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Our Ref: PSM3402-003L

17 October 2017

JBS&G
Level 1, 50 Margaret Street
SYDNEY NSW 2000

ATTENTION: JOANNE ROSNER
By email: JRosner@jbsg.com.au

Dear Joanne

RE: URBANGROWTH SYDNEY METRO SITE, EPPING
GEOTECHNICAL INVESTIGATION

1 INTRODUCTION

Pells Sullivan Meynink (PSM) is pleased to present the results of our geotechnical site investigation at Epping for the Sydney Metro Northwest Urban Transformation Project. This work has been undertaken in accordance with our email proposal dated 9 August 2017.

Prior to the fieldwork, we were supplied with the following documents:

- North West Rail Link – Major Civil Construction Works– Chapter 8 and 9 – “Soils and Groundwater“
- AECOM Technical Paper – “Surface Water and Hydrology – Major Civil and Construction Works – EIS 1” dated 26 March 2012
- Coffey and AECOM Figure – “Draft Diagrammatic Geotechnical Long Section” dated 29 February 2017
- Insites Survey – “Topographic Survey Information – North West Rail Link – Epping” Drawing No. NWRL-10045-10-SWD-DRG-LS-60999-A-MAPSHEETS, dated 23 January 2012

PSM note the following about the site and proposed works:

- The majority of the site is currently used for construction access to the Sydney Metro tunnels.
- The proposed land use is for high-density residential buildings.

2 PREVIOUS INVESTIGATION

A number of previous geotechnical investigations have been completed in the area surrounding the UrbanGrowth development site as part of development of the geotechnical ground model for the North West Rail Link tunnels. Locations and inferred RL of top of bedrock at relevant boreholes are shown in Figure 1.

Previous investigations indicate that the top of the bedrock is highly weathered Sandstone of medium strength. The depth to the top of bedrock is relatively shallow varying from 2.0 m to 4.0 m below the existing ground surface. The strength and weathering of the bedrock rapidly improves with depth. Results of previous investigations have been considered when providing advice for the proposed development in Section 5.

3 FIELDWORK

The fieldwork was undertaken by PSM on 25 September 2017 and comprised the following:

- Eight (8) boreholes
- Two (2) dynamic cone penetrometer (DCP) tests

Test locations were measured using a tape measure relative to existing site features. Approximate test locations are shown in Figure 2. Selected site photographs taken during the fieldwork are included in Figures 3 and 4.

3.1 Boreholes

A total of eight (8) boreholes were drilled on 25 September 2017 under the supervision of PSM. Six (6) boreholes (BH2, BH7, BH8, BH9, BH10 and BH11) were drilled using a 3.5 tonne excavator with 200 mm diameter auger attachment. The existing concrete slab was cored where required. Two (2) boreholes (BH3 and BH6) were excavated using a hand auger. All boreholes were drilled to 3 m or prior refusal. Prior to excavation, test locations were scanned for buried services.

Tabulation of material encountered is provided for each borehole in Attachment A.

We understand a further three (3) boreholes were drilled as part of environmental investigations. PSM were not present during the drilling and have not provided logs for these holes.

3.2 DCP Testing

Two (2) Dynamic Cone Penetrometer (DCP) tests were completed at the site. Depth to termination of each test is summarised in Table 1. A tabulation of DCP test results is included as Attachment B.

We have inferred that practical refusal indicates the likely presence of:

- Floaters (boulders) within a unit of fill at BH3.
- Bedrock or a boulder within the fill at BH6.

TABLE 1
DEPTH TO TERMINATION OF TEST

TEST	DEPTH TO REFUSAL OF TEST (m)
DCP01	0.7 R
DCP02	2.7 R

R – Indicates refusal

4 SITE CONDITIONS

4.1 Geological Setting

The 1:100,000 geological map for the Sydney region indicates that the site is underlain by Ashfield Shale of the Wianamatta Group.

4.2 Surface Conditions

At the time of fieldwork, majority of the site was being used for construction access to the Sydney Metro tunnels and comprised concrete slabs, car parks, driveways, grassed and landscaped areas.

4.3 Subsurface Conditions

The subsurface conditions encountered within the boreholes are summarised in Table 2 and Table 3. The encountered subsurface conditions were generally consistent with the published information and previous investigations. Tabulated logs of material encountered are provided in Attachment A.

TABLE 2
SUMMARY OF INFERRED SUBSURFACE CONDITIONS ENCOUNTERED IN GEOTECHNICAL BOREHOLES

INFERRED UNIT	INFERRED TOP OF UNIT DEPTH BELOW GROUND SURFACE (m)	DESCRIPTION
TOPSOIL	0.0	Sandy SILT with some gravel; brown, fine grained sand, sub-angular gravel up to 25 mm, loose consistency, dry, organics and bark observed.
FILL	0.0	Existing concrete slab. Sandy GRAVEL to gravelly sandy CLAY, sub-rounded to angular gravel up to 40 mm, fine to medium grained sand, low to high plasticity clay, medium dense to firm to stiff consistency, dry to moist.
NATURAL SOIL	0.1	Clayey SAND with some gravel; medium grained, yellow and orange, low plasticity clay, sub-angular gravel up to 30 mm, medium dense consistency, moist.
BEDROCK	0.8 to > 3.0	SANDSTONE: reddish brown to brown and grey, extremely weathered to highly weathered, extremely low to low strength.

Previous site investigations indicate the Sandstone bedrock strength rapidly improves with depth with medium and high strength sandstone present below 3.0 m of the surface.

TABLE 3
APPROXIMATE DEPTH OF TOP OF INFERRED GEOTECHNICAL UNITS ENCOUNTERED IN GEOTECHNICAL BOREHOLES

BOREHOLE ID	APPROXIMATE DEPTH TO TOP OF INFERRED GEOTECHNICAL UNITS (m)				
	TOPSOIL	FILL	NATURAL SOIL	BEDROCK	EOH
BH02	N.E.	0.0	N.E.	1.7	1.8^{R}
BH03	0.0	N.E.	N.E.	N.E.	0.3^{R*}
BH06	0.0	N.E.	N.E.	N.E.	0.3^{R*}
BH07	N.E.	0.0	N.E.	N.E.	1.0^{R}
BH08	N.E.	0.0	0.11	0.9	1.0^{R}
BH09	N.E.	0.0	0.14	0.8	1.0^{R}
BH10	N.E.	0.0	0.12	0.8	0.8^{R}
BH11	N.E.	0.0	N.E.	N.E.	3.0

Note: 'R' denotes practical refusal with 3.5 tonne excavator
'R*' denotes practical refusal with hand auger
'N.E.' denotes not encountered

4.4 Groundwater

Groundwater was not observed within the boreholes.

5 ADVICE AND RECOMMENDATIONS

5.1 Excavation Conditions

Excavation within the soil units (TOPSOIL, FILL and NATURAL SOIL) should be achieved with conventional earthmoving equipment.

Excavation within the BEDROCK unit is likely to require rock breaking equipment.

We note that an experienced contractor should make their own assessment of the appropriate excavation equipment. The contractor should recognise that there is a potential for damage to adjacent buildings and consider this in planning and executing its work. It is our experience that excavatability is heavily dependent on both the operator and the plant used.

Heavy rock breaking equipment will generate vibrations that may impact on neighbouring structures. Where controls on vibrations are required, the contractor should consider the use of smaller hammers, rock saws and grinders to undertake the excavation. The use of “pre-split” cuts along the boundaries using a rock saw can provide a “buffer” for vibrations.

5.2 Earthworks

Details of proposed earthworks are currently unclear. Any minor filling (filling up to 1500 mm deep) required to bring the exposed subgrade to the finished level, should be placed as follows:

- Engineered Fill material to be inspected and approved by PSM.
- Engineered Fill to be placed and compacted as follows:
  - For cohesive material (clayey sand to clay): a dry or Hilf density ratio of between 98% and 102% (Standard) and moisture variation of between 2% dry and 2% wet, unless otherwise directed by PSM.
  - For cohesionless material (sand): a minimum Density Index of 75%.
- Engineered Fill to be placed in compacted layers not greater than 300 mm in thickness.
- Engineered Fill to be placed in Lots that are defined as a single layer of Engineered Fill consisting of uniform material which has undergone similar treatment.
- The minimum density testing frequency to be taken as follows:
  - For Lots less than 30 m^3 – 1 test per Lot
  - For Lots between 30 m^3 to 150 m^3 – 2 tests per Lot
  - For Lots greater than 150 m^3 – shall not be less than the greater of:
    - 1 test per 500 m^3 of material placed
    - 3 tests per lot
- If any one test undertaken within a Lot fails, the whole of the Lot shall be reworked and retested, i.e. “a none to fail basis”.
- A Geotechnical Inspection and Testing Authority (GITA) shall be engaged to undertake the Level 1 role and certify that the earthworks have been completed in accordance with this letter.
- Upon completion of the earthworks, PSM should be provided with the test results and GITA certificate for review.
- PSM should be requested to proof roll the finished surface.

Should filling be required to depths greater than 500 mm, an appropriate Bulk Earthworks Specification should be developed. The Specification is generally tailored for the performance requirements of the proposed development. At this stage, we are not aware of any performance requirements. If required, PSM are happy to work with UrbanGrowth to develop an appropriate Specification. This is beyond the scope of this report.

5.3 Permanent and temporary batters

The batter slope angles shown in Table 4 are recommended for the design of batters up to 3 m height and above the groundwater table; subject to the following recommendations:

1. The batters shall be protected from erosion.
2. Permanent batters shall be drained.
3. Temporary batters shall not be left unsupported for more than 1 month without further advice, and inspection by a geotechnical engineer should be undertaken following significant rain events.
4. Where loads are imposed or structures/services are located within one batter height of the crest of the batter, further advice should be sought.
5. Exposed rock faces should be inspected by an experienced geotechnical engineer or engineering geologist during excavation at 1.5m lifts to assess the need for localised rock bolting and/or shotcreting to control adverse jointing in the BEDROCK unit and for overall face support.

TABLE 4
BATTER SLOPE ANGLES

UNIT	TEMPORARY	PERMANENT
FILL	2.0H : 1V	2.5H : 1V
NATURAL SOIL	2.0H : 1V	2.5H : 1V
BEDROCK	Vertical*	Vertical*

Note: *: See above requirements regarding inspections and local support.

Steeper batters may be possible subject to further advice, probably including inspection during construction.

5.4 Excavation support

Permanent cuts, particularly for basement excavations in soil units (FILL and NATURAL SOIL), and BEDROCK units steeper than the recommended permanent batter slopes in Table 4 will need to be supported by some form of retaining structure.

The design of retaining structures, including basements, should be based on the following:

- Effective soil strength parameters in Table 5, and
- Water pressure (depending on the type of the structure).

With regards to the BEDROCK unit, the designer shall allow a minimum lateral pressure of 10 kPa for the BEDROCK unit when cut vertical. This is to allow for blocks and rock wedges formed due to adverse defects that may exist within the unit. These loads may be able to be reduced by specifying inspections during the works and provision of additional support (rock bolts, shotcrete etc) should the inspection indicate that support is required.

Note that design of retention systems may be based on either K_a or K_o earth pressures. Design using active earth pressures provides the minimum lateral earth pressure that must be supported to avoid failure and requires a wall that can rotate or translate to allow the pressures to reduce to these values (vertical and lateral movements up to 2% of height may occur, typical movements will be much less).

Where the design is based on K_o pressures to control excavation induced deformations, the construction should be carefully controlled. It should be noted that designing for K_o pressures does not, of itself, ensure that movement does not occur. Movements are controlled by the construction method, especially sequence.

If relying on passive support from embedment of piles into the BEDROCK unit (e.g. cantilever piled wall or propped or anchored piled wall), the designer shall ignore the support provided in the upper 1.0 m of embedment and can adopt a lateral resistance of one third of the ABP in Table 5.

Both surface and sub-surface drainage needs to be designed and constructed properly to prevent pore water pressures from building up behind the retaining walls or appropriate water pressures must be included in the design.

5.5 Groundwater and Effect on Basements

We note groundwater was not observed during the investigation. However, based on previous investigations and PSM's experience with the Northwest Rail Link tunnels, high groundwater inflows are possible at this site. Where excavation below the water table is proposed for the proposed basements, construction stage dewatering will be required.

We note that recent experience indicates that the New South Wales Office of Water (NoW) have been conditioning approval of basement excavation on the basis of:

- Temporary dewatering allowed during excavation. Permits will need to be sought for both extraction of the water and disposal.
- No inflows into the basement allowed in the permanent condition. That is the final basement needed to be water tight, i.e. tanked. Such a requirement results in the basement floor slab and the walls needing to be designed for full hydrostatic load below a maximum foreseeable water table. This requirement, if enforced by the regulatory authorities, may influence decisions regarding the feasibility of deep basement excavations. It is our experience that NoW may relax the requirement for tanking where monitoring of groundwater levels in combination with analysis of inflows in the permanent condition indicate yearly inflow into the excavation of less than 3 ML/yr and that the extraction of groundwater has no adverse impact on other groundwater users. Please refer to NoW Aquifer Interference Policy for details.

- The developments are usually conditioned on monitoring of groundwater levels, assessment and estimation of temporary inflows during construction, and assessment of effect on neighbouring structures.

PSM can undertake the above, but this is outside the scope of this report.

5.6 Foundation

5.6.1 Shallow footings

Pad footings can be proportioned on the basis of an allowable bearing pressure (ABP) for centric vertical loads provided in Table 5. Higher ABPs in soil units may be available, but these depend on the size, depth, loads, etc and would be subject to specific advice. The ABP needs to be confirmed by a geotechnical engineer during an inspection.

Settlements in soil units can be estimated using the elastic parameters provided in Table 5. We note that allowable bearing pressures presented in Table 5 assume a settlement of approximately 1% (or less) of the least footing dimension for footings in the BEDROCK unit.

TABLE 5
ENGINEERING PARAMETERS OF INFERRED GEOTECHNICAL UNITS

INFERRED UNIT	BULK UNIT WEIGHT (kN/m^3)	SOIL EFFECTIVE STRENGTH PARAMETERS		ULTIMATE BEARING PRESSURE UNDER VERTICAL CENTRIC LOADING (kPa)	ALLOWABLE BEARING PRESSURE (ABP) UNDER VERTICAL CENTRIC LOADING (kPa)	ULTIMATE SHAFT ADHESION (kPa)	ELASTIC PARAMETERS	
		c' (kPa)	*ϕ'* (deg)				LONG TERM YOUNG MODULUS (MPa)	POISSON'S RATIO
FILL	18	0	30	420[1]	150[1]	NA	10	0.3
NATURAL SOIL	18	0	30	420[1]	150[1]	NA	15	0.3
BEDROCK	22	NA	NA	30,000[2]	4,500[3]	1,000	750	0.2

Note:
1. Pad footings in SOIL units should have a minimum horizontal dimension of 1.0 m and a minimum embedment depth of 0.5 m.
2. Ultimate values occur at large settlement (>5% of minimum footing dimensions).
3. End bearing pressure to cause settlement of <1% of minimum footing dimensions.

5.6.2 Piles

Piles should be designed in accordance with the requirements in AS 2159-2009, *Piling - Design and Installation*. The parameters provided in Table 5 may assist in the design of piles within the BEDROCK unit.

In general, the designer should note the following with regards to pile design:

- The ABP needs to be confirmed by a geotechnical engineer during a pile inspection.
- Under permanent load, the contribution of side adhesion for soils including FILL and NATURAL SOIL should be ignored.

- Deflection needs to be checked using the recommended elastic parameters in Table 5.

The bearing capacities provided are contingent on piles or footings being vertically and centrally loaded. Further advice should be sought if the footings are not vertically centrically loaded.

With regards to the pile design, we recommend that:

- A geotechnical strength reduction factor, Φ_g = 0.60 (AS2159 CL. 4.3.2) be adopted for a high redundancy system for an assessed average risk rating (ARR) between 2.5 and 3.0. This should be reviewed to suit the specific design and appropriate pile testing proposed by the structural designers in accord with the requirements of AS2159.
- It may be possible to increase the pile reduction factors, if the details of the proposed pile installation procedures indicate a high level of quality control with regards to concrete placement, base cleanliness, etc.
- If a geotechnical strength reduction factor, Φ_g = 0.40 is adopted then no pile testing will be required (AS2159 CL. 8.2.4 (b)).

5.7 Slab on ground

The design of slabs on ground on the FILL and NATURAL SOIL units can be based on a subgrade with a long term Young's Modulus recommended in Table 5.

The soil subgrade will need to be prepared or compacted using a smooth drum vibratory roller, e.g. with a 10 tonne roller.

5.8 Possible Effects on Neighbouring Structures Including Rail Tunnels

Developments adjacent or above rail tunnels need to comply with TfNSW standard T HR CI 12051 ST – "Development Near Rail Tunnels", version 1.0 dated 14 November 2016. Typically this involves the following geotechnical components:

- Assessment of foundation geometry, excavations and ground support with respect to rail protection reserves.
- Assessment of foundation loads with respect to load limits. The load limit for shallow footings above the ECRL tunnels is 500 kPa above the top of the second reserve.
- Engineering assessment including:
  - Impact assessment report
  - Vibration and monitoring assessment
  - Ground monitoring plan

PSM can assist with the above when details of the proposed development are well defined. This is outside the scope of this report.

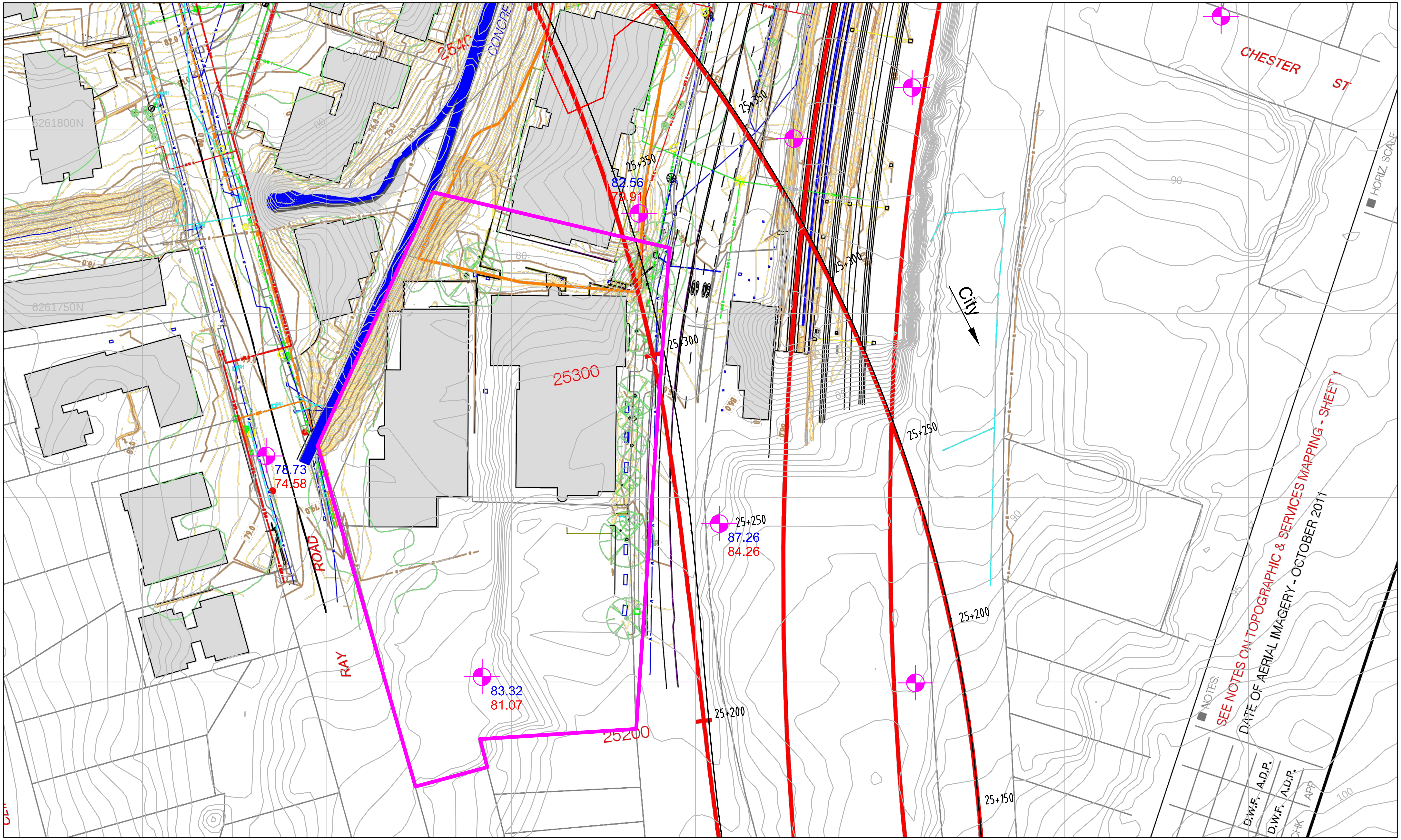
Please do not hesitate to contact the undersigned if you have any queries.

For and on behalf of
PELLS SULLIVAN MEYNINK

MATTHEW HAERTSCH
Geotechnical Engineer

DAVID PICCOLO
Principal

Encl.	Figure 1	Previous Geotechnical Investigations – Locality Plan
	Figure 2	Geotechnical Investigation – Locality Plan
	Figure 3	Selected Site Photos (1 of 2)
	Figure 4	Selected Site Photos (2 of 2)
	Attachment A	Tabulated Borehole Logs
	Attachment B	DCP Results



LEGEND

83.32
81.07

BH Location, Collar RL (m AHD), Inferred Top of Rock RL (m AHD)

Approximate Site Extents

0 10 20 30 40 50
Scale(m)

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UrbanGrowth Site
Epping, NSW

PREVIOUS GEOTECHNICAL INVESTIGATIONS
LOCALITY PLAN

PSM3402-003L	Figure 1

Notes:

1. Borehole location (approximate)

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JBS&G UrbanGrowth Site Epping, NSW GEOTECHNICAL INVESTIGATION LOCALITY PLAN	
PSM3402-003L	FIGURE 2

Photo 1 - General Site Conditions - View to the North

Photo 2 - General Site Conditions - View from the East

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Epping, NSW

SELECTED SITE PHOTOS [1 OF 2]

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PSM3402-003L	**Figure 3**

Photo 3 - General Site Conditions - View from the North

Photo 4 - General Site Conditions - Car Park

Photo 5 - General Site Conditions - View from the South

JBS&G **UrbanGrowth Site** **Epping, NSW** **SELECTED SITE PHOTOS [2 OF 2]**	
PSM3402-003L	**Figure 4**

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ATTACHMENT A

TABULATED BOREHOLE LOGS

TABLE 1
SUMMARY OF SUBSURFACE CONDITIONS

TEST PIT	DEPTH	MATERIAL ENCOUNTERED
BH02	0.0 – 0.05 m	Asphalt / Bitumen wearing course (50 mm thick)
	0.05 – 0.3 m	FILL: Sandy GRAVEL; sub-angular to angular up to 25 mm, grey, fine to medium grained sand, medium dense consistency, moist.
	0.3 – 1.0 m	FILL: Sandy CLAY with some gravel; medium to high plasticity, pale brown and grey, fine to medium grained sand, sub-rounded to angular gravel up to 35 mm, firm consistency, moist. Concrete observed in fill at approximately 0.5 m.
	1.0 – 1.7 m	FILL: Clayey SAND with some gravel; fine to medium grained, pale brown and grey, low plasticity clay, sub-rounded gravel up to 40 mm, medium dense consistency, moist. Crushed sandstone observed throughout fill.
	1.7 – 1.8 m	SANDSTONE: reddish brown, extremely weathered, extremely low to very low strength.
	1.8 m	Hole Terminated at 1.8 m due to practical refusal with 3.5 tonne excavator.
BH03	0.0 – 0.3 m	TOPSOIL: Sandy SILT with some gravel; brown, fine grained sand, sub-angular gravel up to 25 mm, loose consistency, dry, organics and bark observed down to 0.3 m.
	0.3 m	Hole terminated at 0.3 m due to refusal with hand auger.
BH06	0.0 – 0.4 m	TOPSOIL: Sandy SILT with some gravel; brown, fine grained sand, sub-angular gravel up to 25 mm, loose consistency, dry, organics and bark observed down to 0.4 m.
	0.4 m	Hole terminated at 0.4 m due to refusal with hand auger.

TEST PIT	DEPTH	MATERIAL ENCOUNTERED
BH07	0.0 – 0.13 m	Concrete slab (130 mm thick)
	0.13 – 0.4 m	FILL: Gravelly SAND; medium grained, orange and brown, sub-rounded to sub-angular gravel up to 40 mm, medium dense consistency, moist. Crushed sandstone observed throughout fill.
	0.4 – 1.0 m	FILL: Clayey SAND with some gravel; medium grained, brown, orange and black, low plasticity clay, sub-rounded to sub-angular gravel up to 30 mm, medium dense consistency, moist. Timber and bricks observed at 0.8m.
	1.0 m	Hole terminated at 1.0 m due to practical refusal on concrete with 3.5 tonne excavator.
BH08	0.0 – 0.11 m	Concrete slab (110 mm thick)
	0.11 – 0.9 m	Clayey SAND with some gravel; medium grained, yellow and orange, low plasticity clay, sub-angular gravel up to 30 mm, medium dense consistency, moist.
	0.9 – 1.0 m	SANDSTONE; grey, low strength, highly weathered.
	1.0 m	Hole Terminated at 1.0 m due to practical refusal with 3.5 tonne excavator.
BH09	0.0 – 0.14 m	Concrete slab (140 mm thick)
	0.14 – 0.8 m	Clayey SAND; fine to medium grained, brown and grey, low to medium plasticity clay, medium dense consistency, moist.
	0.8 – 1.0 m	SANDSTONE; brown and grey, extremely low strength, extremely weathered. Becomes low strength, highly weathered from 0.9 m.
	1.0 m	Hole Terminated at 1.0 m due to practical refusal with 3.5 tonne excavator.

TEST PIT	DEPTH	MATERIAL ENCOUNTERED
BH10	0.0 – 0.12 m	Concrete slab (120 mm thick)
	0.12 – 0.7 m	FILL: Gravelly SAND with some clay; fine to medium grained, orange, sub-rounded to sub-angular gravel up to 40 mm, medium plasticity clay, medium dense consistency, moist. Crushed sandstone observed throughout fill. Fragments of brick and terracotta observed at 0.5m.
	0.7 – 0.8 m	SANDSTONE; grey, low strength, highly weathered.
	0.8 m	Hole Terminated at 0.8 m due to practical refusal with 3.5 tonne excavator.
BH11	0.0 – 3.0 m	FILL: Gravelly sandy CLAY; low to medium plasticity, brown, medium grained sand, sub-angular to angular gravel up to 25 mm, firm to stiff consistency, dry. Becomes moist from 0.1 m. Metal and concrete observed at 0.3 m. Becomes low plasticity from 1.0 m. Sub-angular sandstone gravel up to 30 mm observed in fill from 1 m.
	3.0 m	Hole Terminated at 3.0 m.

ATTACHMENT B

DCP TEST RESULTS

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DYNAMIC CONE PENETROMETER TEST RESULTS

Job No.	PSM3402	**Sheet**	1 **of** 1
Project	UrbanGrowth Sydney Metro Site, Epping	**Date**	25-Sep-17
Test Method	*AS 1289.6.3.2. - 1997 Methods of Testing Soils for Engineering Purposes - 9 kg Dynamic Cone Penetrometer Test*	Drop Height	*510 mm*
		Hammer Mass	*9 kg*
Tested by		Tip Type	*CONICAL*

Test Depth	*DCP01*	*DCP02*	*DCP*	*DCP*	*DCP*	*DCP*
LOCATION	BH 3	BH 6				
0.10	-	-				
0.20	5	1				
0.30	22	5				
0.40	30	8				
0.50	12	17				
0.60	12	15				
0.70	25	0				
0.80	Refusal	6				
0.90	Bouncing	18				
1.00		6				
1.10		9				
1.20		13				
1.30		9				
1.40		10				
1.50		23				
1.60		17				
1.70		17				
1.80		18				
1.90		16				
2.00		16				
2.10		15				
2.20		17				
2.30		10				
2.40		10				
2.50		14				
2.60		16				
2.70		25+				
2.80		Refusal				
2.90		Bouncing				
3.00						
3.10						
3.20						
3.30						
3.40						
3.50						
3.60						
3.70						
3.80						
3.90						
4.00						

Comments: