (A_{st}/(b_v * d_o))^{1/3} =$	110.4 kN	$V_{ucy} =$	132.5 kN
Capacity reduction factor (ϕ) =	0.70 Table 2.2.2		Cl 8.2.7.1
$\phi V_{ucx} =$	77.3 kN	$\phi V_{ucy} =$	92.7 kN
$0.5 * \phi V_{ucx} =$	38.6 kN	$0.5 * \phi V_{ucy} =$	46.4 kN
			Cl 8.2.5

Shear reinforcement contribution - Cl 8.2.10

Angle (θ_{vx}) =	30.0 °	$\theta_{vy} =$	30.0 °	
$V_{us} = (A_{sv} * f_{sy} * f * d_o / s) * \cot(\theta_v) =$	221.8 kN	$V_{usy} =$	250.3 kN	Eq 8.2.10(1)
$\phi V_{usx} =$	155.3 kN	$\phi V_{usy} =$	175.2 kN	
Ultimate shear strength ($\phi V_{ux} = \phi V_{uc} + \phi V_{us}$) =	232.6 kN	$\phi V_{uy} =$	268.0 kN	Cl 8.2.2(a)
$V_{u.min} < V_u < V_{u.max}$		$V_{u.min} < V_u < V_{u.max}$		

Minimum Shear reinforcement - Cl 8.2.8

$As_{vx1.min} = 0.35 * b_v * s / f_{sy}.f =$	55.8 mm ²	$As_{vy1.min} =$	55.8 mm ²	
$As_{vx2.min} = 0.06 * v_f'c * b_v * s / f_{sy}.f =$	54.1 mm ²	$As_{vy2.min} =$	54.1 mm ²	
$As_{vx.min} = \text{Max}(As_{vx1.min}, As_{vx2.min}) =$	55.8 mm ²	$As_{vy.min} =$	55.8 mm ²	Cl 8.2.8
$V_{ux1.min} = V_{ucx} + 0.6 * b_v * s * d_o =$	198.5 kN	$V_{uy1.min} =$	220.6 kN	
$V_{ux2.min} = V_{ucx} + 0.10 * v_f'c * b_v * s * d_o =$	193.4 kN	$V_{uy1.min} =$	206.1 kN	
$V_{ux.min} = \text{Max}(V_{ux1.min}, V_{ux2.min}) =$	198.5 kN	$V_{uy.min} =$	220.6 kN	Cl 8.2.9
$\phi V_{ux.min} =$	138.9 kN	$\phi V_{uy.min} =$	154.4 kN	

Maximum Shear reinforcement

$As_{vx.max} = b_v * s / f_{sy} * (0.2 * f'_c - V_{uc} / (b_v * d_o)) =$	885 mm ²	$As_{vy.max} =$	877 mm ²	
$V_{ux.max} = 0.2 * f'_c * b_v * d_o =$	832.4 kN	$V_{uy.max} =$	939.3 kN	Cl 8.2.6

$\phi V_{ux.max} = 582.7 \text{ kN}$ $\phi V_{uy.max} = 657.5 \text{ kN}$

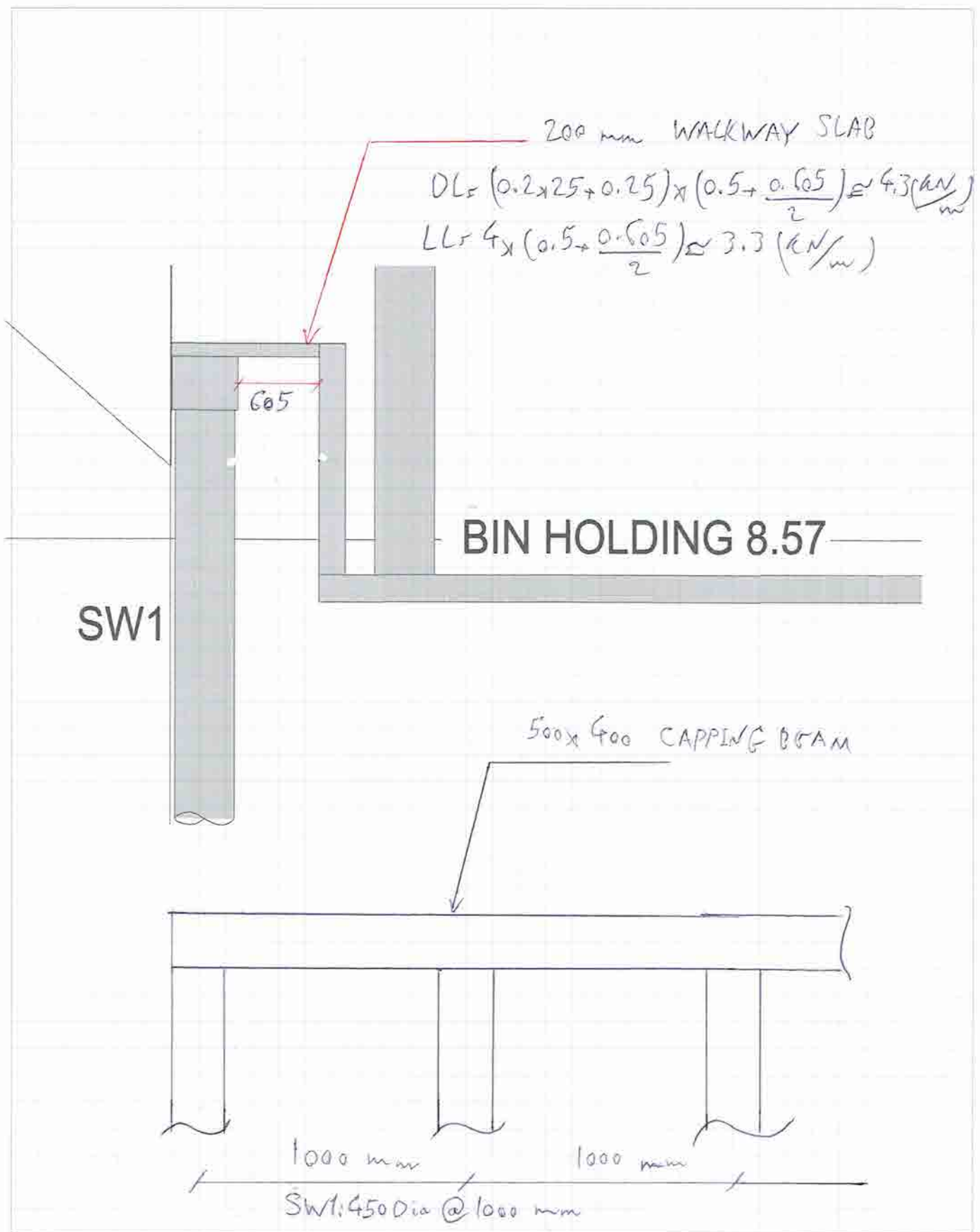
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PROJECT 21 ATKINSON ST LIVERPOOL JOB NO 160507

PAGE NO 1

SUBJECT DESIGN OF CAPPING BEAM BY N.V.

DATE 12/08/16



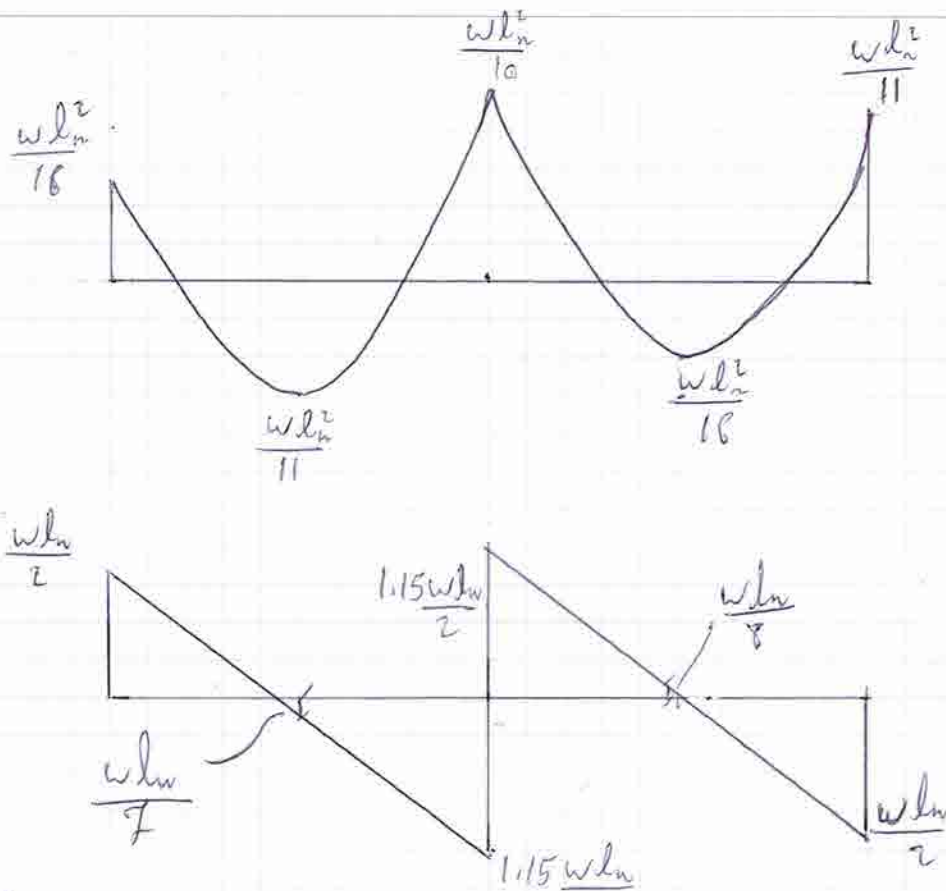
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PROJECT 2 | ATKINSON ST LIVERPOOL JOB NO 160507

PAGE NO 2

SUBJECT DESIGN OF CAPPING BEAM BY N.V.

DATE 12/08/2016



$$l_n \approx l = 2 \times \frac{D}{2} = 1000 - 2 \times \frac{450}{2} = 550 \text{ (mm)}$$

$$w_u^* = 1.2(SW + SDL) + 1.5 LL = 1.2(25 \times 0.4 \times 0.5 + 4.3) + 1.5 \times 3.3 = 16.11 \text{ (kN/m)}$$

$$M^* = \frac{w_u^* l_n^2}{10} = \frac{16.11 \times (0.55)^2}{10} = 0.5 \text{ (kN-m)}$$

$$V^* = \frac{1.15 w_u^* l_n}{2} = 1.15 \times \frac{16.11 \times 0.55}{2} = 5.1 \text{ (kN)}$$

$$M^* = 5 \text{ (kN-m)}$$

$$V^* = 10 \text{ (kN)}$$

→ DESIGN ACTIONS FOR CAPPING BEAMS

VERTICAL LOAD ON PILLS;

$$DL = 1.15 \times (25 \times 0.4 \times 0.5 + 4.3) \times \frac{1.0}{2} = 5.35 \text{ (kN)} \approx 6.0 \text{ (kN)}$$

$$LL = 1.15 \times 3.3 \times \frac{1.0}{2} = 1.90 \text{ (kN)} \approx 2.0 \text{ (kN)}$$

Concrete Member V4.07

Xavier Knight Consulting Engineers Pty Ltd

Section: (CappingBeam) 400mm (D) x 500mm (W) beam, $f'c=32\text{MPa}$
Reinf't: 4.0-N16 top, 4.0-N16 bottom, $k_u = 0.10$
Strength: (+ve M) $M^* = 5.0\text{kNm} < \phi M_{uo} = 107.5\text{kNm}$ ($\phi M_{uo,\min} = 47.0\text{kNm}$) **OK (0.05)**
Ast.req'd = 36mm^2 (0.2-N16)
Cracking: Fully enclosed where wider cracks tolerated
Ast.min: $Ast.\min = 310\text{mm}^2 < Ast = 804\text{mm}^2$ (Minimum of Deemed and actual) **OK (0.39)**

Geometry S.Wt = 5.00 kN/m L/D ratio = 2.5

Concrete strength ($f'c$) = 32 MPa
 Span (L) = 1000 mm
 Depth (D) = 400 mm
 Web width (W) = 500 mm, (S)lab
 Flange width (Bf) = 0 mm



Comp.
Tension

Side cover = 75 mm Formwork = S (S)tandard,(R)igid
 Concrete weight = 25.0 kN/m³ Exposure Top = A1 Tab 4.10.3.2
 Fully enclosed = Y (Y)es,(N)o Bottom = A2 Tab 4.10.3.2
 Gross area (Ag) = 200000 mm² Side = (B2)* **Tab 4.10.3.2 [*Cl 4.3.2]**

Design actions

Analysis values = M (M)anual, (L)eft, Position (X) from analysis, (R)ight

	Manual values		Manual	Units
Design (M^*) =	5.0 kNm	M^*	5.0	kNm
Design (M_{s1}^*) =	d kNm	M_{s1}^*	3.8	kNm
Design (M_s^*) =	d kNm	M_s^*	3.3	kNm
		Ast req'd	36	mm ²
		Ast	804	mm ²
		Reinf't req'd	0.2-N16	

Estimates for M_{s1}^* and M_s^* used - To be verified

Reinforcement

Ligs = 2 legs N12-300 cts

Bottom steel = 4.0-N16		Top steel = 4.0-N16	
Bar size =	16 mm	Bar size =	16 mm
Bar cts/No/mm ² =	4 No	Bar cts/No/mm ² =	4 No
Yield strength (fsy) =	500 MPa	Yield strength (fsyc) =	500 MPa
Bottom cover to ligs =	30 mm	Top cover to ligs =	20 mm
Steel area (Ast) =	804 mm ²	Steel area (Asc) =	804 mm ²
Ductility class =	A (N)ormal,(L)ow,(A)uto	Ductility class =	A (N)ormal,(L)ow,(A)uto
Reinf't ductility class =	N (N)ormal,(L)ow	Reinf't ductility class =	N (N)ormal,(L)ow
Depth to bottom steel layer (ds,max) =	350 mm	Depth to top steel layer =	40 mm
Depth to bottom steel (ds) =	350 mm	Depth to top steel =	40 mm
D-ds =	50 mm	D-ds =	360 mm
No. bars =	4.0 No.	No. bars =	4.0 No.
Bar centres =	103 mm	Bar centres =	103 mm
Max bars per layer =	5	Max bars per layer =	5
Layers required =	1	Max bars pers 2nd layer =	5
		Layers required =	1

Strength in +ve bending for beams - Cl 8.1

Design width (W) = 500 mm Design flange (bef) = 500 mm
 Tensile steel area (As) = 804 mm² ds = 350 mm
 Comp. steel area (Ac) = 0 mm² dc = 40 mm
 Ultimate Moment (Mu) = 134.4 kNm ku = 0.102
 Design capacity (ϕM_{uo}) = 107.5 kNm ϕ = 0.800 Table 2.2.2
 (M_{uo})min = 58.7 kNm Ast.min = 310 mm²

Crack control - Fully enclosed where wider cracks tolerated - Ignored

Steel stress (fscr) = 12 MPa Max. stress (Fscr) = 317 MPa OK (0.04)
 Steel stress (fscr1) = 14 MPa Max. stress (Fscr1) = 400 MPa OK (0.04)

Concrete Member V4.07

Xavier Knight Consulting Engineers Pty Ltd

Section: (CappingBeam) 400mm (D) x 500mm (W) beam, $f'_c=32\text{MPa}$
Reinf't: 804mm²
Strength: $V^* = 10.0\text{kN} \leq 0.5\phi V_{uc}$ - No ligs req'd
 $0.5\phi V_{uc} = 44.4\text{kN}$, $\phi V_{uc} = 88.9\text{kN}$, $\phi V_{us} = 160.0\text{kN}$, $V^*/\phi V_{uc} = 0.11$
 $\phi V_{u,\min} = 162.4\text{kN}$, $\phi V_{u,\max} = 784.0\text{kN}$

OK (0.22)

Design actions

Analysis values = M (M)anual, (L)eft, Position (X) from analysis, (R)ight
Adopt (V^*) = 10.0 kN Design (V^*) = 10.0 kN
Ast = 804 mm² Tensile steel (Ast) = 804 mm² (blank = design)
Design M* sign = 1 (1)=+ve,(-1)=-ve
Compression (N^*) = 0 kN (-ve = tension)

Reinforcement

Description = 2 legs N12-300 cts Spacing: $V^* < \phi V_{u,\min}$, 0.75D - Cl 8.2.12.2
Lig size = 12 mm $V^* > \phi V_{u,\min}$, 0.5*D = 200 mm or 300mm
Spacing (s) = 300 mm $V^* \leq \phi V_{u,\min}$, 0.75*D = 300 mm of 500mm
Legs crossing shear plane = 2 (Asv/Asv.min = 2.15) Max. cts = 300 mm
Asv = 226 mm²
Steel yield strength (f_{sy}) = 500 MPa Transverse spacing = 338 mm
Max.transverse spacing = 400 mm Cl 8.2.12.2

Shear strength excluding reinforcement - Cl 8.2.7.1

Strength reduction factor (ϕ) = 0.70 Table 2.2.2 do = ds.max = 350 mm
 $\beta_1 = 1.1 \cdot (1.6 - d_o/1000) \geq 1.1 = 1.38$ Effective flange (bv) = 500 mm
 β_1 minimum = 1.10 Tensile steel (Ast) = 804 mm²
Comp./Tens. effects (β_2) = 1.00
 $\beta_3 = 1.00$ $1.0 \leq \beta_3 \leq 2.0$ (Taken as 1.0)
 $f_{cv} = f'_c \cdot \beta_3 \leq 4\text{MPa} = 3.175\text{MPa}$
 $V_{uc} = \beta_1 \cdot \beta_2 \cdot \beta_3 \cdot b_v \cdot d_o \cdot f_{cv} \cdot (A_{st}/(b_v \cdot d_o))^{1/3} = 127.0\text{ kN}$ Cl 8.2.7.1
 $\phi V_{uc} = 88.9\text{ kN}$
 $0.5 \cdot \phi V_{uc} = 44.4\text{ kN}$ Cl 8.2.5

Shear reinforcement contribution - Cl 8.2.10

Angle (θ_v) = 30.0 ° (Perpendicular reinforcement)
 $V_{us} = (A_{sv} \cdot f_{sy} \cdot d_o / s) \cdot \cot(\theta_v) = 228.5\text{ kN}$ Eq 8.2.10(1) Min. Angle (θ_v) = 30.0 °
 $\phi V_{us} = 160.0\text{ kN}$ Other Angle (θ_v) = °
Ultimate shear strength ($\phi V_u = \phi V_{uc} + \phi V_{us}$) = 248.9 kN Cl 8.2.2(a) OK (0.04)

Legs	Asv	Cts	øVus for reinforcement (kN) for spacing (mm)					Max	mm
	mm²	300	50	100	150	200	250	300	
2	226	241.8	709.6	479.0	370.5	308.9	269.3	241.8	kN
4	452	370.5	784.0	709.6	570.2	479.0	416.2	370.5	kN
6	679	479.0	784.0	784.0	709.6	610.1	535.6	479.0	kN
8	905	570.2	784.0	784.0	784.0	709.6	632.3	570.2	kN

Minimum Shear reinforcement - Cl 8.2.8 and Cl 8.2.9

$A_{sv1,\min} = 0.35 \cdot b_v \cdot s / f_{sy} \cdot f = 105\text{ mm}^2$ Eq 8.2.8
 $A_{sv2,\min} = 0.06 \cdot \sqrt{f'_c} \cdot b_v \cdot s / f_{sy} \cdot f = 102\text{ mm}^2$
 $A_{sv,\min} = \text{Max}(A_{sv1,\min}, A_{sv2,\min}) = 105\text{ mm}^2$
 $V_{u1,\min} = V_{uc} + 0.6 \cdot b_v \cdot d_o = 232.0\text{ kN}$ Eq 8.2.9
 $V_{u2,\min} = V_{uc} + 0.10 \cdot \sqrt{f'_c} \cdot b_v \cdot d_o = 226.0$
 $V_{u,\min} = \text{Max}(V_{u1,\min}, V_{u2,\min}) = 232.0$
 $\phi V_{u,\min} = 162.4\text{ kN}$

Maximum Shear strength (Limited by crushing) - Cl 8.2.6

$V_{u,\max} = 0.2 \cdot \sqrt{f'_c} \cdot b_v \cdot d_o = 1120.0\text{ kN}$ Eq 8.2.6
 $\phi V_{u,\max} = 784.0\text{ kN}$

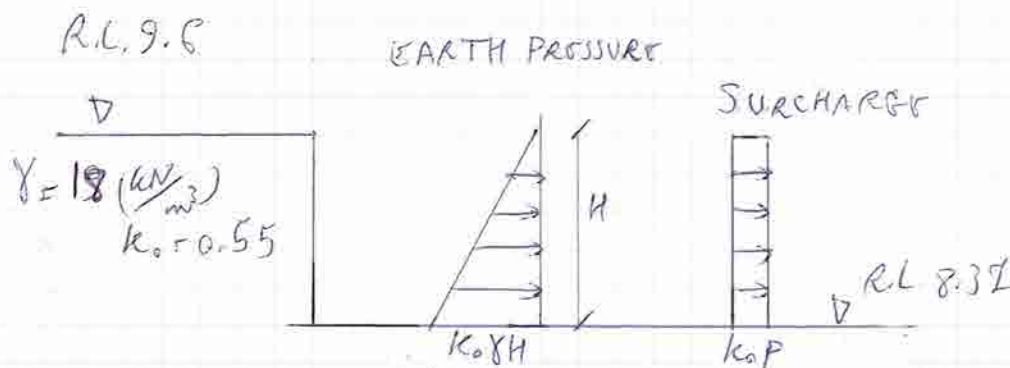
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PROJECT 21 AT KINSON ST LIVERPOOL JOB NO 160507

PAGE NO 1

SUBJECT DESIGN OF SHOTCRETE WALL BY N.V.

DATE 12/08/2016



$$\text{EARTH PRESSURE} = 0.55 \times 18 \times (9.6 - 8.37) = 12.2 \text{ (kPa)}$$

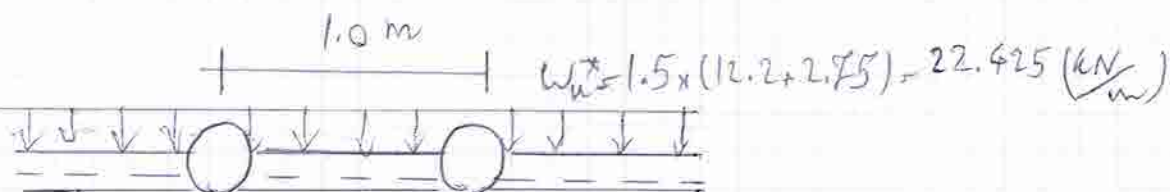
$$\text{SURCHARGE PRESSURE} = 0.55 \times 5 = 2.75 \text{ (kPa)}$$

SHOTCRETE WALL IS DESIGNED TO SPAN HORIZONTALLY BETWEEN THE SHORING PILES.

TO BE CONSERVATIVE, THE DESIGN IS BASED ON MAXIMUM PRESSURE WHICH IS AT THE 'BASE',

$$M^* = (12.2 + 2.75) \times \frac{(1.0)^2}{8} \times 1.5 = 2.81 \text{ (kN-m)} \approx 5.0 \text{ (kN-m)}$$

$$V^* = (12.2 + 2.75) \times \frac{1.0}{2} \times 1.5 = 11.22 \text{ (kN)} \approx 15.0 \text{ (kN)}$$



SHOTCRETE SPANS HORIZONTALLY BETWEEN SHORING PILES.

Concrete Member V4.07

Xavier Knight Consulting Engineers Pty Ltd

Section: (ShotcreteWall) 150mm thk slab, $f'c=25\text{MPa}$
Reinf't: SL92 (main wires in secondary) bottom, $k_u = 0.07$
Strength: (+ve M) $M^* = 5.0\text{kNm} < \phi M_{uo} = 10.4\text{kNm}$ ($\phi M_{uo.min} = 11.0\text{kNm}$) **OK (0.48)**
 $A_{st.req'd} = 138\text{mm}^2$ (N9-420)
Cracking: Fully enclosed where wider cracks tolerated
Ast.min: $A_{st.min} = 233\text{mm}^2 < A_{st} = 290\text{mm}^2$ (Minimum of Deemed and actual) **OK (0.80)**

Geometry S.Wt = 3.75 kN/m L/D ratio = 6.7

Concrete strength ($f'c$) = 25 MPa
 Span (L) = 1000 mm
 Depth (D) = 150 mm
 Web width (W) = S mm, (S)lab



Comp.
Tension

Slab type = O (O)ne way, Two way & (C)ols, (T)wo way & walls, (F)ooting
 Formwork = S (S)andard, (R)igid
 Concrete weight = 25.0 kN/m³
 Fully enclosed = Y (Y)es, (N)o
 Gross area (A_g) = 150000 mm²
 Exposure Top = (B1)* **Tab 4.10.3.2 [*Cl 4.3.2]**
 Bottom = A2 **Tab 4.10.3.2**

Design actions

Analysis values = M (M)anual, (L)eft, Position (X) from analysis, (R)ight

	Manual values		Manual	Units
Design (M^*) =	5.0 kNm/m	M^*	5.0	kNm
Design (M_{s1}^*) =	d kNm/m	M_{s1}^*	3.8	kNm
Design (M_s^*) =	d kNm/m	M_s^*	3.3	kNm
		Ast req'd	138	mm ²
		Ast	290	mm ²
		Reinf't req'd	N9-420	

Estimates for M_{s1}^* and M_s^* used - To be verified

Reinforcement

Bottom steel = SL92 (main wires in secondary)		Top steel = None	
Bar size =	8.6 mm	Bar size =	mm
Bar cts/No/mm ² =	200 mm	Bar cts/No/mm ² =	No
Yield strength (f_{sy}) =	500 MPa	Yield strength (f_{syc}) =	500 MPa
Bottom cover to steel =	30 mm	Top cover to steel =	75 mm
Steel area (A_{st}) =	290 mm ² /m	Steel area (A_{sc}) =	0 mm ² /m
Ductility class =	A (N)ormal, (L)ow, (A)uto	Ductility class =	A (N)ormal, (L)ow, (A)uto
Reinf't ductility class =	L (N)ormal, (L)ow	Reinf't ductility class =	N (N)ormal, (L)ow
Depth to bottom steel layer ($d_{s,max}$) =	116 mm	Depth to top steel =	75 mm
Depth to bottom steel (d_s) =	116 mm	Depth to top steel =	75 mm
D-ds =	34 mm	D-ds =	75 mm
No. bars =	5.0 No.	No. bars =	0.0 No.
Bar centres =	200 mm	Bar centres =	0 mm
Max bars per layer =	29	Max bars per layer =	1
Layers required =	1	Max bars pers 2nd layer =	0
		Layers required =	0

Strength in +ve bending for slabs - Cl 9.1

Design width (W) =	1000 mm	Design flange (bef) =	1000 mm
Tensile steel area (A_s) =	290 mm ² /m	ds =	116 mm
Comp. steel area (A_c) =	0 mm ² /m	dc =	75 mm
Ultimate Moment (M_u) =	16.3 kNm/m	k_u =	0.069
Design capacity (ϕM_{uo}) =	10.4 kNm/m	ϕ =	0.640 Table 2.2.2
(M_{uo})min =	13.7 kNm/m	Ast.min =	233 mm ² /m

Crack control - Fully enclosed where wider cracks tolerated - Ignored

Steel stress (f_{scr}) =	103 MPa	Max. stress (F_{scr}) =	335 MPa	OK (0.31)
Steel stress (f_{scr1}) =	119 MPa	Max. stress (F_{scr1}) =	400 MPa	OK (0.30)

Concrete Member V4.07

Xavier Knight Consulting Engineers Pty Ltd

Section: (ShotcreteWall) 150mm thk slab, $f'_c=25\text{MPa}$
Reinf't: SL92 (main wires in secondary) bottom
Strength: $V^* = 15.0\text{kN/m} < \phi V_{uc} = 48.2\text{kN/m}$

OK (0.31)

Geometry

Concrete strength (f'_c) =	25 MPa	$d_o = d_{s.\max} =$	100 mm
Depth (D) =	150 mm	Effective flange (b_v) =	1000 mm
Web width (W) =	1000 mm	Tensile steel (A_{st}) =	0 mm

Design actions

Analysis values =	M (M)anual, (L)eft, Position (X) from analysis, (R)ight
Adopt (V^*) =	15.0 kN/m
A_{st} =	290 mm ² /m
Design (V^*) =	15.0 kN/m
Tensile steel (A_{st}) =	290 mm ² (blank = design)
Design M^* sign =	-1 (1)=+ve, (-1)=-ve
Compression (N^*) =	0 kN (-ve = tension)

Shear strength excluding reinf't - Cl 8.2.7.1

Strength reduction factor (ϕ) =	0.70 Table 2.2.2
$\beta_1 = 1.1 \cdot (1.6 - d_o/1000) \geq 0.8 =$	1.65
Comp. / tension effects (β_2) =	1.00
$\beta_3 =$	1.00 Taken as constant
$f_{cv} = f'_c{}^{1/3} \leq 4\text{MPa} =$	2.924 MPa
$V_{uc} = \beta_1 \cdot \beta_2 \cdot \beta_3 \cdot b_v \cdot d_o \cdot f_{cv} \cdot (A_{st}/(b_v \cdot d_o))^{1/3} =$	68.8 kN
$\phi V_{uc} =$	48.2 kN
$0.5 \cdot \phi V_{uc} =$	24.1 kN

Eq 8.2.7.1

Concrete Column V4.02

Xavier Knight Consulting Engineers Pty Ltd

Column: (P1-600Dia-32MPa-4N28-N10-350) 600mm diameter, $f'_c=32\text{MPa}$
Reinf't: 4-N28, 75mm cover, R10 ties at 420mm cts
Capacity: $\phi N_u = 4588\text{kN}$ at min. ecc., $\phi M_{uy} = 138\text{kNm}$, $\phi M_{ux} = 0\text{kNm}$

Geometry

Concrete strength (f'_c) = 32 MPa
Diameter (cX) = 600 mm
Circle (cY > 0 for Rect.) = 0 mm

Cover to ties (cover) = 75 mm
Bar size = 28 mm
No. bars = 4 **Warning**

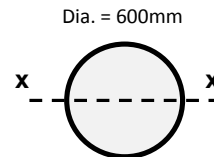
Steel yield strength (f_{sy}) = 500 MPa (Class N)
Axis distance (as) = 99.0 mm CI 5.2.2
Total number of bars = 4
Bar area (A_b) = 616 mm²
Total steel area (A_{st}) = 2463 mm²
Gross column area (A_g) = 282743 mm²
Reinforcement ratio (ρ) = 0.87 %

Warning - Ast < 1% reinf't - Refer CI 10.7.1(a)

Formwork = S (S)standard,(R)igid
Exposure classification = (B2)* **Tab 4.10.3.2 [*CI 4.3.2]**

Min. ecc. ($0.05 \cdot c_X$) = 30 mm CI 10.1.2
Min. ecc. ($0.05 \cdot c_Y$) = 0 mm CI 10.1.2

Tie diameter = 10 mm
Min tie = 10 mm CI 10.7.4.3(a)
Max tie spacing = 420 mm CI 10.7.4.3(b)(i)
Clear gap (x) = 288 mm
Spacing (x) = 316 mm



α_1 = 0.850 ($0.72 \leq \alpha_1 \leq 0.85$) CI 10.6.2.2
 α_2 = 0.850 ($0.67 \leq \alpha_2 \leq 0.85$) Eq 10.6.2.5(1)
 γ = 0.826 ($0.67 \leq \gamma \leq 0.85$) Eq 10.6.2.5(2)
 $E_c = \rho^{1.5} \cdot 0.043 \cdot f_{cmi} = 28600 \text{ MPa} \pm 20\% \text{ CI 3.1.2}$

FRP = Refer CI 5.6

Capacity

About Y-Y (Axis rotated for circular bar setout)

	ϕN_{uy}	e.min	Decomp ku at bottom	Limit ku at dmaxy	Balanced ku=kuby	Ductile	ϕM_{uy}	ϕM_{ud}	
ϕN_{uy}	5314	4588	4480	3206	1103	467	0	0	kN
ϕM_{uy}	0	137.6	157.9	310.8	316.8	267.0	206.2	124.1	kNm
Ecc.	0	30	35	97	287	573	∞		mm
ku	∞	1.394	1.357	1.000	0.545	0.360	0.273	0.352	mm
							d.effy =	256.9	mm

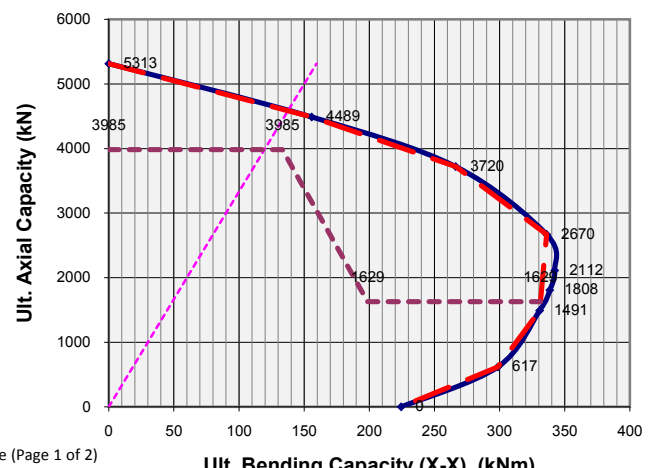
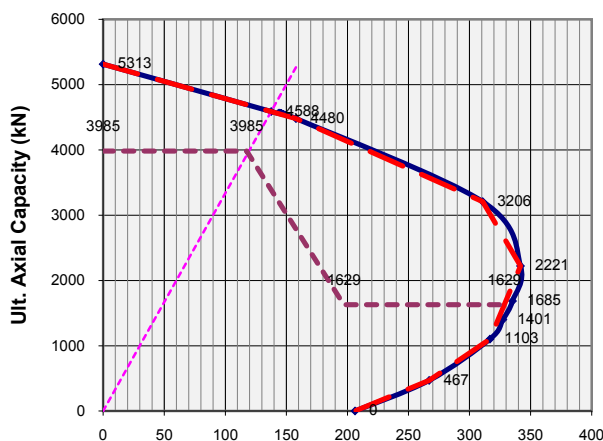
About X-X

	ϕN_{ux}	e.min	Decomp ku at bottom	Limit ku at dmaxx	Balanced ku=kuby	Ductile	ϕM_{ux}	ϕM_{ud}	
ϕN_{ux}	5314	5313	4489	3720	1491	617	0	0	kN
ϕM_{ux}	0	0.0	155.8	266.4	330.6	298.6	224.5	124.1	kNm
Ecc.	0	0	35	72	222	484	∞		mm
ku	∞	7.100	1.198	1.000	0.545	0.360	0.246	0.352	mm
							d.effx =	256.9	mm

$\phi N_{uo} = \phi(0.6) \cdot (\alpha_1 \cdot f'_c \cdot (A_g - A_{st}) + f_{sy} \cdot A_{st}) = 5313 \text{ kN}$

$\phi N_{ut} = 985 \text{ kN}$ (If all bars are anchored - Class N)

Graphs



Concrete Column V4.02

Xavier Knight Consulting Engineers Pty Ltd

Capacity: (P1-600Dia-32MPa-4N28-N10-350) 600mm Diameter, $f'_c=32\text{MPa}$
 Reinf't: 4-N28, 75mm cover, R10 ties at 420mm cts
 Y-Axis: $L_y = 4000\text{mm}$, key = 0.85, $N^* = 4349\text{kN}$, $M_y^* = 130\text{kNm}$ < $\phi M_{uy} = 142\text{kNm}$ OK (0.92)
 X-Axis: $L_x = 4000\text{mm}$, key = 0.85, $N^* = 4349\text{kN}$, $M_x^* = 130\text{kNm}$ < $\phi M_{ux} = 161\text{kNm}$ OK (0.81)
 Biaxial: Not required ($c_x/c_y \leq 3$ & within shaded limits)

Loading

Core confinement not required - Cl 10.7.3.1

For a short column:

Dead load ($N_{dl}=G$) =	2224 kN	Max L_{ey} =	3750 mm (Braced)
Live load ($N_{ll}=Q$) =	1120 kN	Max L_{ex} =	3750 mm (Braced)
$N^* = 1.2*G + 1.5*Q$ =	4349 kN	Min Asc = $0.15*N^*/f_{sy}$ =	1305 mm ² = 0.46% Cl 10.7.1
$G/(G+Q)$ =	0.67	$N^*/\phi N_{uo}$ =	0.819

About Y-Y (Strong axis) - Short column - OK (0.92)

Core confinement not required - Cl 10.7.3.1

L_y =	4000 mm	r_y =	150 mm	For a short column
key =	0.85 Cl 10.5.3	L_{ey}/r_y =	22.7	Max L_{ey}/r_y =
L_{ey} =	3400 mm			$\alpha_c = \sqrt{(2.25 - 2.5*N^*/\phi N_{uo})}$ =
		Min. M_y^* =	130.5 kNm	β_{dy} =
M_{top}^* =	kNm	M_y^* =	130.5 kNm	d_{oy} =
M_{bot}^* =	kNm	Ecc.y =	30 mm	ϕM_{uby} =
Type =	B (B)raccd, (U)nbraced			Buckling (N_{cy}) =
Transv. Load =	N (Y)es,(N)o	δ_{by} =	1.00 Eq 10.4.2	
X =	600 mm	δ_{sy} =	1.00 Cl 10.4.3	$M1^*/M2^*$ =
Min. Ecc. =	30 mm	Design M_y^* =	130 kNm	k_{my} =
		ϕM_{uy} =	142 kNm	

About X-X (Weak axis) - Short column - OK (0.81)

Core confinement not required - Cl 10.7.3.1

L_x =	4000 mm	r_x =	150 mm	For a short column
key =	0.85 Cl 10.5.3	L_{ex}/r_x =	22.7	Max L_{ex}/r_x =
L_{ex} =	3400 mm			$\alpha_c = \sqrt{(2.25 - 2.5*N^*/\phi N_{uo})}$ =
		Min. M_x^* =	130.5 kNm	β_{dx} =
M_{top}^* =	kNm	M_x^* =	130.5 kNm	d_{ox} =
M_{bot}^* =	kNm	Ecc.x =	30 mm	ϕM_{ubx} =
Type =	B (B)raccd/(U)nbraced			Buckling (N_{cx}) =
Transv. Load =	N (Y)es,(N)o	δ_{bx} =	1.00 Eq 10.4.2	
Y =	600 mm	δ_{sx} =	1.00 Cl 10.4.3	$M1^*/M2^*$ =
Min. Ecc. =	30 mm	Design M_x^* =	130 kNm	k_{mx} =
		ϕM_{ux} =	161 kNm	

Biaxial Check - Cl 10.6.3 - Not required ($c_x/c_y \leq 3$ & within shaded limits)

c_x/c_y =	1.00
$\alpha_n = 1 \leq 0.7 + 1.7*N^*/\phi N_{uo} \leq 2$ =	2.00
Biaxial ratio = $(M_x^*/\phi M_{ux})^n + (M_y^*/\phi M_{uy})^n$ =	1.50 Eq 10.6.4

Axial Shortening

Column Length =	4000 mm
Load type =	N (N)ormal, (S)torage
Short term LL factor (Ψ_s) =	0.7 (Normal load use) AS/NZS 1170.0 - Table 4.1
Long term LL factor (Ψ_l) =	0.4
Transformed area ($A_t = A_g + A_{st}*(E_s/E_c - 1)$) =	297504 mm ²
Short term shortening ($d_s = (G + \Psi_s*Q)*L/(E_c*A_t)$) =	1.4 mm
Long term shortening ($d_l = (G + \Psi_l*Q)*L/(E_c*A_t)$) =	1.3 mm (Excluding creep and shrinkage)

Concrete Column V4.02

Xavier Knight Consulting Engineers Pty Ltd

Column: (P2-750Dia-32MPa-4N28-N10-400) 750mm diameter, $f'_c=32\text{MPa}$
Reinf't: 4-N28, 75mm cover, R10 ties at 420mm cts
Capacity: $\phi N_u = 6920\text{kN}$ at min. ecc., $\phi M_{uy} = 259\text{kNm}$, $\phi M_{ux} = 0\text{kNm}$

Geometry

Concrete strength (f'_c) = 32 MPa
Diameter (cX) = 750 mm
Circle (cY > 0 for Rect.) = 0 mm

Cover to ties (cover) = 75 mm
Bar size = 28 mm
No. bars = 4 **Warning**

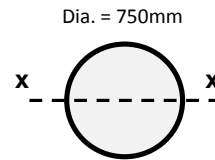
Steel yield strength (f_{sy}) = 500 MPa (Class N)
Axis distance (as) = 99.0 mm CI 5.2.2
Total number of bars = 4
Bar area (A_b) = 616 mm²
Total steel area (A_{st}) = 2463 mm²
Gross column area (A_g) = 441786 mm²
Reinforcement ratio (ρ) = 0.56 %

Warning - Ast < 1% reinf't - Refer CI 10.7.1(a)

Formwork = S (S)standard,(R)igid
Exposure classification = (B2)* **Tab 4.10.3.2 [*CI 4.3.2]**

Min. ecc. ($0.05 \cdot c_X$) = 38 mm CI 10.1.2
Min. ecc. ($0.05 \cdot c_Y$) = 0 mm CI 10.1.2

Tie diameter = 10 mm
Min tie = 10 mm CI 10.7.4.3(a)
Max tie spacing = 420 mm CI 10.7.4.3(b)(i)
Clear gap (x) = 406 mm
Spacing (x) = 434 mm



α_1 (α_1) = 0.850 ($0.72 \leq \alpha_1 \leq 0.85$) CI 10.6.2.2
 α_2 (α_2) = 0.850 ($0.67 \leq \alpha_2 \leq 0.85$) Eq 10.6.2.5(1)
 γ (γ) = 0.826 ($0.67 \leq \gamma \leq 0.85$) Eq 10.6.2.5(2)
 $E_c = \rho^{1.5} \cdot 0.043 \cdot f_{cmi} = 28600 \text{ MPa} \pm 20\% \text{ CI 3.1.2}$

FRP = Refer CI 5.6

Capacity

About Y-Y (Axis rotated for circular bar setout)

	ϕN_{uy}	e.min	Decomp ku at bottom	Limit ku at dmaxy	Balanced ku=kuby	Ductile	ϕM_{uy}	ϕM_{udy}	
ϕN_{uy}	7909	6920	6773	5050	1982	1073	0	0	kN
ϕM_{uy}	0	259.5	295.1	573.6	595.1	494.2	284.5	124.1	kNm
Ecc.	0	37	44	114	300	461	∞		mm
ku	∞	1.348	1.315	1.000	0.545	0.360	0.214	0.352	mm
							d.effy =	256.9	mm

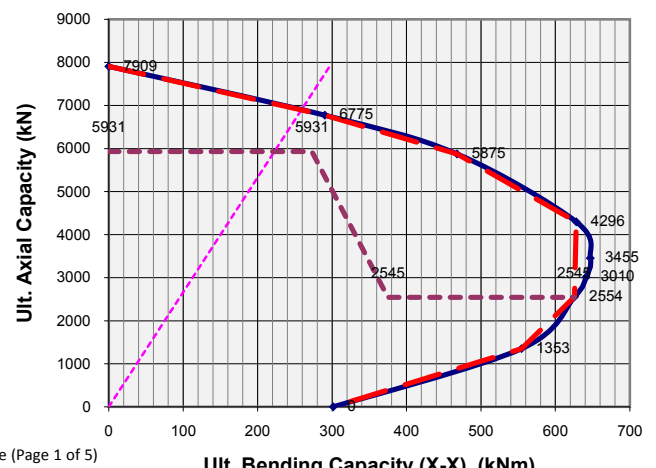
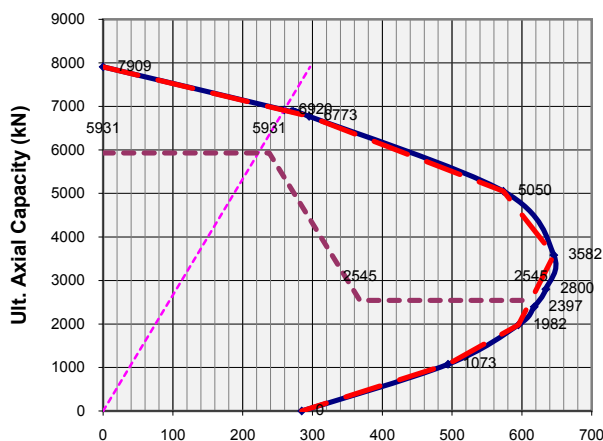
About X-X

	ϕN_{ux}	e.min	Decomp ku at bottom	Limit ku at dmaxx	Balanced ku=kuby	Ductile	ϕM_{ux}	ϕM_{udx}	
ϕN_{ux}	7909	7908	6775	5875	2554	1353	0	0	kN
ϕM_{ux}	0	0.0	290.2	467.7	625.6	554.3	301.6	124.1	kNm
Ecc.	0	0	43	80	245	410	∞		mm
ku	∞	7.100	1.152	1.000	0.545	0.360	0.175	0.352	mm
							d.effx =	256.9	mm

$$\phi N_{uo} = \phi(=0.6) \cdot (\alpha_1 \cdot f'_c \cdot (A_g - A_{st}) + f_{sy} \cdot A_{st}) = 7909 \text{ kN}$$

$$\phi N_{uot} = 985 \text{ kN (If all bars are anchored - Class N)}$$

Graphs



Concrete Column V4.02

Xavier Knight Consulting Engineers Pty Ltd

Column: (P2-750Dia-32MPa-4N28-N10-400) 750mm diameter, $f'_c=32\text{MPa}$
Reinf't: 4-N28, 75mm cover, R10 ties at 420mm cts
Capacity: $\phi N_u = 6920\text{kN}$ at min. ecc., $\phi M_{uy} = 259\text{kNm}$, $\phi M_{ux} = 0\text{kNm}$

Geometry

Concrete strength (f'_c) = 32 MPa
Diameter (cX) = 750 mm
Circle (cY > 0 for Rect.) = 0 mm

Cover to ties (cover) = 75 mm
Bar size = 28 mm
No. bars = 4 **Warning**

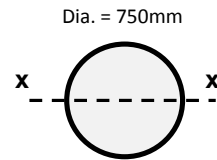
Steel yield strength (f_{sy}) = 500 MPa (Class N)
Axis distance (as) = 99.0 mm CI 5.2.2
Total number of bars = 4
Bar area (A_b) = 616 mm²
Total steel area (A_{st}) = 2463 mm²
Gross column area (A_g) = 441786 mm²
Reinforcement ratio (ρ) = 0.56 %

Warning - Ast < 1% reinf't - Refer CI 10.7.1(a)

Formwork = S (S)standard,(R)igid
Exposure classification = (B2)* **Tab 4.10.3.2 [*CI 4.3.2]**

Min. ecc. ($0.05 \cdot c_X$) = 38 mm CI 10.1.2
Min. ecc. ($0.05 \cdot c_Y$) = 0 mm CI 10.1.2

Tie diameter = 10 mm
Min tie = 10 mm CI 10.7.4.3(a)
Max tie spacing = 420 mm CI 10.7.4.3(b)(i)
Clear gap (x) = 406 mm
Spacing (x) = 434 mm



α_1 (α_1) = 0.850 ($0.72 \leq \alpha_1 \leq 0.85$) CI 10.6.2.2
 α_2 (α_2) = 0.850 ($0.67 \leq \alpha_2 \leq 0.85$) Eq 10.6.2.5(1)
 γ (γ) = 0.826 ($0.67 \leq \gamma \leq 0.85$) Eq 10.6.2.5(2)
 $E_c = \rho^{1.5} \cdot 0.043 \cdot f_{cmi} = 28600 \text{ MPa} \pm 20\% \text{ CI 3.1.2}$

FRP = Refer CI 5.6

Capacity

About Y-Y (Axis rotated for circular bar setout)

	ϕN_{uoy}	e.min	Decomp ku at bottom	Limit ku at dmaxy	Balanced ku=kuby	Ductile	ϕM_{uoy}	ϕM_{udy}	
ϕN_{uy}	7909	6920	6773	5050	1982	1073	0	0	kN
ϕM_{uy}	0	259.5	295.1	573.6	595.1	494.2	284.5	124.1	kNm
Ecc.	0	37	44	114	300	461	∞		mm
ku	∞	1.348	1.315	1.000	0.545	0.360	0.214	0.352	mm
							d.effy =	256.9	mm

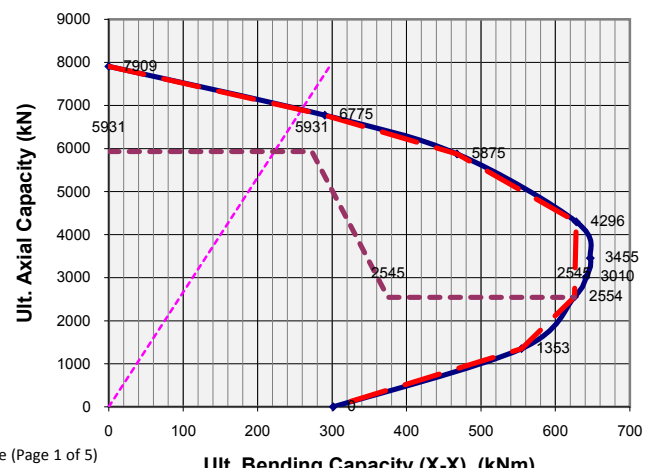
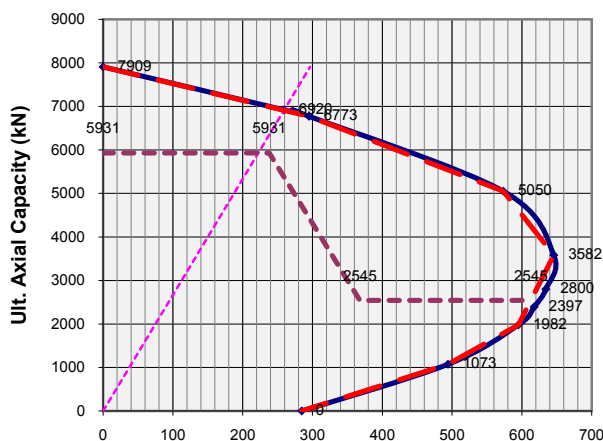
About X-X

	ϕN_{uo}	e.min	Decomp ku at bottom	Limit ku at dmaxx	Balanced ku=kuby	Ductile	ϕM_{uox}	ϕM_{udx}	
ϕN_{ux}	7909	7908	6775	5875	2554	1353	0	0	kN
ϕM_{ux}	0	0.0	290.2	467.7	625.6	554.3	301.6	124.1	kNm
Ecc.	0	0	43	80	245	410	∞		mm
ku	∞	7.100	1.152	1.000	0.545	0.360	0.175	0.352	mm
							d.effx =	256.9	mm

$$\phi N_{uo} = \phi(=0.6) \cdot (\alpha_1 \cdot f'_c \cdot (A_g - A_{st}) + f_{sy} \cdot A_{st}) = 7909 \text{ kN}$$

$$\phi N_{uot} = 985 \text{ kN (If all bars are anchored - Class N)}$$

Graphs



Concrete Column V4.02

Xavier Knight Consulting Engineers Pty Ltd

Capacity: (P2-750Dia-32MPa-4N28-N10-400) 750mm Diameter, $f'_c=32\text{MPa}$
 Reinf't: 4-N28, 75mm cover, R10 ties at 420mm cts
 Y-Axis: $L_y = 4000\text{mm}$, key = 0.85, $N^* = 6319\text{kN}$, $M_y^* = 237\text{kNm}$ < $\phi M_{uy} = 319\text{kNm}$ OK (0.74)
 X-Axis: $L_x = 4000\text{mm}$, key = 0.85, $N^* = 6319\text{kN}$, $M_x^* = 237\text{kNm}$ < $\phi M_{ux} = 366\text{kNm}$ OK (0.65)
 Biaxial: Not required ($c_x/c_y \leq 3$ & within shaded limits)

Loading

Core confinement not required - Cl 10.7.3.1

For a short column:

Dead load ($N_{dl}=G$) =	3948 kN	Max L_{ey} =	4688 mm (Braced)
Live load ($N_{ll}=Q$) =	1054 kN	Max L_{ex} =	4688 mm (Braced)
$N^* = 1.2*G + 1.5*Q$ =	6319 kN	Min Asc = $0.15*N^*/f_{sy}$ =	1896 mm ² = 0.43% Cl 10.7.1
$G/(G+Q)$ =	0.79	$N^*/\phi N_{uo}$ =	0.799

About Y-Y (Strong axis) - Short column - OK (0.74)

Core confinement not required - Cl 10.7.3.1

L_y =	4000 mm	r_y =	188 mm	For a short column
key =	0.85 Cl 10.5.3	L_{ey}/r_y =	18.1	Max L_{ey}/r_y =
L_{ey} =	3400 mm			$\alpha_c = \sqrt{(2.25 - 2.5*N^*/\phi N_{uo})}$ =
		Min. M_y^* =	236.9 kNm	β_{dy} =
M_{top}^* =	kNm	M_y^* =	236.9 kNm	d_{oy} =
M_{bot}^* =	kNm	Ecc.y =	38 mm	ϕM_{uby} =
Type =	B (B)raccd, (U)nbraced			Buckling (N_{cy}) =
Transv. Load =	N (Y)es,(N)o	δ_{by} =	1.00 Eq 10.4.2	
X =	750 mm	δ_{sy} =	1.00 Cl 10.4.3	$M1^*/M2^*$ =
Min. Ecc. =	37.5 mm	Design M_y^* =	237 kNm	k_{my} =
		ϕM_{uy} =	319 kNm	

About X-X (Weak axis) - Short column - OK (0.65)

Core confinement not required - Cl 10.7.3.1

L_x =	4000 mm	r_x =	188 mm	For a short column
key =	0.85 Cl 10.5.3	L_{ex}/r_x =	18.1	Max L_{ex}/r_x =
L_{ex} =	3400 mm			$\alpha_c = \sqrt{(2.25 - 2.5*N^*/\phi N_{uo})}$ =
		Min. M_x^* =	236.9 kNm	β_{dx} =
M_{top}^* =	kNm	M_x^* =	236.9 kNm	d_{ox} =
M_{bot}^* =	kNm	Ecc.x =	38 mm	ϕM_{ubx} =
Type =	B (B)raccd/(U)nbraced			Buckling (N_{cx}) =
Transv. Load =	N (Y)es,(N)o	δ_{bx} =	1.00 Eq 10.4.2	
Y =	750 mm	δ_{sx} =	1.00 Cl 10.4.3	$M1^*/M2^*$ =
Min. Ecc. =	37.5 mm	Design M_x^* =	237 kNm	k_{mx} =
		ϕM_{ux} =	366 kNm	

Biaxial Check - Cl 10.6.3 - Not required ($c_x/c_y \leq 3$ & within shaded limits)

c_x/c_y =	1.00
$\alpha_n = 1 \leq 0.7 + 1.7*N^*/\phi N_{uo} \leq 2$ =	2.00
Biaxial ratio = $(M_x^*/\phi M_{ux})^n + (M_y^*/\phi M_{uy})^n$ =	0.97 Eq 10.6.4

Axial Shortening

Column Length =	4000 mm
Load type =	N (N)ormal, (S)torage
Short term LL factor (Ψ_s) =	0.7 (Normal load use) AS/NZS 1170.0 - Table 4.1
Long term LL factor (Ψ_l) =	0.4
Transformed area ($A_t = A_g + A_{st}*(E_s/E_c - 1)$) =	456547 mm ²
Short term shortening ($d_s = (G + \Psi_s*Q)*L/(E_c*A_t)$) =	1.4 mm
Long term shortening ($d_l = (G + \Psi_l*Q)*L/(E_c*A_t)$) =	1.3 mm (Excluding creep and shrinkage)

>XAVIER <NIGHT

PROJECT 21 ATKINSON RD LIVERPOOL JOB NO 160502

PAGE NO 1/2

SUBJECT DESIGN OF SHORING PILES BY N.V.

DATE 12/08/2016

LOADS ON THE SHORING PILES;

UDL (FROM L1 SLAB);

TRIBUTARY LENGTH = 3.0 (m)

$$DL = (0.3 \times 25 + 1.5) \times 3.0 = 27.0 \text{ (kN/m)}$$

$$LL = 5.0 \times 3.0 = 15 \text{ (kN/m)}$$

IN ORDER TO TAKE THE EFFECTS OF THE TRANSFERRING LOADS ON THE L1 SLAB ON THE PILES, THE ABOVE LOADS ARE INCREASED BY 50% AS BELOW;

$$DL = 27 \times 1.5 = 40.5 \approx 45 \text{ (kN/m)}$$

$$LL = 15 \times 1.5 = 22.5 \approx 25 \text{ (kN/m)}$$

POINT LOADS DUE TO COLUMNS SITTING ON THE SHORING;

$$\text{TRIBUTARY AREA} = 3.0 \times 6.0 = 18.0 \text{ (m}^2\text{)} \approx 20 \text{ (m}^2\text{)}$$

$$DL = (0.5 \times 25 + 0.3 \times 25 + 2 \times 1.5) \times 20 = 460 \text{ (kN)}$$

$$LL = 2 \times 5 \times 20 = 200 \text{ (kN)}$$

ASSUMING THE LOADS OF THE COLUMNS TAKEN BY ONE PILE;

$$DL = 45.0 \times 1.0 + 460 = 505 \text{ (kN)} \Rightarrow N^* = 1.2 \times 505 + 1.5 \times 225 = 943.5 \text{ (kN)}$$

$$LL = 25.0 \times 1.0 + 200 = 225 \text{ (kN)}$$

FOR THE PILES NOT UNDER THE CONCENTRATED LOADS OF COLUMNS;

$$DL = 45 \times 1.0 = 45 \text{ (kN/m)}$$

$$LL = 25 \times 1.0 = 25 \text{ (kN/m)} \Rightarrow N^* = 1.2 \times 45 + 1.5 \times 25 = 91.5 \text{ (kN)}$$

REQUIRED LENGTH FOR THE SHORING PILES;

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

$\phi_g = 0.52$ (RECOMMENDED BY THE GEOTECHNICAL ENGINEER)

$$R_{d,ug} = f_{ms} A_s + f_b A_b$$

LAYER	R.L.		$f_{b,ULS}$	$f_{ms,ULS}$
	BH1	BH2		
4a	7.5	0.8	—	100
4b	5.4	-2	10000	400

FOR PILES UNDER CONCENTRATED LOADS;

ASSUMING 1.0 M LENGTH IN UNIT 4b

$$R_{d,ug} = 10000 \times \frac{\pi}{4} (0.45)^2 + 1 \times 400 \times \pi (0.45) + 100 \times \pi (0.45) \times 2.1 = 2452 \text{ (kN)}$$

$$R_{d,g} = \phi_g R_{d,ug} = 0.52 \times 2452 = 1275 \text{ (kN)} > N^* = 943.5 \text{ (kN)}$$

FOR PILES UNDER ONLY UDL:

ASSUMING 1.5 (m) LENGTH IN UNIT 4a

$$R_{d,g} = 0.52 \times 100 \times \pi \times 0.45 \times 1.5 = 110 \text{ (kN)} > 91.5 \text{ (kN)}$$

>XAVIER <NIGHT

PROJECT 21 ATKINSON ST LIVERPOOL JOB NO 16050Z

PAGE NO 1/3

SUBJECT DESIGN OF THE PILED FOOTING BY N.V.

DATE 12/08/2016

CALCULATION OF PRESSURE AT EACH TOWER;

a) North Tower;

Level	Slab THK (mm)	Tower's (kpa)		PODIUM		ULS = 1.2(SW + SOL) + 1.5LL	
		SD (kpa)	LL (kpa)	SD (kpa)	LL (kpa)	Tower's	PODIUM
Roof	200	1.5	1.5	—	—	10.05	—
L6	200	1.5	2.0	—	—	10.8	—
L5	200	1.5	2.0	—	—	10.8	—
L4	200	1.5	2.0	—	—	10.8	—
L3	500	1.5	2.0	0.25	2.5	19.8	19.05
L2	300	1.5	2.0	0.25	2.5	13.8	13.05
L1	300	1.5	3.0	0.25	10	15.3	24.3
GL	250	1.5	3.0	0.25	2.5	13.8	11.55

b) South Tower;

Level	Slab THK (mm)	Tower's		PODIUM		ULS = 1.2(SW + SOL) + 1.5LL	
		SD (kpa)	LL (kpa)	SD (kpa)	LL (kpa)	Tower's	PODIUM
Roof	200	1.5	1.5	—	—	10.05	—
L8	200	1.5	2.0	—	—	10.8	—
L7	200	1.5	2.0	—	—	10.8	—
L6	200	1.5	2.0	—	—	10.8	—
L5	200	1.5	2.0	—	—	10.8	—
L4	200	1.5	2.0	—	—	10.8	—
L3	500	1.5	2.0	0.25	2.5	19.8	19.05
L2	300	1.5	2.0	0.25	2.5	13.8	13.05
L1	300	1.5	2.0	0.25	10	15.3	24.3
GL	250	1.5	3.0	0.25	2.5	13.8	11.55

>XAVIER <NIGHT

PROJECT 21 ATKINSON ST LIVERPOOL JOB NO 16050Z

PAGE NO 2/3

SUBJECT DESIGN OF THE PILED FOOTING BY N.V.

DATE 12/08/2016

LOADS ON PILES:

Column below L3; $A = 8 \times 8 = 64 \text{ (m}^2\text{)} \text{ (TRIBUTARY AREA)}$

Column Above L3; $A = 6.0 \times 6.0 = 36 \text{ (m}^2\text{)} \text{ (TRIBUTARY AREA)}$

Loads on P1 pile (PILES UNDER COLUMNS IN PODIUM)

$$DL = 64 \times [(0.25 + 2 \times 0.3 + 0.5) \times 25 + 4 \times 0.25] = 2224 \text{ (kN)}$$

$$LL = 64 \times (3 \times 2.5 + 1.0) = 1120 \text{ (kN)}$$

P2 (PILES UNDER COLUMNS IN THE TOWER AREA)

North Tower;

$$DL = 64 \times (0.25 \times 25 + 2 \times 0.3 \times 25 + 0.5 \times 25 + 4 \times 1.5) + 36 \times (4 \times 0.2 \times 25 + 4 \times 1.5) = 3480 \text{ (kN)}$$

$$LL = 64 \times (2 \times 2.0 + 3 \times 3.0) + 36 \times (3 \times 2.0 + 1.5) = 910 \text{ (kN)}$$

South Tower;

$$DL = 64 \times (0.25 \times 25 + 2 \times 0.3 \times 25 + 0.5 \times 25 + 4 \times 1.5) + 36 \times (6 \times 0.2 \times 25 + 6 \times 1.5) = 3948 \text{ (kN)}$$

$$LL = 64 \times (2 \times 2.0 + 2 \times 3.0) + 36 \times (5 \times 2.0 + 1.5) = 1054 \text{ (kN)}$$

Pile	DL (kN)	LL (kN)	ULS (kN)	SOCKET LENGTH IN UNIT 46 (mm)
P1	2224	1120	4349	7
P2	3948	1054	6319	8

$R_{d,gr} = \phi_g R_{d,gr} = \phi_g (f_b A_b + f_{ms} A_s)$ (THE CALCULATION IS THE SAME AS WHAT DISPLAYED FOR SHORING PILES)

>XAVIER <NIGHT

PROJECT 21 ATKINSON ST LIVERPOOL JOB NO 160507

PAGE NO 3/3

SUBJECT DESIGN OF THE PILE FOOTING BY M.V.

DATE 12/08/2016

Loads on Stair & Lft Shafts;

Assuming tributary length of 4.0 (m)

Stair shaft = $5.0 \times 2.5 \text{ m}^2 \Rightarrow$ Tributary = $124.0 \text{ (m}^2\text{)}$

Lft shaft = $2.5 \times 2.5 \text{ m}^2 \Rightarrow$ Tributary = $104 \text{ (m}^2\text{)}$

Double Lft Shaft = $5.0 \times 2.5 \Rightarrow$ Tributary = $124.0 \text{ (m}^2\text{)}$

Level	Slab THK mm	S.O.L (kPa)	DL kPa	LL
Roof	200	1.5	6.5	1.5
L8	200	1.5	6.5	4.0
L7	200	1.5	6.5	4.0
L6	200	1.5	6.5	4.0
L5	200	1.5	6.5	4.0
L4	200	1.5	6.5	4.0
L3	500	1.5	14.0	4.0
L2	300	1.5	9.0	4.0
L1	300	1.5	9.0	4.0
GL	250	1.5	7.75	4.0
SUM			78.75	37.5

Stair Shaft & Double Lft shaft (6 Piles Under);

DL = $78.75 \times 124 = 9765 \text{ (kN)} \Rightarrow \frac{\text{DL}}{\text{Pile}} = 16275 \text{ (kN)} \Rightarrow \text{PI piles}$

LL = $37.5 \times 124 = 4650 \text{ (kN)} \Rightarrow \frac{\text{LL}}{\text{Pile}} = 775 \text{ (kN)}$

Single Lft shaft (North Tower with two level less); (4 Piles Under)

DL = $78.75 - 2 \times 6.5 = 65.75 \text{ (kPa)}$ Commercial Area on Roof

LL = $37.5 - 2 \times 4.0 - 1.5 + 4.0 = 32 \text{ (kPa)}$

DL = $65.75 \times 104 = 7046 \text{ (kN)} \Rightarrow \frac{\text{DL}}{\text{Pile}} = 1761.5 \text{ (kN)} \text{ (PI Piles)}$

LL = $32 \times 104 = 3328 \text{ (kN)} \Rightarrow \frac{\text{LL}}{\text{Pile}} = 832 \text{ (kN)}$

8.3. GEOTECHNICAL REPORT

SYNERGY DEVELOPMENT GROUP PTY LTD



GEOTECHNICAL ASSESSMENT REPORT

21 ATKINSON STREET, LIVERPOOL, NSW

Report Distribution

Geotechnical Assessment Report

21 Atkinson Street, Liverpool, NSW

El Report No. E23001 GA

Date: 17 June 2016

Copies	Recipient
1 Soft Copy (PDF – Secured, issued by email)	Mr Amen Zoabi Synergy Development Group Pty Ltd Suite 7, L3, 26 Castlereagh Street <u>LIVERPOOL NSW 2170</u>
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Revision	Details	Date	Amended By
	Original	17 June 2016	

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- 1 Site Locality Plan
- 2 Borehole Location Plan

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- A Borehole Logs and Explanatory Notes
- B Laboratory Certificates
- C Important Information

1 INTRODUCTION

1.1 BACKGROUND

At the request of Synergy Development Group Pty Ltd (Synergy), Environmental Investigations Australia Pty Ltd (EI) has carried out a Geotechnical Assessment (GA) for the proposed development at 21 Atkinson Street, Liverpool, NSW (the Site).

This GA report has been prepared to provide preliminary advice and recommendations to assist in the preparation of initial designs for the proposed development. The assessment has been carried out in accordance with the agreed scope of works outlined in EI's proposal referenced P13802.1, dated 20 April 2016.

1.2 PROPOSED DEVELOPMENT

To assist us with the preparation of this GA report, Synergy has supplied EI with:

- Architectural drawings prepared by Mosca Pserras Architects – Reference No. 15001, Drawing Nos. AP01 to AP13, dated 1 February 2016; and
- Detailed site survey prepared by Lawrence Group – Job Number 142981, Drawing No. DETL-001, Issue A, dated 14 October 2014.

Based on the above, we understand that the proposed development will involve the demolition of the existing site structures and the construction of three, eight to ten-storey mixed-use apartment buildings over a common three-level podium. The ground floor is proposed to have a finished floor level of Reduced Level (RL) 10.3m Australian Height Datum (AHD). Based on the levels shown on the survey plan, filling of about 0.5m will be required across the site to achieve the finish floor level. Local excavations will be required for footings, service trenches and lift overrun pits.

1.3 ASSESSMENT OBJECTIVES

The objective of the GA was to assess site surface and subsurface conditions, and to provide preliminary geotechnical advice and recommendations addressing the following:

- Construction considerations
- Site classification
- Earthworks
- Subgrade preparation and engineered fill
- Building foundation options, including;
 - ▶ Preliminary design parameters.
 - ▶ Earthquake loading factor in accordance with AS1170.4:2007.
- The requirement for additional geotechnical works.

1.4 SCOPE OF WORKS

The scope of works for the GA included:

- Preparation of a Work Health and Safety Plan;
- Review of relevant geological maps for the project area;
- Site walkover inspection by a Geotechnical Engineer to assess topographical features and site conditions;
- Scanning of proposed borehole locations for buried conductive services using a licensed service locator with reference to Dial Before You Dig (DBYD) plans;

- Auger drilling of two boreholes (BH1M and BH2) by a track-mounted drill rig using solid flight augers equipped with a 'Tungsten-Carbide' (T-C) bit. BH1M and BH2 were auger drilled to a depth of about 4.0m (or RL of about 5.7m AHD) and 10.8m BEGL (or RL of about -1.0m AHD), respectively. The approximate surface levels shown on the borehole logs were interpolated from spot levels shown on the supplied survey plan which formed the basis of **Figure 2**. Approximate borehole locations are shown on **Figure 2**;
- Standard Penetration Testing (SPT) was carried out during auger drilling of the boreholes to assess soil strength/relative densities. These were augmented, where possible, by hand penetrometer readings on cohesive soil samples collected in the SPT split tube sampler. Soil samples were sent to Macquarie Geotechnical Pty Ltd (Macquarie) and SGS Australia Pty Ltd (SGS), which are National Australian Testing Authority (NATA) accredited laboratories, for testing and storage.
- The strength of the shale bedrock in the augered sections of the boreholes was assessed by observation of the auger penetration resistance using a T-C drill bit, examination of the recovered rock cuttings, and rock moisture content tests. It should be noted that rock strengths assessed from augered boreholes are approximate and strength variances can be expected.
- Continuation of BH1M and BH2 using NMLC diamond coring techniques to termination depths of about 9.0m (or about RL 0.7m AHD) and 13.9m BEGL (or about RL -4.1m AHD), respectively. Rock cores recovered from the boreholes were boxed, logged, photographed and sent to Macquarie for point load strength index testing and storage. The test results are presented in **Appendix A** and **Appendix B**, and the rock core photographs are presented in **Appendix A**;
- Measurements of groundwater seepage/levels, where possible, in the augered sections of the boreholes during and shortly after completion of auger drilling.
- Installation of one PVC standpipe in BH1M to allow for long term groundwater monitoring; and
- Preparation of this GA report.

An EI Geotechnical Engineer was present on site to set out the borehole locations, direct the testing and sampling, log the subsurface conditions and record groundwater levels.

1.5 ASSESSMENT CONSTRAINTS

The GA was limited by the intent of the assessment and the presence of existing site structures covering the majority of the site. The discussions and advice presented in this report are preliminary and intended to assist in the preparation of initial designs for the proposed development. Further geotechnical investigations should be carried out prior to final design and following demolition to confirm the geotechnical and groundwater models, and the design parameters provided in this report.

2 SITE DESCRIPTION

2.1 SITE DESCRIPTION AND IDENTIFICATION

The site identification details and associated information are presented in **Table 2-1** below while the site locality is shown on **Figure 1**.

Table 2-1 Summary of Site Information

Information	Detail
Street Address	21 Atkinson Street, Liverpool, NSW 2170
Lot and Deposited Plan (DP) Identification	Lot 101 in DP 777404
Local Government Authority	Liverpool City Council
Parish	St Luke
County	Cumberland
Current Zoning	R4 – High Density Residential (Liverpool Local Environment Plan 2008)
Site Description	At the time of our assessment, the majority of the site was occupied by a two-storey glass and concrete commercial building and warehouse. The building appeared to be in good external condition upon a cursory inspection. The south and east of the building were concrete covered, which appeared to be in good condition. A grassed area located to the north of the building. A low height fill embankment was located north of the building which fell towards the south at an angle of about 10°. At the south-western corner of the site, a 4m high concrete retaining wall retained up to 3m of soil. The concrete block retaining wall appeared to be in good condition.
Site Area	The site area is approximately 5412m ² (based on the provided site survey map referenced above).

2.2 LOCAL LAND USE

The site is situated within an area of mixed use. Current uses on surrounding land are described in **Table 2-2** below.

Table 2-2 Summary of Local Land Use

Direction Relative to Site	Land Use Description
North	Shepherd Street, a two-lane asphaltic-concrete road, followed by two to four-storey brick residential buildings with no basements. The residential buildings have a setback of about 20m from the northern site boundary.
East	Shepherd Street, followed by three-storey residential brick buildings with no basements. The residential buildings have a setback of about 24m from the eastern site boundary.
South	Atkinson Street, a two-lane asphaltic-concrete road, followed by an industrial warehouse. The warehouse has a setback of about 20m from the southern site boundary. Beyond Atkinson Street in the south-eastern direction lies a construction site for an eight to ten-storey apartment building with three basement levels. At the time of the assessment, excavation was taking place and was supported by a combination of soldier pile and secant pile shoring wall systems with two rows of anchors, and shotcrete panels. The construction site has a setback of about 30m from the south-eastern site boundary.
West	Immediately west of the site, a Sydney Trains T2 and T5 rail corridor, running in a roughly north-south direction. The corridor consists of three lines and is located on a ballast embankment at a higher level than the site. The closest rail line has a setback of about 7m from the western site boundary.

2.3 REGIONAL SETTING

The site topography and geological information for the locality is summarised in **Table 2-3** below.

Table 2-3 Topographic and Geological Information

Attribute	Description
Topography	Regionally, the topography falls towards the north-northeast at an angle of about 2-3°. The site levels slopes down from RL of 13.0m AHD at the south-western corner of the site to RL 8.4m AHD at the northern corner.
Regional Geology	Information on regional sub-surface conditions, referenced from the Department of Mineral Resources Geological Map Penrith 1:100,000 Geological Series Sheet 9030 (DMR 1991) indicates the site to lie on the border of Bringelly Shale to the west and Tertiary Pliocene aged fluvial soil to the east. Bringelly Shale consists of shale, carbonaceous claystone, laminite, fine to medium grained lithic sandstone, and rare coal, while the fluvial soil consists of clayey quartzose sand, and clay.

3 ASSESSMENT RESULTS

3.1 STRATIGRAPHY

For the development of a site-specific geotechnical model, the observed stratigraphy during the GA has been grouped into four geotechnical units, with Unit 4 subdivided into two sub-units. A summary of the subsurface conditions across the site, interpreted from the assessment results, is presented in **Table 3-1** below. More detailed descriptions of subsurface conditions at each borehole location are available on the borehole logs presented in **Appendix A**. The details of the method of soil and rock classification, explanatory notes and abbreviations adopted on the borehole logs are also presented in **Appendix A**.

Table 3-1 Summary of Subsurface Conditions

Unit	Material ²	Depth to top of Unit (m BEGL) ¹	Approximate RL of top of Unit (m AHD) ¹	Observed Thickness (m)	Material Description ²	Comments
1	Fill / Topsoil	Surface	9.7 and 9.8	0.5 and 2.5	CONCRETE over FILL or Topsoil / Fill	Concrete pavement of 120mm thickness overlying silty clayey fill with gravel was observed in BH1. Silty sand topsoil with roots, rootlets, and gravel overlying silty gravel and gravelly silty clay fill with slag was observed in BH2. Based on our observations, the fill in BH1 appeared to be well compacted, while the fill in BH2 appeared to be poorly compacted. In the absence of any records that document the placement, control and compaction of the fill, the fill is considered 'uncontrolled'.
2	Residual Soil	0.5	9.2	1.7	Silty CLAY	Hard silty clay of medium plasticity grading into extremely low strength shale. Unit 2 was observed in BH1 only. SPT refusal in Unit 2, and hand penetrometer readings on the SPT samples were >600kPa.
3	Fluvial Soil	2.5	7.3	6.5	Sandy / Silty CLAY	Firm to stiff sandy clay of low plasticity. At a depth of 3.4m, became stiff to very stiff silty clay of medium to high plasticity. Unit 3 was observed in BH2 only. SPT N values of 7 to 13, and hand penetrometer readings on the SPT samples were between 100 to 350kPa.
4a	Extremely Low to Very Low Strength Shale	2.2 and 9.0	7.5 and 0.8	2.1 and 2.8	SHALE	Extremely low to very low strength, extremely to distinctly weathered shale with iron indurated bands and clay.
4b	Medium Strength Shale	4.3 and 11.8	5.4 and -2.0	4.7 and 2.1 ³	SHALE	Generally medium strength, slightly weathered to fresh shale. Defects in Unit 3c are generally moderately to widely spaced (200-2000mm), including sub-horizontal bedding partings, joints inclined up to 5°, and up to 2% decomposed and crushed seams.

Notes:

- 1 Approximate depth and level at the time of our assessment. Depths and levels may vary across the site.
- 2 For more detailed descriptions of the subsurface conditions, reference should be made to the borehole logs attached to **Appendix A**.
- 3 Observed up to termination depth in BH1M and BH2

3.2 GROUNDWATER OBSERVATIONS

No groundwater seepage was encountered in BH1M during auger drilling. Groundwater seepage was observed in BH2 at a depth of about 3.0m, or RL of about 6.8m AHD, during auger drilling. The water induced during the coring process of the boreholes precluded further observations of the groundwater levels.

However, following the completion of fieldwork, a monitoring well was installed in BH1M for further monitoring, and was bailed dry on the day of installation. The groundwater within the well was measured during fieldwork 5 hours following well installation. We note that the groundwater level may have not become stabilised during the limited time in site.

A summary of groundwater levels measured during fieldwork is presented in **Table 3-2** below.

Table 3-2 Summary of Groundwater Levels

Monitoring Well / Borehole ID	Date of Observation	Approximate Depth to Groundwater (m BEGL)	Approximate RL of Groundwater (m AHD) ¹
BH1M	16 May 2016 (5 hours following well development)	3.6	6.1

Notes:

1 Interpolated from spot levels shown on the supplied survey plan.

3.3 TEST RESULTS

Three soil samples, two rock chip samples, and one groundwater sample were selected for laboratory testing to assess the following:

- Atterberg Limits and Linear Shrinkage;
- Rock Moisture Content; and
- Soil and groundwater aggressivity (pH, Chloride and Sulfate content and electrical conductivity).

A summary of soil and groundwater test results is provided in **Table 3-3** below.

16 selected rock core samples were tested by Macquarie to estimate the Point Load Strength Index (Is_{50}) values to assist with rock strength assessment. The results of the testing are summarised on the attached borehole logs.

Laboratory test certificates are presented in **Appendix B**.

Table 3-3 Summary of Laboratory Test Results

Test/ Sample ID	BH1_3.0-3.1	BH1_3.8-4.0	BH2_7.5-7.95	BH1_1.5-1.75	BH2_3.0-3.4	BH1_GW1
Unit	4a	4a	3	2	3	-
Material Description ¹	SHALE	SHALE	Silty CLAY	Silty CLAY	Sandy CLAY	GROUNDWATER
Atterberg Limits	Liquid Limit (%)	-	-	-	22	-
	Plastic Limit (%)	-	-	-	16	-
	Plasticity Index (%)	-	-	-	6	-
	Linear Shrinkage (%)	-	-	-	2.0	-
Aggressivity	pH	-	-	8.7	8.1	6.8
	Electrical Conductivity (µS/cm)	-	-	100	58	7500
	Sulfate SO ₄ (mg/kg)	-	-	32	13	310
	Chloride Cl (mg/kg)	-	-	73	11	1900
Moisture Content (%)	6.7	6.8	19.9	14.0	19.9	-

Notes:

More detailed descriptions of the subsurface conditions at each borehole location are available on the borehole logs presented in **Appendix A**.

The Atterberg Limits and Linear Shrinkage results on Unit 3 indicated these clays to be inorganic clays and slightly reactive with moisture change (Class S).

The point load strength index tests correlated reasonably well with our field assessments of rock strength. The approximate Unconfined Compressive Strength (UCS) of the rock core, estimated from correlations with the point load strength index test results, varied from <1 MPa to 20 MPa.

The assessment indicated low permeability soils were present above and below the groundwater table. In accordance with Tables 6.4.2(C) and 6.5.2(C) of AS 2159:2009 'Piling – Design and Installation', the results of the pH, chloride and sulfate content and electrical conductivity of the soils and groundwater, provided the following exposure classifications:

- 'Non-aggressive' for buried concrete structural elements; and
- 'Non-aggressive' for buried steel structural elements.

In accordance with Table 4.8.1 of AS3600-2009 'Concrete Structures' these soils would be classified as exposure classification 'A1' for concrete in sulfate soils.

4 RECOMMENDATIONS

4.1 GEOTECHNICAL ISSUES

Based on the results of the assessment, we consider the following to be the main geotechnical issues for the proposed development:

- Varying depth to rock and quality across the site;
- Presence of deep fill on site;
- Presence of compressible firm clay;
- Foundation design for building loads; and
- Sydney Trains Corridor is located within close vicinity of the site to the west.

4.2 CONSTRUCTION CONSIDERATIONS

Should the existing concrete retaining wall be retained, a structural engineer should assess its condition and ascertain its suitability for the proposed development. Should a new retaining wall be required, further advice can be provided on the design of the retaining wall.

4.3 SITE CLASSIFICATION

The fill observed in the boreholes is deep and variable in compaction and hence, considered to be uncontrolled. Because of this fill and the presence of the compressible firm clays, the site is considered to be Class 'P' in accordance with AS2870-2011. We advise that in strict sense AS2870 site classification does not apply to this development. As a guide, reference should also be made to AS2870 for design, construction, performance criteria and maintenance precautions on Class P sites. The fill is deemed unsuitable as a bearing stratum for, at least the footings and is considered a 'moderate risk' (of poor performance) as a supporting subgrade under the floor slab. Our recommendation is that the slab is suspended on piles bearing into the natural bedrock.

4.4 EARTHWORKS

The architectural drawings provided by Synergy indicate that as part of the proposed works filling (estimated to be up to 0.5m) will be required to achieve the proposed FFL of RL 10.3m AHD. The following are some general guidelines and recommendations regarding these proposed earthworks.

4.4.1 Excavation Materials and Methods

Prior to any excavation commencing, we recommend that reference be made to the WorkCover Excavation Work Code of Practice – July 2015.

EI assumes that local excavations for footings, service trenches and lifts overrun pits may be required.

Based on the borehole logs, any local excavations may encounter Units 1, 2, and/or 3 material.

Units 1, 2, and 3 materials should be readily achievable using conventional earthmoving equipment, such as hydraulic excavators.

4.4.2 Groundwater

Groundwater seepage does not appear to be a significant issue as levels were measured within the open boreholes after completion at depths of 4.3m BGL to 5.3m BGL which are well below the proposed development.

Groundwater run-off inflows into the excavation will occur following periods of rainfall. Such inflow of water is expected to be drained through the Unit 1 sandy fill layer

4.4.3 Subgrade Preparation and Engineered Fill

If the slab is to be fully suspended on footings then it would be unnecessary to complete any particular subgrade preparation other than stripping any top layer of root affected soil in Unit 1. Alternatively, if the slab is non-suspended then we recommend that the top 1m of the existing uncontrolled fill and be fully removed and replaced

with engineered fill as recommended below. Earthworks recommendations provided in this report should be complemented by reference to AS3798.

1. Strip the upper layer of fill that contains deleterious materials or organics, and stockpile this separately since these materials are not suitable for re-use as engineered fill.
2. The remaining existing fill should be excavated down to at least 1m BGL and replaced with engineered fill. The replacement should extend to at least 2m beyond the boundaries of the floor area. Such an excavation might have to be completed with battered sides of not steeper than 1 vertical to 2 horizontal. The earthworks contractor must ensure that during the backfilling earthworks that the engineered fill is well 'keyed' into the side batters of the excavation.
3. The exposed subgrade at the base of the excavation should be proof rolled with at least 10 passes of a heavy (at least 12 tonne) smooth drum roller used in static or non-vibratory mode of operation. Caution is required when proof rolling near the outhouse south of the site. The purpose of the proof rolling is to detect any soft or heaving areas.
4. The final pass should be undertaken in the presence of a geotechnician or a geotechnical engineer, to detect any unstable areas or soft subgrade areas and to allow for some further improvement in strength/compaction
5. Unstable subgrade detected during proof rolling should be locally excavated down to a sound base and replaced with engineered fill or further advice should be sought. Any fill placed to raise site levels should also be engineered fill.

4.4.3.1 Engineered Fill Specifications

Any fill used to backfill unstable subgrade areas, raise surface levels or backfill service trenches should be engineered fill. Materials preferred for use as engineered fill are well-graded granular materials, such as ripped or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75 mm. such fill should be compacted in layers not greater than 200 mm loose thickness, to a minimum density of 98% of Standard Maximum Dry Density (SMDD).

The existing sand fill and clays at the site are acceptable for re-use on conditions that the soils used are clean (i.e. free of organics and inclusions greater than 75mm size), and free of contaminants. These soils should be compacted in maximum 200mm loose layers to the required specifications. All clayey fill should preferably be used in the lower fill layers. Thus, the use of any existing clayey fill for engineered fill will entail more rigorous earthwork supervision and compaction control, time for drying out soils and thus, possibly a greater eventual cost for earthworks.

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per material type per 2500 m² or three tests per lot or 1 test per 500 m³ distributed reasonably evenly throughout full depth and area, whichever requires the most tests. We recommend that Level 1 control of fill compaction, as defined in AS3798-2007, be adhered to on this site. Preferably, the geotechnical testing authority (GTA) should be engaged directly on behalf of the client and not by the earthworks subcontractor.

During construction of the fill, platform runoff should be enhanced by providing suitable falls to reduce ponding of water on the surface of the fill. Ponding of water may lead to softening of the fill and subsequent delays in the earthworks program.

4.5 EARTHQUAKE SITE RISK CLASSIFICATION

AS 1170.4:2007 indicates that the site is classified with an earthquake subsoil class of Class C_e (Shallow soil site). The hazard factor (z) for Sydney is 0.08.

4.6 FOUNDATIONS

Considering the presence of deep fill, firm clay which is susceptible to settlement and the expected medium to high column loads of the proposed development, it is recommended that all footings for the building and retaining walls (if any) be founded within Unit 4b shale bedrock of similar strength to provide uniform support and reduce the potential for differential settlements.

Footings founded within Unit 4b may be designed for an allowable bearing capacity of 3500kPa, based on serviceability. An allowable shaft adhesion of 10% (5% in uplift) of the allowable bearing capacity may be used. Piles will be required to reach the shale bedrock.

Bored piers are not recommended for this site due to potential difficulties resulting from collapse of the sandy fill and soft soils and the presence of the relatively shallow groundwater levels. However, the use of temporary/sacrificial liners together with tremmie methods may assist in overcoming such difficulties. If bored piers are adopted, we recommend that trial piers be drilled to assess their suitability. Our recommendation is the use of grout injected (CFA) piles.

For piles socketed into the shale bedrock, large capacity piling rigs (e.g. Soilmec SR-40 or larger) will be required. We note that the use of conventional uncased CFA rigs will have a risk of 'draw in' of soils whilst drilling rock sockets. Such 'draw in' of soils can lead to settlement adjacent to the pile which may damage adjacent structures. We recommend further advice be sought from piling contractors who should be provided with a copy of this report. Working platforms may also be required. The proposed pile locations should take into account the presence of buried services.

EI recommends the drilling of additional cored boreholes (at least three) to confirm the depth and quality of bedrock across the site, following demolition of the existing warehouse building.

Geotechnical inspections of foundations are recommended to determine that the required socket lengths have been achieved and founding material has been reached and determine any variations that may occur between the boreholes and inspected locations.

4.7 WALLAP PARAMETERS

Table 4-1 WALLAP Parameters

Unit	Material ¹	Bulk Unit Weight (kN/m ³)	Effective Cohesion, C' (kPa)	Angle of Internal Friction, Φ' (degrees)	Poisson's Ratio	Elastic Modulus E'(MPa)
1	Fill / Topsoil	18	0	25	0.3	5-8
2	Residual Soil	20	3	25	0.3	20
3	Fluvial Soil	19	0	25	0.3	15
4a	Extremely Low to Very Low Strength Shale	22	10	30	0.3	100
4b	Medium Strength Shale	24	80	40	0.2	800

Notes:

1 For more detailed descriptions of the subsurface conditions, reference should be made to the borehole logs attached to **Appendix A**.

5 RECOMMENDATIONS FOR FURTHER GEOTECHNICAL SERVICES

Below is a summary of the previously recommended additional work that needs to be carried out:

- Additional geotechnical investigation in the form of at least three cored boreholes to confirm the depth and quality of shale bedrock across the site;
- Design of working platforms (if required) for construction plant by an experienced and qualified geotechnical engineer;
- Classification of all excavated material transported off site; and
- Geotechnical inspections of foundations.

We recommend that a meeting be held after initial structural design has been completed to confirm that our recommendations have been correctly interpreted. We also recommend a meeting at the commencement of construction to discuss the primary geotechnical issues and inspection requirements.

6 STATEMENT OF LIMITATIONS

This report has been prepared for the exclusive use of Synergy Development Group Pty Ltd who is the only intended beneficiary of EI's work. The scope of the assessment carried out for the purpose of this report is limited to those agreed with Synergy Development Group Pty Ltd

No other party should rely on the document without the prior written consent of EI, and EI undertakes no duty, or accepts any responsibility or liability, to any third party who purports to rely upon this document without EI's approval.

EI has used a degree of care and skill ordinarily exercised in similar investigations by reputable members of the geotechnical industry in Australia as at the date of this document. No other warranty, expressed or implied, is made or intended. Each section of this report must be read in conjunction with the whole of this report, including its appendices and attachments.

The conclusions presented in this report are based on a limited assessment of conditions, with specific sampling and test locations chosen to be as representative as possible under the given circumstances.

EI's professional opinions are reasonable and based on its professional judgment, experience, training and results from analytical data. EI may also have relied upon information provided by the Client and other third parties to prepare this document, some of which may not have been verified by EI.

EI's professional opinions contained in this document are subject to modification if additional information is obtained through further investigation, observations, or validation testing and analysis during construction. In some cases, further testing and analysis may be required, which may result in a further report with different conclusions.

We draw your attention to the document "Important Information", which is included in **Appendix C** of this report. The statements presented in this document are intended to advise you of what your realistic expectations of this report should be. The document is not intended to reduce the level of responsibility accepted by EI, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in so doing.

Should you have any queries regarding this report, please do not hesitate to contact EI.

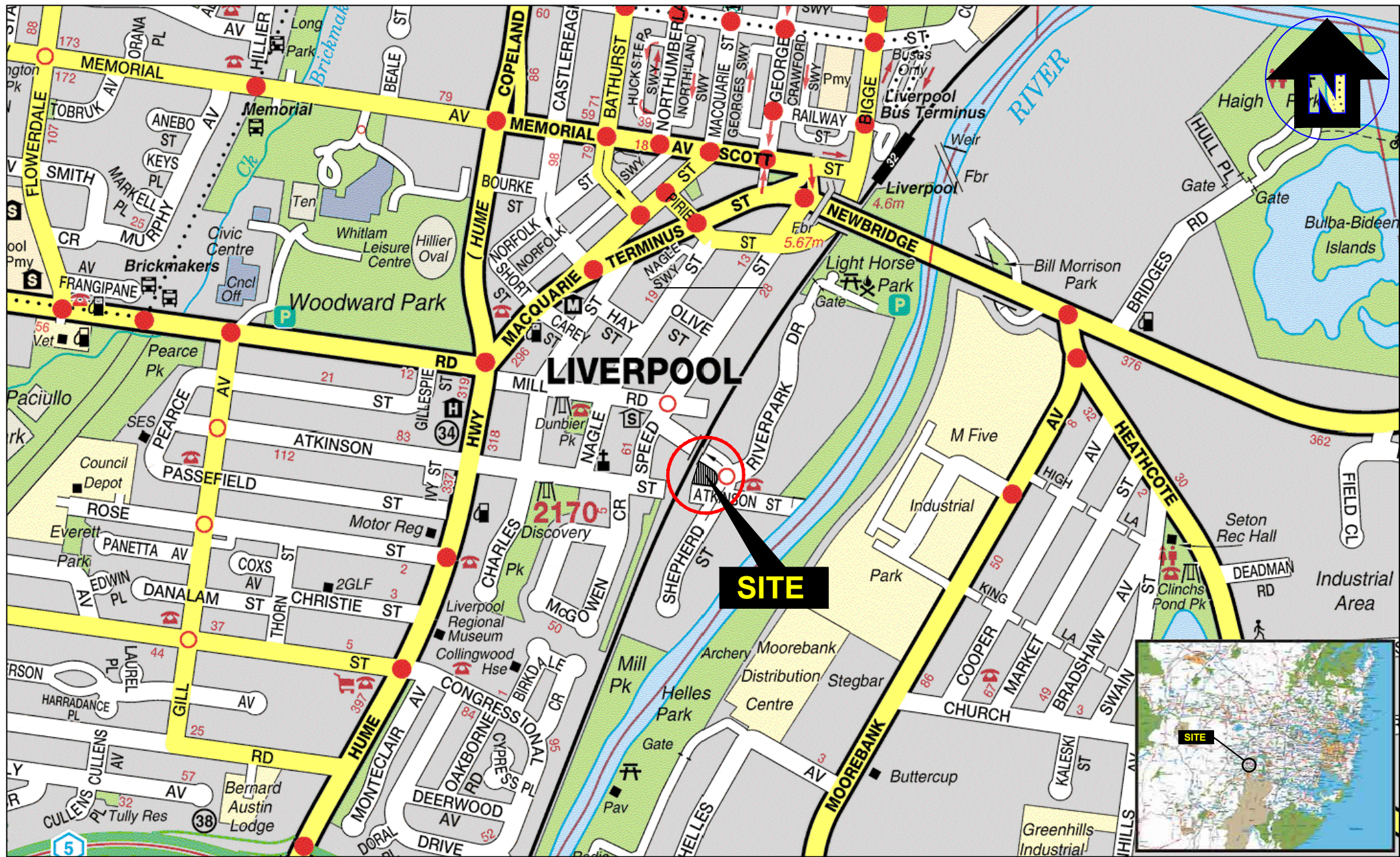
7 REFERENCES

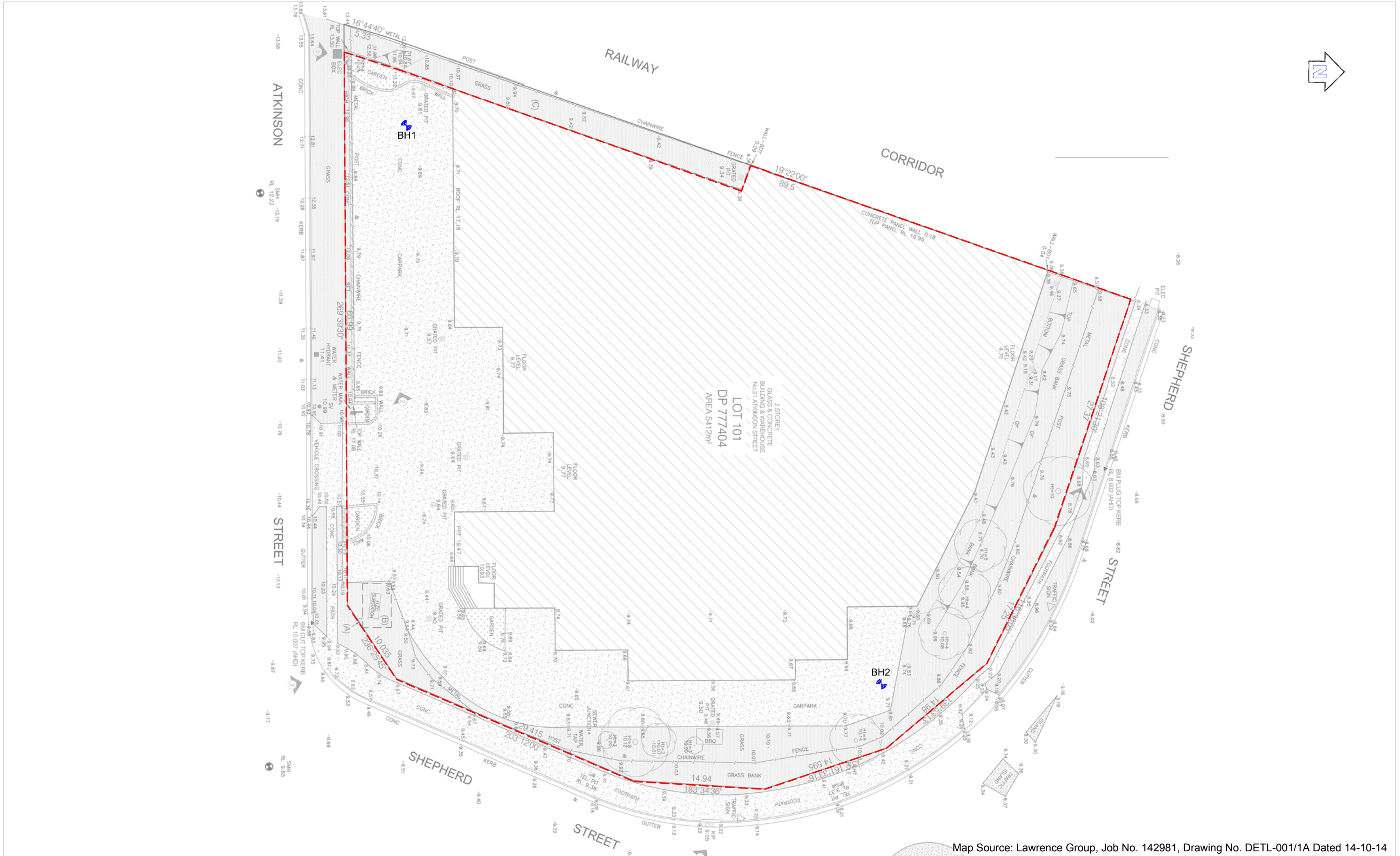
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AS1726:1993, *Geotechnical Site Investigations*, Standards Australia.
AS2159:2009, *Piling – Design and Installation*, Standards Australia.
AS3600:2009, *Concrete Structures*, Standards Australia
Excavation Work Code of Practice – July 2015 – WorkCover NSW,
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Geological Survey of New South Wales, Department of Mineral Resources.

8 ABBREVIATIONS

AHD	Australian Height Datum
AS	Australian Standard
BEL	Bulk Excavation Level
B EGL	Below Existing Ground Level
BH	Borehole
DBYD	Dial Before You Dig
DP	Deposited Plan
EI	Environmental Investigations Australia Pty Ltd
GA	Geotechnical Assessment
NATA	National Association of Testing Authorities, Australia
RL	Reduced Level
SPT	Standard Penetration Testing
T-C	Tungsten-Carbide
UCS	Unconfined Compressive Strength

FIGURES





Map Source: Lawrence Group, Job No. 142981, Drawing No. DETL-001/1A Dated 14-10-14

LEGEND  Approximate borehole location  Approximate site boundary	 Suite 6.01, 55 Miller Street, PYRMONT 2009 Ph (02) 9516 0722 Fax (02) 9518 5088	Drawn: V.L. Approved: S.K. Date: 17-06-16 Approx Scale: 1:400 @ A3 or as shown	Synergy Development Group Pty Ltd Geotechnical Assessment 21 Atkinson Street, Liverpool NSW Borehole Location Plan	Figure: 2
				Project: E23001 GA

APPENDIX A

BOREHOLE LOGS AND EXPLANATORY NOTES

BOREHOLE: BH1M

Project	Proposed Mixed-Use Development	East	307979.3 m	Sheet	1 OF 2
Location	21 Atkinson Street, Liverpool, NSW	North	6243520.9 m MGA94 Zone 56	Date Started	16/5/16
Position	Refer to Figure 2	Surface RL	9.70 m AHD	Date Completed	16/5/16
Job No.	E23001	Contractor	Rockwell Drilling Pty Ltd	Logged BY	Date: 16/5/16
Client	Synergy Development Group Pty Ltd	Drill Rig	Hanjin DB8	Checked SK	Date: 17/6/16
		Inclination	-90°		

Drilling				Sampling		Field Material Description						
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
DT			0	0.12				-	CONCRETE; 120mm thick.	-	-	CONCRETE HARDSTAND
AD/T	-	GWNE on completion of augering	9.58					-	FILL: Silty CLAY; medium to high plasticity, pale grey, brown with orange-brown to red-brown mottling, with fine to medium subangular gravel.	M (>PL)	-	FILL
			0.50	9.20	SPT 0.50-0.95 m 6, 10, 14 N=24 BH1_0.5-0.95 PP ≈300 kPa		CI	Silty CLAY; medium plasticity, pale grey-brown with orange-brown to red-brown mottling, grading into extremely low strength shale.			RESIDUAL SOIL	
			1									
			2		SPT 1.50-1.75 m 13,20/100mm HB N>20 BH1_1.5-1.75 PP >600 kPa				M (>PL)	H		
			2.20	7.50			-	SHALE; extremely weathered, extremely low strength, pale grey to grey with orange-red iron staining.			WEATHERED ROCK	
			3		SPT 3.00-3.10 m 10/100mm HB N=SPT BH1_3.0-3.1				-	-		
			4	4.00	BH1_3.8-4.0 D 3.80-4.00 m							
								Continued as Cored Borehole				

This borehole log should be read in conjunction with Environmental Investigations Australia's accompanying standard notes.

BOREHOLE: BH1M

Project	Proposed Mixed-Use Development	East	307979.3 m		
Location	21 Atkinson Street, Liverpool, NSW	North	6243520.9 m MGA94 Zone 56	Sheet	2 OF 2
Position	Refer to Figure 2	Surface RL	9.70 m AHD	Date Started	16/5/16
Job No.	E23001	Contractor	Rockwell Drilling Pty Ltd	Date Completed	16/5/16
Client	Synergy Development Group Pty Ltd	Drill Rig	Hanjin DB8	Logged BY	Date: 16/5/16
		Inclination	-90°	Checked SK	Date: 17/6/16

Drilling					Field Material Description					Defect Information				
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	DEPTH RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $I_{s(50)}$ MPa	DEFECT DESCRIPTION & Additional Observations			AVERAGE DEFECT SPACING (mm)	
				0										
				1										
				2										
				3										
				4	4.00		Continuation from non-cored borehole							
				5.70			SHALE; orange-brown, interbedded with clay.	XW						
				4.32										
				5.38			SHALE; bedding dipping 0-5 degrees, <1-5mm thick, <1-10mm average spacing, grey to dark grey with pale-grey siltstone laminations.	SW - FR		4.41: JT 0° PR RF Fe SN				
				5						4.72-4.74: DS 20mm thick				
										4.83-5.40: BPx3 0° PR RF CN avg sp = 250mm				
				6						5.68-5.70: CS 20mm thick				
				7						6.70: BP 0° PR RF CN				
										7.09: JT 0° PR RF CN				
										7.10: BP 0° PR RF CN				
				8						7.63-7.65: DS 20mm thick				
										8.38: BP 0° PR RF CN				
										8.82-8.84: DS 20mm thick				
				9	9.00		Hole Terminated at 9.00 m							
				0.70			Borehole converted to monitoring well.							
				10										

This borehole log should be read in conjunction with Environmental Investigations Australia's accompanying standard notes.

CORE PHOTOGRAPH OF BOREHOLE: BH1M

Project: Proposed Mixed Use Development
Location: 21 Atkinson Street, Liverpool, NSW
Position: Refer to Figure 2
Job No. : E23001
Client: Synergy Development Group Pty Ltd

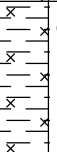
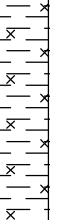
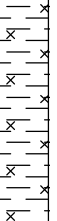
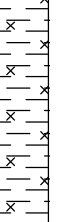
East: 307979.3 m
North: 6243520.9 m MGA94 Zone 56
Surface RL: 9.70 m AHD
Inclination: -90°
Box: 1 of 1
Hole Depth: 9.00 m

Depth Range: 4.00 m to 9.00 m
Contractor: Rockwell Drilling Pty Ltd
Drill Rig: Hanjin DB8
LOGGED BY: DATE: 16/05/16
CHECKED: SK - DATE: 17/06/16



BOREHOLE: BH2

Project	Proposed Mixed-Use Development	East	308046.9 m	Sheet	1 OF 3
Location	21 Atkinson Street, Liverpool, NSW	North	6243571.1 m MGA94 Zone 56	Date Started	16/5/16
Position	Refer to Figure 2	Surface RL	9.80 m AHD	Date Completed	16/5/16
Job No.	E23001	Contractor	Rockwell Drilling Pty Ltd	Logged BY	Date: 16/5/16
Client	Synergy Development Group Pty Ltd	Drill Rig	Hanjin DB8	Checked SK	Date: 17/6/16
		Inclination	-90°		

Drilling				Sampling		Field Material Description														
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS								
AD/T	-	▽	0	9.80	SPT 0.50-0.65 m 12 HB N=SPT BH2_0.5-0.65			-	TOPSOIL: Silty SAND; fine grained, brown, with roots and root fibres, trace fine to medium subangular gravel.	D	-	TOPSOIL								
			0.40	9.40				-	FILL: Silty GRAVEL; dark grey/black slag.			FILL								
			1	8.80				-	FILL: Gravelly Silty CLAY; low to medium plasticity, orange-brown to yellow-brown, with fine to medium gravel and trace of slag.											
			2	2.50	SPT 1.50-1.95 m 5,6,9 N=15 BH2_1.5-1.95 PP =400 kPa			M (>PL)												
			7.30	CL								Sandy CLAY; low plasticity, fine to medium grained, pale grey to yellow-brown.	M (<PL)	F - St	ALLUVIUM					
			3	3.40								SPT 3.00-3.45 m 4,4,3 N=7 BH2_3.0-3.4 PP =100 kPa BH2_3.4-3.45 PP =200 kPa			CL-CH	Silty CLAY; high plasticity, grey, yellow-grown with orage-brown mottling, with roots and root fibres.				
			4	4.50	SPT 4.50-4.95 m 3,6,7 N=13 BH2_4.5-4.95 PP =350 kPa				From 4.5m; medium plasticity, grey to yellow-brown											
			5	6.00												SPT 6.00-6.45 m 3,5,7 N=12 BH2_6.0-6.45				From 6.0m, high plasticity, grey to yellow-brown with orange-brown mottling.
			6	3.80								SPT 7.50-7.95 m 4,4,8 N=12 BH2_7.5-7.95								
			7	9.00	SPT 9.20-9.40 m 9,15/50mm HB N>15 BH2_9.2-9.4															
			8	0.80												-	SHALE; grey to pale grey with red-brown ironstaining, extremely weathered, extremely low strength.	-	-	WEATHERED ROCK
			9																	

This borehole log should be read in conjunction with Environmental Investigations Australia's accompanying standard notes.



BOREHOLE: BH2

Project	Proposed Mixed-Use Development	East	308046.9 m	Sheet	2 OF 3
Location	21 Atkinson Street, Liverpool, NSW	North	6243571.1 m MGA94 Zone 56	Date Started	16/5/16
Position	Refer to Figure 2	Surface RL	9.80 m AHD	Date Completed	16/5/16
Job No.	E23001	Contractor	Rockwell Drilling Pty Ltd	Logged BY	Date: 16/5/16
Client	Synergy Development Group Pty Ltd	Drill Rig	Hanjin DB8	Checked SK	Date: 17/6/16
		Inclination	-90°		

Drilling				Sampling		Field Material Description						
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/T	-		10					-	SHALE; grey to pale grey with red-brown ironstaining, extremely weathered, extremely low strength.	-	-	WEATHERED ROCK
			10.75									
			11						Continued as Cored Borehole			
			12									
			13									
			14									
			15									
			16									
			17									
			18									
			19									
			20									

This borehole log should be read in conjunction with Environmental Investigations Australia's accompanying standard notes.

BOREHOLE: BH2

Project	Proposed Mixed-Use Development	East	308046.9 m		
Location	21 Atkinson Street, Liverpool, NSW	North	6243571.1 m MGA94 Zone 56	Sheet	3 OF 3
Position	Refer to Figure 2	Surface RL	9.80 m AHD	Date Started	16/5/16
Job No.	E23001	Contractor	Rockwell Drilling Pty Ltd	Date Completed	16/5/16
Client	Synergy Development Group Pty Ltd	Drill Rig	Hanjin DB8	Logged BY	Date: 16/5/16
		Inclination	-90°	Checked SK	Date: 17/6/16

Drilling					Field Material Description				Defect Information			
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	DEPTH RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH Is(50) MPa	DEFECT DESCRIPTION & Additional Observations		AVERAGE DEFECT SPACING (mm)
				10								
				10.75	-0.95		Continuation from non-cored borehole					
				11			SHALE; brown to orange-red.	XW				
				11.81	-2.01							
				12			SHALE; bedding dipping 0-5 degrees, <1-6mm thick, <1-15mm average spacing, grey to dark grey with pale-grey siltstone laminations and orange iron-staining.	SW				
				12.40	-2.60		From 12.4m, no iron staining.	FR				
				13								
				13.89	-4.09		Hole Terminated at 13.89 m Backfilled with drilling spoil.					
				14								
				15								
				16								
				17								
				18								
				19								
				20								

This borehole log should be read in conjunction with Environmental Investigations Australia's accompanying standard notes.

CORE PHOTOGRAPH OF BOREHOLE: BH2

Project: Proposed Mixed Use Development
Location: 21 Atkinson Street, Liverpool, NSW
Position: Refer to Figure 2
Job No. : E23001
Client: Synergy Development Group Pty Ltd

East: 308046.9 m
North: 6243571.1 m MGA94 Zone 56
Surface RL: 9.80 m AHD
Inclination: -90°
Box: 1 of 1
Hole Depth: 13.89 m

Depth Range: 10.75 m to 13.89 m
Contractor: Rockwell Drilling Pty Ltd
Drill Rig: Hanjin DB8
LOGGED: BY DATE: 16/05/16
CHECKED: SK - DATE: 17/06/16



EXPLANATION OF NOTES, ABBREVIATIONS & TERMS USED ON BOREHOLE AND TEST PIT LOGS

DRILLING/EXCAVATION METHOD

HA	Hand Auger	RD	Rotary blade or drag bit	NQ	Diamond Core - 47 mm
DTC	Diatube Coring	RT	Rotary Tricone bit	NMLC	Diamond Core - 52 mm
NDD	Non-destructive digging	RAB	Rotary Air Blast	HQ	Diamond Core - 63 mm
AS*	Auger Screwing	RC	Reverse Circulation	HMLC	Diamond Core - 63 mm
AD*	Auger Drilling	PT	Push Tube	BH	Tractor Mounted Backhoe
*V	V-Bit	CT	Cable Tool Rig	EX	Tracked Hydraulic Excavator
*T	TC-Bit, e.g. AD/T	JET	Jetting	EE	Existing Excavation
ADH	Hollow Auger	WB	Washbore or Bailer	HAND	Excavated by Hand Methods

PENETRATION RESISTANCE

L	Low Resistance	Rapid penetration/ excavation possible with little effort from equipment used.
M	Medium Resistance	Penetration/ excavation possible at an acceptable rate with moderate effort from equipment used.
H	High Resistance	Penetration/ excavation is possible but at a slow rate and requires significant effort from equipment used.
R	Refusal/Practical Refusal	No further progress possible without risk of damage or unacceptable wear to equipment used.

These assessments are subjective and are dependent on many factors, including equipment power and weight, condition of excavation or drilling tools and experience of the operator.

WATER

 **Water level at date shown**

 **Partial water loss**

 **Water inflow**

 **Complete Water Loss**

GWNE	GROUNDWATER NOT OBSERVED - Observation of groundwater, whether present or not, was not possible due to drilling water, surface seepage or cave-in of the borehole/ test pit.
GWNO	GROUNDWATER NOT ENCOUNTERED - Borehole/ test pit was dry soon after excavation. However, groundwater could be present in less permeable strata. Inflow may have been observed had the borehole/ test pit been left open for a longer period.

SAMPLING AND TESTING

SPT	Standard Penetration Test to AS1289.6.3.1-2004
4,7,11 N=18 seating	4,7,11 = Blows per 150mm. N = Blows per 300mm penetration following 150mm
30/80mm	Where practical refusal occurs, the blows and penetration for that interval are reported
RW	Penetration occurred under the rod weight only
HW	Penetration occurred under the hammer and rod weight only
HB	Hammer double bouncing on anvil
Sampling	
DS	Disturbed Sample
BDS	Bulk disturbed Sample
GS	Gas Sample
WS	Water Sample
U63	Thin walled tube sample - number indicates nominal sample diameter in millimetres
Testing	
FP	Field Permeability test over section noted
FVS	Field Vane Shear test expressed as uncorrected shear strength (sv= peak value, sr= residual value)
PID	Photoionisation Detector reading in ppm
PM	Pressuremeter test over section noted
PP	Pocket Penetrometer test expressed as instrument reading in kPa
WPT	Water Pressure tests
DCP	Dynamic Cone Penetrometer test
CPT	Static Cone Penetration test
CPTu	Static Cone Penetration test with pore pressure (u) measurement

ROCK CORE RECOVERY

TCR=Total Core Recovery

SCR=Solid Core Recovery (%)

RQD = Rock Quality Designation (%)

$$= \frac{\text{Length of core recovered}}{\text{Length of core run}} \times 100$$

$$= \frac{\sum \text{Length of cylindrical core recovered}}{\text{Length of core run}} \times 100$$

$$= \frac{\sum \text{Axial lengths of core} > 100\text{mm}}{\text{Length of core run}} \times 100$$

MATERIAL BOUNDARIES

————— = Inferred Boundary

- - - - - = Probable Boundary

- ? - ? - ? - ? - ? = Possible Boundary

METHOD OF SOIL DESCRIPTION USED ON BOREHOLE AND TEST PIT LOGS



FILL



COUBLES or
BOULDERS



GRAVEL (GP or
GW)



ORGANIC SOILS
(OL, OH or Pt)



SILT (ML or MH)



CLAY (CL, CI or CH)

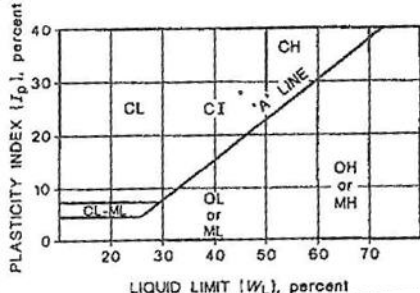


SAND (SP or SW)

Combinations of these basic symbols may be used to indicate mixed materials such as sandy clay

CLASSIFICATION AND INFERRED STRATIGRAPHY

Soil is broadly classified and described in Borehole and Test Pit Logs using the preferred method given in AS1726 – 1993, (Amdt1 – 1994 and Amdt2 – 1994), Appendix A. Material properties are assessed in the field by visual/tactile methods.

PARTICLE SIZE CHARACTERISTICS			USCS SYMBOLS				
Major Division	Sub Division	Particle Size	Major Divisions		Symbol	Description	
BOULDERS		>200 mm	COARSE GRAINED SOILS More than 50% by dry mass less than 63mm is greater than 0.075mm	More than 50% of coarse grains are >2mm	GW	Well graded gravel and gravel-sand mixtures, little or no fines.	
COBBLES		63 to 200 mm			GP	Poorly graded gravel and gravel-sand mixtures, little or no fines.	
GRAVEL	Coarse	20 to 63 mm			GM	Silty gravel, gravel-sand-silt mixtures.	
	Medium	6 to 20 mm			GC	Clayey gravel, gravel-sand-clay mixtures.	
SAND	Fine	2 to 6 mm		More than 50% of coarse grains are <2 mm	SW	Well graded sand and gravelly sand, little or no fines.	
	Coarse	0.6 to 2 mm			SP	Poorly graded sand and gravelly sand, little or no fines.	
	Medium	0.2 to 0.6 mm			SM	Silty sand, sand-silt mixtures.	
SILT		0.002 to 0.075 mm			SC	Clayey sand, sandy-clay mixtures.	
CLAY		<0.002 mm		FINE GRAINED SOILS More than 50% by dry mass less than 63mm is less than 0.075mm	Liquid Limit less < 50%	ML	Inorganic silts of low plasticity, very fine sands, rock flour, silty or clayey fine sands.
PLASTICITY PROPERTIES 		CL				Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.	
		OL	Organic silts and organic silty clays of low plasticity.				
		MH	Inorganic silts of high plasticity.				
		CH	Inorganic clays of high plasticity.				
		Liquid Limit > 50%	OH		Organic clays of medium to high plasticity.		
			PT		Peat muck and other highly organic soils.		

MOISTURE CONDITION

Symbol	Term	Description
D	Dry	Sands and gravels are free flowing. Clays & Silts may be brittle or friable and powdery.
M	Moist	Soils are darker than in the dry condition & may feel cool. Sands and gravels tend to cohere.
W	Wet	Soils exude free water. Sands and gravels tend to cohere.

Moisture content of cohesive soils may also be described in relation to plastic limit (WP) or liquid limit (WL) [» much greater than, > greater than, < less than, « much less than].

CONSISTENCY			DENSITY			
Symbol	Term	Undrained Shear Strength	Symbol	Term	Density Index %	SPT "N" #
VS	Very Soft	0. to 12 kPa	VL	Very Loose	< 15	0 to 4
S	Soft	12 to 25 kPa	L	Loose	15 to 35	4 to 10
F	Firm	25 to 50 kPa	MD	Medium Density	35 to 65	10 to 30
St	Stiff	50 to 100 kPa	D	Dense	65 to 85	30 to 50
VSt	Very Stiff	100 to 200 kPa	VD	Very Dense	Above 85	Above 50
H	Hard	Above 200 kPa				

In the absence of test results, consistency and density may be assessed from correlations with the observed behaviour of the material. # SPT correlations are not stated in AS1726 – 1993, and may be subject to corrections for overburden pressure and equipment type.

MINOR COMPONENTS

Term	Assessment Guide	Proportion by Mass
Trace	Presence just detectable by feel or eye but soil properties little or no different to general properties of primary component	Coarse grained soils: ≤ 5% Fine grained soil: ≤15%
Some	Presence easily detectable by feel or eye but soil properties little or no different to general properties of primary component	Coarse grained soils: 5 - 12% Fine grained soil: 15 - 30%

TERMS FOR ROCK MATERIAL STRENGTH AND WEATHERING

CLASSIFICATION AND INFERRED STRATIGRAPHY

Soil is broadly classified and described in Borehole and Test Pit Logs using the preferred method given in AS1726 – 1993, (Amdt1 – 1994 and Amdt2 – 1994), Appendix A. Material properties are assessed in the field by visual/ tactile methods.

STRENGTH

Symbol	Term	Point Load Index, $Is_{(50)}$ (MPa) #	Field Guide
EL	Extremely Low	< 0.03	Easily remoulded by hand to a material with soil properties.
VL	Very Low	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30 mm can be broken by finger pressure.
L	Low	0.1 to 0.3	Easily scored with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of pick point; has dull sound under hammer. A piece of core 150 mm long by 50 mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
M	Medium	0.3 to 1	Readily scored with a knife; a piece of core 150 mm long by 50 mm diameter can be broken by hand with difficulty.
H	High	1 to 3	A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken with pick with a single firm blow; rock rings under hammer.
VH	Very High	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
EH	Extremely High	>10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Rock Strength Test Results



Point Load Strength Index, $Is_{(50)}$, Axial test (MPa)



Point Load Strength Index, $Is_{(50)}$, Diametral test (MPa)

Relationship between rock strength test result ($Is_{(50)}$) and unconfined compressive strength (UCS) will vary with rock type and strength, and should be determined on a site-specific basis. UCS is typically 10 to 30 x $Is_{(50)}$, but can be as low as 5 MPa.

ROCK MATERIAL WEATHERING

Symbol	Term	Field Guide
RS	Residual Soil	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
EW	Extremely Weathered	Rock is weathered to such an extent that it has soil properties - i.e. it either disintegrates or can be remoulded, in water.
DW	Distinctly Weathered	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores. In some environments it is convenient to subdivide into Highly Weathered and Moderately Weathered, with the degree of alteration typically less for MW.
SW	Slightly Weathered	Rock slightly discoloured but shows little or no change of strength relative to fresh rock.
FR	Fresh	Rock shows no sign of decomposition or staining.

ABBREVIATIONS AND DESCRIPTIONS FOR ROCK MATERIAL AND DEFECTS

CLASSIFICATION AND INFERRED STRATIGRAPHY

Rock is broadly classified and described in Borehole Logs using the preferred method given in AS1726 – 1993, (Amdt1 – 1994 and Amdt2 – 1994), Appendix A. Material properties are assessed in the field by visual/ tactile methods.

ROCK MATERIAL DESCRIPTION

Layering		Structure	
Term	Description	Term	Spacing (mm)
Massive	No layering apparent	Thinly laminated	<6
		Laminated	6 – 20
Poorly Developed	Layering just visible; little effect on properties	Very thinly bedded	20 – 60
		Thinly bedded	60 – 200
Well Developed	Layering (bedding, foliation, cleavage) distinct; rock breaks more easily parallel to layering	Medium bedded	200 – 600
		Thickly bedded	600 – 2,000
		Very thickly bedded	> 2,000

ABBREVIATIONS AND DESCRIPTIONS FOR DEFECT TYPES

Defect Type	Abbr.	Description
Joint	JT	Surface of a fracture or parting, formed without displacement, across which the rock has little or no tensile strength. May be closed or filled by air, water or soil or rock substance, which acts as cement.
Bedding Parting	BP	Surface of fracture or parting, across which the rock has little or no tensile strength, parallel or sub-parallel to layering/ bedding. Bedding refers to the layering or stratification of a rock, indicating orientation during deposition, resulting in planar anisotropy in the rock material.
Foliation	FL	Repetitive planar structure parallel to the shear direction or perpendicular to the direction of higher pressure, especially in metamorphic rock, e.g. Schistosity (SH) and Gneissosity.
Contact	CO	The surface between two types or ages of rock.
Cleavage	CL	Cleavage planes appear as parallel, closely spaced and planar surfaces resulting from mechanical fracturing of rock through deformation or metamorphism, independent of bedding.
Sheared Seam/ Zone (Fault)	SS/SZ	Seam or zone with roughly parallel almost planar boundaries of rock substance cut by closely spaced (often <50 mm) parallel and usually smooth or slickensided joints or cleavage planes.
Crushed Seam/ Zone (Fault)	CS/CZ	Seam or zone composed of disoriented usually angular fragments of the host rock substance, with roughly parallel near-planar boundaries. The brecciated fragments may be of clay, silt, sand or gravel sizes or mixtures of these.
Decomposed Seam/ Zone	DS/DZ	Seam of soil substance, often with gradational boundaries, formed by weathering of the rock material in places.
Infilled Seam	IS	Seam of soil substance, usually clay or clayey, with very distinct roughly parallel boundaries, formed by soil migrating into joint or open cavity.
Schistosity	SH	The foliation in schist or other coarse grained crystalline rock due to the parallel arrangement of platy or prismatic mineral grains, such as mica.
Vein	VN	Distinct sheet-like body of minerals crystallised within rock through typically open-space filling or crack-seal growth.

ABBREVIATIONS AND DESCRIPTIONS FOR DEFECT SHAPE AND ROUGHNESS

Shape	Abbr.	Description	Roughness	Abbr.	Description
Planar	PI	Consistent orientation	Polished	Pol	Shiny smooth surface
Curved	Cu	Gradual change in orientation	Slickensided	SL	Grooved or striated surface, usually polished
Undulating	Un	Wavy surface	Smooth	S	Smooth to touch. Few or no surface irregularities
Stepped	St	One or more well defined steps	Rough	RF	Many small surface irregularities (amplitude generally <1mm). Feels like fine to coarse sandpaper
Irregular	Ir	Many sharp changes in orientation	Very Rough	VR	Many large surface irregularities, amplitude generally >1mm. Feels like very coarse sandpaper

Orientation:

Vertical Boreholes – The dip (inclination from horizontal) of the defect.

Inclined Boreholes – The inclination is measured as the acute angle to the core axis.

ABBREVIATIONS AND DESCRIPTIONS FOR DEFECT COATING

DEFECT APERTURE

Coating	Abbr.	Description	Aperture	Abbr.	Description
Clean	CN	No visible coating or infilling	Closed	CL	Closed.
Stain	SN	No visible coating but surfaces are discoloured by staining, often limonite (orange-brown)	Open	O	Without any infill material.
Veneer	VNR	A visible coating of soil or mineral substance, usually too thin to measure (< 1 mm); may be patchy	Infilled	-	Soil or rock i.e. clay, talc, pyrite, quartz, etc.

APPENDIX B

LABORATORY CERTIFICATES

MOISTURE CONTENT TEST REPORT

Client:	Environmental Investigations	Job No:	S16193
Address:	Suite 6.01, 55 Miller Street, Pyrmont, NSW 2009	Report No:	S11636-MC
Project:	21 Atkinson St, Liverpool (E23001)		

Test Procedure:	☒	AS 1289 2.1.1 Soil moisture content tests - Determination of the moisture content of a soil - Oven drying method (Standard method).
	☐	AS4133 1.1.1 Rock moisture content tests - Determination of the moisture content of rock - Oven drying method (standard method)
	☐	RMS T120 Moisture content of road construction materials (Standard method)
	☐	RMS T262 Determination of moisture content of aggregates (Standard method)

Sampling:	Sampled by Client	Date Sampled:	Unknown
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Preparation:	Prepared in accordance with the test method
---------------------	---

[illegible]

Notes:



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NATA Accredited Laboratory Number: 14874

Authorised Signatory:

[Signature]

31/05/2016

Chris Lloyd

Date:



Macquarie Geotechnical
Unit 8/10
Bradford Street
Alexandria NSW 2015

SOIL CLASSIFICATION REPORT

Client:	Environmental Investigations	Source:	BH2_3.0-3.4m
Address:	Suite 6.01, 55 Miller Street, Pyrmont, NSW 2009	Sample Description:	sandy CLAY
Project:	21 Atkinson St, Liverpool (E23001)	Report No:	S11638-PI
Job No:	S16193	Lab No:	S11638

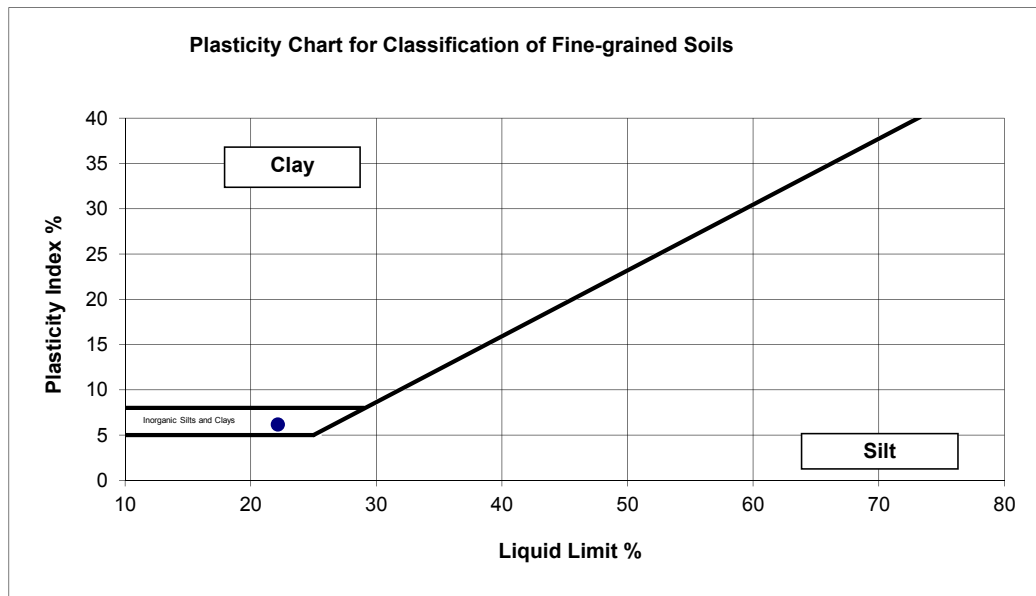
Test Procedure:	☒ AS1289 2.1.1 Soil moisture content tests (Oven drying method)
	☐ AS1289 3.1.1 Soil classification tests - Determination of the liquid limit of a soil - Four point casagrande method
	☒ AS1289 3.1.2 Soil classification tests - Determination of the liquid limit if a soil - One point Casagrande method (subsidiary method)
	☒ AS1289 3.2.1 Soil classification tests - Determination of the plastic limit of a soil - Standard method
	☒ AS1289 3.3.1 Soil classification tests - Calculation of the plasticity Index of a soil
	☒ AS1289 3.4.1 Soil classification tests - Determination of the linear shrinkage of a soil - Standard method

Sampling:	Sampled by Client	Date Sampled:	Unknown
Preparation:	Prepared in accordance with the test method		

Liquid Limit (%):
Linear Shrinkage (%):

Plastic Limit (%):
Field Moisture Content (%):

Plastic Index:



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Authorised Signatory:

Chris Lloyd

31/05/2016

Date:



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Unit 8/10
Bradford Street
Alexandria NSW 2015

POINT LOAD STRENGTH INDEX REPORT

Client:	Environmental Investigations	Moisture Content Condition:	As Received
Address:	Suite 6.01, 55 Miller Street, Pyrmont, NSW 2009	Storage History:	Core Box
Project:	21 Atkinson St, Liverpool (E23001)	Report No:	S11640-PL
Job No:	S16193	Date Tested:	23.05.16

Test Procedure: ☒ AS4133 4.1 Rock strength tests - Determination of point load strength index

Sampling: Sampled by Client **Date Sampled:** Unknown

Preparation:

Sample Number	Sample Source	Sample Description	Test Type	Average Width (mm)	Platen Separation (mm)	Failure Load (kN)	Point Load Index Is (MPa)	Point Load Index Is ₍₅₀₎ (MPa)	Notes
S11640	BH1 4.05m	Shale	Diametral	-	45.0	0.00	0.00	0.00	
			Axial	52.0	13.0	0.01	0.01	0.01	
S11641	BH1 4.60m	Shale	Diametral	-	50.0	0.38	0.15	0.15	
			Axial	52.0	31.0	0.71	0.35	0.33	
S11642	BH1 5.25m	Shale	Diametral	-	51.0	0.02	0.01	0.01	
			Axial	52.0	40.0	1.20	0.45	0.46	
S11643	BH1 5.75m	Shale	Diametral	-	51.0	0.04	0.02	0.02	
			Axial	52.0	39.0	0.74	0.29	0.29	
S11644	BH1 6.25m	Shale	Diametral	-	50.0	1.08	0.43	0.43	
			Axial	52.0	42.0	2.69	0.97	0.99	
S11645	BH1 6.75m	Shale	Diametral	-	50.0	0.45	0.18	0.18	
			Axial	52.0	45.0	1.60	0.54	0.56	
S11646	BH1 7.20 m	Shale	Diametral	-	50.0	0.39	0.16	0.16	
			Axial	52.0	39.0	1.06	0.41	0.41	
S11647	BH1 7.75m	Shale	Diametral	-	50.0	0.87	0.35	0.35	
			Axial	52.0	36.0	1.35	0.57	0.56	
S11648	BH1 8.25m	Shale	Diametral	-	51.0	0.60	0.23	0.23	
			Axial	52.0	36.0	2.30	0.96	0.95	
S11649	BH1 8.75m	Shale	Diametral	-	50.0	0.43	0.17	0.17	
			Axial	52.0	32.0	2.12	1.00	0.96	

Comments:



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31/05/2016

Date:



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Unit 8/10
Bradford Street
Alexandria NSW

POINT LOAD STRENGTH INDEX REPORT

Client:	Environmental Investigations	Moisture Content Condition:	As Received
Address:	Suite 6.01, 55 Miller Street, Pyrmont, NSW 2009	Storage History:	Core Box
Project:	21 Atkinson St, Liverpool (E23001)	Report No:	S11650-PL
Job No:	S16193	Date Tested:	23.05.16

Test Procedure:	☒ AS4133 4.1	Rock strength tests - Determination of point load strength index	
Sampling:	Sampled by Client	Date Sampled:	Unknown
Preparation:			

Sample Number	Sample Source	Sample Description	Test Type	Average Width (mm)	Platen Separation (mm)	Failure Load (kN)	Point Load Index Is (MPa)	Point Load Index Is(50) (MPa)	Notes
S11650	BH2 11.45m	Shale	Diametral	-	47.0	0.00	0.00	0.00	
			Axial	52.0	19.0	0.02	0.02	0.01	
S11651	BH2 11.95m	Shale	Diametral	-	50.0	0.28	0.11	0.11	
			Axial	52.0	17.0	0.53	0.47	0.39	
S11652	BH2 12.45m	Shale	Diametral	-	50.0	0.58	0.23	0.23	
			Axial	52.0	38.0	1.54	0.61	0.61	
S11653	BH2 12.95m	Shale	Diametral	-	51.0	0.52	0.20	0.20	
			Axial	52.0	39.0	1.76	0.68	0.69	
S11654	BH2 13.40m	Shale	Diametral	-	51.0	0.58	0.22	0.22	
			Axial	52.0	45.0	1.83	0.61	0.64	
S11655	BH2 13.80m	Shale	Diametral	-	50.0	0.58	0.23	0.23	
			Axial	52.0	32.0	0.97	0.46	0.44	

Comments:



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Chris Lloyd

31/05/2016

Date:



Macquarie Geotechnical
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CLIENT DETAILS

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Client Environmental Investigations
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Project **E23001 21 Atkinson St, Liverpool NSW**
Order Number **E23001**
Samples 3

LABORATORY DETAILS

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SGS Reference **SE152317 R0**
Date Received 18/5/2016
Date Reported 24/5/2016

COMMENTS

Accredited for compliance with ISO/IEC 17025. NATA accredited laboratory 2562(4354).

SIGNATORIES



Andy Sutton
 Senior Organic Chemist



Dong Liang
 Metals/Inorganics Team Leader



ANALYTICAL RESULTS

SE152317 R0

pH in soil (1:5) [AN101] Tested: 23/5/2016

			BH1_1.5-1.75	BH2_3.0-3.4
			SOIL	SOIL
			-	-
			16/5/2016	16/5/2016
			SE152317.001	SE152317.002
PARAMETER	UOM	LOR		
pH	pH Units	-	8.7	8.1



ANALYTICAL RESULTS

SE152317 R0

Conductivity and TDS by Calculation - Soil [AN106] Tested: 23/5/2016

			BH1_1.5-1.75	BH2_3.0-3.4
			SOIL	SOIL
			-	-
			16/5/2016	16/5/2016
			SE152317.001	SE152317.002
PARAMETER	UOM	LOR		
Conductivity of Extract (1:5 dry sample basis)	µS/cm	1	100	58



ANALYTICAL RESULTS

SE152317 R0

Soluble Anions (1:5) in Soil by Ion Chromatography [AN245] Tested: 19/5/2016

			BH1_1.5-1.75	BH2_3.0-3.4
			SOIL	SOIL
			-	-
			16/5/2016	16/5/2016
			SE152317.001	SE152317.002
PARAMETER	UOM	LOR		
Chloride	mg/kg	0.25	73	11
Sulphate	mg/kg	5	32	13



ANALYTICAL RESULTS

SE152317 R0

Moisture Content [AN002] Tested: 19/5/2016

			BH1_1.5-1.75	BH2_3.0-3.4
			SOIL	SOIL
			-	-
			16/5/2016	16/5/2016
			SE152317.001	SE152317.002
PARAMETER	UOM	LOR		
% Moisture	%w/w	0.5	14	14



ANALYTICAL RESULTS

SE152317 R0

pH in water [AN101] Tested: 20/5/2016

			BH1_GW1
			WATER
			-
			16/5/2016
			SE152317.003
PARAMETER	UOM	LOR	
pH**	No unit	-	6.8



ANALYTICAL RESULTS

SE152317 R0

Conductivity and TDS by Calculation - Water [AN106] Tested: 20/5/2016

			BH1_GW1
			WATER
			-
			16/5/2016
			SE152317.003
PARAMETER	UOM	LOR	
Conductivity @ 25 C	µS/cm	2	7500



ANALYTICAL RESULTS

SE152317 R0

Anions by Ion Chromatography in Water [AN245] Tested: 19/5/2016

			BH1_GW1
			WATER
			-
			16/5/2016
			SE152317.003
PARAMETER	UOM	LOR	
Chloride	mg/L	0.05	1900
Sulphate, SO4	mg/L	1	310

METHOD

METHODOLOGY SUMMARY

AN002

The test is carried out by drying (at either 40°C or 105°C) a known mass of sample in a weighed evaporating basin. After fully dry the sample is re-weighed. Samples such as sludge and sediment having high percentages of moisture will take some time in a drying oven for complete removal of water.

AN101

pH in Soil Sludge Sediment and Water: pH is measured electrometrically using a combination electrode and is calibrated against 3 buffers purchased commercially. For soils, sediments and sludges, an extract with water (or 0.01M CaCl₂) is made at a ratio of 1:5 and the pH determined and reported on the extract. Reference APHA 4500-H⁺.

AN106

Conductivity and TDS by Calculation: Conductivity is measured by meter with temperature compensation and is calibrated against a standard solution of potassium chloride. Conductivity is generally reported as µmhos/cm or µS/cm @ 25°C. For soils, an extract with water is made at a ratio of 1:5 and the EC determined and reported on the extract, or calculated back to the as-received sample. Salinity can be estimated from conductivity using a conversion factor, which for natural waters, is in the range 0.55 to 0.75. Reference APHA 2510 B.

AN245

Anions by Ion Chromatography: A water sample is injected into an eluent stream that passes through the ion chromatographic system where the anions of interest ie Br, Cl, NO₂, NO₃ and SO₄ are separated on their relative affinities for the active sites on the column packing material. Changes to the conductivity and the UV-visible absorbance of the eluent enable identification and quantitation of the anions based on their retention time and peak height or area. APHA 4110 B

FOOTNOTES

*	NATA accreditation does not cover the performance of this service.	-	Not analysed.	UOM	Unit of Measure.
**	Indicative data, theoretical holding time exceeded.	NVL	Not validated.	LOR	Limit of Reporting.
		IS	Insufficient sample for analysis.	↑↓	Raised/lowered Limit of Reporting.
		LNR	Sample listed, but not received.		

Samples analysed as received.
Solid samples expressed on a dry weight basis.

Where "Total" analyte groups are reported (for example, Total PAHs, Total OC Pesticides) the total will be calculated as the sum of the individual analytes, with those analytes that are reported as <LOR being assumed to be zero. The summed (Total) limit of reporting is calculated by summing the individual analyte LORs and dividing by two. For example, where 16 individual analytes are being summed and each has an LOR of 0.1 mg/kg, the "Totals" LOR will be 1.6 / 2 (0.8 mg/kg). Where only 2 analytes are being summed, the "Total" LOR will be the sum of those two LORs.

Some totals may not appear to add up because the total is rounded after adding up the raw values.

If reported, measurement uncertainty follow the ± sign after the analytical result and is expressed as the expanded uncertainty calculated using a coverage factor of 2, providing a level of confidence of approximately 95%, unless stated otherwise in the comments section of this report.

Results reported for samples tested under test methods with codes starting with ARS-SOP, radionuclide or gross radioactivity concentrations are expressed in becquerel (Bq) per unit of mass or volume or per wipe as stated on the report. Becquerel is the SI unit for activity and equals one nuclear transformation per second.

Note that in terms of units of radioactivity:

- 1 Bq is equivalent to 27 pCi
- 37 MBq is equivalent to 1 mCi

For results reported for samples tested under test methods with codes starting with ARS-SOP, less than (<) values indicate the detection limit for each radionuclide or parameter for the measurement system used. The respective detection limits have been calculated in accordance with ISO 11929.

The QC criteria are subject to internal review according to the SGS QAQC plan and may be provided on request or alternatively can be found here : <http://www.sgs.com.au/~media/Local/Australia/Documents/Technical Documents/MP-AU-ENV-QU-022 QA QC Plan.pdf>

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APPENDIX C

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SCOPE OF SERVICES

The geotechnical report ("the report") has been prepared in accordance with the scope of services as set out in the contract, or as otherwise agreed, between the Client And Environmental Investigations Pty Ltd ("EI"). The scope of work may have been limited by a range of factors such as time, budget, access and/or site disturbance constraints.

RELIANCE ON DATA

EI has relied on data provided by the Client and other individuals and organizations, to prepare the report. Such data may include surveys, analyses, designs, maps and plans. EI has not verified the accuracy or completeness of the data except as stated in the report. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations ("conclusions") are based in whole or part on the data, EI will not be liable in relation to incorrect conclusions should any data, information or condition be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to EI.

GEOTECHNICAL ENGINEERING

Geotechnical engineering is based extensively on judgment and opinion. It is far less exact than other engineering disciplines. Geotechnical engineering reports are prepared for a specific client, for a specific project and to meet specific needs, and may not be adequate for other clients or other purposes (e.g. a report prepared for a consulting civil engineer may not be adequate for a construction contractor). The report should not be used for other than its intended purpose without seeking additional geotechnical advice. Also, unless further geotechnical advice is obtained, the report cannot be used where the nature and/or details of the proposed development are changed.

LIMITATIONS OF SITE INVESTIGATION

The investigation programme undertaken is a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions. The data derived from the site investigation programme and subsequent laboratory testing are extrapolated across the site to form an inferred geological model, and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite investigation, the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies. The engineering logs are the subjective interpretation of subsurface conditions at a particular location and time, made by trained personnel. The actual interface between materials may be more gradual or abrupt than a report indicates.

SUBSURFACE CONDITIONS ARE TIME DEPENDENT

Subsurface conditions can be modified by changing natural forces or man-made influences. The report is based on conditions that existed at the time of subsurface exploration. Construction operations adjacent to the site, and natural events such as floods, or ground water fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report. EI should be kept apprised of any such events, and should be consulted to determine if any additional tests are necessary.

VERIFICATION OF SITE CONDITIONS

Where ground conditions encountered at the site differ significantly from those anticipated in the report, either due to natural variability of subsurface conditions or construction activities, it is a condition of the report that EI be notified of any variations and be provided with an opportunity to review the recommendations of this report. Recognition of change of soil and rock conditions requires experience and it is recommended that a suitably experienced geotechnical engineer be engaged to visit the site with sufficient frequency to detect if conditions have changed significantly.

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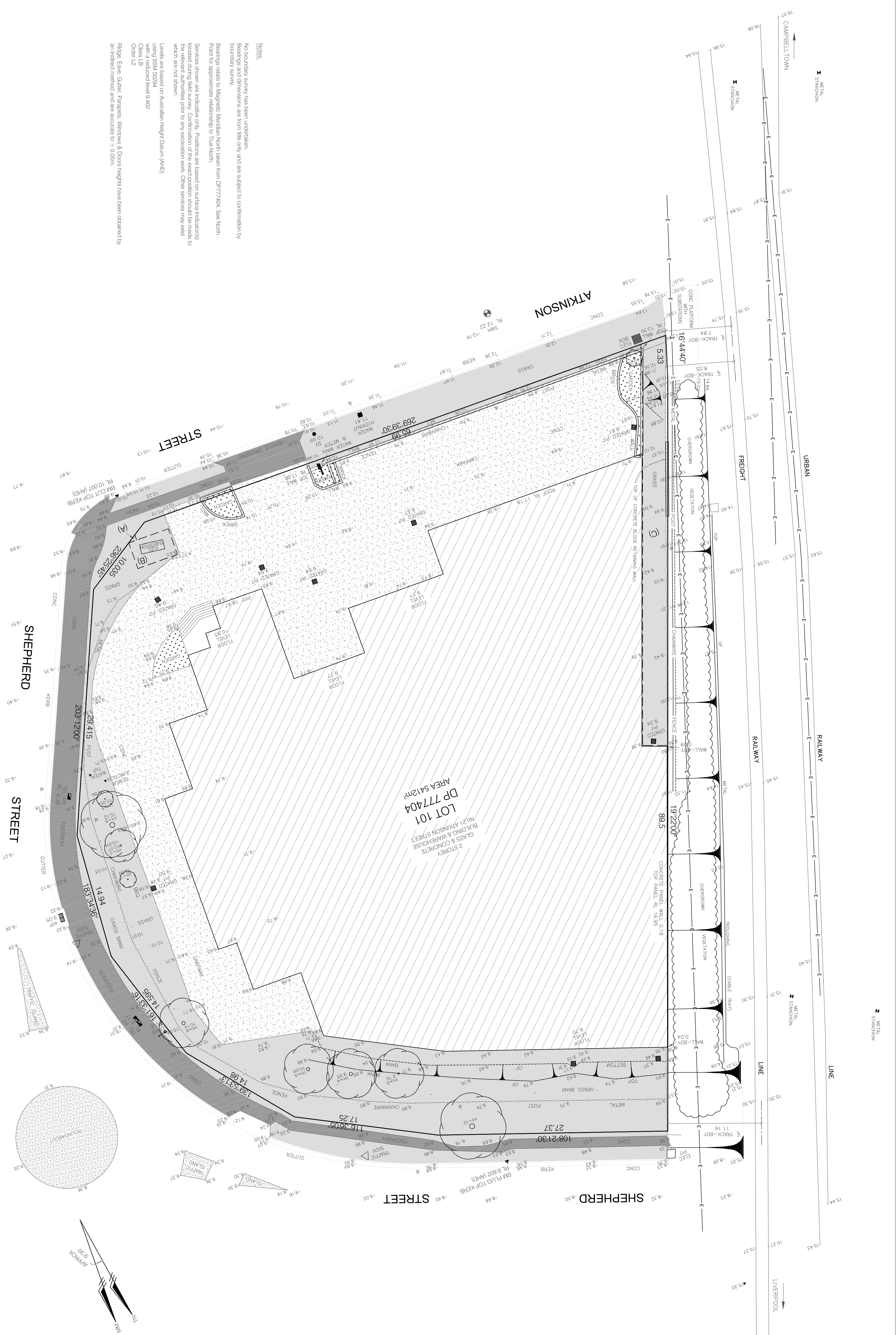
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The report has been prepared for the benefit of the Client and no other party. EI assumes no responsibility and will not be liable to any other person or organisation for or in relation to any matter dealt with or conclusions expressed in the report, or for any loss or damage suffered by any other person or organisation arising from matters dealt with or conclusions expressed in the report (including without limitation matters arising from any negligent act or omission of EI or for any loss or damage suffered by any other party relying upon the matters dealt with or conclusions expressed in the report). Other parties should not rely upon the report or the accuracy or completeness of any conclusions and should make their own inquiries and obtain independent advice in relation to such matters.

OTHER LIMITATIONS

EI will not be liable to update or revise the report to take into account any events or emergent circumstances or fact occurring or becoming apparent after the date of the report.

8.4. SURVEY

[illegible]

LG

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LEGEND

KIP - KERB INLET PIT

PPT - PARAPET

SMH - SEWER MANHOLE

SV - STOP VALVE

CLIENT

SYNERGY DEVELOPMENT GROUP

PROJECT

21 ATKINSON STREET LIVERPOOL

DRAWING TITLE

DETAIL & LEVEL SURVEY

TITLE INFORMATION

LOT 101

DP 777404

101/777404

LIVERPOOL

ST LUKE

CUMBERLAND

QUALITY ASSURANCE

Surveyed by: CE

Drawn by: AK

Checked by: SL

Approved by: SL

Date: 04/07/16

Date: 04/07/16

Date: 04/07/16

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Date of Survey: 02/07/16

Date of Title: 04/07/16

Drawn: AHD

Scale: 1:200

Sheet: 1 of 1

163408

DRAWING NUMBER ISSUE

DETL-001/B

8.5. RISK ASSESSMENT REPORT



RISK ASSESSMENT

Derailment Scenarios on Southern Sydney Freight Line adjacent

Development 21 Atkinson Street Liverpool

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**Version 1.1
19 August 2016**

DOCUMENT REVISION RECORD

Document / Version	Prepared By	Date	Checked By	Date	Approved By	Date
Initial Risk Management Report Derailment Scenarios Draft version 0.A For review by stakeholders	David Cotton	27/06/16	Andrew Barnett	27/06/16		
Initial Risk Management Report Derailment Scenarios Draft version 1.0 Following review & revised Architectural Drawings	David Cotton	25/07/16	Andrew Barnett	26/07/16		
Risk Management Report Derailment Scenarios Draft version 1.1 Following review of Xavier Knight 'Retention Wall Design' Report 17/8/2015	David Cotton	18/08/16	Andrew Barnett	19/08/16		

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1 OVERVIEW

A derailment risk assessment has been undertaken for structures at 21 Atkinson Street, Liverpool located within 10-20m of an adjacent rail track centreline. The study has been conducted in accordance with the Asset Standards Authority (ASA) T-HR-CI-12080-ST, version 1.0 of February 2015. The purpose is to determine appropriate load requirements specified in AS 5100 as mitigation to manage the risk of collision from a derailed freight train on the adjacent bi-directional Southern Sydney Freight Line. The risk assessment takes into account:

- Site condition (elevation below level of rail line)
- Derailment history (nil records of train derailments or collisions on the section adjacent the development)
- Type of structure and potential for collapse damage to trains and building occupants
- Track geometry (maintained fit for purpose, straight section)
- Track speed (sign board max approach 115kph coasting)
- Type of rolling stock (non electric freight trains)
- Future use and growth and developments

The unit developments do not impact the likelihood of a derailment but do alter the profile of persons at risk by introducing:

- Additional persons exposed in (occupants) and around the buildings
- Additional exposure to freight train crew (driver and observer) in the event of a collapse of a section of the building onto a train following a derailment and collision with a support structure

The **quantitative** risk model analysis for this rail section as a whole estimated:

- Potential derailment incident once every 241 years
- Average derailment frequency 1.70E-07 per train movement
- Fatal incident every 53,466 years (as a consequence of impact with buildings)

The desktop **qualitative** assessment using the specified risk matrix found that:

- Likelihood of a derailment on Up Main = L5 (Very Unlikely)
- Likelihood derailment proceeding to site boundary = L6 (Almost Unprecedented)
- Most likely case consequence following derailment as a result of building
1Fatality – 10-20 Major Injuries = C2 (Severe)
- Risk Level = L6 x C2 = C (Tolerable)

T-HR-CI-12080-ST applies the result of the risk analysis to determine the category of loading requirement from AS 5100 that is to be applied to a support. In accordance with T-HR-CI-12080-ST where the supporting elements are located within 10-20 metres of the rail line they shall comply as per Table 1, clause 5.13, the AS5100 Collision Loading Requirement for support structures:

Safety Risk Matrix Ranking	AS 5100 Collision Loading Requirements
A	AS 5100.2 clause 10.4.3 (using loading for between 10m and 20m from centre line of track), clause 10.4.4, clause 10.4.5 and clause 10.4.6
B	AS 5100.2 clause 10.4.4 and clause 10.4.6
C	Nil
D	Nil

**DERAILMENT SCENARIOS on SSFL adjacent
Development at 21 ATKINSON STREET, LIVERPOOL**

In conclusion, the risk assessment finds that the likelihood of a derailed train travelling and displacing laterally an adequate distance down an embankment to strike and collapse a structural column resulting in fatality and major injuries is less than once in 1,000 years. The consequence is a risk level of 'C' in the TERM risk matrix that is nominated in T-HR-CI-12080-ST. It is noted that given the unprecedented likelihood of an impact, this risk ranking remains 'C' even if more fatalities are factored into a worse case outcome event.

In accordance with this and the link reference to specification in AS5100:

- (Rail) Collision Loading Requirement for the support structures of the buildings adjacent the rail line is 'Nil'.



Furthermore, a derailment on the adjacent Down Main South Line (see diagram section 3.3) would be laterally contained by the SSFL and is a lower risk.

2 INTRODUCTION

The development is located in a property adjacent the SSFL previously occupied by Rheem Pool Heating between Liverpool and Casula Stations.

The purpose of this risk assessment is to determine appropriate load requirements of AS 5100 for the unit developments with regards to the risk of collision from a derailed train on the adjacent Up Main West rail line. It is undertaken in accordance with T-HR-CL-12080-ST External Developments Clause 5.13 that applies the risk ranking assessed in the analysis to determine the category of collision loading in AS5100 to be applied to design of supports. This table categorises safety risk rankings and specifies that 'C' and 'D' require nil collision loading requirements in addition to other applicable design requirements in AS5100.

The risk assessment takes into account:

- Site configuration - embankment elevates track above existing industrial building level



- Centreline of nearest rail line to nearest buildings is 11-14m (refer drawings in section 3.2 and Xavier Knight Retention Wall Design Report, 17 August 2016 sections 1.3 and 8.1)
- Track is 60kg railed, bidirectional conveying freight traffic from 385 to 1500-1800 intermodal metre length consists conveying cement, steel, general freight, grain, coals and containers, plus some Diesel Passenger Trains (Xplorer, Endeavour, XPT) as scheduled in ARTC Master Plan, Mar 2016
- Track on approach has no discontinuity components e.g. catchpoints; turnouts (facing or trailing); slips; diamonds; scissor crossovers that would increase the hazard of derailment above the network average during normal operations
- Track is level, rising and falling 1:100 on either approach to the site
- Track is straight adjacent the site and past towards Casula, while curve is gradual away from the site on the Liverpool approach (ie throw of derailed train away from site towards Down Main South line)
- Track speed 110kph board approaching the site (SSFL boarded 110kph to Glenfield, 80kph to Leightonfield)
- Type of rolling stock electric and diesel passenger and freight
- Future usage and growth reviewed in section 4.1
- Derailment history – since commencement of operations in January 2013, no derailments on this section

**DERAILMENT SCENARIOS on SSFL adjacent
Development at 21 ATKINSON STREET, LIVERPOOL**

In summary, the proposed building development will not impact the likelihood of a derailment but may alter the profile of persons potentially at risk by introducing:

- Freight train crew (driver and observer in cab) who may be exposed in the event of a collapse of a building onto a train following a derailment and collision with a building support structure
- Building occupants (max 20 in apartments within 20m facing the SSFL rail line) a given number of whom may be exposed at a given time to a potential impact of a train collision event

The assessment estimates the number of persons in these groups who could be injured following a derailment incident impacting the new building. Event Tree and Collision Analyses (sections 5.7, 5.8) estimate the likelihood of partial and total collapse per train movement that derails and impacts the development structure(s), and the number of persons who could be exposed are then considered in estimating the potential fatalities per train movement.

3 BACKGROUND

3.1 LOCATION

The development site is adjacent the rail corridor and SSFL section between Liverpool and Casula Stations:



3.2 DEVELOPMENT

The development comprises the demolition of the existing warehouse/light-industrial buildings and construction of a part 3, part 7 and part 9 storey (8 levels and roof) residential flat building with:

- 1 level of partial basement parking and 2 above ground parking levels
- 143 residential apartments
- 4 neighborhood shops are situated at the south-eastern corner of the site

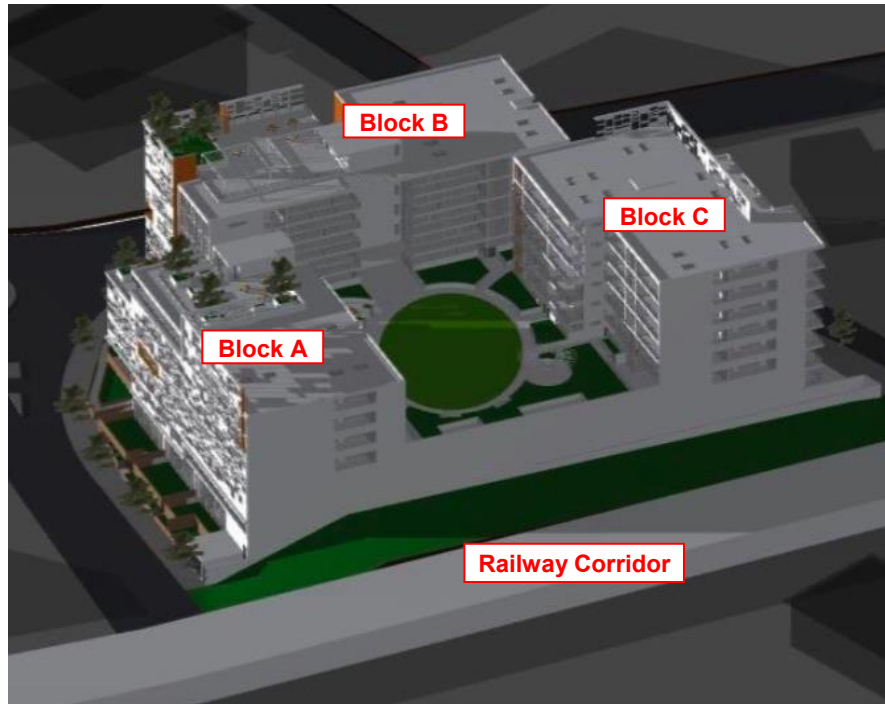
Third floor and above comprises three tower buildings, blocks A, B and C, arranged around a large, centrally located communal open space area.

Block A is 4 storeys above podium and Blocks B and C are each 7 storeys above podium.

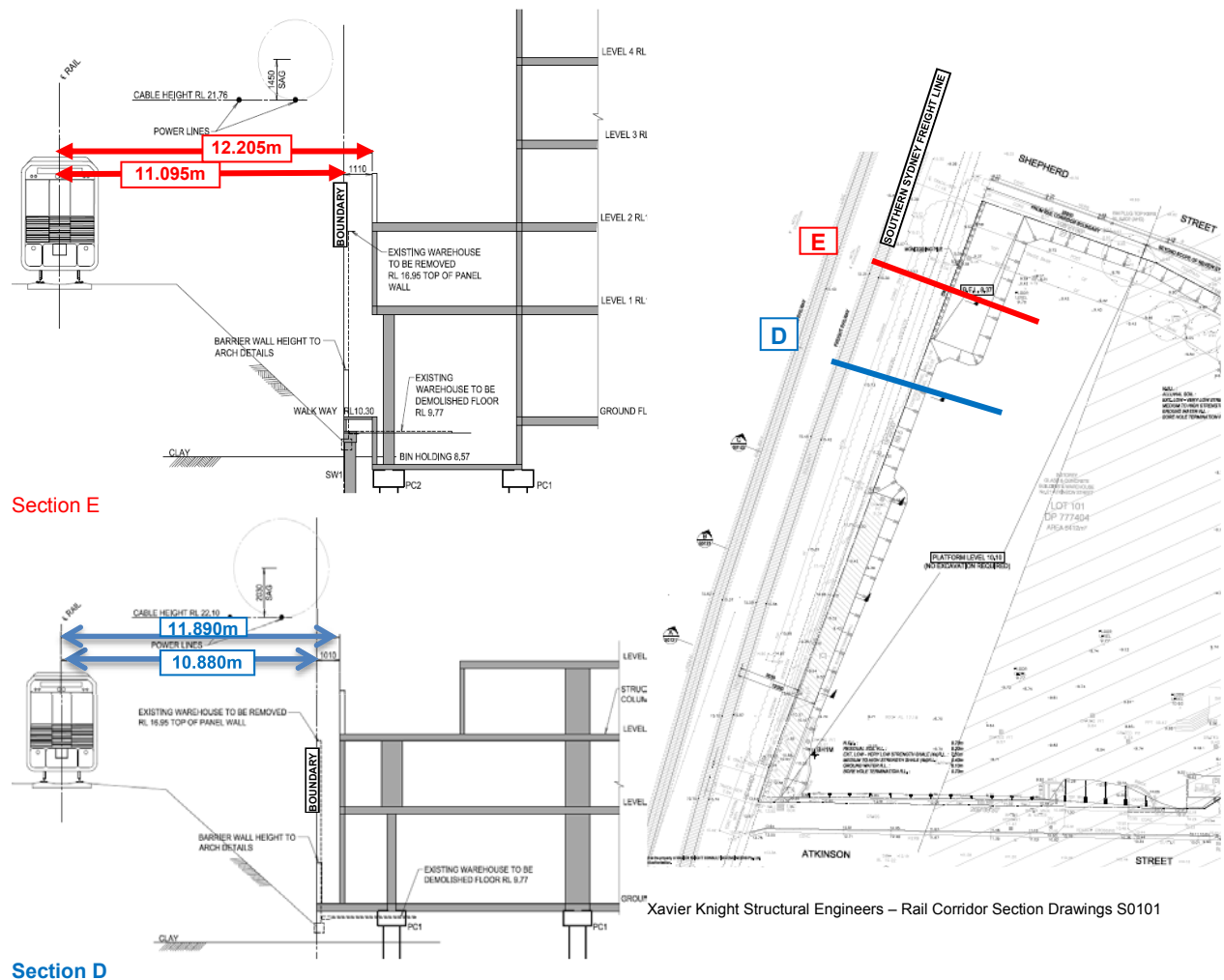
A roof top communal open space area is provided at the north-eastern corner of Block A and at the rooftop of Block B. From third floor and above, the development delivers 20 x 1 bedroom, 82 x 2 bedroom and 11 x 3 bedroom dwellings.

The following is a perspective of the development fronting the SSFL:

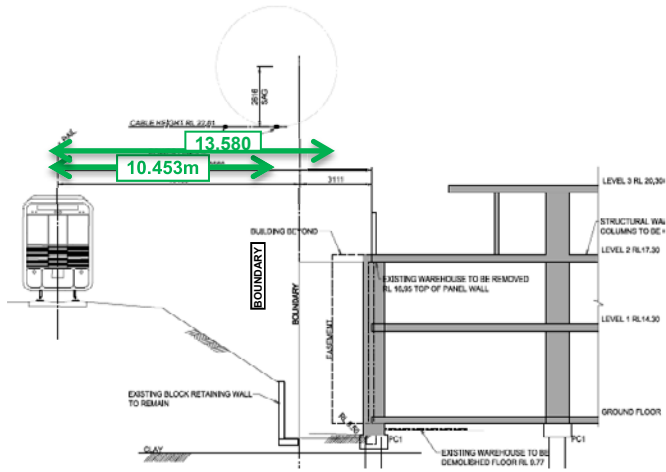
**DERAILMENT SCENARIOS on SSFL adjacent
Development at 21 ATKINSON STREET, LIVERPOOL**



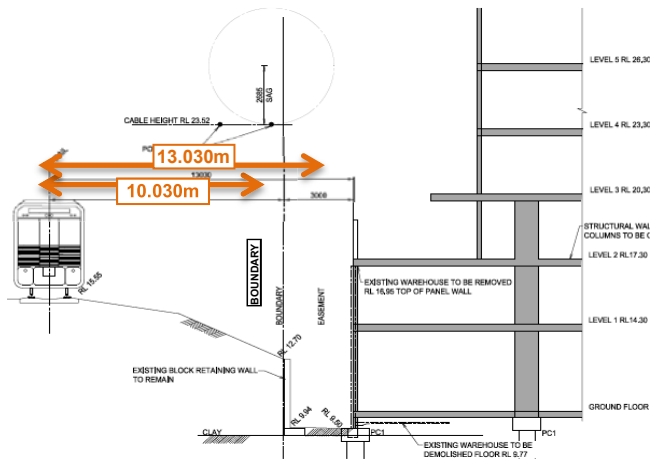
Xavier Knight Structural Engineers – Rail Corridor Section Drawings S0101 building shoring and earthworks plan indicates configuration clearances as follows:



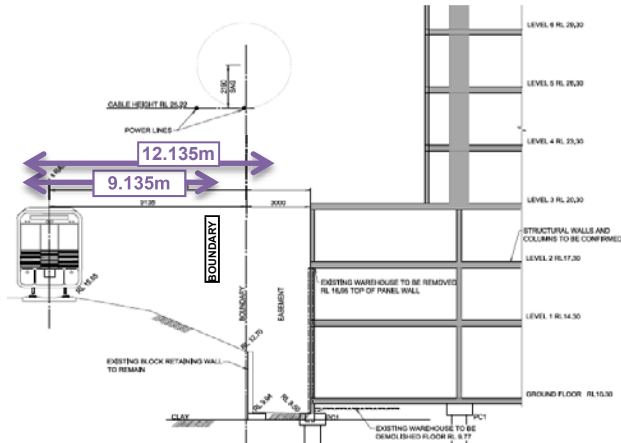
DERAILMENT SCENARIOS on SSFL adjacent Development at 21 ATKINSON STREET, LIVERPOOL



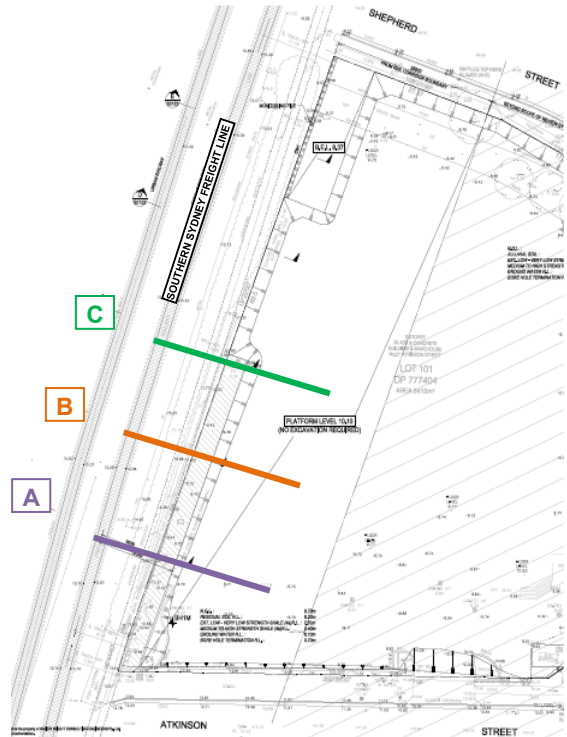
Section C



Section B



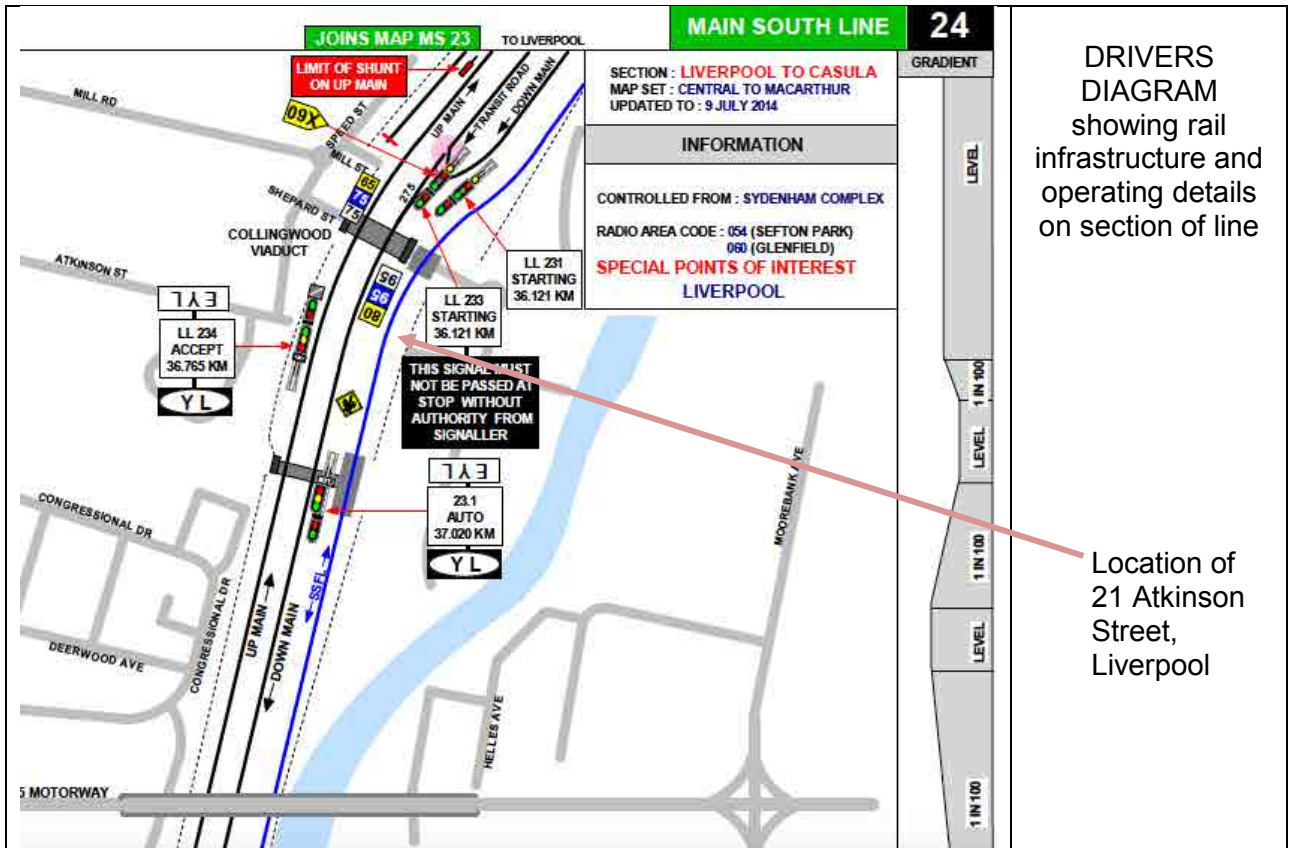
Section A



Xavier Knight Structural Engineers – Rail Corridor Section Drawings S0101

3.3 TRAIN DRIVERS DIAGRAM

The drivers diagram shows the track infrastructure and operating conditions – SSFL in blue:



4 RISK ASSESSMENT METHODOLOGY

4.1 HAZARD IDENTIFICATION

A hazard and risk worksheet has been completed as part of desk-top exercise in accordance with Safety Management System SMS-06-PR-0030, 'Hazard Identification and Safety Risk Assessment Guide SMS-06-GD-0031 and AS/NZ 4360-2004. It was conducted in consultation with appropriate stakeholders and Subject Matter Experts and reference to clauses in Engineering Standard T-HR-CL-12080-ST External Developments.

4.2 STAKEHOLDERS & SUBJECT MATTER EXPERTS

Information has been sourced from:

ARTC, Discussion with Michael Walsh, ARTC Sydney Operations

- No freight train derailment or collision incidents on this section since commissioning into operation in January 2013

Rail Infrastructure Corporation Incident Database reviews 1998-2004:

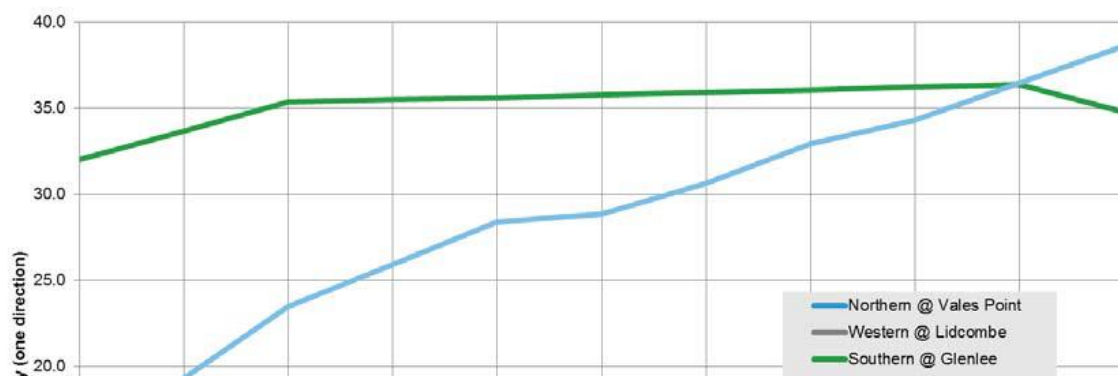
- Liverpool during possession shunting derailed at points to shed (9/1/1993) – note this would have no impact on site being analysed in this study, and furthermore the section under review in this study is straight rail with no points ie this scenario does not exist)
Only other incidents involved loose skirting on wagon demolished signal at Liverpool (24/10/1994); signal points relay box damaged by rolling stock at Liverpool (10/7/1997); train struck by object at Casula (9/11/1999); and closest derailment was incident at Sefton Junction points 26/8/2001 (note the section under review in this study is straight rail with no points ie this scenario does not exist)

RailCorp IIMS review 2004-2013 and Sydney Trains 2014-2015:

- No train derailment or collision incidents on the adjacent Down Main South through this section

NSW Freight and Ports Strategy 2015 and ARTC 2015 Sydney Metropolitan Freight Strategy, October 2015

- The SSFL runs 36 kilometres between Sefton Park Junction and Macarthur, linking the Interstate Network between Sydney and Melbourne with the Metropolitan Freight Network (MFN) managed by the ARTC. It provides access to the MFN and Port Botany from the south. Forecast traffic on SSFL from the South is shown in green (Southern @ Glenlee) includes diesel passenger trains



**DERAILMENT SCENARIOS on SSFL adjacent
Development at 21 ATKINSON STREET, LIVERPOOL**

4.3 RISK ASSESSMENT

In accordance T-HR-CI-12080-ST External Develops the Standard 30-ST-164 v2.0, TERM Matrix has been used in the assessment process of the impact of estimated frequency of hazardous events and their potential safety consequences:

TERM Health & Safety (Injury) Risk Matrix			Consequence						
			Workplace Historical (Likelihood)	Illness, first aid or injury not requiring treatment.	Illness or Minor Injuries requiring medical treatment	Single recoverable Loss Time Injury or illness, alternate / restricted duties injury, or short term occupational illness	1-10 Multiple Injuries requiring hospitalization and numerous days lost, or medium term occupational illness	Single Fatality and/or 10-20 major injuries / permanent disabilities, chronic diseases	Multiple Fatalities and/or >20 major injuries / permanent disabilities, chronic diseases
				C6	C5	C4	C3	C2	C1
Likelihood / Frequency			Insignificant	Minor	Moderate	Major	Severe	Catastrophic	
10 times or more per year	Expected to occur frequently during time of activity or project	L1 Almost Certain	C – Medium	B - High	B - High	A – Very High	A – Very High	A – Very High	
1 to 10 times a year	Expected to occur occasionally during time of activity or project	L2 Very Likely	C – Medium	C – Medium	B - High	B - High	A – Very High	A – Very High	
Once each year	More likely to occur than not occur during time of activity or project	L3 Likely	D - Low	C – Medium	C – Medium	B - High	B - High	A – Very High	
Once every 1 to 10 years	More likely not to occur than occur during time of activity or project	L4 Unlikely	D - Low	D - Low	C – Medium	C – Medium	B - High	B - High	
Once every 10 to 100 years	Not expected to occur during time of activity or project	L5 Very Unlikely	D - Low	D - Low	D - Low	C – Medium	C – Medium	B - High	
Less than once every 100 years	Not expected to ever occur during time of activity or project	L6 Almost Unprecedented	D - Low	D - Low	D - Low	D - Low	C – Medium	C – Medium	

4.4 RISK CRITERIA

In accordance with T-HR-CI-12080-ST and previous T-HR-CL-12080-ST Clause 5.13, the results of the risk analysis will determine the category of collision loading in AS 5100 that is to be applied to a support. The risk ranking determined from the risk analysis shall be equated to a loading requirement from AS 5100 as tabulated below:

Safety Risk Matrix Ranking	AS 5100 Collision Loading Requirements
A	AS 5100.2 clause 10.4.3 (using loading for between 10m and 20m from centre line of track), clause 10.4.4, clause 10.4.5 and clause 10.4.6
B	AS 5100.2 clause 10.4.4 and clause 10.4.6
C	Nil
D	Nil

5. DERAILMENT RISK

5.1 DATA and INFORMATION

The Incident Information Management System has no recorded derailment or collision incidents since 1998 at this section of the Main South Line that would impact the proposed development, and information from ARTC indicates no derailment incidents on the SSFL since it was commissioned into operation in January 2013.

5.2 RISK ASSESSMENT

The process of review and discussions with stakeholders and Subject Matter Experts has been to compare:

- Site Specific Hazards to average on Freight Network (MFN, SSFL)
- Operating Specific Hazards to average on Freight Network (MFN, SSFL)

5.3 SITE SPECIFIC FEATURES

The SSFL is concrete sleepered track with 60kg rail and is designed and in a condition to carry the traffic at the speeds indicated (ref ARTC Sydney Operations).

Current freight traffic comprises freight trains 650m to 1500m (40%) and 1800m interstate (approx. 20%) in length with projected traffic 35 trains per day in one direction, Taking account of closedowns on the line that equates for up to 15 days per annum, this gives:

$$= (35 \times 2 \times 350) = 24,500 \text{ freight services per year}$$

Line	SSFL
Maximum approach speed	115
Recommended approach Speed (200m) ex Liverpool	100
Train Status	Coasting
Number of Signals on approach	1
Freight and Diesel Passenger Service Movements per annum	24,500
Infrastructure Features	
Rail Curvature	Straight through site, curve away on approach from Liverpool
Gradient	Level, 1:100 falling/ rising either approach
Facing Points	0

References for above include:

- Drivers Diagrams – Track Diagram Main South Line, Liverpool – Casula
- Location of Speed Signs Train Operating Conditions (TOC)

5.4 REVIEW OF HAZARDS

The likelihood of a derailment occurring is a function of the presence of hazards identified as having the potential to cause an incident at this location, and modifications to the generic derailment incident frequency have been reviewed in a desk-top exercise following discussions with Subject Matter Experts (SMEs) and a review of the historical incident information sources that indicated no derailment or collision incidents on this section of the Main West Line that would impact the development.

5.5 ROOT CAUSE ANALYSIS

The fit for purpose condition of the track was confirmed by Regional Track & Civil Maintenance who indicated that the track in this section is concrete sleepers with 60kg rail, with no derailment incidents and a track that is maintained fit for purpose. The incident review was used to modify the Generic Network Incident Count sample to reflect Site Specific hazards as follows:

Hazard Area Environmental Factors:

SSFL	Historical Frequency per Train Movement Km	Site Factor *	Total
FREIGHT & DIESEL PASSENGER TRAFFIC			
Spread road	4.27E-07	1	4.27E-07
Vandalism	2.05E-08	1	2.05E-08
Heat buckle	6.14E-08	1	6.14E-08
Rock or fill on line	3.41E-09	0.1	3.41E-10
Subsiding Embankment	3.41E-09	1	3.41E-09
			<u>5.12E-07</u>

* Scores from Risk Model range from Highly probable 10 to Probable 1, to Improbable 0.1 to Nil 0

Hazard Area Infrastructure Factors:

SSFL	Historical Frequency per Train Movement Km	Site Factor *	Total
FREIGHT & DIESEL PASSENGER TRAFFIC			
Broken Rail	3.75E-08	1	3.75E-08
Broken Joints	5.46E-08	1	5.46E-08
Geometry Design	3.41E-09	1	3.41E-09
Excessive Speed	6.83E-09	0.5	3.41E-09
			<u>9.90E-08</u>

* Scores from Risk Model range from Highly probable 10 to Probable 1, to Improbable 0.1 to Nil 0

**DERAILMENT SCENARIOS on SSFL adjacent
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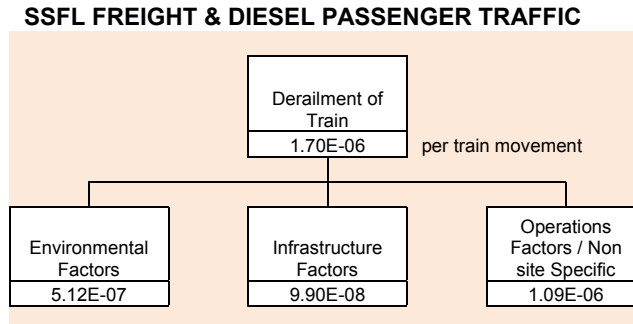
Hazard Area Rolling stock and operational hazards:

Rolling stock and operational hazards have been assumed to be consistent across the network and have not been modified for line or site specific factors as follows:

		Historical Frequency per Train Movement Km
Operations and Rolling Stock Dependant Hazards	Private Party	2.90E-07
	Crew performance	2.35E-07
	Related maintenance work	1.30E-07
	Faulty brakes	2.05E-08
	Screwed journal	1.19E-07
	Scaled wheels	4.44E-08
	Train - track dynamics (not compatible)	2.05E-08
	Faulty Wheels	3.07E-08
	Irregular alignment	2.73E-08
	Bearer clearance incorrect	1.71E-08
	Incorrect dampening	2.73E-08
	Uneven loading	1.71E-08
	Broken equipment falling	1.02E-08
	Dragging gear	6.83E-09
	Faulty coupler	1.37E-08
	Faulty axles	1.02E-08
	Jammed axle box	1.02E-08
	Driver performance	1.02E-08
	Employee performance	0.00E+00
	Lashings Loose	6.83E-09
	Operator performance	6.83E-09
	Worksite protection	1.02E-08
	Broken springs	3.41E-09
	Cracked bogie frame	0.00E+00
	Damaged rolling stock	3.41E-09
	Damaged structure	3.41E-09
	Faulty King Pin	0.00E+00
	Irregularity released ground frame	3.41E-09
	Outside agent	3.41E-09
	Structure collapse	3.41E-09
		1.09E-06

5.6 FAULT TREE ANALYSIS

The site specific frequency of derailment within the nominated track profile adjacent the support structures of the development adjacent the rail corridor between Westmead and Wentworthville Stations is illustrated on fault trees:



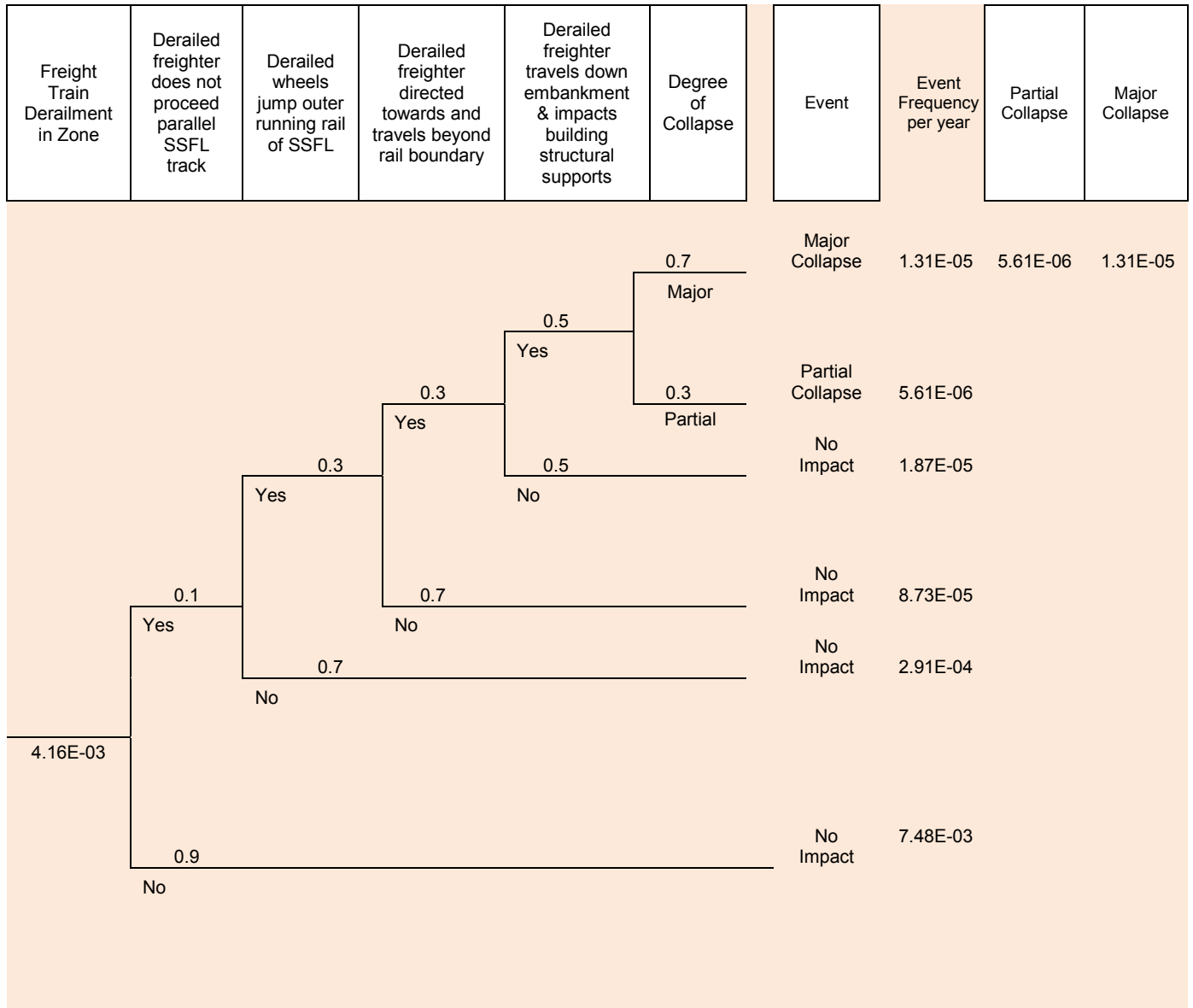
Thus the generic frequency of derailment of 1.09E-06 per freight train km per year on the open metropolitan freight network has been modified on the basis of the site-specific derailment hazards to 1.70E-06 per train km per year for freight traffic on the SSFL on the approaches and adjacent the development site..

5.7 EVENT TREE ANALYSIS

The units of the frequency analysis have been transformed from per train movement km to per train movement via the study area by the application of a train length track of interest equivalent to the braking distance at the approach track speed. As a consequence, the Train Derailment Frequency in the Zone (first column) is smaller than the reported Site Specific Derailment Frequency in the Fault Tree. The worse case scenario is estimated as follows:

Freight Trains

Track Features	SSFL	
DEFLECTION WALL	No	
Freight Line Derailment Frequency	1.70E-06	per movement
Freight Movements	24,500	per year
Per Study Length (Derailment Zone)	0.1	km
Total Derailment Frequency	4.16E-03	per year



5.8 TRAIN COLLISION ANALYSIS

In the event of an incident, the risk to an individual is a function of the probability of fatality per incident is estimated as:

- Line Frequency = Frequency in Study Zone / Passenger movements per year

Thus for freight trains on the SSFL:

- Major Collapse = $1.31\text{E-}05$ (ex Event Tree) / 24,500 (sect 5.3) = $5.34\text{E-}10$
- Partial Collapse = $5.61\text{E-}06$ (ex Event Tree) / 24,500 (sect 5.3) = $2.29\text{E-}10$

Applying this to the SSFL and major collapse scenario we have:

Freight & Diesel Passenger Movements

Partial Collapse	2.29E-10	per movement
Major Collapse	5.34E-10	per movement
	7.63E-10	

5.9 FATALITY RISK per YEAR

In the event of an incident, the risk to an individual is a function of the probability of fatality per incident, based upon groups and numbers of persons potentially exposed to the incident frequency and train movements. The number of individuals at risk is calculated from:

(Frequency Collapse x Probability of Fatality x Probability of Exposure) x Train Movements)

For a structure such as the residential development the probability of fatality has been estimated at 0.4 for a driver or observer in the train cab when a structure collapses on the train and the probability of exposure is 0.125 for a partial collapse and 0.25 for a major collapse. The probability of exposure to either scenario is 0.1. A train (path) is available 2 trips per day (up and down), 5 days per week, for 52 weeks minus 4 weeks possessions and track outages, that is $(2 \times 5 \times 48) = 480$ train paths utilisation by nominated trains per annum.

Thus daily risk per annum:

(Major Collapse Freq x Prob Fatality Multistorey Collapse x Prob Exposure to Fatality x Probability Exposure to Event) + (Partial Collapse Freq x Prob Fatality Multistorey Collapse x Prob Exposure to Fatality x Probability Exposure to Event))
x Train Movements on SSFL

$$= ((5.34\text{E-}10 \times 0.4 \times 0.250 \vee 0.1) + (2.29\text{E-}10 \times 0.8 \times 0.125 \times 0.1)) \times 480 = 3.11\text{E-}09$$

Applying this to the categories of individuals at risk we have:

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Individual Risk (Fatality) Per Annum

Freight Movements

Train (Loco) Driver	3.11E-09	per year
Building Occupant	7.24E-07	per year

The frequency of a fatal incident has been calculated for partial and major collapse scenarios for which incident frequency has been estimated on the Event Trees (section 5.4) for both peak (35%) on off-peak (65%) services. For instance:

- Up Main West Line Partial Collapse during Peak Services = $5.61\text{E-}06 \times 35\%$
= $1.96\text{E-}06$

Applying this for both Partial and Major Collapse scenarios during peak and off-peak services we have:

Freight Movements	Fatal Incident Frequency
Peak - Partial Collapse	1.96E-06
Peak - Major Collapse	4.58E-06
Off-Peak - Partial Collapse	3.65E-06
Off-Peak - Major Collapse	8.51E-06
	1.87E-05
	<i>per year</i>

The societal risk can be assessed by assuming a range of exposures for persons in trains and building occupants. Thus in the case of a Partial Collapse during peak services the number of potential fatalities may be calculated from:

- Number of person exposed in train x probability of Exposure to Partial Collapse x Probability of Fatality

For freight trains:

- 2 persons in driver cab x 0.5 (building partial collapse on train) x 0.2 (Fatality) = 0.2, say 1 train cab occupant

While for persons in the building who could be impacted in the case of partial collapse we have:

- (20 persons exposed in the building) x 0.125 (likelihood strike by partial collapse) x 0.4 (multi-storey structure) = 1

Applying this for both Partial and Major Collapse scenarios during peak and off-peak services we have:

<u>Societal Risk-</u> Scenarios	Potential Number Persons Exposed in the environ or vicinity of:		
	Train	Development	Total
Freight Movements			
Peak - Partial Collapse	1	1	2
Peak - Major Collapse	1	2	3
Off-Peak - Partial Collapse	1	1	2
Off-Peak - Major Collapse	1	2	3

**DERAILMENT SCENARIOS on SSFL adjacent
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By multiplying the fatal incident frequency by societal risk we can see that the highest risk exposure will be to train passengers during the peak:

Scenario	Risk Contribution	
Freight Movements		
Partial Collapse - Train Fatalities - Peak	3.93E-07	2.9%
Partial Collapse - Train Fatalities - Off-Peak	7.29E-07	5.4%
Partial Collapse - Bridge Fatalities - Peak	7.36E-07	5.4%
Partial Collapse - Bridge Fatalities - Off-Peak	5.47E-07	4.0%
Major Collapse - Train Fatalities - Peak	1.83E-06	13.4%
Major Collapse - Train Fatalities - Off-Peak	3.40E-06	25.0%
Major Collapse - Bridge Fatalities - Peak	3.44E-06	25.2%
Major Collapse - Bridge Fatalities - Off-Peak	2.55E-06	18.7%
Total Potential Loss of Life per year	1.36E-05	100.0%

5.10 RISK ASSESSMENT

It has been estimated a derailment could occur once every 241 years, and following such an incident, the consequences analysed in the Event Tree forecasts that:

- A derailed loco hauled train is most likely to proceed along the track parallel the site because:
 - The track is straight parallel the site, and on the approach from Liverpool curves slightly away towards the adjacent Down Main South Line
 - the track is fit for purpose, and maintained fit for purpose, is continuous with no signal catchpoints, crossovers or other discontinuities that are potential causes of derailment following a train passing its limit of authority (SPAD) or owing to track deterioration
 - the Fault Tree indicates the most likely cause is a Rollingstock fault, and these most commonly occur with wagons eg cracked wheels, and in locos tend to 'drag' these derailed wagons in the consist without derailing the locos eg GM Freight Network Efficiency, Freight & Regional Development, TfNSW, advised of freight incident at Wolli Creek in September 2011 when a hopper door on a coal train dislodged under a wheel resulting in a derailment and damaged infrastructure – but the derailed wagons remained within the cess; confirmed by Asset Engineer, Track, T4 (Illawarra) Asset Planning & Performance, confirmed derailment of freight train owing to hopper door falling off under wheel, and that train on the Up Illawarra Local remained contained between the tracks with subsequent damage to track

The event of a train actually 'jumping' the outer rail of the SSFL, crashing through the signal troughing (GST) and beyond the rail boundary thence down the embankment and actually impacting with building support structure and incurring fatalities has been assessed in the Collision Analysis as a possible occurrence every 53,466 years.

Thus, in the qualitative assessment (see Appendix A), applying the scenario of a fatality every 53,488 years – in the SMS TERM as specified in T-HR-CL-12080-ST this is rated as 1 person (C2) less than once every 1,000 years (L6) from train occupants and persons in and about the new buildings, that equates to a risk level of 'C'.

6. CONCLUSION

The existing SSFL track profile on the approach to the unit developments at 21 Atkinson Street, Liverpool is consistent with the fact that:

- Track is configured and maintained fit for purpose for the speeds currently placed as per Train Operating Conditions Manual
- Current risk profile from hazards of a derailment or collision is managed As Low As Reasonably Practicable (ALARP)

The hazard scenarios have been quantitatively estimated as:

- Derailment incident once every 241 years
- Average derailment frequency 1.70E-07 per train movement
- Fatal incident every 53,466 years (as a consequence of development)

The desk-top qualitative assessment in Appendix A is based upon the likelihood of a derailment once every 100 - 1,000 years and subsequent collision with building support less than once every 1,000 years and a consequence of impact with building of 1 fatality and/or 10 major injuries, giving a risk ranking 'C' – given the unprecedented likelihood of an impact, this risk ranking remains 'C' even if more fatalities are factored into the outcome event.

Then design specifications that comply with T-HR-CI-12080-ST and AS5100 practices and code will manage the risk So Far As Is Reasonably Practicable (SFAIRP). These specifications are based upon the risk assessment and qualitative ranking in the TERM risk matrix and indicate that:

- Based upon risk assessment of 'C' for likelihood and consequence of derailed train impacting building structures then AS5100 the additional (Rail) Collision Loading Requirement for the support structures of the building adjacent the rail line is 'Nil'

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7. APPENDIX A – HAZARD AND RISK WORKSHEETS

Based upon conversations with Stakeholders and SMEs and historical incident data records and information eg IIMS

Hazard Information			Existing Risk Controls	Original Risk			Proposed Additional Risk Control	Residual Risk			
Hzd Ref.	POTENTIAL CAUSES	POTENTIAL CONSEQUENCES (Worse Case)	CONTROLS	Likelihood	Consequence	Risk	ADDITIONAL CONTROLS	Likelihood	Consequence	Risk	COMMENTS
STRUCTURAL COLLAPSE OF BUILDING OVER TRAIN FOLLOWING:											
1	TRAIN DERAILMENT owing to track condition on SSFL, derailed train travels: 1. Beyond tracks ie outside 4 ft, thence 2. Beyond cess, GST and boundary, thence 3. Down embankment to nearest structures thence 4. Strikes building supports located at min 11m from centre line of SSFL	DERAILED TRAIN IMPACTS BUILDING located between 11-14m from centre line of SSFL on the embankment; hence direct impact causing structural collapse of multi storey building unlikely. Most likely case estimate of consequence of derailment travelling beyond rail boundary and base of embankment is 1 Fatality / 10 Major Injuries	- Track designed to ENGINEERING STANDARDS - Line Speed @ 115/100kph between Liverpool and Casula stations - Class 1 track (60kg rail) and concrete sleepers - No structure (crossovers, catchpoints) in track on approach to site - Signal headway designed for line speeds - No points, crossovers or catchpoints that would direct derailed train towards site - No shunting on this section of track - Inspection / Maintenance conducted in accordance with Technical Maintenance Plans and Works Orders, track fit for purpose - Line speed / traffic restrictions during inclement weather, noting that track is well drained and ballasted on an embankment - WOLO procedures for installing track speed restrictions in hot weather - Building Support Structure located at least 16m from SSFL running line - Operations Rail operations and rollingstock infrastructure referred section 5.3, and consequence of building development is that more persons are potentially exposed such as occupants of the new building - Track Curved section of track away from site on approach from Liverpool, straight through site and thence to Casula, concrete sleepered and fit for purpose at indicated track speeds - Rollingstock Freight with some diesel passenger trains scheduled	L6	C2	C	Nil on basis that: - Design & configuration of Building complies with Good Practice / Law / Codes of Practice in particular risk analysis as per T-HR-CL-12080-ST Table 1, clause 5.13, for specifying AS5100 Collision Loading Requirement for support structures located between 10 to 20 metres from the rail line meets ALARP requirement - Design is Reasonably Practicable On this basis SFAIRP achieved	L6	C2	C	Historical records indicate no derailments on record in ARTC, IIMS or RIC databases since 1998 owing to infrastructure cause on this section through the site

**DERAILMENT SCENARIOS on SSFL adjacent
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Hazard Information			Existing Risk Controls	Original Risk			Proposed Additional Risk Control	Residual Risk			
Hzd Ref.	POTENTIAL CAUSES	POTENTIAL CONSEQUENCES (Worse Case)	CONTROLS	Likelihood	Consequence	Risk	ADDITIONAL CONTROLS	Likelihood	Consequence	Risk	COMMENTS
2	REAR-END TRAIN COLLISION causes derailment of leading train that could strike building	DERAILED TRAIN IMPACTS BUILDING located between 11-14m from centre line of SSFL on the embankment; hence direct impact causing structural collapse of multi storey building unlikely. Most likely case estimate of consequence of derailment travelling beyond rail boundary and base of embankment is 1 Fatality / 10 Major Injuries	- Signal headway designed for line speeds; signals on approach to site - Track designed to ENGINEERING STANDARDS - Line Speed @ 115/100kph between Liverpool and Casula stations - Class 1 track (60kg rail) and concrete sleepers - No structure (crossovers, catchpoints) in track on approach to site - No shunting on this section of track - Inspection / Maintenance conducted in accordance with Technical Maintenance Plans and Works Orders, track fit for purpose - Line speed / traffic restrictions during inclement weather, noting that track is well drained and ballasted on an embankment - WOLO procedures for installing track speed restrictions in hot weather - Building Support Structure located at least 16m from SSFL running line - Operations Rail operations and rollingstock infrastructure referred section 5.3, and consequence of building development is that more persons are potentially exposed such as occupants of the new building - Track Curved section of track away from site on approach from Liverpool, straight through site and thence to Casula, concrete sleepers and fit for purpose at indicated track speeds - Rollingstock Freight with some diesel passenger trains scheduled	L6	C2	C	Nil on basis that: - Design & configuration of Building complies with Good Practice / Law / Codes of Practice in particular risk analysis as per T-HR-CL-12080-ST Table 1, clause 5.13, for specifying AS5100 Collision Loading Requirement for support structures located between 10 to 20 metres from the rail line meets ALARP requirement - Design is Reasonably Practicable On this basis SFAIRP achieved	L6	C2	C	Historical records indicate no rear end collisions on record in IIMS or RIC databases since 1998 owing to infrastructure cause on this section through the site
3	Incidental Causes of DERAILMENT or COLLISION Train on FIRE on track adjacent development	BUILDING located between 11-14m from centre line of SSFL on the embankment; hence direct impact causing structural collapse of multi storey building unlikely. Most likely case	Design Fire & Life Safety to comply with appropriate Fire Rating requirements: - BCA minimum fire rating ref egress from building - Also fire rating under Sydney Trains / RailCorp codes to comply with Sydney Trains / RailCorp obligations under the RSNL 2014 (Rail Safety Act and Regulations) in particular for provision of safe egress for persons on railway property	L6	C2	C	Nil on basis that: Design & configuration complies with relevant codes and standards and SFAIRP achieved	L6	C2	C	
	HEAT Buckle during hot weather		WOLO procedures for installing track speed restrictions in hot weather	L6	C2	C	Nil on basis as per 1 previous	L6	C2	C	

**DERAILMENT SCENARIOS on SSFL adjacent
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Hazard Information			Existing Risk Controls	Original Risk			Proposed Additional Risk Control	Residual Risk			
Hzd Ref.	POTENTIAL CAUSES	POTENTIAL CONSEQUENCES (Worse Case)	CONTROLS	Likelihood	Consequence	Risk	ADDITIONAL CONTROLS	Likelihood	Consequence	Risk	COMMENTS
	FLOODING during heavy rains	estimate of consequence of derailment travelling beyond rail boundary and base of embankment is 1 Fatality / 10 Major Injuries	- Track designed to ENGINEERING STANDARDS - Infrastructure Maintenance conducted Inspection / Maintenance in accordance with Technical Maintenance Plans and Works Orders - Line speed / traffic restrictions during floods, noting that track is well drained and ballasted	L6	C2	C	Nil on basis as per 1 previous	L6	C2	C	

8.6. STRUCTURAL DESIGN CERTIFICATE

17/08/16

STRUCTURAL DESIGN CERTIFICATE

Project No: 160507

Project Title: 21 Atkinson St Liverpool

Design Phase

We, Xavier Knight Consulting Engineers Pty Ltd, hereby certify that we are responsible for the structural design of the works shown on the following drawing No's;

Drawing Number	Drawing Title
S0101	BUILDING SHORING AND EARTHWORKS PLAN
S0111	SHORING WALL ELEVATION
S0121	RAIL CORRIDOR SECTIONS
S0122	RAIL CORRIDOR SECTIONS
S0123	RAIL CORRIDOR SECTIONS
S0131	SHORING DETAILS
S0201	FOOTING PLAN
S0211	FOOTING DETAILS
S0901	GROUND LEVEL CONCRETE OUTLINE PLAN
S1001	FIRST FLOOR CONCRETE OUTLINE PLAN
S2001	SECOND FLOOR CONCRETE OUTLINE PLAN
S3001	THIRD FLOOR CONCRETE OUTLINE PLAN
S4001	FOURTH FLOOR CONCRETE OUTLINE PLAN

We certify that this work has been designed to the requirements of letter named "STATE ENVIRONMENTAL PLANNING POLICY (INFRASTRUCTURE) 2007, DEVELOPMENT APPLICATION- DA - 84/2016 - 21 Atkinson Street, Liverpool dated 23rd March 2016.

We certify that this work has been designed in accordance with the relevant clauses of the BCA and the following Australian Standards:

AS 1170.0 Structural design actions – General principles
AS 1170.1 Structural design actions – Permanent, imposed and other actions
AS 1170.4 Structural design actions – Earthquake actions in Australia
AS 3600 Concrete Structures
AS 2159 Piling- Design and Installation
AS 4678 Earth Retaining Structures

For and on behalf of

XAVIER KNIGHT CONSULTING ENGINEERS PTY LTD



FERIS CHEHADE BE(Hons) MIEAust CPEng NPER
Director