GCA

Geotechnical Consultants Australia

J & CG Con NSW Pty Ltd

Geotechnical Investigation Report

Proposed Development at:

21 Whistler Street

Manly NSW 2095

G21699-1-Rev B

2nd June 2023

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Report Distribution

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Address: 21 Whistler Street Manly NSW 2095

GCA Report No.: G21699-1-Rev B

Date: 2nd June 2023

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Report Revision	Details	Report No.	Date	Amended By
0	Original Report	G21699-1	24th November 2021	-
1	Revision A	G21699-1-Rev A	22nd August 2022	GN/JN/MG
2	Revision B	G21699-1-Rev B	2nd June 2023	GA/JN

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1. INTRODUCTION

1.1 Background

This geotechnical engineering report presents the results of a geotechnical investigation undertaken by Geotechnical Consultants Australia Pty Ltd (GCA) for a proposed development at No. 21 Whistler Street Manly NSW 2095 (the site). The investigation was commissioned by Mr. Mark Guerreiro of J & CG Con NSW Pty Ltd (the client) and was carried out on the 11th November 2021.

The purpose of the investigation was to assess the subsurface conditions over the site at the selected boreholes and testing locations (where accessible and feasible) and provide necessary recommendations from a geotechnical perspective for the proposed development.

The findings presented in this report are based on our subsurface investigation, laboratory testing results and our experience with subsurface conditions in the area and local region. This report presents our assessment of the geotechnical conditions and has been prepared to provide preliminary geotechnical advice and recommendations to assist in the preparation of designs and construction of the ground structures for the proposed development.

For your review, **Appendix A** contains a document prepared by GCA entitled “Important Information About Your Geotechnical Report”, which summarises the general limitations, responsibilities and use of geotechnical engineering reports.

1.2 Proposed Development

Information provided by the client indicates the proposed development comprises demolition of existing infrastructures onsite, followed by construction of a new multi-unit residential development, overlying a basement level including a car turntable.

The Finished Floor Levels (FFL)s of the proposed developments basement and ground floor levels are set to be at Reduced Levels (RL)s of:

- Basement level: RL1.190m Australian Height Datum (AHD).
- Ground floor level: RL5.840m AHD.

Based on this information and the existing site levels and topography, maximum excavation depths varying from approximately 4.7m to 7.3m are expected to be required for construction of the proposed development. Locally deeper excavations for the lift shafts, and building footings and service trenches are also anticipated to be required as part of the planned development.

It should be noted that excavation depths are expected to vary across the site and are inferred off the FFLs shown on the architectural drawings and existing levels on the site survey plan, referenced in Section 1.3 below.

1.3 Provided Information

The following relevant information was provided to GCA prior to the geotechnical investigation and during preparation of this report:

- Architectural drawings prepared by Wolski Coppin Architecture, titled "Multi Unit Residential", and referenced project No. 21806.
- Site survey plan prepared by Norton Survey Partners, titled "Plan Showing Existing Building Locations No. 21 Whistler Street, Manly", referenced No. 53011, sheet 2 and dated 28th June 2021.
- Site survey plan prepared by Norton Survey Partners, titled "Plan Showing Selected Detail & Level Over No. 21 Whistler Street, Manly", referenced No. 53011, sheet 1 of 1 and dated 13th March 2018.
- Service protection report prepared by Olsen Infrastructure Pty Ltd, titled "Service Protection Report by Olsen Infrastructure Pty Ltd", referenced job No. 21491 and dated 23rd August 2021.

1.4 Geotechnical Assessment Objectives

The objective of the geotechnical investigation was to assess the site surface and subsurface conditions at the selected boreholes and testing locations within the site (where accessible and feasible), and to provide professional geotechnical advice and recommendations on the following based on requirements provided to GCA by the client:

- General assessment of any potential geotechnical issues that may affect any surrounding infrastructures, buildings, council assets, etc., along with the proposed development.
- Excavation conditions and recommendations on excavation methods in soils to restrict any ground vibrations.
- Recommendations on suitable shoring (retention) systems for the site.
- Design parameters based on the ground conditions within the site for retaining walls, cantilever shoring walls and propped shoring.
- Recommendations on suitable foundation types and design for the site.
- End bearing capacities and shaft adhesion for shallow and deep foundations based on the ground conditions within the site.
- Groundwater levels which may be determined during the geotechnical investigation.
- Recommendations on groundwater maintenance and limiting.
- Preliminary site lot classification in accordance with Australian Standards (AS) 2870-2011.
- Preliminary aggressivity and salinity assessment within the site based on laboratory testing results at the selected borehole locations.
- General geotechnical advice on site preparation, filling and subgrade preparation.

1.5 Scope of Works

Fieldwork for the geotechnical investigation was undertaken by an experienced geotechnical engineer/engineering geologist, following in general the guidelines outlined in AS 1726-2017. The scope of works included:

- Submit and review Dial Before You Dig (DBYD) plans and any other plans provided by the client of existing buried services on the site.
- Service locating carried out using electromagnetic detection equipment to ensure the area is free of any underground services at the selected boreholes and testing locations.
- Review of site plans and drawings to determine appropriate testing locations (where accessible and feasible) and identify any relevant features of the site.
- Machine drilling of two (2) boreholes at selected locations within the site (where accessible and feasible) by a specialised trailer mounted drilling rig, using solid flight augers equipped with a 'Tungsten Carbide' (TC) bit, and identified as boreholes BH1 and BH2. The drilling rig is owned and operated by a specialist subcontractor.
    - The boreholes were drilled to varying practical TC bit terminated depths of approximately 6.0m to 10.0m below the existing ground level within the site (bgl).
- Dynamic Cone Penetrometer (DCP) testing immediately adjacent to borehole BH2 and at a selected location within the site (where accessible and feasible), using hand operated equipment to varying practical terminated depths of approximately 5.3m to 5.8m bgl. The DCP tests are identified as DCP1 and DCP2.
        - The approximate locations of the boreholes and DCP tests are shown on **Figure 1, Appendix B** of this report.
- Collection of soil samples during drilling for the following laboratory testing required:
    - Laboratory testing by a National Association of Testing Authorities, Australia (NATA) accredited laboratory (ALS Environmental) on six (6) selected samples collected during drilling of the boreholes to determine the pH, chloride and sulphate content, and electrical conductivity of the selected samples. Laboratory analysis was undertaken for the purpose of a preliminary aggressivity and salinity assessment within the site.
- Reinstatement of the boreholes with available soil displaced during drilling.
- Preparation of this geotechnical engineering report.

1.6 Constraints

The discussions and recommendations provided in this report have been based on the results obtained during machine drilling and DCP testing at the selected boreholes and testing locations within the site (where accessible and feasible). It is recommended that further geotechnical inspections should be carried out during construction to confirm the subsurface conditions across the site and foundation bearing capacities have been achieved.

Additional machine drilled boreholes and Cone Penetrometer Testing (CPT) are recommended prior to construction in order to confirm the ground conditions and nature of groundwater, and to help assist in final designs of the proposed development.

2. SITE DESCRIPTION

2.1 Overall Site Description

The overall site description and its surrounding are presented in Table 1 below.

Table 1. Overall Site Description and Site Surroundings

Information	Details
Overall Site Location	The site located within a residential/commercial area along Whistler Street carriageway, approximately 20m east of Belgrave Street thoroughfare.
Site Address	21 Whistler Street Manly NSW 2095
Approximate Site Area[1]	285m^2
Local Government Authority	Northern Beaches Council
Site Description	At the time of the investigation, the site was predominately covered in fill material, a partially demolished building and a number of mature trees. We note that a Sydney Water pipeline (150 VC) is present within the northern portion of the site, running from Whistler Street carriageway (eastern boundary) into the property at nos. 35-39 Belgrave Street to the west. We understand the invert depth of the 150 VC pipeline is approximately 1.78m to 2.35m. The Sydney Water documents are attached in **Appendix H**.
Approximate Distances to Nearest Watercourses (i.e. rivers, lakes, creeks, etc.)	• South Pacific Ocean – 250m east of the site. • Manly Cove – 400m south-west of the site.
Site Surroundings	The site is located within an area of residential/ commercial use and is bounded by: • Commercial property at No. 40 Belgrave Street to the north. • Whistler Street carriageway to the east. • Commercial property at No. 33 Belgrave Street to the south. • Residential property at nos. 35-39 Belgrave Street to the west.

[1]Site area is approximate and obtained from the site survey plan referenced in Section 1.3.

2.2 Topography

The local and the site topography generally falls towards the north to north-east. Levels within the site vary from approximately RL6.1m to RL7.3m AHD.

It should be noted that the site topography, levels and slopes are approximate and based off the site survey plan referenced in Section 1.3, observations made during the geotechnical investigation and reference to NSW Six Maps (https://maps.six.nsw.gov.au/). The actual topography in areas inaccessible during the site investigation, including areas under the existing infrastructures, along with the site and local topography and levels are expected to vary from those outlined in this report.

2.3 Regional Geology

Information obtained on the local regional subsurface conditions, referenced from the Department of Mineral Resources, Sydney 1:100,000 Geological Series Sheet 9130 First Edition, dated 1983, by the Geological Survey of New South Wales, indicates the site is located within a geological region generally underlain by Quaternary Aged Holocene Deposits (Qhf). The Holocene Deposits (Qhf) normally comprise "medium to fine "marine" sand".

The site is also located approximately 100m south-east of a geological boundary/region generally underlain by Triassic Aged Hawkesbury Sandstone (Rh). The Hawkesbury Sandstone (Rh) normally comprises "medium to coarse grained quartz sandstone, very minor shale and laminite lenses".

Furthermore, reference made to MinView by the State of New South Wales through Regional NSW 2021 indicates the site is positioned mainly within a geological region underlain by Sand (QH_bd) and in close proximity to Sandstone (Tuth).

A review of the regional maps by the NSW Government Environment and Heritage shows the site is set within the Woy Woy (ww) landscape group. The Woy Woy (ww) landscape is generally recognised by level to gently undulating non-tidal beach ridges on marine sands. Soils of the Woy Woy group have permanently high water tables (<200cm), localised flooding, periodic waterlogging in depressions, very low to low soil fertility and localised areas of high soil erosion hazard. Beach plains mainly have reliefs of generally less than 3m and slopes of less than 5% in gradient. Soils of the Woy Woy landscape group are generally slightly (pH 6.5) to strongly (pH 4.0) acidic.

The site is also noted to be situated in close proximity to the Narrabeen (na) landscape group, which is typically recognised by beaches and coastal foredunes on marine sands. Soils of the Narrabeen group have extreme wind and wave erosion hazard, very low soil fertility, high soil permeability and non-cohesive soils. Beach plains mainly have reliefs generally up to 6m and slopes of less than 3% in gradient, whereas foredune reliefs are normally less than 20m and slope gradients of up to 45%. Soils of the Narrabeen landscape group are generally moderately (pH 9.0) alkaline to slightly (pH 6.5) acidic.

The Woy Woy (ww) and Narrabeen (na) landscape group reports are attached in **Appendix I**.

3. SUBSURFACE CONDITIONS AND ASSESSMENT RESULTS

3.1 Stratigraphy

A summary of the surface and subsurface conditions within the investigation area of the proposed development are summarised below and in the detailed engineering borehole logs presented in **Appendix D**, and should be read in conjunction with the geotechnical explanatory notes detailed in **Appendix C**. Any rock description has been based on Pells P.J.N, Mostyn G. & Walker B.F. Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998.

It should be noted that estimated soil consistency/strength assessed by DCP testing in the site during the geotechnical investigation are approximate and variances should be expected throughout. Due to the variable ground conditions throughout the site, it is recommended that confirmation of the subsurface materials be carried out prior to construction by additional boreholes and CPT.

It should also be noted that ground conditions within the site are expected to differ from those encountered and inferred in this report, since no geotechnical or geological exploration program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site.

From the boreholes (BH1 and BH2) carried out within the site, the subsurface conditions at the test locations (where accessible and feasible) generally comprised:

- (Unit 1): Fill material predominately comprising SAND and Gravelly SAND, fine to medium grained, from the existing ground level within the site and extending to depths of approximately 0.5m to 0.9m bgl (varying throughout), generally underlain by:
- (Unit 2): Marine SAND, fine to medium grained, estimated very loose to loose, becoming medium dense, then dense to very dense with depth, and present to at least 6.0m to 10.0m bgl.

Based on the geotechnical investigation at the selected borehole and testing locations, along with our experience and observations made within the site and local region, it is inferred that majority of the site area is underlain by relatively deeper natural sandy soils extending to depths of greater than approximately 6.0m to 10.0m bgl, and are expected to vary throughout.

Furthermore, results of the geotechnical investigation indicate natural soils underlying the site to vary throughout with variable composition and consistency/strength, predominately at locations and depths not assessed during the geotechnical investigation.

It should be noted that the estimated consistency/strength of the underlying soils are based on DCP testing to the maximum terminated depths at the selected testing locations within the site. The potential for weak or softer layers throughout the unit should be considered, predominately below the practical DCP testing depths of approximately 5.3m to 5.8m bgl. As part of our geotechnical assessment, we have assumed a similar subsurface profile (i.e. sandy soils) to be present throughout the proposed development area, extending to the maximum testing depths throughout.

Information obtained from the property at No. 26 Whistler Street shows the presence of poor subsurface materials (Clayey SAND) from depths of about 9.0m to 11.0m bgl, with natural sandy soils present to at least 18.2m bgl. It is therefore recommended that additional borehole drilling and CPT be undertaken prior to construction, in order to confirm the ground conditions, and depths and consistency/strength of the soils underlying the site.

A summary of the inferred subsurface conditions encountered and inferred during DCP testing are summarised in Table 2 below, with the DCP testing results attached in **Appendix E**. Ground conditions depicted in Table 2 below are inferred based on the DCP testing results and assumes a similar subsurface profile observed during the geotechnical investigation to be present over the remainder of the site and throughout the testing depths indicated.

It should be noted that DCP testing and higher blow counts encountered may be affected by factors such as gravels, ironstone bands, well consolidated soils and highly cemented sands, and other deleterious materials which may be present within the underlying soils, along with tree rootlets extending throughout the soils from trees and vegetation within the vicinity. These results should be read in conjunction with the boreholes and geotechnical confirmation should be made during construction by additional borehole drilling and appropriate testing (i.e. CPT) as site conditions may vary.

Table 2. Summary of Inferred Subsurface Conditions From DCP Testing

<table>
<tr><th></th><th></th><th>DCP ID</th><th>DCP1</th><th>DCP2 (BH2)</th></tr>
<tr><th>Unit</th><th>Unit Type</th><th>Estimated Consistency/ Strength[1]</th><th colspan="2">Depth/Thickness of Unit (m bgl)</th></tr>
<tr><td>1</td><td>Fill</td><td>N/A</td><td rowspan="2">0.0 – 0.6?</td><td>0.0 – 0.5</td></tr>
<tr><td rowspan="5">2</td><td rowspan="5">Marine Sands[2]</td><td>Very loose</td><td rowspan="2">0.5 – 2.5</td></tr>
<tr><td>Loose</td><td>0.6 – 3.8</td></tr>
<tr><td>Medium Dense</td><td>3.8 – 4.3</td><td>2.5 – 4.0</td></tr>
<tr><td>Dense</td><td>4.3 – 5.2</td><td>4.0 – 5.5</td></tr>
<tr><td>Dense to Very Dense</td><td>5.2 – 5.3</td><td>5.5 – 5.8</td></tr>
</table>

[1]Estimated soil consistency/strength is based on DCP testing to the maximum practical terminated depths at the selected testing locations within the site. The potential for weak or softer layers throughout the unit should be considered, predominately below the practical DCP testing depths of approximately 5.3m to 5.8m bgl.
[2]Confirmation of the underlying soil composition and consistency/strength should be made by additional boreholes and CPT. Precaution should be made when considering these layers throughout the site.

3.2 Groundwater

Groundwater was encountered drilling of boreholes BH1 and BH2 at depths of approximately 5.6m and 6.0m bgl, respectively. We note that soils were observed to be "moist to wet" from a depth of about 5.0m bgl in borehole BH2, which may indicate groundwater levels to be at shallower depths within the site.

It should be noted that the boreholes were immediately backfilled following completion of fieldwork which precluded longer term monitoring of groundwater levels. Furthermore, it is worth noting that the nature of groundwater within the local region and underlying soils is expected to vary throughout and may be encountered at shallower depths within the site (i.e. <2.0m to 3.0m bgl).

Thus, based on the above observations and data available at the time of reporting, groundwater which may be present within the site is expected to be in through voids within the underlying fill material and through the pore spaces between particles of unconsolidated natural sandy soils underlying the site. It should further be noted that groundwater levels have the potential to elevate during daily or seasonal influences such as tidal fluctuations, heavy rainfall, damaged services, flooding, etc., and moisture content within soils may be influenced by events within the site and adjoining properties.

We note that no provision was made for longer term groundwater monitoring within the site and its presence should not be precluded. Therefore, groundwater monitoring should be carried out prior to construction in order to confirm the nature and level of groundwater within the site.

Where groundwater conditions vary from those outlined in this report, GCA should be contacted for further advice.

4. LABORATORY TEST RESULTS

4.1 Aggressivity and Salinity

Six (6) selected samples were sent to a NATA accredited testing laboratory, ALS Environmental, to determine the pH, chloride and sulphate content, and electrical conductivity of the samples.

A summary of the laboratory tests results is provided in Table 3 below with laboratory certificates presented in **Appendix G** of this report.

Table 3. Summary of Laboratory Test Results (Aggressivity and Salinity)

Borehole ID		**BH1**	**BH1**	**BH1**	**BH2**	**BH2**	**BH2**
Approximate Depth (m bgl)		0.9 – 1.0	2.9 – 3.0	3.9 – 4.0	4.9 – 5.0	6.9 – 7.0	9.9 – 10.0
Strata Type		Natural Sandy Soils					
Aggressivity and Salinity	**pH**	6.3	8.8	8.7	9.3	9.2	8.3
	Moisture Content (%)	2.5	1.6	1.9	24.0	24.4	18.0
	Chloride (mg/kg)	10	<10	<10	10	20	10
	Sulphate SO_4 (mg/kg)	20	<10	<10	<10	20	20
Electrical Conductivity (µS/cm)	**EC (µS/cm)**	54	43	26	78	86	140
	EC (dS/m)	0.054	0.043	0.026	0.078	0.086	0.14
	Multiplication Factor[1]	25	25	25	25	25	25
	Saturation Extract ECe (dS/m)	1.35	1.08	0.65	1.95	2.15	3.5

5. GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

5.1 Dilapidation Survey

It is recommended that prior to demolition, excavation and construction, a detailed dilapidation survey be carried out on all adjacent buildings, structures, council assets, road reserves and infrastructures that fall within the "zone of influence" of the proposed excavation and vicinity of the proposed development. A dilapidation survey will record the condition of existing defects prior to any works being carried out within the site. Preparation of a dilapidation report should constitute as a "Hold Point".

5.2 General Geotechnical Issues

The following aspects have been considered main geotechnical issues for the proposed development:

- Preliminary aggressivity and salinity assessment.
- Preliminary site lot classification.
- Excavation conditions.
- Groundwater management.
- Stability of excavation and retention of adjoining properties and infrastructures.
- Foundations.
- Sydney Water pipeline.

Based on results of our assessment, a summary of the geotechnical aspects above and recommendations for construction and designs are presented below.

5.3 Preliminary Aggressivity and Salinity Assessment

In accordance to AS 2159-2009 "Piling – Design and Installation" (as outlined in Table 4 below), results of laboratory tests and introduction of a multiplication factor for electrical conductivity indicates the following classification:

Table 4. Aggressivity and Salinity Reference Table

Reference	Element Type	High Perm. Soils	Low Perm. Soils	pH	Chloride (mg/kg)	Sulphate SO_4 (mg/kg)
AS 2159-2009	**Concrete Elements**	Mild	Non	>5.5	N/A	<5,000
		Moderately	Mild	4.5 – 5.5		5,000 – 10,000
		Severely	Moderately	4.0 – 4.5		10,000 – 20,000
		Very Severely	Severely	<4.0		>20,000
	Steel Elements	Non	Non	>5.0	<5,000	N/A
		Mild	Non	4.0 – 5.0	5,000 – 20,000	
		Moderately	Mild	3.0 – 4.0	20,000 – 50,000	
		Severely	Moderately	<3.0	>50,000	
Dry Salinity 1993	Electrical Conductivity Saturation Extract ECe (dS/m) value range, based on an introduction of a multiplication factor from DNR publication.				Non-Saline <2 Slightly Saline 2 – 4 Moderately Saline 4 – 8 Very Saline 8 – 16 Highly Saline >16	

- Underlying natural sandy soils:
  - Non aggressive for buried steel structural elements in low and high permeability soils.
  - Non aggressive for buried concrete structural elements in low permeability soils.
  - Mildly aggressive for buried concrete structural elements in high permeability soils.
  - Electrical conductivity of saturated extract (ECe) ranging from approximately 0.65ds/m to 3.5ds/m, indicating generally "slightly" saline natural sandy soils underlying the site.

It should be note that soil aggressivity and salinity may vary throughout the site and is based on testing at the selected borehole locations to the maximum depths indicated, in conjunction with multiplication factors for electrical conductivity, as described above. Ground conditions and soil aggressivity and salinity are expected to vary across the site as discussed in this report since no geotechnical or geological exploration program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site.

Consideration should be given to additional borehole drilling and laboratory testing in order to confirm the findings presented above.

5.4 Preliminary Site Lot Classification

Based on the geotechnical investigation and observations made at the selected testing locations, it is inferred that the site is generally underlain by relatively deeper natural soils, as discussed in Section 3 above.

The governing site lot classification in accordance with AS 2870-2011 has been identified as "Class P" (Problematic Site) for the overall site, due to:

- The nature of very loose to loose sandy soils underlying the proposed development area, providing inadequate bearing capacities for the proposed structure.
- The presence of deep fill material in certain areas of the site, considered as "uncontrolled fill".

Based on the boreholes and DCP tests carried out within the site, AS 2870-2011 indicates the site may be classified as a "Class A" site for the design and construction of the foundation system, founded below any loose/soft soils, topsoil, slopewash, fill or other deleterious material, being <u>entirely</u> on appropriate consistency/strength natural sandy soils (i.e. estimated dense to very dense, or better) underlying the proposed development area (subject to confirmation).

This classification is solely based on assessment of the subsurface conditions are the selected borehole and testing locations/depths within the site and assumes a similar subsurface profile (i.e. sandy soils) to be present throughout the proposed development area. Where clayey soils are encountered during construction or subsurface conditions vary from those outlined in this report, GCA should be contacted immediately, to re-assess the findings and recommendations presented in this report (i.e. reclassification of the site, etc.). Confirmation should be carried out as outlined in this report.

Foundation design and construction should be carried out as outlined in Section 5.9 below, with reference made to AS 2870-2011. Geotechnical inspections and confirmation of the actual depth of underlying fill material and estimated consistency/strength of the natural soils should be made prior to construction by additional borehole drilling and appropriate testing, and by inspection during construction.

GCA should be contacted where ground conditions vary from those outlined in this report at the borehole and testing locations. Where the building foundations are not proposed to be constructed on appropriate material underlying the site, GCA should also be contacted immediately and the building foundations be designed and constructed as a "Class P" site.

Footing designs should take into consideration the effect of recent removal and planting of trees, along with any future tree removal within the vicinity of the proposed development on soil moisture conditions. Sufficient time should be given for soil moisture to re-equilibrate following any removal or planting of trees within the proposed development area, or specific engineering assessment and design will be required on the foundation design.

Although trees and vegetation are considered to contribute to the stability of the site, we recommend that planting of trees around the development area (i.e. in close proximity to the proposed building foundations) be limited as they can also affect moisture changes within the soil and cause significant displacement/damage within the building foundations by extensive tree root system movement.

Based on the preliminary site lot classification outlined above, it is recommended that reference is made to the recommendations provided by CSIRO "Guide to Home Owners on Foundation Maintenance and Footing Performance", attached as **Appendix F**.

5.5 Inspection Pits and Underpinning

Consideration should be given to inspection pits carried out for the existing adjacent buildings and infrastructures, particularly where they fall within the "zone of influence" (obtained by drawing a line 45º above horizontal from the base of the proposed excavations) of the proposed development. This should be carried out prior to any demolition, excavation or construction activities, and will provide an assessment of the existing foundations of the adjacent buildings.

The assessment of the adjacent building footings should include assessment of the underlying soils, which will determine the need for additional support, such as underpinning, prior to installation of shoring piles, or any demolition, excavation and construction activities.

An excavation monitoring report/plan should be implemented for the subject site prior to excavation and construction activities (mainly for adjoining infrastructures such as Sydney Water assets and road reserves).

5.6 Excavation

Maximum excavation depths varying from approximately 4.7m to 7.3m are expected to be required for construction of the proposed development. Locally deeper excavations for the lift shafts, and building footings and service trenches are also anticipated to be required as part of the planned development.

Based on this information and existing ground conditions as encountered during the geotechnical investigation, it is anticipated that excavations will extend predominately through Unit 1 (fill) and Unit 2 (marine sands) throughout majority of the proposed development area, as discussed in Section 3.

Consultation should be made with subcontractors to discuss the feasibility and capability of machinery for the proposed development for the existing site conditions.

5.6.1 Excavation Assessment

Excavation through softer soils should be feasible using conventional earth moving excavators, typically medium to large hydraulic excavators. Smaller sized excavators may encounter difficulty in high strength bands of soils which may be encountered. Where high strengths bands are encountered, rock breaking or ripping should be allowed for. Removal of the existing pavements and associated infrastructures within the site are also expected to require larger excavators and rock breaking and ripping.

Demolition, excavation and construction activities (or the like) will generate both vibration and noise whilst being carried out within the site. Therefore, particular care will be required to ensure that adjacent buildings and infrastructures (i.e. road reserves, buildings, etc.), are not damaged such activities, mainly where excavations are expected to be conducted within the vicinity or "zone of influence" of adjoining infrastructures (i.e. Sydney Water pipeline).

Should sheet piles be considered as a shoring solution (as outlined in Section 5.8), vibration limits are to be maintained to limits not exceeding the following maximum Peak Particle Velocity (PPV) for adjacent structures, as outlined in AS 2187.2-2006:

- Sensitive (i.e. Sydney Water pipeline) and/or historical structures – 2mm/sec.
- Residential and/or low rise structures – 5mm/sec.
- Unreinforced and/or brick structures – 10mm/sec.
- Reinforced and/or steel structures – 25mm/sec.
- Commercial and/or industrial buildings – 25mm/sec.

A vibration monitoring plan should be implemented to monitor construction activities and their effects on adjoining infrastructures (including Sydney Water assets). A vibration monitoring plan may be carried out attended or unattended. An unattended vibration monitoring must be fitted with alarms in the form of strobe lights, sirens or live alerts sent to the vibration monitoring supervisor, which are activated when the vibration limit is exceeded. If adopted/considered, consultation should be made with appropriate subcontractors/consultants for the installation of vibration monitoring instruments.

A geotechnical engineer should be contacted immediately if vibrations during construction or in adjacent structures exceed the values outlined above and work should immediately cease. Further advice should be sought from GCA on vibration control measures by implementation of a vibration monitoring report/plan, in consultation with a specialist consultant/contractor.

All excavation works should be carried out in accordance with the NSW WorkCover code of practice for excavation work. It is recommended a dilapidation report be carried out prior to any excavation or construction, as discussed in Section 5.1. This should be considered a "Hold Point".

5.7 Groundwater Management

Based on the geotechnical investigation at the selected boreholes and testing locations, it is *anticipated* that groundwater levels may be present at depths above the proposed basement FFL.

It should be noted that the presence of groundwater should not be precluded during construction and in the long term design life of the proposed building. We note that no provision was made for longer term groundwater monitoring within the site as part of this investigation and this should be undertaken prior to construction.

It should also be noted that these groundwater levels have the potential to elevate during daily or seasonal influences such as tidal fluctuations, heavy rainfall, damaged services, flooding, etc. Thus, we *anticipate* groundwater within site to be through the voids and particle spacing within the underlying soils.

As noted, groundwater levels are subject to fluctuations on a daily and seasonal basis, and should be considered as part of the long term design life of the building. Therefore, consideration should be given to (not limited to):

- A dewatering program to at least 1.0m below the proposed basement FFL where groundwater is expected to be present within the basement level, in order to allow excavation and construction of the basement under controlled "dry" conditions.
    - It is recommended that in order to reduce the extent of any groundwater drawdown effects within the region, the retention system which is adopted be constructed as a "fully tanked" retention shoring wall and be extended into appropriate consistency/strength natural sandy soils (i.e. estimated dense to very dense, or better) underlying the site. This should be confirmed by additional testing and investigation within the perimeter of the proposed basement following demolition of existing infrastructures.
- If required (subject to confirmation):
    - Spear points and wells should be installed within the perimeter of the proposed retention system and should be connected to a header pipe to allow for discharge during pump out.
    - Consultation should be made with specialist subcontractors for dewatering in order to confirm the capacity of the dewatering system and assess the effects of any drawdown which may occur following this process. The specialist subcontractors should make the client aware of any effects which may arise during and following dewatering.
    - It is recommended that groundwater levels be monitored during dewatering with care taken for the quality of the groundwater being discharged. Care should also be taken for any settlement in adjacent buildings with consideration made for settlement monitoring points on adjacent buildings and infrastructures, and an excavation management plan implemented as part of the proposed development.

Waterproofing of the proposed basement walls and slabs, and allowance given for nominal hydrostatic uplift should also be implemented as part of the design of the proposed development. This should be carried out by a suitably qualified structural engineer and should take into consideration the possibility for groundwater levels to elevate at depths above those outlined in this report.

Longer term groundwater monitoring is necessary prior to construction to ensure a suitable retention system has been implemented for the proposed development, as discussed in Section 5.8 below, and to provide confirmation of the hydrogeological characteristics within the site.

Recommendations on groundwater management and retention support systems (Section 5.8) are based on observations made during the geotechnical investigation and data available at the time of reporting on groundwater conditions within the site. Where groundwater conditions vary from those outlined in this report, GCA should be contacted for further advice.

5.8 Excavation Stability

Maximum excavation depths are expected vary within the site from approximately 4.7m to 7.3m for construction of the proposed development. Locally deeper excavations for the lift shafts, and building footings and service trenches are also anticipated to be required as part of the planned development.

Based on the ground conditions within the site, the total depth of excavation and the extent of the development walls to the site boundaries and adjoining infrastructures, it is critical from geotechnical perspective to maintain the stability of the adjacent structures and infrastructures during demolition, excavation and construction.

5.8.1 Excavation Retention Support Systems

Based on the proposed development, assessment of the subsurface and current groundwater conditions within the site, and adjoining properties and infrastructures (i.e. Sydney Water pipeline), it is assessed that the use of temporary or permanent batter slopes are not suitable for the proposed development and consideration should be given to a "fully tanked" suitable retention system such as a secant pile wall solution. We recommend Continuous Flight Auger (CFA) piles be considered for construction of this proposed development.

The retention system which is adopted should be constructed as a "fully tanked" retention shoring wall and be extended/sufficiently embedded into appropriate consistency/strength natural sandy soils (i.e. estimated dense to very dense, or better) underlying the site, in order to reduce the lateral movements and risk of potential damage to the exiting building and adjacent infrastructures (i.e. adjacent road reserves and infrastructures).

Care is be taken when considering steel sheet piles for the shoring walls, as installation may cause excessive vibrations when driven into medium dense to dense sandy soils (or better). Consultation should be made with a suitable contractor on suitability of steel sheet piles for the subject site and ground conditions (including assessment of potential damage to adjoining infrastructures). Vibration limits are to be maintained to those generally presented in Section 5.6, in consultation with a specialist consultant/contractor who should confirm these limits and provide any additional advice which is out of our area of expertise.

Consultation should be made with specialist subcontractors on the design and construction of the above retention system shoring wall. A suitably qualified structural engineer should also oversee the design and construction of the proposed retention system, with all structural elements inspected and approved by the project structural engineer.

The retention system should also be analysed and designed by the project structural engineer taking into account horizontal pressures due to surcharge loads from adjacent infrastructures (i.e. buildings, road reserves, etc.), and long term loadings. Soil anchors such as "manta ray" or "sting ray" type anchors and other temporary support for the adopted shoring system should be considered, and are also recommended to provide additional support where a cantilevered wall arrangement is shown to be insufficient.

As noted, the selected retention system should be sufficiently embedded into appropriate consistency/strength natural sandy soils (i.e. estimated dense to very dense, or better) underlying the site, and should be inspected and approved by a suitably qualified geotechnical engineer.

Shoring design should also take into consideration both short term (during construction) and permanent conditions, along with surcharge loading and footing loads from adjacent infrastructures. Groundwater levels within the site should be confirmed by measures discussed in Section 5.7 of this report. Where the nature of groundwater varies from that inferred in this report, GCA should be made aware.

The design of the basement retaining walls will depend on the method of constructed being adopted. Common methods include (not limited to):

- Top-down construction.
- Bottom-up construction.
- Staged excavation and installation of props and/or partial berms.

In cases where anchoring is impractical, other temporary support for the adopted shoring system should be considered. This may include the staged excavation and installation of temporary berms or props in front of the retaining wall.

If considered, the shoring wall can be designed using the recommended design parameters provided in Section 5.8.2. Bulk excavation and foundations (including pile installations) should be supervised, monitored and inspected by a geotechnical engineer, with all structural elements of the development by a structural engineer. Inspections should be considered as “Hold Points” to the project.

5.8.2 Design Parameters

Excavation pressures acting on the support will depend on a number of factors including external forces from surcharge loading, the stiffness of the support, varying groundwater levels within the site, and the construction sequence of the proposed development. Therefore, the following parameters may be used for the design of temporary and permanent retaining walls at the subject site:

- A triangular earth pressure distribution may be adopted for derivation of active pressures where a simple support system (i.e. cantilevered wall or propped/anchored wall with only one row of props/anchors are required) is adopted. Cantilevered walls are typically less than 2.5m in height, and should ensure deflections remain within tolerable limits.
    - Flexible retaining structures (i.e. cantilevered walls or walls with only one row of anchors), should be based on active lateral earth pressure. “At rest” earth pressure coefficient should be considered to limit the horizontal deformation of the retaining structure. Lateral active (or at rest) and passive earth pressures for cantilever walls or walls with only one row of anchors may be determined as follows:

  Lateral active or “at rest” earth pressure:

  $$P_a = K\,\gamma\,H - \,2c\sqrt{K}$$

  Passive earth pressure:

  $$P_p = K_p\,\gamma\,H + \,2c\sqrt{K_p}$$

- Where lateral deflection exceeds tolerable limits, or where two or more rows of anchors are required, the retention/shoring system should be designed as a braced structure. This more complex support system should utilise advanced numerical analysis tools such as WALLAP or PLAXIS which can ensure deflections in the walls remain within tolerable limits and to model the sequence of anchor installation and excavation. For braced retaining walls, a uniform lateral earth pressure should be adopted as follows:

Active earth pressure:

$$P_a = 0.65\,K\,\gamma\,H$$

Where:

P_a = Active (or at rest) Earth Pressure (kN/m^2)
P_p = Passive Earth Pressure (kN/m^2)
γ = Bulk density (kN/m^3)
K = Coefficient of Earth Pressure (K_a or K_o)
K_p = Coefficient of Passive Earth Pressure
H = Retained height (m)
c = Effective Cohesion (kN/m^2)

- Support systems and retaining structures 'should be designed to withstand hydrostatic pressures, lateral earth pressures and earthquake pressures (if applicable). The applied surcharge loads in their "zone of influence" should also be considered as part of the design, where the "zone of influence" may be obtained by drawing a line 45º above horizontal from the base of the proposed excavations.

Support system designed using the earth pressure approach may be based on the parameters given in Table 5 below for soil horizons underlying the site. Table 5 also provides preliminary coefficients of lateral earth pressure for the soil horizons encountered in the site. These are based on fully drained conditions and that the ground behind the retention walls is horizontal.

Table 5. Preliminary Geotechnical Design Parameters

Material	Fill (Unit 1)	Marine Sands (Unit 2)[4]		
		Very Loose to Loose	Medium Dense	Dense to Very Dense
Unit Weight (kN/m^3)[3]	16	16	17	18
Effective Cohesion c' (kPa)	0	0	0	0
Angle of Friction ϕ' (°)	26	28	30	34
Modulus of Elasticity E_{sh} (MPa)	3	8	15	20
Earth Pressure Coefficient At Rest Ko[1]	0.56	0.53	0.5	0.44
Earth Pressure Coefficient Active Ka[2]	0.39	0.36	0.33	0.28
Earth Pressure Coefficient Passive Kp[2]	2.56	2.77	3.0	3.54
Poisson Ratio v	0.4	0.35	0.35	0.35

[1]Earth pressure coefficient at rest (Ko) can be calculated using Jacky's equation.
[2]Earth pressure coefficient of active (Ka) and passive (Kp) can be calculated using Rankine's or Coulomb's equation.
[3]Above groundwater levels. GCA should be contacted for further advice on unit weight values of soils which are below the groundwater level.
[4]Subject to confirmation by a geotechnical engineer by additional borehole drilling and appropriate testing (CPT).

5.9 Foundations

Following excavation to the FFLs of the proposed development, and based on the boreholes and DCP tests carried out within the site, we expect varying ground conditions comprising predominately Unit 2 (marine sands) to be exposed at bulk excavation level across the site.

It should be noted that construction on fill or loose soils present throughout the site can lead to total and differential settlement under working loads, and not adequately support shallow foundations for the proposed development. Removal of any fill material within the proposed development area prior to foundation construction is recommended.

It is noted that ground conditions within the site is expected to differ from those encountered and inferred in this report, since no geotechnical or geological exploration program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site. It is therefore recommended that confirmation of the underlying ground conditions be confirmed by a geotechnical engineer prior to construction by additional borehole drilling and appropriate testing (i.e. CPT).

5.9.1 Geotechnical Assessment

Based on the proposed development and assessment of the subsurface conditions, it is recommended that a piled foundation system be adopted, with the proposed building footings supported on piles sufficiently embedded into the underlying estimated dense to very dense (or better) natural sandy soils underlying the site.

Piles sufficiently embedded into the underlying estimated dense to very dense (or better) sandy soils may achieve a preliminary allowable bearing capacity of 700kPa, depending on the pile dimensions and actual depth of embedment (subject to confirmation by geotechnical engineer). It is noted that the presence of groundwater can significantly reduce the bearing capacities of the underlying soils and confirmation should be made as outlined in this report.

Piles drilled deeper into the underlying estimated very dense (or better) sandy soils may achieve higher allowable bearing capacities than that provided above, however a detailed geotechnical assessment will be required.

The toe of the piles should be installed to suitable depths such that the thickness of the estimated dense to very dense (or better) sandy soils are present to at least 3 to 4 times the diameter of the pile below the pile toe. Piles should be socketed into the underlying estimated dense to very dense (or better) sandy soils for a length of about 3 pile diameters in order to mobilise both skin friction and the end bearing pressure.

Installation of piles should be complemented by inspections carried out by a geotechnical engineer during construction to confirm the allowable bearing capacities have been achieved and inferred ground conditions are consistent throughout. The actual depth and embedment of the piles should be assessed by the project structural engineer, with all structural elements also inspected and certified by a suitably qualified structural engineer.

It should be noted that the preliminary allowable bearing capacities have taken into consideration similar subsurface profile (i.e. sandy soils) to be present throughout the proposed development area with skin friction in the very loose to loose sands being ignored, piles are founded at suitable depths (i.e. at least 8.0m bgl) and into appropriate materials within the site, and the possibility of groundwater within the site to be at depths of up to 3.0m bgl (varying throughout and subject to fluctuations).

Alternatively, where the use of bored or CFA piles are not economically feasible for the proposed development, consideration may be given to screw piles extending into suitable material underlying the site. If adopted, specialist subcontractors should be contacted to assess the suitability of screw piles and allowable pile loads for the current subsurface conditions within the proposed development area. This option may be best suitable for the proposed development based on ground conditions encountered during the site investigation.

Furthermore, the design and specifications (i.e. length of pile, number of helixes, etc.) of the screw piles should be carried out by suitably qualified subcontractors taking into consideration the subsurface conditions and working loads of the proposed development. A more detailed aggressivity and salinity assessment should be considered if screw piles are adopted for the proposed development. The piling contractor who installs the screw piles should be responsible for certifying the load capacity of the piles.

Where groundwater is encountered during construction at shallower depths than those outlined in this report, or ground conditions vary within the site, GCA should be contacted immediately to re-assess the geotechnical recommendations provided in this report (such as allowable bearing capacities, design parameters, etc.).

Due to variable ground conditions and soil reactivity within the site (as discussed in Section 5.4), it is recommended that all foundations are constructed on consistent material and reactivity throughout the proposed development area to provide uniform support and reduce the potential for total and differential settlement. Reference should be made to the estimated levels of the subsurface conditions outlined in this report, and compared to the final bulk excavation levels across the site.

Footings designed using ultimate values and limit state design will need to consider serviceability which usually governs designs in these cases. For pile designs, a basic geotechnical reduction factor (Φ_{gb}) should be calculated by the structural engineer from AS 2159-2009, taking into consideration the design, installation method and associated risk rating. Furthermore. the design structural engineer should check both ‘piston’ pull-out and ‘cone’ pull-out mechanics in accordance with AS 4678-2002.

5.9.2 Geotechnical Comments

It is recommended that suitable compaction of the subgrade soils be carried out prior to construction of any foundations, as granular subgrade soils usually become loose following excavation. It should be noted that the settlement behaviour, and pile and bearing capacities will vary significantly depending on the pile dimensions and actual depth of embedment, along with the method of installation. Consultation should be made with specialist subcontractors to discuss the feasibility of piles for the existing site conditions, predominately screw piles (if adopted).

It should be noted that higher bearing capacities may be justified for the proposed foundations subject to confirmation by additional borehole drilling and appropriate testing (i.e. CPT). Confirmation of the actual subsurface conditions underlying the proposed development area should also be carried out by a geotechnical engineer during construction, predominately the thickness, consistency/strength and extent of the underlying natural soils.

Specific geotechnical advice should be obtained for footing deigns and end bearing capacities, and design of the foundation system (shallow and pile foundations) should be carried out in accordance with AS 2870-2011 and AS 2159-2009.

Foundations located within the “zone of influence” of any services or sensitive structures (i.e. Sydney Water pipeline) should be supported by a piled foundation. The depths of the piles should extend below the “zone of influence” and should ignore any shaft adhesion. Appropriate measures should be taken to ensure that any services or sensitive structures located within the “zone of influence” of the proposed development are not damaged during and following construction.

Furthermore, we recommend that any works in the vicinity of Sydney Water assets, including excavation and piling works be undertaken in accordance with the Sydney Water technical guidelines for building over and adjacent to pipe assets.

If required, consideration may be given to the concrete encasement of the existing pipeline within the site. This should be carried out by a suitably qualified, licensed and accredited subcontractor, with assessment and inspections provided by a geotechnical engineer and appropriate asset owners, co-ordinators and/or stakeholders.

It is recommended that suitable drainage and the use of impermeable surfaces be implemented as a precaution as part of the design and construction of the proposed development in order to divert surface water away from the building, and help eliminate or minimise surface water infiltration to minimise moisture within the soils. Although trees and vegetation are considered to contribute to the stability of the site, we recommend that planting of trees around the development area (i.e. in close proximity to the proposed building foundations) be limited as they can also affect moisture changes within the soil and cause significant displacement/damage within the building foundations by extensive tree root system movement.

The design and construction of the foundations should take into consideration the potential of flooding. All foundation excavations should be free of any loose debris and wet soils, and if groundwater seepage or runoff is encountered dewatering should be carried out prior to pouring concrete in the foundations. Due to the possibility of groundwater being encountered or possible groundwater seepage during installation of bored piles within the site, as well as the nature of collapsing sandy soils, it is recommended that consideration be given to other piling methods such as CFA piles.

Shaft adhesion should be ignored or reduced within socket lengths that are smeared or fail to satisfy cleanliness requirements (i.e. at least 80%). It is recommended that where piles penetrate expansive soils present within the site, which are susceptible to shrink and swell due to daily and seasonal moisture, shaft adhesion be ignored due to the potential of shrinkage cracking. Pile inspections should be complemented by downhole CCTV camera.

We recommend that geotechnical inspections of foundations be completed by an experienced geotechnical engineer to determine that the designed socket materials have been reached and the required bearing capacity has been achieved. The geotechnical engineer should also determine any variations between the boreholes carried out and inspected locations. Inspections should be carried out in dewatered foundations for a more accurate examination, and inspections should be carried out under satisfactory WHS requirements. Geotechnical inspections for verification capacities of the foundations should constitute as a “Hold Point”.

5.10 Sydney Water Pipeline

We note that a Sydney Water pipeline (150 VC) is present within the middle to rear (north to north-eastern) portion of the site. The 150 VC pipeline intersects the site eastern boundary, running westerly, and falls in within the proposed development footprint.

We understand the invert depth of the 150 VC pipeline is approximately 1.78m to 2.35m. The Sydney Water documents are attached in **Appendix H**.

Based on our site investigation and provided information, we envisage the pipeline to be present within the underlying sandy soils (depending on the actual depth of the pipe). Thus, we recommend that this service, along with any other services within the site be positively identified prior to any construction activities. This should include CCTV footage by an accredited surveyor showcasing that the Sydney Water pipelines are in good condition.

It is also recommended that any works in the vicinity of Sydney Water assets, including excavation and construction activities be undertaken in accordance with the Sydney Water technical guidelines for building over and adjacent to pipe assets. Sydney Water assets should be monitored with an appropriate monitoring regime.

Furthermore, foundations located within the "zone of influence" of the Sydney Water pipeline should be supported by a piled foundation and should follow in general the recommendations outlined in Section 5.9 of this report. The depths of the piles should extend below the "zone of influence" of the pipeline and should ignore any shaft adhesion.

In addition, an appropriate retention system should be designed and constructed along the Sydney Water pipeline and all other perimeter walls of the proposed development, following in general the recommendations drawn in Section 5.8 of this report. All piles should extend below the "zone of influence" of the Sydney Water pipeline and as discussed above, ignoring any shaft adhesion.

Vibrations should be maintained to suitable limits, predominately where excavation and construction activities are within close proximity (i.e. within the "zone of influence") to the Sydney Water asset. Appropriate measures should be taken to ensure that the sewer line is not damaged during and following construction (i.e. suitable machinery, etc.).

In the event of an unexpected incident onsite, the builder or site manager will need to stop works immediately and contact Sydney Water and the Water Service Coordinator to seek emergency actions. We recommend that a post dilapidation survey be carried out after completion of the proposed development.

5.11 Filling

Where filling is required, the following recommended compaction targets should be considered:

- Place horizontal loose layers not more than 150mm thickness over the prepared subgrade.
- Compact to a minimum dry density ratio not less than 98% of the maximum dry density for the building platforms.
- The moisture content during compaction should be maintained at ±2% of the Optimal Moisture Content (OMC).
- The upper 150mm of the subgrade should be compacted to a dry density ratio not less than 100% of the maximum dry density.

Any soils which are imported onto the site for the purpose of filling and compaction of the excavated areas should be free of deleterious materials and contamination. The imported soils should also include appropriate validation documentation in accordance with current regulatory authority requirements. The design and construction of earthworks should be carried out in accordance with AS 3798-2007 and AS 1289. Inspections of the prepared subgrade should be carried out by a geotechnical engineer, and should include proof rolling as a minimum. These inspections should be established as “Hold Points”.

5.12 Subgrade Preparation

The following are general recommendations on subgrade preparation for earthworks, slab on ground constructions and pavements:

- Remove existing fill and topsoil, including all materials which are unsuitable from the site.
- Excavate natural soils and rock.
    - Excavated material may be used for engineered fill.
    - Rock may be used for subgrade material underlying pavements.
- Any natural soils (predominately clayey soils) exposed at the bulk excavation level should be treated and have a moisture condition of 2% OMC. This should be followed by proof rolling and compaction of the upper 150mm layer.
    - Any soft or loose areas should be removed and replaced with engineered or approved fill material.
- Any rock exposed at the bulk excavation level should be clear of any deleterious materials (and free of loose or softened materials). As a guideline, remove an additional 150mm from the bulk excavation level.
- Ensure the foundations and excavated areas are free of water prior to concrete pouring.
- Areas which show visible heaving under compaction or proof rolling should be excavated at least 300mm and replaced with engineered or approved fill, and compacted to a minimum dry density ratio not less than 98% of the maximum dry density.

6. FINITE ELEMENT ANALYSIS

Based on ground conditions encountered during the geotechnical investigation at the selected boreholes and testing locations within the site, a Finite Element Analysis (FEA) was undertaken using PLAXIS by GCA.

The aim of the analysis is to predict any impacts on the Sydney Water sewer pipeline and water main, and the proposed sheet piles located within the site and proposed development area, in particular the possible ground displacements that may occur as a result of the proposed development.

The following assumptions have been made as part of the FEA analysis for the heavier loaded fences and should be confirmed by the project structural engineer:

- Maximum excavation of 7.26m.
- Groundwater levels within the site are assumed to be varying depths of approximately 5.0m to 5.5m bgl, however subject to fluctuation and should be confirmed by longer term groundwater monitoring prior to construction.
- Sheet pile will be designed, installed and certified by the shoring wall contractor.
- Sheet piles will be installed to a minimum depth of 3.0m below the bulk excavation level.
- Three (3) rows of temporary anchors are to be installed throughout the excavation of the basement at varying depths and into estimated very dense natural sandy soils (subject to confirmation by further boreholes and CPT). The anchors are proposed to be R15B anchors with a minimum yield load of 630kN. The anchors should be designed, installed and certified by a specialist subcontractor.
- Excavations to occur with applied surcharged loads of 10kPa and 20kPa for the adjoining footpath and Whistler Street, respectively.

A summary of the PLAXIS analysis results are presented in **Appendix J** of this report, including assumption drawings. Based on result from the FEA and above assumptions, the proposed development is anticipated to induce **(Figure 5, Appendix J)**:

- Less than 11mm horizontal movement at the middle of the sheet pile adjoining the Sydney Water assets.
- Maximum estimated movement of 6mm at the top of the Sydney Water main.
- Maximum estimated movement of 5mm at the top of the Sydney Water sewer pipeline.

The purpose of this assessment is to provide estimated movements within the sheet pile wall adjoining the Sydney Water assets and also for the Sydney Water pipelines. GCA is not responsible or liable for the design of the proposed development shoring system and this assessment is solely based on information provided by the project structural engineer and shoring wall contractor (MESO Solutions). It is the responsibility of the shoring wall contractor to ensure the system is appropriately designed, installed and certified (including all anchors), in consultation with the project structural engineer.

It is recommended that an excavation monitoring report/plan be implemented to monitor movements within the shoring wall and adjoining infrastructures. Should any of the above assumptions change or different requirements are necessary for the proposed development, GCA is to be made aware.

7. ADDITIONAL GEOTECHNICAL RECOMMENDATIONS

Furthermore, following completion of the geotechnical investigation and report, GCA recommends the following additional work to be carried out:

- Dilapidation survey report on adjacent properties and infrastructures.
- Monitoring and supervision of excavations within the site.
- The composition, depth/thickness and consistency/strength of the underlying soils should be confirmed prior to construction by further borehole drilling and appropriate testing (i.e. CPT), predominately in areas and at depths not assessed during the geotechnical investigation.
- Confirmation of design assumptions made for the FEA by the project structural engineer.
- Geotechnical inspections of exposed materials at bulk excavation level.
- Geotechnical inspections of shoring wall piles installations.
- Geotechnical inspections of foundations (shallow and pile foundations) to confirm the preliminary bearing capacities have been achieved.
- Monitoring of any groundwater inflows into the excavation areas within the site.
- Provision for longer term groundwater monitoring to confirm the nature and level of groundwater within the site.
- Implementation of an excavation and vibration monitoring report/plan to monitor all vibrations to adjoining infrastructures (including Sydney Water assets), and movements within the shoring wall and adjoining infrastructures.
- Classification of all excavated material transported from the site.
- A meeting to be carried out to discuss any geotechnical issues and inspection requirements.
- Final architectural and structural design drawings are provided to GCA for further assessment.

8. LIMITATIONS

Geotechnical Consultants Australia Pty Ltd (GCA) has based its geotechnical assessment on available information obtained prior and during the site inspection/investigation. The geotechnical assessment and recommendations provided in this report, along with the surface, subsurface and geotechnical conditions are limited to the inspection and test areas during the site inspection/investigation, and then only to the depths investigated at the time the work was carried out. Subsurface conditions can change abruptly, and may occur after GCA's field testing has been completed.

It is recommended that if for any reason, the site surface, subsurface and geotechnical conditions (including groundwater conditions) encountered during the site inspection/investigation vary substantially during construction, and from GCA's recommendations and conclusions, GCA should be contacted immediately for further testing and advice. This may be carried out as necessary, and a review of recommendations and conclusions may be provided at additional fees. GCA's advice and accuracy may be limited by undetected variations in ground conditions between sampling locations.

GCA does not accept any liability for any varying site conditions which have not been observed, and were out of the inspection or test areas, or accessible during the time of the investigation. This report and any associated information and documentations have been prepared solely for **J & CG Con NSW Pty Ltd,** and any misinterpretations or reliances by third parties of this report shall be at their own risk. Any legal or other liabilities resulting from the use of this report by other parties can not be religated to GCA.

This letter does not constitute as a detailed geotechnical engineering report or has included any geotechnical assessment on the structural design/suitability of building elements. The preliminary FEA undertaken by GCA is to assess the findings provided by the structural engineer/shoring wall contractor and provide any necessary geotechnical recommendations on the Sydney Water asset only. GCA will not be responsible for any structural elements and/or designs of the structure, including (not limited to) the retention system and foundations. All findinings and assumptions presented in this report are to be reviewed and confirmed by the project structural engineer and shoring wall contractor.

It is the clients responsibility to ensure all the recommendations provided by the structural engineer/ shoring wall contractor and geotechnical engineer are undertaken/implemented for the proposed development. All necessary geotechnical (and structural) inspections should be implemented during construction in order to confirm ground conditions within the site. These should constitute as "Hold Points".

This report should be read in full, including all conclusions and recommendations. Consultation should be made to GCA for any misundertandings or misinterpretations of this report.

For and behalf of

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eSPADE NSW Environment & Heritage.

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Sydney Water Technical Guidelines for Building Over and Adjacent to Pipe Assets (July 2015).

APPENDIX A

GCA
Geotechnical Consultants Australia

Important Information About Your

Geotechnical Report

This geotechnical report has been prepared based on the scopes outlined in the project proposal. The works carried out by Geotechnical Consultants Australia Pty Ltd (GCA), have limitations during the site investigation, and may be affected by a number of factors. Please read the geotechnical invesitgation report in conjunction with this "Important Information About Your Geotechnical Report".

Geotechnical Services Are Performed for Specicif Projects, Clients and Purposes.
Due to the fact that each geotechnical investigation is unique and varies from sites, each geotechnical report is unique, and is prepared soley for the client. A geotechnical report may satisfy the needs of structural engineer, where is will not for a civil engineer or construction contractor. No one except the client should rely on the geotechnical report without first conferring with the specific geotechnical consultant who prepared the report. The report is prepared for the contemplated project or original purpose of the investigation. No one should apply this report to any other or similar project.

Reading The Full Report.
Do not read selected elements of the report or tables/figures only. Serious problems have occurred because those relying on the specially prepared geotechnical invesitgation report did not read it all in full context.

The Geotechnical Report is Based on a Unique Set of Project And Specific Factors.
When preparing a geotechnical report, the geotechnical engineering consultant considers a number of unique factors for the specific project. These typially include:

- Clients objectives, goals and risk management preferences;
- The general proposed development or nature of the structure involved (size, location, etc.); and
- Future planned or existing site improvements (parking lots, roads, underground services, etc.);

Care should be taken into identifying the reason of the geotechnical report, where you should not rely on a geotechnical engineering report that was:

- Not prepared for your project;
- Not prepared for the specific site;
- Not prepared for you;
- Does not take into consideration any important changes made to the project; or
- Was carried out prior to any new infrastructure on your subject site.

Typical changes that can affect the reliabiliy if an existing geotechical investigation report include those that affect:

- The function of the proposed structure, where it may change from one basement level to two basement levels, or from a light structure to a heavy loaded structure;
- Location, size, elevation or configuration of the proposed development;
- Changes in the structural design occur; or
- The owner of the proposed development/project has changed.

The geotecnical engineer of the project should always be notified of any changes – even minor – and be asked to evaluate if this has any impact. GCA does not accept responsibility or liability for problems that occur because its report did not consider developments which it was not informed of.

Subsurface Conditions Can Change
This report is based on conditions that existed at the time of the investigation, at the locations of the subsurface tests (i.e. boreholes) carried out during the site investigation. Subfurface conditions can be affected and modified by a number of factores including, but not limited to, the passage of time, man-made influences such as construction on or adjacent to the site, by natural forces such as floods, groundwater fluctuations or earthquakes. GCA should be contacted prior to submitting its report to determine if any further testing may be required. A minor amount of additional testing may prevent any major problems.

Geotechnical Findings Are Professional Opinions
Results of subsurface conditions are limited only to the points where the subsurface tests were carried out, or where samples were collected. The field and laboratory data is analysed and reviewed by a geotechnical engineer, who then applys their professional experience and recommendations about the site's subsurface conditions. Despite investigation, the actual subsurface conditions may differ – in some cases significantly – from the results presented in the geotechnical investigation report, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface anomalies and details.

Therefore, the recommendations in this report can only be used as preliminary. Retaining GCA as your geotechnical consultants on your project to provide construction observations is the most effective method of managing the risks associated with unanticipated subsurface conditions.

Geotechnical Report's Recommendations Are Not Final

Because geotechnical engineers provide recommendations based on experience and judgement, you should not overrely on the recommendations provided – they are not final. Only by observing the actual subsurface conditions revealed during construction may a geotechnical engineer finalise their recommendations. GCA does not assume responsibility or liability for the report's recommendations if no additional observations or testing is carried out.

Geotechnical Report's Are Subject to Misinterpretations

The project geotechnical engineer should consult with appropriate members of the design team following submission of the report. You should review your design teams plans and drawings, in conjunction with the geotechnical report to ensure they have all be incorporated. Due to many issues arising from misinterpretation of geotechnical reports between design teams and building contractors, GCA should participate in pre-construction meetings, and provide adequate construction observations.

Engineering Borehole Logs And Data Should Not be Redrawn

Geotechnical engineers prepare final borehole and testing logs, figure, etc. based on results and interpretation of field logs and laboratory data following the site investigation. The logs, figure, etc. provided in the geotechnical report should never be redrawn or altered for inclusion in any other documents from this report, includined architectural or other design drawings.

Providing The Full Geotechnical Report For Guidance

The project design teams, subcontactors and building contractors should have a copy of the full geotechnical investigation report to help prevent any costly issues. This should be prefaced with a clearly written letter of transmittal. The letter should clearly advise the aforementioned that the report was prepared for proposed development/project requirements, and the report accuracy is limited. The letter should also encourage them to confer with GCA, and/or carry out further testing as may be required. Providing the report to your project team will help share the financial responsibilities stemming from any unanticipated issues or conditions in the site.

Understanding Limitation Provisions

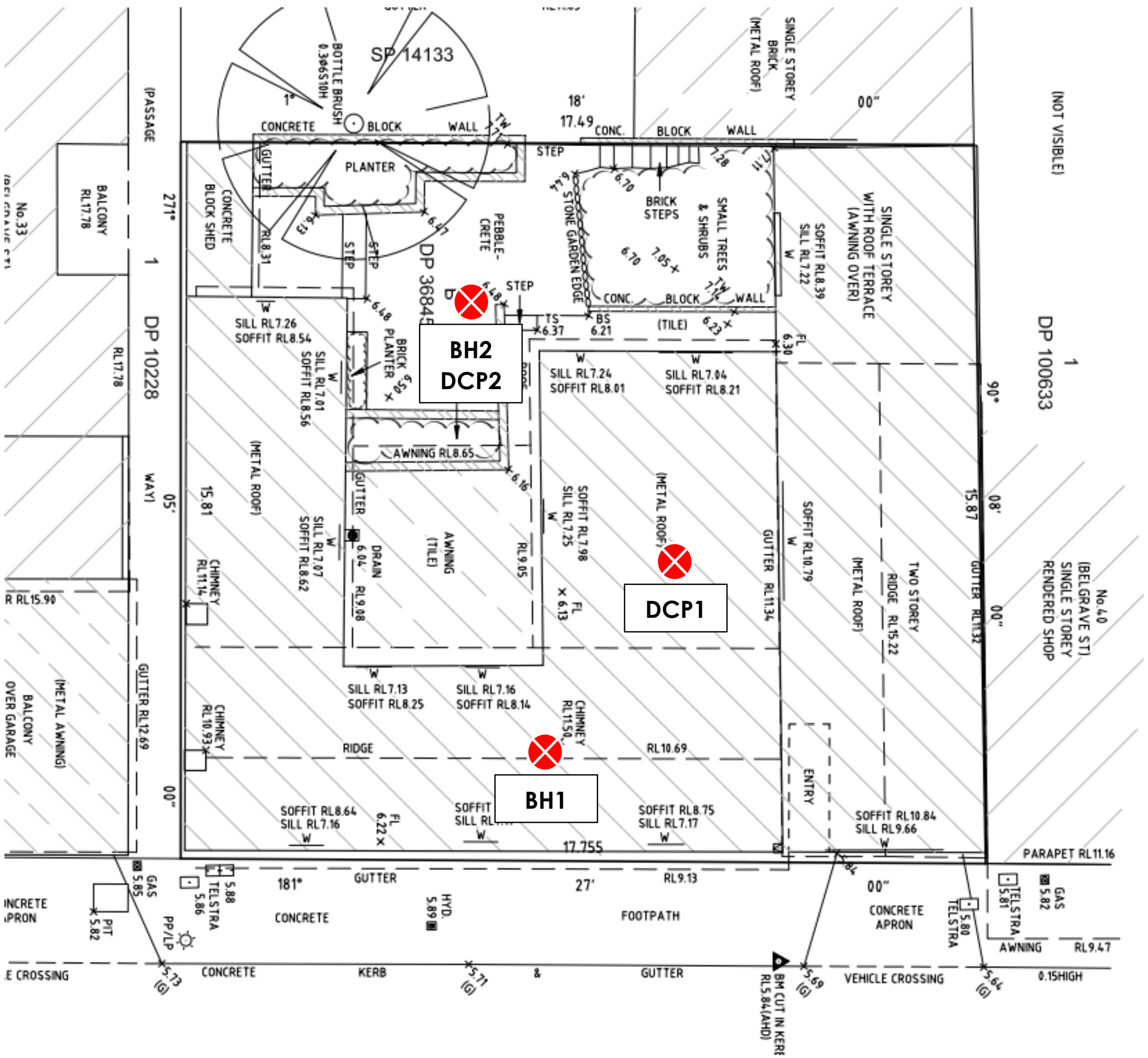
As some clients, contractors and design professionals do not recognise geotechnical engineering is much broader and less exact than other engineering disciplines, this creates unrealistic expectations that lead to claims, disputs and other disappointments. As part of the geotechnical report, (in most cases) a 'limitations' explanatory provision is included, outlining the geotechnical engineers' limitations for your project – with the geotechnical engineers responsibilites to help other reduce their own. This should be read closely as part of your report.

Other Limitations

GCA will not be liable to revise or update the report to take into account any events or circumstances (seen or unforeseen), or any fact occurring or becoming apparent after the date of the report. This report is the subject of copyright and shall not be reproduced either totally or in part without the express permission of GCA. The report should not be used if there have been changes to the project, without first consulting with GCA to assess if the report's recommendations are still valid. GCA does not accept any responsibility for problems that occur due to project changes which have not been consulted.

APPENDIX B

Legend: Approximate Borehole/DCP Testing Location



GCA Geotechnical Consultants Australia	Figure 1 Site Plan	**Geotechnical Investigation**	Drawn: GN/GA	
		J & CG Con NSW Pty Ltd	Date: 02/06/2023	
	Job No.: G21699-1-Rev B	21 Whistler Street Manly NSW 2095	Scale: NTS	

Image source: Site survey plan prepared by Norton Survey Partners, titled "Plan Showing Selected Detail & Level Over No. 21 Whistler Street, Manly", referenced No. 53011, sheet 1 of 1 and dated 13th March 2018.

APPENDIX C

Explanation of Notes, Abbreviations and Terms Used on Borehole and Test Pit Reports

DRILLING/EXCAVATION METHOD

Method	Description
AS	Auger Screwing
BH	Backhoe
CT	Cable Tool Rig
EE	Existing Excavation/Cutting
EX	Excavator
HA	Hand Auger
HQ	Diamond Core – 63mm
JET	Jetting
NMLC	Diamond Core – 52mm
NQ	Diamond Core – 47mm
PT	Push Tube
RAB	Rotary Air Blast
RB	Rotary Blade
RT	Rotary Tricone Bit
TC	Auger TC Bit
V	Auger V Bit
WB	Washbore
DT	Diatube
CC	Concrete Coring

PENETRATIION/EXCAVATION RESISTANCE

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

L **Low Resistance.** Rapid penetration possible with little effort from the equipment used.

M **Medium Resistance.** Excavation possible at an acceptable rate with moderate effort required from the equipment used.

H **High Resistance.** Further penetration is possible at a slow rate and required significant effort from the equipment.

R **Refusal or Practical Refusal.** No further progress possible within the risk of damage or excessive wear to the equipment used.

WATER

▽ Water level at date shown ◁ Partial water loss

▷ Water inflow ◀ Complete water loss

Groundwater not observed: The observation of groundwater, whether present or not, was not possible due to drilling water, surface seepage or cave in of the borehole/test pit.

Groundwater not encountered: No free-flowing (springs or seepage) was intercepted, although the soil may be moist due to capillary water. Water may be observed in low permeable soils if the test pits/boreholes had been left open for at least 12-24 hours.

MOISTURE CONDITION (AS 1726-2017)

Dry - Cohesive soils are friable or powdery
Cohesionless soil grains are free-running

Moist - Soil feels cool, darkened in colour
Cohesive soils can be moulded
Cohesionless soil grains tend to adhere

Wet - Cohesive soils usually weakened
Free water forms on hands when handling

For cohesive soils the following codes may also be used:

MC>PL Moisture Content greater than the Plastic Limit.
MC~PL Moisture Content near the Plastic Limit.
MC<PL Moisture Content less than the Plastic Limit.

SAMPLING AND TESTING

Sample	Description
B	Bulk Disturbed Sample
DS	Disturbed Sample
Jar	Jar Sample
SPT*	Standard Penetration Test
U50	Undisturbed Sample – 50mm
U75	Undisturbed Sample – 75mm

*SPT (4, 7, 11 N=18). 4, 7, 11 = Blows per 150mm. N= Blows per 300mm penetration following 150mm sealing.
SPT (30/80mm). Where practical refusal occurs, the blows and penetration for that interval is recorded.

ROCK QUALITY

The fracture spacing is shown where applicable and the Rock Quality Designation (RQD) or Total Core Recovery (TCR) is given where:

$$\text{TCR (\%)} = \frac{\text{length of core recovered}}{\text{length of core run}}$$

$$\text{RQD (\%)} = \frac{\text{sum of axial lengths of core > 100mm long}}{\text{length of core run}}$$

ROCK STRENGTH TEST RESULTS

● Diametral Point Load Index test

■ Axial Point Load Index test

SOIL ORIGINS

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- **Residual soils**: derived from in-situ weathering of the underlying rock (see "rock material weathering" below).
- **Transported soils**: formed somewhere else and transported by nature to the site.
- **Filling**: moved/placed by man.

Transported soils may be further subdivided into:

- **Alluvium/alluvial**: river deposits.
- **Lacustrine**: lake deposits.
- **Aeolian**: wind deposits.
- **Littoral**: beach deposits.
- **Estuarine**: tidal river deposits.
- **Talus**: scree or coarse colluvium.
- **Slopewash or colluvium/colluvial**: transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

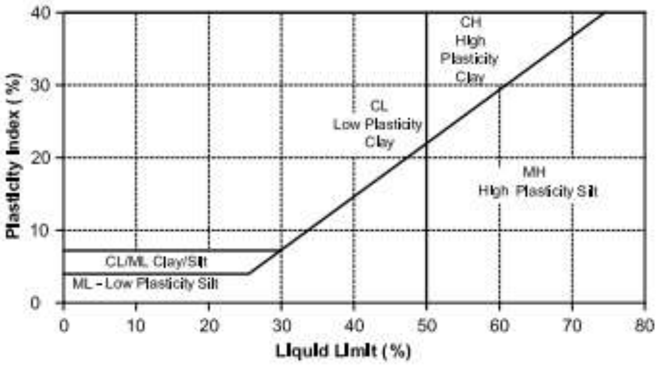
Method and Terms for Soil and Rock Descriptions Used on Borehole and Test Pit Reports

Soil and Rock is classified and described in reports of boreholes and test pits using the preferred method given in AS 1726-2017, Appendix A. The material properties are assessed in the field by visual/tactile methods. The appropriate symbols in the Unified Soil Classification are selected on the result of visual examination, field tests and available laboratory tests, such as, sieve analysis, liquid limit and plasticity index.

COHESIONLESS SOILS PARTICLE SIZE DESCRIPTIVE TERMS

Name	Subdivision	Size
Boulders		>200mm
Cobbles		63mm to 200mm
Gravel	coarse	20mm to 63mm
	medium	6mm to 20mm
	fine	2.36mm to 6mm
Sand	coarse	600µm to 2.36mm
	medium	200µm to 600µm
	fine	75µm to 200µm

PLASTICITY PROPERTIES



COHESIVE SOILS – CONSISTENCY (AS 1726-2017)

Strength	Symbol	Undrained Shear Strength, C_u (kPa)
Very Soft	VS	< 12
Soft	S	12 to 25
Firm	F	25 to 50
Stiff	St	50 to 100
Very Stiff	VSt	100 to 200
Hard	H	> 200
Friable	Fr	Easily crumbled or broken into small pieces by hand

PLASTICITY

Description of Plasticity	LL (%)
Low	<35
Medium	35 to 50
High	>50

COHESIONLESS SOILS - RELATIVE DENSITY

Term	Symbol	Density Index	N Value (blows/0.3 m)
Very Loose	VL	0 to 15	0 to 4
Loose	L	15 to 35	4 to 10
Medium Dense	MD	35 to 65	10 to 30
Dense	D	65 to 85	30 to 50
Very Dense	VD	>85	>50

UNIFIED SOIL CLASSIFICATION

USC Symbol	Description
GW	Well graded gravel
GP	Poorly graded gravel
GM	Silty gravel
GC	Clayey gravel
SW	Well graded sand
SP	Poorly graded sand
SM	Silty sand
SC	Clayey sand
ML	Silt of low plasticity
CL	Clay of low plasticity
OL	Organic soil of low plasticity
MH	Silt of high plasticity
CH	Clay of high plasticity
OH	Organic soil of high plasticity
Pt	Peaty Soil

ROCK MATERIAL WEATHERING

Symbol	Term	Definition
RS	Residual Soil	Soil definition on extremely weathered rock; the mass structure and substance 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
HW *DW*	Highly Weathered *Distinctly Weathered (as per AS 1726)*	The rock substance is affected by weathering to the extent that limonite staining or bleaching affects the whole rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength is usually decreased compared to the fresh rock. The colour and strength of the fresh rock is no longer recognisable.
MW	Moderately Weathered	The whole of the rock substance is discoloured, usually by iron staining or bleaching, to the extent that the colour of the fresh rock is no longer recognisable
SW	Slightly Weathered	Rock is slightly discoloured but shows little or no change of strength from fresh rock
FR	Fresh	Rock shows no sign of decomposition or staining

ROCK STRENGTH (AS 1726-2017 and ISRM)

Term	Symbol	Point Load Index $I_{s(50)}$ (MPa)
Extremely Low	EL	<0.03
Very Low	VL	0.03 to 0.1
Low	L	0.1 to 0.3
Medium	M	0.3 to 1
High	H	1 to 3
Very High	VH	3 to 10
Extremely High	EH	>10

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ABREVIATIONS FOR DEFECT TYPES AND DECRIPTIONS

Term	Defect Spacing	Bedding
Extremely closely spaced	<6mm	Thinly Laminated
	6mm to 20mm	Laminated
Very closely spaced	20mm to 60mm	Very Thin
Closely spaced	0.06m to 0.2m	Thin
Moderately widely spaced	0.2m to 0.6m	Medium
Widely spaced	0.6m to 2m	Thick
Very widely spaced	>2m	Very Thick

Type	Definition
B	Bedding
J	Joint
HJ	Horizontal to Sub-Horizontal Joint
VJ	Vertical to Sub-Vertical Joint
F	Fault
Cle	Cleavage
SZ	Shear Zone
SM	Shear Seam
FZ	Fractured Zone
CZ	Crushed Zone
CS	Crushed Seam
MB	Mechanical Break
HB	Handling Break

Planarity	Roughness
P – Planar	C – Clean
Ir – Irregular	Cl – Clay
St – Stepped	VR – Very Rough
U – Undulating	R – Rough
	S – Smooth
	Sl – Slickensides
	Po – Polished
	Fe – Iron

Coating or Infill	Description
Clean (C)	No visible coating or infilling
Stain	No visible coating or infilling but surfaces are discoloured by mineral staining
Veneer	A visible coating or infilling of soil or mineral substance but usually unable to be measured (<1mm). If discontinuous over the plane, patchy veneer
Coating	A visible coating or infilling of soil or mineral substance, >1mm thick. Describe composition and thickness
Iron (Fe)	Iron Staining or Infill.

APPENDIX D

BOREHOLE NUMBER BH1

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(02) 9788 2829

CLIENT J & CG Con NSW Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21699-1
PROJECT LOCATION 21 Whistler Street Manly NSW 2095

DATE STARTED 11/11/21 **COMPLETED** 11/11/21 **R.L. SURFACE** **DATUM**
DRILLING CONTRACTOR A P Smith **SLOPE** 90° **BEARING** ---
EQUIPMENT Trailer Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT			0.0			Gravelly SAND, fine to medium grained, brown to dark brown, fine to coarse gravel, moist.		FILL
			0.5			SAND, fine to medium grained, brown to pale brown, pale grey, some silt, moist.		
			1.0		SW	SAND, fine to medium grained, brown to brownish orange, some silt, moist.	DS	MARINE SANDS
			2.5		SW	SAND, fine to medium grained, brown to brownish orange, pale brown, moist.		
			3.0				DS	
			4.0				DS	
			5.0					

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BOREHOLE NUMBER BH1

CLIENT J & CG Con NSW Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21699-1
PROJECT LOCATION 21 Whistler Street Manly NSW 2095

DATE STARTED 11/11/21 **COMPLETED** 11/11/21 **R.L. SURFACE** **DATUM**
DRILLING CONTRACTOR A P Smith **SLOPE** 90° **BEARING** ---
EQUIPMENT Trailer Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT					SW	SAND, fine to medium grained, brown to brownish orange, pale brown, moist. *(continued)* occasional grey laminations, sea shells from 4.0m bgl.		
			5.5					
	►					becoming wet from 5.6m bgl.		
			6.0					
						Borehole BH1 terminated at 6m		
			6.5					
			7.0					
			7.5					
			8.0					
			8.5					
			9.0					
			9.5					
			10.0					

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BOREHOLE NUMBER BH2

CLIENT J & CG Con NSW Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21699-1
PROJECT LOCATION 21 Whistler Street Manly NSW 2095

DATE STARTED 11/11/21 **COMPLETED** 11/11/21 **R.L. SURFACE** **DATUM**
DRILLING CONTRACTOR A P Smith **SLOPE** 90° **BEARING** ---
EQUIPMENT Trailer Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT						SAND, fine to medium grained, brown to dark brown, with fine to coarse gravel, moist.		FILL
						SAND, fine to medium grained, brown to pale brown, greyish white, with crushed sandstone, moist.		
			0.5		SW	SAND, fine to medium grained, brown, grey, some silt, moist, estimated very loose to loose.		MARINE SANDS
			1.0					
					SW	SAND, fine to medium grained, brown to brownish orange, some silt, moist, estimated loose.		
			1.5					
			2.0					
			2.5		SW	SAND, fine to medium grained, brown to pale brown, greyish white and brownish orange laminations, sea shells, moist, estimated medium dense.		
			3.0					
			3.5					
			4.0			becoming estimated dense from 4.0m bgl.		
			4.5					
			5.0				DS	

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BOREHOLE NUMBER BH2

CLIENT J & CG Con NSW Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21699-1
PROJECT LOCATION 21 Whistler Street Manly NSW 2095

DATE STARTED 11/11/21 **COMPLETED** 11/11/21
R.L. SURFACE **DATUM**
DRILLING CONTRACTOR A P Smith
SLOPE 90° **BEARING** ---
EQUIPMENT Trailer Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT					SW	SAND, fine to medium grained, brown to pale brown, greyish white and brownish orange laminations, sea shells, moist, estimated medium dense. *(continued)* becoming moist to wet from 5.0m bgl.		
			5.5			becoming estimated very dense from 5.5m bgl.		
								Maximum DCP testing depth at 5.8m bgl.
	►		6.0		SW	SAND, fine to medium grained, brown to pale brown, grey, some silt, sea shells, wet.		
			6.5					
			7.0				DS	
			7.5					
			8.0		SW	SAND, fine to medium grained, grey, some silt, trace of clay, wet.		
			8.5					
			9.0					
			9.5					
			10.0				DS	

Borehole BH2 terminated at 10m

APPENDIX E

DYNAMIC CONE PENETOMETER RESULTS

Client:	J & CG Con NSW Pty Ltd	Test Date:	11/11/2021
Address:	21 Whistler Street Manly NSW 2095	Job No.:	G21699-1-Rev B

Depths (mm bgl)	DCP No.				Depths (mm bgl)	DCP No.			
	1	2							
0-100	1	2			0-100				
100-200	2	2			100-200				
200-300	2	2			200-300				
300-400	1	3			300-400				
400-500	2	1			400-500				
500-600	2	1			500-600				
600-700	1	1			600-700				
700-800	2	1			700-800				
800-900	1	1			800-900				
900-1000	2	2			900-1000				
1000-1100	1	1			1000-1100				
1100-1200	2	2			1100-1200				
1200-1300	1	1			1200-1300				
1300-1400	2	3			1300-1400				
1400-1500	2	3			1400-1500				
1500-1600	1	2			1500-1600				
1600-1700	2	1			1600-1700				
1700-1800	2	1			1700-1800				
1800-1900	1	2			1800-1900				
1900-2000	2	2			1900-2000				
2000-2100	2	3			2000-2100				
2100-2200	2	2			2100-2200				
2200-2300	2	4			2200-2300				
2300-2400	2	2			2300-2400				
2400-2500	3	2			2400-2500				
2500-2600	2	3			2500-2600				
2600-2700	2	3			2600-2700				
2700-2800	3	5			2700-2800				
2800-2900	2	4			2800-2900				
2900-3000	2	3			2900-3000				
3000-3100	2	3			3000-3100				
3100-3200	2	3			3100-3200				
3200-3300	2	5			3200-3300				
3300-3400	1	6			3300-3400				
3400-3500	2	5			3400-3500				
3500-3600	2	5			3500-3600				
3600-3700	3	8			3600-3700				
3700-3800	3	6			3700-3800				
3800-3900	7	7			3800-3900				
3900-4000	8	8			3900-4000				

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Tested:	GA/GN/AS		Sheet:	1 of 2

DYNAMIC CONE PENETOMETER RESULTS

Client:	J & CG Con NSW Pty Ltd	**Test Date:**	11/11/2021
Address:	21 Whistler Street Manly NSW 2095	**Job No.:**	G21699-1-Rev B

Depths (mm bgl)	DCP No. 1	DCP No. 2	DCP No.	DCP No.	Depths (mm bgl)	DCP No.	DCP No.	DCP No.	DCP No.
4000-4100	6	9			4000-4100				
4100-4200	7	9			4100-4200				
4200-4300	7	10			4200-4300				
4300-4400	13	10			4300-4400				
4400-4500	15	10			4400-4500				
4500-4600	18	13			4500-4600				
4600-4700	16	12			4600-4700				
4700-4800	17	10			4700-4800				
4800-4900	18	14			4800-4900				
4900-5000	18	15			4900-5000				
5000-5100	17	16			5000-5100				
5100-5200	18	17			5100-5200				
5200-5300	20	18			5200-5300				
5300-5400	**Terminated**	17			5300-5400				
5400-5500		17			5400-5500				
5500-5600		20			5500-5600				
5600-5700		19			5600-5700				
5700-5800		22			5700-5800				
5800-5900		**Terminated**			5800-5900				
5900-6000					5900-6000				
6000-6100					6000-6100				
6100-6200					6100-6200				
6200-6300					6200-6300				
6300-6400					6300-6400				
6400-6500					6400-6500				
6500-6600					6500-6600				
6600-6700					6600-6700				
6700-6800					6700-6800				
6800-6900					6800-6900				
6900-7000					6900-7000				
7000-7100					7000-7100				
7100-7200					7100-7200				
7200-7300					7200-7300				
7300-7400					7300-7400				
7400-7500					7400-7500				
7500-7600					7500-7600				
7600-7700					7600-7700				
7700-7800					7700-7800				
7800-7900					7800-7900				
7900-8000					7900-8000				

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Tested:	GA/GN/AS	**©Geotechnical Consultants Australia Pty Ltd**	**Sheet:**	**2 of 2**

APPENDIX F

Foundation Maintenance and Footing Performance: A Homeowner's Guide

CSIRO

BTF 18 replaces Information Sheet 10/91

Buildings can and often do move. This movement can be up, down, lateral or rotational. The fundamental cause of movement in buildings can usually be related to one or more problems in the foundation soil. It is important for the homeowner to identify the soil type in order to ascertain the measures that should be put in place in order to ensure that problems in the foundation soil can be prevented, thus protecting against building movement.

This Building Technology File is designed to identify causes of soil-related building movement, and to suggest methods of prevention of resultant cracking in buildings.

Soil Types

The types of soils usually present under the topsoil in land zoned for residential buildings can be split into two approximate groups – granular and clay. Quite often, foundation soil is a mixture of both types. The general problems associated with soils having granular content are usually caused by erosion. Clay soils are subject to saturation and swell/shrink problems.

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870, the Residential Slab and Footing Code.

Causes of Movement

Settlement due to construction

There are two types of settlement that occur as a result of construction:

- Immediate settlement occurs when a building is first placed on its foundation soil, as a result of compaction of the soil under the weight of the structure. The cohesive quality of clay soil mitigates against this, but granular (particularly sandy) soil is susceptible.
- Consolidation settlement is a feature of clay soil and may take place because of the expulsion of moisture from the soil or because of the soil's lack of resistance to local compressive or shear stresses. This will usually take place during the first few months after construction, but has been known to take many years in exceptional cases.

These problems are the province of the builder and should be taken into consideration as part of the preparation of the site for construction. Building Technology File 19 (BTF 19) deals with these problems.

Erosion

All soils are prone to erosion, but sandy soil is particularly susceptible to being washed away. Even clay with a sand component of say 10% or more can suffer from erosion.

Saturation

This is particularly a problem in clay soils. Saturation creates a bog-like suspension of the soil that causes it to lose virtually all of its bearing capacity. To a lesser degree, sand is affected by saturation because saturated sand may undergo a reduction in volume – particularly imported sand fill for bedding and blinding layers. However, this usually occurs as immediate settlement and should normally be the province of the builder.

Seasonal swelling and shrinkage of soil

All clays react to the presence of water by slowly absorbing it, making the soil increase in volume (see table below). The degree of increase varies considerably between different clays, as does the degree of decrease during the subsequent drying out caused by fair weather periods. Because of the low absorption and expulsion rate, this phenomenon will not usually be noticeable unless there are prolonged rainy or dry periods, usually of weeks or months, depending on the land and soil characteristics.

The swelling of soil creates an upward force on the footings of the building, and shrinkage creates subsidence that takes away the support needed by the footing to retain equilibrium.

Shear failure

This phenomenon occurs when the foundation soil does not have sufficient strength to support the weight of the footing. There are two major post-construction causes:

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.
- In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes
H	Highly reactive clay sites, which can experience high ground movement from moisture changes
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes
A to P	Filled sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

Tree root growth
Trees and shrubs that are allowed to grow in the vicinity of footings can cause foundation soil movement in two ways:

- Roots that grow under footings may increase in cross-sectional size, exerting upward pressure on footings.
- Roots in the vicinity of footings will absorb much of the moisture in the foundation soil, causing shrinkage or subsidence.

Unevenness of Movement

The types of ground movement described above usually occur unevenly throughout the building's foundation soil. Settlement due to construction tends to be uneven because of:

- Differing compaction of foundation soil prior to construction.
- Differing moisture content of foundation soil prior to construction.

Movement due to non-construction causes is usually more uneven still. Erosion can undermine a footing that traverses the flow or can create the conditions for shear failure by eroding soil adjacent to a footing that runs in the same direction as the flow.

Saturation of clay foundation soil may occur where subfloor walls create a dam that makes water pond. It can also occur wherever there is a source of water near footings in clay soil. This leads to a severe reduction in the strength of the soil which may create local shear failure.

Seasonal swelling and shrinkage of clay soil affects the perimeter of the building first, then gradually spreads to the interior. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Swelling gradually reaches the interior soil as absorption continues. Shrinkage usually begins where the sun's heat is greatest.

Effects of Uneven Soil Movement on Structures

Erosion and saturation
Erosion removes the support from under footings, tending to create subsidence of the part of the structure under which it occurs. Brickwork walls will resist the stress created by this removal of support by bridging the gap or cantilevering until the bricks or the mortar bedding fail. Older masonry has little resistance. Evidence of failure varies according to circumstances and symptoms may include:

- Step cracking in the mortar beds in the body of the wall or above/below openings such as doors or windows.
- Vertical cracking in the bricks (usually but not necessarily in line with the vertical beds or perpends).

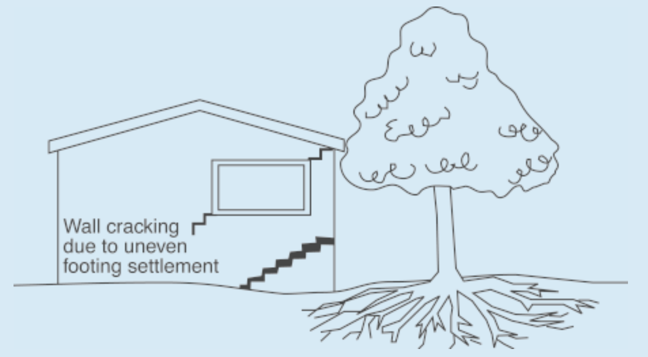
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the external footings are pushed higher than the internal ones.

The first noticeable symptom may be that the floor appears slightly dished. This is often accompanied by some doors binding on the floor or the door head, together with some cracking of cornice mitres. In buildings with timber flooring supported by bearers and joists, the floor can be bouncy. Externally there may be visible dishing of the hip or ridge lines.

As the moisture absorption process completes its journey to the innermost areas of the building, the internal footings will rise. If the spread of moisture is roughly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

Trees can cause shrinkage and damage



As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun's effect is strongest. This has the effect of lowering the external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

Doming and dishing are also affected by weather in other ways. In areas where warm, wet summers and cooler dry winters prevail, water migration tends to be toward the interior and doming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensity is toward dishing.

Movement caused by tree roots
In general, growing roots will exert an upward pressure on footings, whereas soil subject to drying because of tree or shrub roots will tend to remove support from under footings by inducing shrinkage.

Complications caused by the structure itself
Most forces that the soil causes to be exerted on structures are vertical – i.e. either up or down. However, because these forces are seldom spread evenly around the footings, and because the building resists uneven movement because of its rigidity, forces are exerted from one part of the building to another. The net result of all these forces is usually rotational. This resultant force often complicates the diagnosis because the visible symptoms do not simply reflect the original cause. A common symptom is binding of doors on the vertical member of the frame.

Effects on full masonry structures
Brickwork will resist cracking where it can. It will attempt to span areas that lose support because of subsided foundations or raised points. It is therefore usual to see cracking at weak points, such as openings for windows or doors.

In the event of construction settlement, cracking will usually remain unchanged after the process of settlement has ceased.

With local shear or erosion, cracking will usually continue to develop until the original cause has been remedied, or until the subsidence has completely neutralised the affected portion of footing and the structure has stabilised on other footings that remain effective.

In the case of swell/shrink effects, the brickwork will in some cases return to its original position after completion of a cycle, however it is more likely that the rotational effect will not be exactly reversed, and it is also usual that brickwork will settle in its new position and will resist the forces trying to return it to its original position. This means that in a case where swelling takes place after construction and cracking occurs, the cracking is likely to at least partly remain after the shrink segment of the cycle is complete. Thus, each time the cycle is repeated, the likelihood is that the cracking will become wider until the sections of brickwork become virtually independent.

With repeated cycles, once the cracking is established, if there is no other complication, it is normal for the incidence of cracking to stabilise, as the building has the articulation it needs to cope with the problem. This is by no means always the case, however, and monitoring of cracks in walls and floors should always be treated seriously.

Upheaval caused by growth of tree roots under footings is not a simple vertical shear stress. There is a tendency for the root to also exert lateral forces that attempt to separate sections of brickwork after initial cracking has occurred.

The normal structural arrangement is that the inner leaf of brickwork in the external walls and at least some of the internal walls (depending on the roof type) comprise the load-bearing structure on which any upper floors, ceilings and the roof are supported. In these cases, it is internally visible cracking that should be the main focus of attention, however there are a few examples of dwellings whose external leaf of masonry plays some supporting role, so this should be checked if there is any doubt. In any case, externally visible cracking is important as a guide to stresses on the structure generally, and it should also be remembered that the external walls must be capable of supporting themselves.

Effects on framed structures
Timber or steel framed buildings are less likely to exhibit cracking due to swell/shrink than masonry buildings because of their flexibility. Also, the doming/dishing effects tend to be lower because of the lighter weight of walls. The main risks to framed buildings are encountered because of the isolated pier footings used under walls. Where erosion or saturation cause a footing to fall away, this can double the span which a wall must bridge. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by a door or window opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage without first exhibiting some or all of the above symptoms for a considerable period. The same warning period should apply in the case of upheaval. It should be noted, however, that where framed buildings are supported by strip footings there is only one leaf of brickwork and therefore the externally visible walls are the supporting structure for the building. In this case, the subfloor masonry walls can be expected to behave as full brickwork walls.

Effects on brick veneer structures
Because the load-bearing structure of a brick veneer building is the frame that makes up the interior leaf of the external walls plus perhaps the internal walls, depending on the type of roof, the building can be expected to behave as a framed structure, except that the external masonry will behave in a similar way to the external leaf of a full masonry structure.

Water Service and Drainage

Where a water service pipe, a sewer or stormwater drainage pipe is in the vicinity of a building, a water leak can cause erosion, swelling or saturation of susceptible soil. Even a minuscule leak can be enough to saturate a clay foundation. A leaking tap near a building can have the same effect. In addition, trenches containing pipes can become watercourses even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, interstrata seepage into subfloor areas and saturation.

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem.
Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.
- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870.

AS 2870 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

Prevention/Cure

Plumbing
Where building movement is caused by water service, roof plumbing, sewer or stormwater failure, the remedy is to repair the problem. It is prudent, however, to consider also rerouting pipes away from the building where possible, and relocating taps to positions where any leakage will not direct water to the building vicinity. Even where gully traps are present, there is sometimes sufficient spill to create erosion or saturation, particularly in modern installations using smaller diameter PVC fixtures. Indeed, some gully traps are not situated directly under the taps that are installed to charge them, with the result that water from the tap may enter the backfilled trench that houses the sewer piping. If the trench has been poorly backfilled, the water will either pond or flow along the bottom of the trench. As these trenches usually run alongside the footings and can be at a similar depth, it is not hard to see how any water that is thus directed into a trench can easily affect the foundation's ability to support footings or even gain entry to the subfloor area.

Ground drainage
In all soils there is the capacity for water to travel on the surface and below it. Surface water flows can be established by inspection during and after heavy or prolonged rain. If necessary, a grated drain system connected to the stormwater collection system is usually an easy solution.

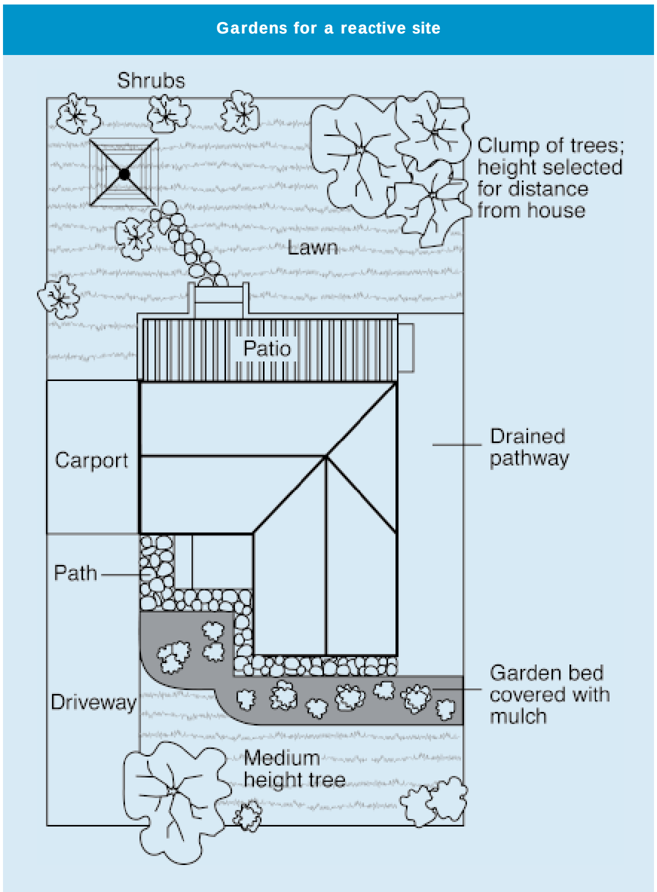
It is, however, sometimes necessary when attempting to prevent water migration that testing be carried out to establish watertable height and subsoil water flows. This subject is referred to in BTF 19 and may properly be regarded as an area for an expert consultant.

Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

For this reason, particularly where problems exist or are likely to occur, it is recommended that an apron of paving be installed around as much of the building perimeter as necessary. This paving

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

Description of typical damage and required repair	Approximate crack width limit (see Note 3)	Damage category
Hairline cracks	<0.1 mm	0
Fine cracks which do not need repair	<1 mm	1
Cracks noticeable but easily filled. Doors and windows stick slightly	<5 mm	2
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weathertightness often impaired	5–15 mm (or a number of cracks 3 mm or more in one group)	3
Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted	15–25 mm but also depend on number of cracks	4



should extend outwards a minimum of 900 mm (more in highly reactive soil) and should have a minimum fall away from the building of 1:60. The finished paving should be no less than 100 mm below brick vent bases.

It is prudent to relocate drainage pipes away from this paving, if possible, to avoid complications from future leakage. If this is not practical, earthenware pipes should be replaced by PVC and backfilling should be of the same soil type as the surrounding soil and compacted to the same density.

Except in areas where freezing of water is an issue, it is wise to remove taps in the building area and relocate them well away from the building – preferably not uphill from it (see BTF 19).

It may be desirable to install a grated drain at the outside edge of the paving on the uphill side of the building. If subsoil drainage is needed this can be installed under the surface drain.

Condensation
In buildings with a subfloor void such as where bearers and joists support flooring, insufficient ventilation creates ideal conditions for condensation, particularly where there is little clearance between the floor and the ground. Condensation adds to the moisture already present in the subfloor and significantly slows the process of drying out. Installation of an adequate subfloor ventilation system, either natural or mechanical, is desirable.

Warning: Although this Building Technology File deals with cracking in buildings, it should be said that subfloor moisture can result in the development of other problems, notably:

- Water that is transmitted into masonry, metal or timber building elements causes damage and/or decay to those elements.
- High subfloor humidity and moisture content create an ideal environment for various pests, including termites and spiders.
- Where high moisture levels are transmitted to the flooring and walls, an increase in the dust mite count can ensue within the living areas. Dust mites, as well as dampness in general, can be a health hazard to inhabitants, particularly those who are abnormally susceptible to respiratory ailments.

The garden
The ideal vegetation layout is to have lawn or plants that require only light watering immediately adjacent to the drainage or paving edge, then more demanding plants, shrubs and trees spread out in that order.

Overwatering due to misuse of automatic watering systems is a common cause of saturation and water migration under footings. If it is necessary to use these systems, it is important to remove garden beds to a completely safe distance from buildings.

Existing trees
Where a tree is causing a problem of soil drying or there is the existence or threat of upheaval of footings, if the offending roots are subsidiary and their removal will not significantly damage the tree, they should be severed and a concrete or metal barrier placed vertically in the soil to prevent future root growth in the direction of the building. If it is not possible to remove the relevant roots without damage to the tree, an application to remove the tree should be made to the local authority. A prudent plan is to transplant likely offenders before they become a problem.

Information on trees, plants and shrubs
State departments overseeing agriculture can give information regarding root patterns, volume of water needed and safe distance from buildings of most species. Botanic gardens are also sources of information. For information on plant roots and drains, see Building Technology File 17.

Excavation
Excavation around footings must be properly engineered. Soil supporting footings can only be safely excavated at an angle that allows the soil under the footing to remain stable. This angle is called the angle of repose (or friction) and varies significantly between soil types and conditions. Removal of soil within the angle of repose will cause subsidence.

Remediation

Where erosion has occurred that has washed away soil adjacent to footings, soil of the same classification should be introduced and compacted to the same density. Where footings have been undermined, augmentation or other specialist work may be required. Remediation of footings and foundations is generally the realm of a specialist consultant.

Where isolated footings rise and fall because of swell/shrink effect, the homeowner may be tempted to alleviate floor bounce by filling the gap that has appeared between the bearer and the pier with blocking. The danger here is that when the next swell segment of the cycle occurs, the extra blocking will push the floor up into an accentuated dome and may also cause local shear failure in the soil. If it is necessary to use blocking, it should be by a pair of fine wedges and monitoring should be carried out fortnightly.

This BTF was prepared by John Lewer FAIB, MIAMA, Partner, Construction Diagnosis.

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APPENDIX G

CERTIFICATE OF ANALYSIS

Work Order	: **ES2141020**	Page	: 1 of 4
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA PTY LTD**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: Suite 5, 5-7 Villiers Street Parramatta NSW 2151	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G21699-1 Geotechnical Investigation	Date Samples Received	: 12-Nov-2021 13:00
Order number	: ----	Date Analysis Commenced	: 17-Nov-2021
C-O-C number	: ----	Issue Date	: 23-Nov-2021 08:38
Sampler	: George A, George N		
Site	: 21 Whistler Street Manly NSW 2095		
Quote number	: EN/333		
No. of samples received	: 6		
No. of samples analysed	: 6		

Accreditation No. 825
Accredited for compliance with
ISO/IEC 17025 - Testing

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted, unless the sampling was conducted by ALS. This document shall not be reproduced, except in full.

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results

Additional information pertinent to this report will be found in the following separate attachments: Quality Control Report, QA/QC Compliance Assessment to assist with Quality Review and Sample Receipt Notification.

Signatories

This document has been electronically signed by the authorized signatories below. Electronic signing is carried out in compliance with procedures specified in 21 CFR Part 11.

Signatories	*Position*	*Accreditation Category*
Franco Lentini	LCMS Coordinator	Sydney Inorganics, Smithfield, NSW
Wisam Marassa	Inorganics Coordinator	Sydney Inorganics, Smithfield, NSW

General Comments

The analytical procedures used by ALS have been developed from established internationally recognised procedures such as those published by the USEPA, APHA, AS and NEPM. In house developed procedures are fully validated and are often at the client request.

Where moisture determination has been performed, results are reported on a dry weight basis.

Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis.

Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

When sampling time information is not provided by the client, sampling dates are shown without a time component. In these instances, the time component has been assumed by the laboratory for processing purposes.

Where a result is required to meet compliance limits the associated uncertainty must be considered. Refer to the ALS Contact for details.

Key : CAS Number = CAS registry number from database maintained by Chemical Abstracts Services. The Chemical Abstracts Service is a division of the American Chemical Society.
LOR = Limit of reporting
^ = This result is computed from individual analyte detections at or above the level of reporting
ø = ALS is not NATA accredited for these tests.
~ = Indicates an estimated value.

Analytical Results

Sub-Matrix: SOIL (Matrix: SOIL)			Sample ID	BH1 0.9m-1.0m	BH1 2.9m-3.0m	BH1 3.9m-4.0m	BH2 4.9m-5.0m	BH2 6.9m-7.0m
			Sampling date / time	11-Nov-2021 00:00	11-Nov-2021 00:00	11-Nov-2021 00:00	11-Nov-2021 00:00	11-Nov-2021 00:00
Compound	CAS Number	LOR	Unit	ES2141020-001	ES2141020-002	ES2141020-003	ES2141020-004	ES2141020-005
				Result	Result	Result	Result	Result
EA002: pH 1:5 (Soils)								
pH Value	----	0.1	pH Unit	6.3	8.8	8.7	9.3	9.2
EA010: Conductivity (1:5)								
Electrical Conductivity @ 25°C	----	1	µS/cm	54	43	26	78	86
EA055: Moisture Content (Dried @ 105-110°C)								
Moisture Content	----	1.0	%	2.5	1.6	1.9	24.0	24.4
ED040S : Soluble Sulfate by ICPAES								
Sulfate as SO4 2-	14808-79-8	10	mg/kg	20	<10	<10	<10	20
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	10	mg/kg	10	<10	<10	10	20

Analytical Results

Sub-Matrix: SOIL (Matrix: SOIL)			Sample ID	BH2 9.9m-10.0m	----	----	----	----
			Sampling date / time	11-Nov-2021 00:00	----	----	----	----
Compound	CAS Number	LOR	Unit	ES2141020-006	--------	--------	--------	--------
				Result	----	----	----	----
EA002: pH 1:5 (Soils)								
pH Value	----	0.1	pH Unit	**8.3**	----	----	----	----
EA010: Conductivity (1:5)								
Electrical Conductivity @ 25°C	----	1	µS/cm	**140**	----	----	----	----
EA055: Moisture Content (Dried @ 105-110°C)								
Moisture Content	----	1.0	%	**18.0**	----	----	----	----
ED040S : Soluble Sulfate by ICPAES								
Sulfate as SO_4 2-	14808-79-8	10	mg/kg	**20**	----	----	----	----
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	10	mg/kg	**10**	----	----	----	----

QUALITY CONTROL REPORT

Work Order	: **ES2141020**	Page	: 1 of 3
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA PTY LTD**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
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This Quality Control Report contains the following information:

- Laboratory Duplicate (DUP) Report; Relative Percentage Difference (RPD) and Acceptance Limits
- Method Blank (MB) and Laboratory Control Spike (LCS) Report; Recovery and Acceptance Limits
- Matrix Spike (MS) Report; Recovery and Acceptance Limits

Signatories

This document has been electronically signed by the authorized signatories below. Electronic signing is carried out in compliance with procedures specified in 21 CFR Part 11.

Signatories	*Position*	*Accreditation Category*
Franco Lentini	LCMS Coordinator	Sydney Inorganics, Smithfield, NSW
Wisam Marassa	Inorganics Coordinator	Sydney Inorganics, Smithfield, NSW

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Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis. Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

Key : Anonymous = Refers to samples which are not specifically part of this work order but formed part of the QC process lot
CAS Number = CAS registry number from database maintained by Chemical Abstracts Services. The Chemical Abstracts Service is a division of the American Chemical Society.
LOR = Limit of reporting
RPD = Relative Percentage Difference
# = Indicates failed QC

Laboratory Duplicate (DUP) Report

The quality control term Laboratory Duplicate refers to a randomly selected intralaboratory split. Laboratory duplicates provide information regarding method precision and sample heterogeneity. The permitted ranges for the Relative Percent Deviation (RPD) of Laboratory Duplicates are specified in ALS Method QWI-EN/38 and are dependent on the magnitude of results in comparison to the level of reporting: Result < 10 times LOR: No Limit; Result between 10 and 20 times LOR: 0% - 50%; Result > 20 times LOR: 0% - 20%.

Sub-Matrix: **SOIL**

				Laboratory Duplicate (DUP) Report					
Laboratory sample ID	*Sample ID*	*Method: Compound*	*CAS Number*	*LOR*	*Unit*	*Original Result*	*Duplicate Result*	*RPD (%)*	*Acceptable RPD (%)*
EA002: pH 1:5 (Soils) (QC Lot: 4018729)									
ES2141117-001	Anonymous	EA002: pH Value	----	0.1	pH Unit	6.1	6.1	0.0	0% - 20%
ES2141020-003	BH1 3.9m-4.0m	EA002: pH Value	----	0.1	pH Unit	8.7	8.6	2.1	0% - 20%
EA010: Conductivity (1:5) (QC Lot: 4018731)									
ES2141117-001	Anonymous	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	86	96	11.2	0% - 20%
ES2141020-003	BH1 3.9m-4.0m	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	26	24	7.5	0% - 20%
EA055: Moisture Content (Dried @ 105-110°C) (QC Lot: 4018744)									
ES2141020-002	BH1 2.9m-3.0m	EA055: Moisture Content	----	0.1	%	1.6	1.5	0.0	No Limit
ES2141117-003	Anonymous	EA055: Moisture Content	----	0.1	%	23.3	21.8	6.8	0% - 20%
ED040S: Soluble Major Anions (QC Lot: 4018727)									
ES2141020-003	BH1 3.9m-4.0m	ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	<10	<10	0.0	No Limit
ED045G: Chloride by Discrete Analyser (QC Lot: 4018728)									
ES2141021-003	Anonymous	ED045G: Chloride	16887-00-6	10	mg/kg	30	30	0.0	No Limit
ES2141020-003	BH1 3.9m-4.0m	ED045G: Chloride	16887-00-6	10	mg/kg	<10	<10	0.0	No Limit

Method Blank (MB) and Laboratory Control Sample (LCS) Report

The quality control term Method / Laboratory Blank refers to an analyte free matrix to which all reagents are added in the same volumes or proportions as used in standard sample preparation. The purpose of this QC parameter is to monitor potential laboratory contamination. The quality control term Laboratory Control Sample (LCS) refers to a certified reference material, or a known interference free matrix spiked with target analytes. The purpose of this QC parameter is to monitor method precision and accuracy independent of sample matrix. Dynamic Recovery Limits are based on statistical evaluation of processed LCS.

Sub-Matrix: **SOIL**

				Method Blank (MB) Report	Laboratory Control Spike (LCS) Report			
					Spike	Spike Recovery (%)	Acceptable Limits (%)	
Method: Compound	CAS Number	LOR	Unit	Result	Concentration	LCS	Low	High
EA010: Conductivity (1:5) (QCLot: 4018731)								
EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	<1	1412 µS/cm	98.9	92.0	108
ED040S: Soluble Major Anions (QCLot: 4018727)								
ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	<10	750 mg/kg	115	80.0	120
ED045G: Chloride by Discrete Analyser (QCLot: 4018728)								
ED045G: Chloride	16887-00-6	10	mg/kg	<10	250 mg/kg	104	75.0	125
				<10	5000 mg/kg	103	79.0	117

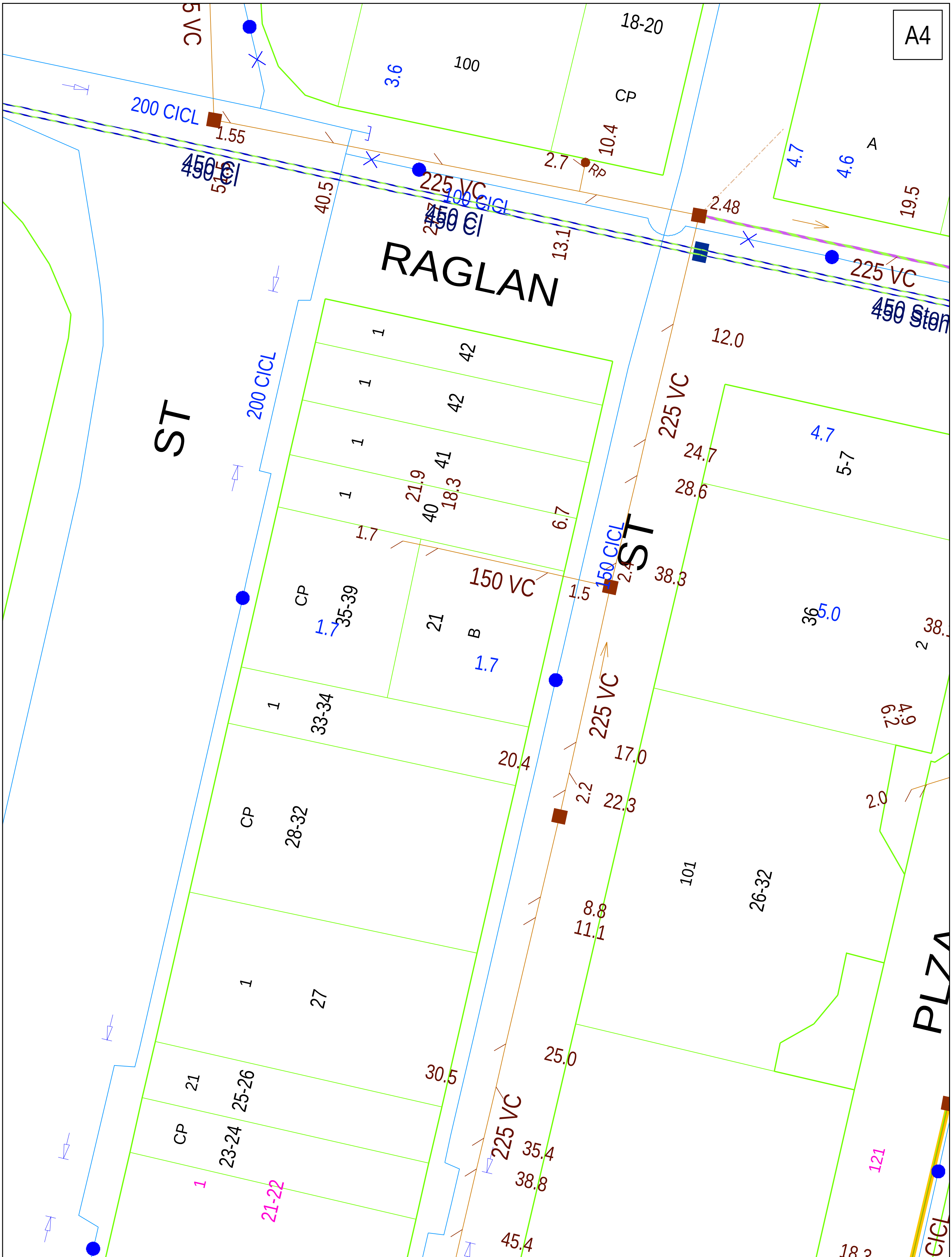
Matrix Spike (MS) Report

The quality control term Matrix Spike (MS) refers to an intralaboratory split sample spiked with a representative set of target analytes. The purpose of this QC parameter is to monitor potential matrix effects on analyte recoveries. Static Recovery Limits as per laboratory Data Quality Objectives (DQOs). Ideal recovery ranges stated may be waived in the event of sample matrix interference.

Sub-Matrix: **SOIL**

				Matrix Spike (MS) Report			
				Spike	SpikeRecovery(%)	Acceptable Limits (%)	
Laboratory sample ID	Sample ID	Method: Compound	CAS Number	Concentration	MS	Low	High
ED045G: Chloride by Discrete Analyser (QCLot: 4018728)							
ES2140936-003	Anonymous	ED045G: Chloride	16887-00-6	250 mg/kg	110	70.0	130

APPENDIX H



DBYD Address:
19a Whistler Street
Manly NSW 2095

DBYD Job No: 30784450
DBYD Sequence No: 204577470

No warranty is given that the information shown is complete or accurate.

SYDNEY WATER CORPORATION

Scale: 1:500

Date of Production: 27/10/2021

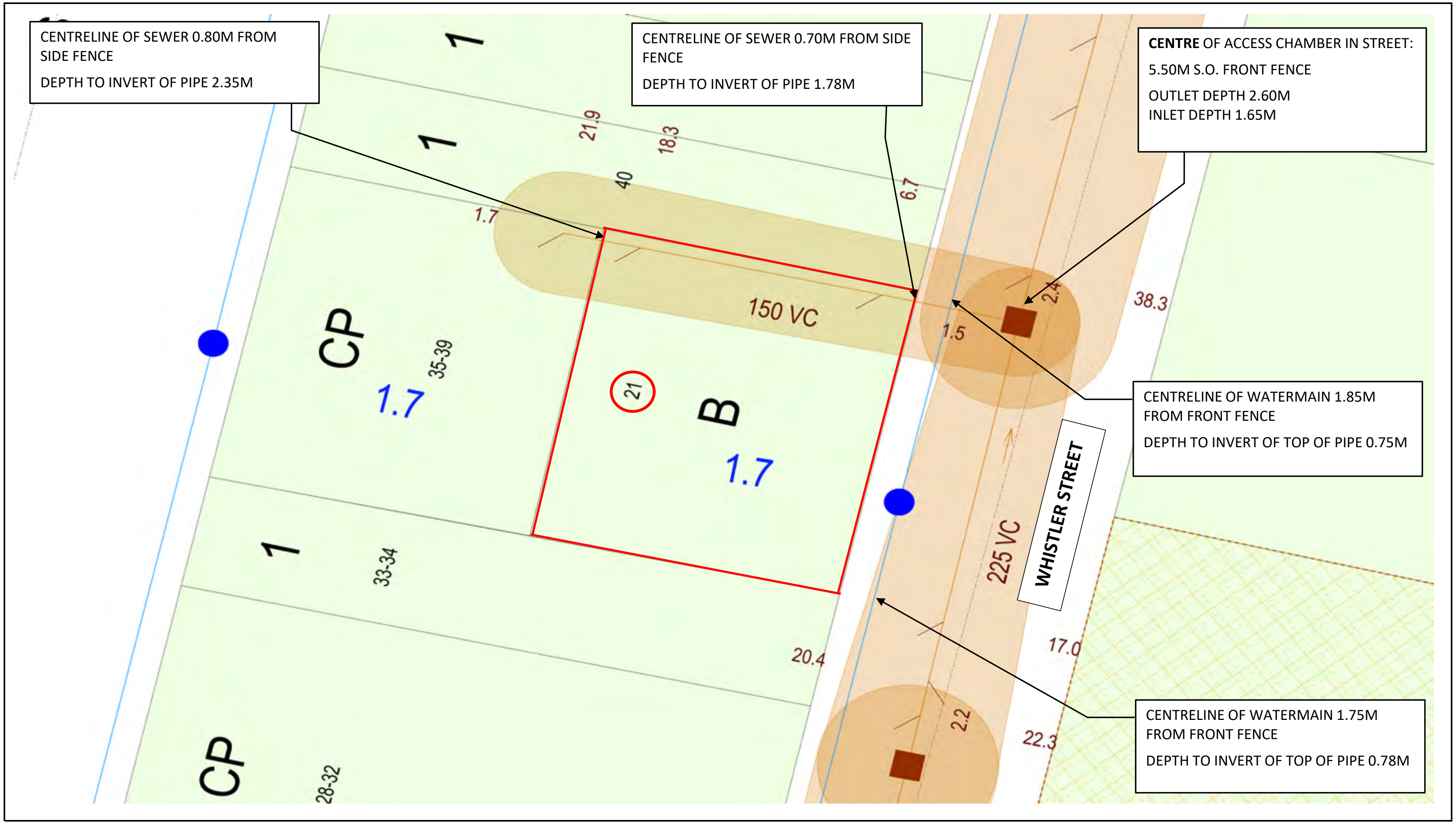
Plan 1 of 1

N

0m 2m 4m 6m 8m 10m

SERVICE PROTECTION REPORT by OLSEN INFRASTRUCTURE PTY LTD

14/10 Gladstone Road, Castle Hill
admin@olseninfrastructure.com.au
Ph 9899 4001   Fax 9899 6005
ABN: 29 132 641 813



CLIENT:

J & CG CONSTRUCTION

PROPERTY ADDRESS:

21 WHISTLER STREET, MANLY

Job No:
21491

Report Date:
23.08.2021

Pipe Size (mm):
150 SEWER
150 WATER

Pipe Type:
VC SEWER
CICL WATER

Strata:
Rock Sand

I Torsten Olsen of Olsen Infrastructure P/L being accredited to carry out a service protection report, certify that the information shown in this report is accurate and has been prepared in accordance with the relevant instructions.

SIGNED **Torsten Olsen**

NOTE:

- Boundaries not surveyed at time of this report
- **Assumed fence lines as boundaries (dimensions taken from fence line)**
- SPRs are valid for a period of 2 years as per Sydney Waters guidelines
- Drawing is NOT to scale (do NOT measure from drawing)
- Physical probing &/or electronic detection used to ascertain asset location

DIMENSIONS REFLECTED IN THIS PLAN MAY NEED TO BE CONFIRMED BY EXCAVATION EXPOSING THE SYDNEY WATER ASSET PRIOR TO CONSTRUCTION COMMENCING ON SITE. SEWER DEPTH NOTED IS BASED UPON GROUND LEVELS AT TIME OF SITE VISIT.

ASSET SIZE AND TYPE IS BASED ON SYDNEY WATER RECORDS.

APPENDIX I

ww	**WOY WOY**	**Marine**

Landscape—level to gently undulating non-tidal beach ridges on marine sands. Local relief <3 m, slopes <5%. Watertable at a depth of <200 cm. Progressive beach ridges in sheltered bays. Extensively cleared closed-scrub and low eucalypt woodland.

Soils—deep (>200 cm) Siliceous Sands (Uc1.22, Uc5.11) and occasional Podzols (Uc2.3) on sandy rises, Humus Podzols (Uc4.2) in poorly drained areas, Calcareous Sands (Uc1.11, Uc1.13) near beaches.

Limitations—permanently high watertables, localised flooding, periodic waterlogging in depressions, very low to low soil fertility, localised areas of high soil erosion hazard.

LOCATION

This unit okccurs on flat low-lying terrain at Woy Woy, Ettalong, Umina, Palm Beach and The Basin in the Narrabeen Hills; also at Clontarf, The Spit and Manly on the harbour foreshores. Most of the beaches in tidal, protected regions have similar small, flat, beach ridge areas. These are usually too small to be included on the map. Examples include Forty Baskets Beach, Sandy Bay and Little Manly.

LANDSCAPE

Geology

Holocene sediments of predominantly coarse to fine quartz sand, with shell fragments and occasionally silt.

Topography

Level to gently undulating, non-tidal, coastal sand flats. Local relief is <3 m. Elevation is <6 m with slopes <5%. Shore parallel beach ridges approximately 100 cm high are common (Hails, 1969). Where residential development has occurred, beach ridges have been levelled and swampy swales filled. The watertable generally occurs within 200 cm of the surface. On the seaward side of this unit the groundwater may be brackish and fluctuate with the tide. In some areas, drainage canals have been excavated to improve drainage.

Vegetation

Extensively cleared open-woodland, with occasional scrub. Dominant native species include coastal banksia *Banksia integrifolia*, old man banksia *B. serrata*, *B. aemula*, red bloodwood *Eucalyptus gummifera*, smooth-barked apple *Angophora costata* and rough-barked apple *A. floribunda*. Common understorey species include bracken *Pteridium esculentum*, blady grass *Imperata cylindrica*, and coastal teatree *Leptospermum laevigatum*. Wetter areas are dominated by Port Jackson fig *Ficus rubiginosa* and bangalay *E. botryoides*.

Land use

This unit is commonly used for urban residential sites such as those at Woy Woy, Umina, Palm Beach, and Clontarf. Recreation areas occur at Clontarf picnic area and The Basin and there are small areas of natural bushland.

Existing Erosion

Little erosion occurs.

Associated Soil Landscapes

Tuggerah (**tg**) soil landscape may be occasionally included within Woy Woy soil landscape; Ettalong (**et**) soil landscape occurs in swamps associated with the Woy Woy soil landscape.

SOILS

Dominant Soil Materials

ww1—Dark brown loose loamy sand. This is dark brown coarse sand to sandy loam with loose apedal single-grained structure and sandy fabric. It occurs as topsoil (A1 horizon).

Colour is black (10YR 2/1) or dark brown (10YR 3/3), due to the presence of organic matter, but can range to a dull yellowish-brown (10YR 5/3). The pH ranges between strongly acid (pH 4.5) and slightly acid (pH 6.5). Occasionally shells and shell fragments are present. Roots are common whilst charcoal fragments are rare, and stones are absent.

ww2—Grey loose sand. This is grey bleached coarse sand with loose apedal single-grained structure and sandy fabric. It commonly occurs as an A2 horizon.

Colour is brownish grey (10YR 5/1), light grey (10YR 7/1) or dull yellowish orange (10YR 7/4). The pH ranges between moderately acid (pH 5.0) and slightly acid (pH 6.5). Roots and charcoal are rare, and stones are absent, but there are occasional shell fragments.

ww3—Brown loose sand. This is brown coarse sand with apedal single-grained structure and sandy fabric. It usually occurs as deep subsoil (B horizon).

Common colours include brown (7.5YR 4/4, 10YR 4/6, 5/6), greyish yellow-brown (10YR 4/2), dull yellow-orange (10YR 6/3). Grey mottles are often present in this material where it is frequently waterlogged. The pH ranges between strongly acid (pH 4.0) and slightly acid (pH 6.0). Shell fragments are occasionally present, but roots, stones and charcoal fragments are absent.

Associated Soil Materials

The following soil materials from Tuggerah soil landscape occasionally occur at depth on old beach ridges: **tg4**—black soft sandy organic pan; and **tg5**—brown soft sandy iron pan.

Occurrence and Relationships

Well-drained beach ridges and sandy rises. Generally, 10–30 cm of dark brown loose loamy sand (**ww1**) overlies >150 cm of brown, loose sand (**ww3**). The total soil depth is >300 cm and the boundary between the soil materials is clear or gradual [Siliceous Sands (Uc 5.11)].

Poorly drained swales and depressions. Generally, 10–30 cm of **ww1** overlies up to 30 cm of grey, bleached loose sand (**ww2**) and >200 cm of **ww3**. Total soil depth is >3 m. Boundaries between soil materials are clear [Humus Podzols (Uc4.21, Uc4.24, Uc2.20)]. The depth of the watertable is usually 50–150 cm.

On some of the larger and older beach ridges, 10–30 cm of **ww1** overlies up to 200 cm of **ww2**. **ww2** overlies a black, soft sandy organic pan (**tg4**) and/or a brown soft sandy iron pan (**tg5**). **tg4** and **tg5** are underlain by **ww3** soil material [Podzols (Uc2.3)]. In swampy areas, 30 cm of dark brown-black, organic rich sandy loam or peaty loam topsoil overlies >100 cm of pale or dark brown sandy subsoil [Acid Peats (O) and Humus Podzols (Uc4.22)].

LIMITATIONS TO DEVELOPMENT

Urban Capability

Low to moderate capability for urban development.

Rural Capability

Not relevant.

Landscape Limitations

Flood hazard (localised)
Wind erosion hazard (localised)
Permanently high watertables
Waterlogging (localised)
Seasonal waterlogging (localised)
Non-cohesive soil

Soil Limitations

ww1 Low available water capacity
Low to very low fertility

ww2 Low available water capacity
Very low fertility

ww3 Low available water capacity
Low to very low fertility
Strongly acid
Strongly alkaline (localised)

Fertility

The general fertility is usually very low. The soil materials have low organic matter content, low available water capacity, very low CEC, and very low nutrient status.

Erodibility

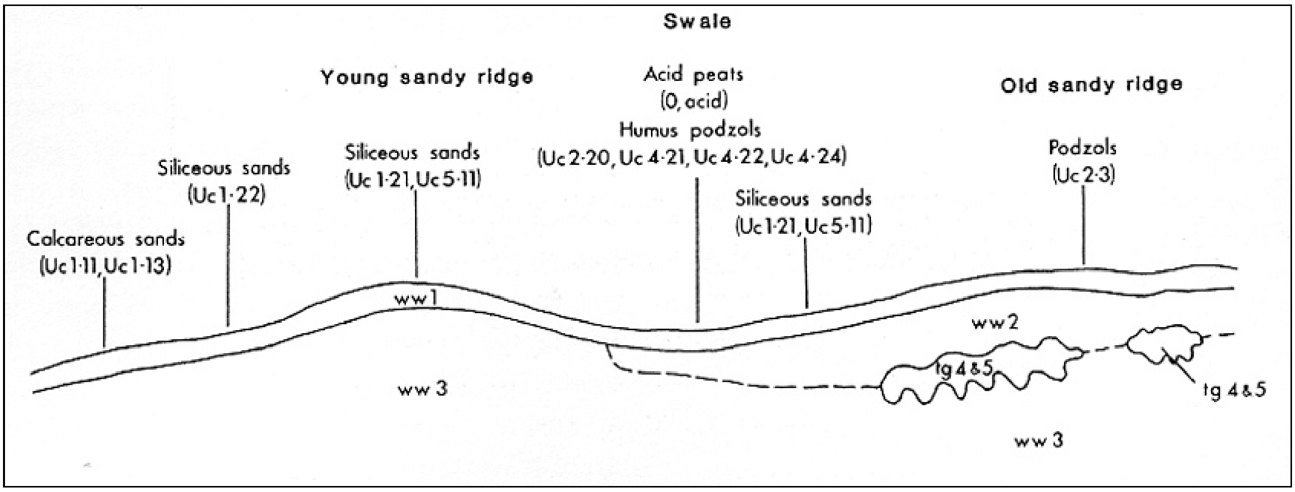
The soil materials all have very low erodibility as they consist of well drained coarse sands.

Erosion Hazard

The erosion hazard for non-concentrated flows is low. Calculated soil loss for the first twelve months of urban development ranges up to 4 t/ha for topsoil and 11 t/ha for subsoil. The erosion hazard for concentrated flows is moderate and for wind is high.

Surface Movement Potential

Sands are stable.



Schematic cross-section of Woy Woy soil landscape illustrating the occurrence and relationship of the dominant soil materials.

na	NARRABEEN	Marine

Landscape— beaches and coastal foredunes on marine sands. Beach plains with relief to 6 m, slopes <3%; foredunes with relief <20 m and slope gradients up to 45%. Spinifex grassland/herbland to closed-scrub on foredunes.

Soils— deep (>200 cm) Calcareous Sands (Uc1.11, Uc1.12) on beaches, Siliceous Sands (Uc1.21, Uc1.22) and occasional calcareous compressed sands on foredunes.

Limitations— extreme wind and wave erosion hazard, non-cohesive soil, very low soil fertility, high soil permeability.

LOCATION

This soil landscape occurs on mainland and barrier beaches exposed to ocean swell and associated wind-blown foredunes. Well known examples include Palm Beach, Whale Beach, Avalon Beach, Newport Beach, Mona Vale Beach, Narrabeen Beach, Long Reef Beach, Dee Why Beach, Curl Curl Beach, Manly Beach, Bondi Beach and Maroubra Beach.

LANDSCAPE

Geology

Quaternary (Holocene) well sorted marine predominantly coarse quartz sands with well sorted, coarse sand-sized, abraded shell fragments.

The foredunes are formed from Quaternary (Holocene) medium and coarse-grained, wind transported, marine sands with traces and bands of well sorted coarse-sand sized, abraded shell and carbonate fragments.

Topography

Beaches are gently inclined to gently undulating plains <100 m wide, but up to several kilometres long. Relief and elevation is <6 m while slopes are usually <3%. Variations in beach characteristics occur both along and across the beachfront. Landform elements include berms, megacusps and cusps that are more often present near the southern end of the beach. Beaches are geomorphically active. Topography is subject to continuous alteration in response to changes in wave energy and tidal dynamics (Short, 1984). The boundary of the beach and foredune is often located close to the level reached by the last significant storm tide.

Foredunes are moderately inclined to steep rises <200 m wide, but up to several kilometres long. Relief ranges from 2–15 m. Slope may be up to 45% on blowout edges and seaward erosion scarps but is more commonly less than 10%.

Vegetation

Except for dead seaweed the beach has no vegetation. The original herbland/grassland of foredunes has been extensively disturbed.

On the foredune grasses and creepers such as hairy spinifex *Spinifex hirsutus*, knobby club-rush *Scirpus nodosus* and beach pennywort *Hydrocotyle bonariensis* are found. In areas that are sheltered from direct salt spray, an open or closed-scrub of species such as Sydney golden wattle *Acacia longifolia*, guinea flower *Hibbertia scandens* and coastal banksia *Banksia integrifolia* occurs. In disturbed areas, the noxious weed bitou bush *Chrysanthemoides monilifera* often dominates.

Many of the foredunes are currently being revegetated to stabilise the sand with community plantings of marram grass *Ammophila arenaria*, hairy spinifex *Spinifex hirsutus* and native dune shrubs.

Land use

Beaches are used for recreation. In some cases, foredunes have been developed for residential purposes at Narrabeen, Collaroy, Manly and Bondi. This includes high rise developments. Parks, playing fields and car parks are often found on foredunes, however some barrier beach foredunes such as those on Dee Why beach are virtually undisturbed.

Existing Erosion

Beaches are dynamic landscapes with common processes of severe wind and wave erosion. Wind erosion can be extreme on foredunes, especially where stabilising vegetation cover is absent or disturbed. Wind erosion on foredunes is characterised by the presence of blowouts.

Extreme wave erosion occurs during high seas. In many cases, the seaward slope of the foredune is significantly eroded resulting in erosion scarps. During storms, large volumes of sand can be removed from the beach and deposited in the near-shore zone. Conversely, sand from the near-shore zone may be deposited on the beachfront during low energy wave conditions.

It is believed that the amount of sand in the beach system is usually close to constant. Sand is moved back and forth from the near-shore zone to the beach and foredune in a cyclic process.

SOILS

Dominant Soil Materials

na1—Loose orange shelly beach sand. This is light orange coarse sand with apedal single-grained structure and sandy fabric. It generally occurs as both topsoil and subsoil.

This material consists of a mixture of quartz sand and tiny shell fragments. Common colours include dull yellow-orange (10YR 7/4), bright yellowish brown (10YR 6/6) and light grey (10YR 8/1). Fragments of shell, pumice and organic material are often found in thin diagonally bedded discontinuous laminations. Similar laminations of well graded marine sands also occur. The pH ranges between slightly acid (pH 6.5) and moderately alkaline (pH 9.0). Rounded pumice fragments are often present. Accumulations of driftwood, seaweed, rubbish and flotsam are found at storm surge and high tide levels.

na2—Loose yellowish-brown quartz sand. This is yellowish-brown quartz sand with apedal single-grained structure and loose sandy fabric.

It consists almost entirely of well sorted medium grained quartz sand. Colour ranges from dark brown (10YR 3/3) on the surface when organic matter is present to more common bright yellowish-brown (10YR 6/6). The pH ranges between neutral (pH 7.0) and moderately alkaline (pH 9.0). There are few roots and pumice and shell fragments are absent.

Associated Soil Materials

Compressed beach sand. This is a sand with apedal massive structure and sandy fabric. Colour is usually brownish yellow (10YR 6/6) or pale brown (10YR 7/4). It is hardsetting and requires a weak to moderate force to disrupt when dry. It is composed mostly of sand size shell fragments and cemented by dissolved lime from other shells. Fine diagonal laminations are characteristic of this material. The pH ranges between neutral (pH 7.0) and slightly alkaline (pH 8.5).

Occurrence and Relationships

Beaches. Generally, in excess of 200 cm of loose orange yellow shelly beach sand (**na1**) occurs over the entire beach [Calcareous Sands (Uc1.11, Uc1.12)]. Some beaches contain fewer shell fragments [Siliceous Sands (Uc1.21, Uc1.22)].

Foredunes. Over 200 cm of loose yellowish brown quartz sand (**na2**) occurs over all foredunes. In some sheltered situations where vegetation has not been disturbed, surface soil texture may approach that of loamy sand and have a slight accumulation of organic matter [Siliceous Sands (Uc1.21, Uc1.22), some Calcareous Sands (Uc1.11, Uc1.12)]. Compressed beach sands may occur at depths >200 cm on some foredunes.

LIMITATIONS TO DEVELOPMENT

Urban Capability

Not capable of urban development.

Rural Capability

Not capable of being cultivated or grazed.

Landscape Limitations

Wave erosion hazard
Wind erosion hazard
Steep slopes (localised)
Waterlogging (beach)
Non-cohesive material

Soil Limitations

na1 High permeability
Low available water capacity

Strongly saline
Strongly sodic
Very low fertility
Very strongly alkaline (localised)

na2 High permeability
Low available water capacity
Very low fertility
Very strongly alkaline (localised)

Fertility

The general fertility is very low. The soils are often strongly saline, with very low organic matter content, low waterholding capacity, very low CEC and very low nutrient status.

Erodibility

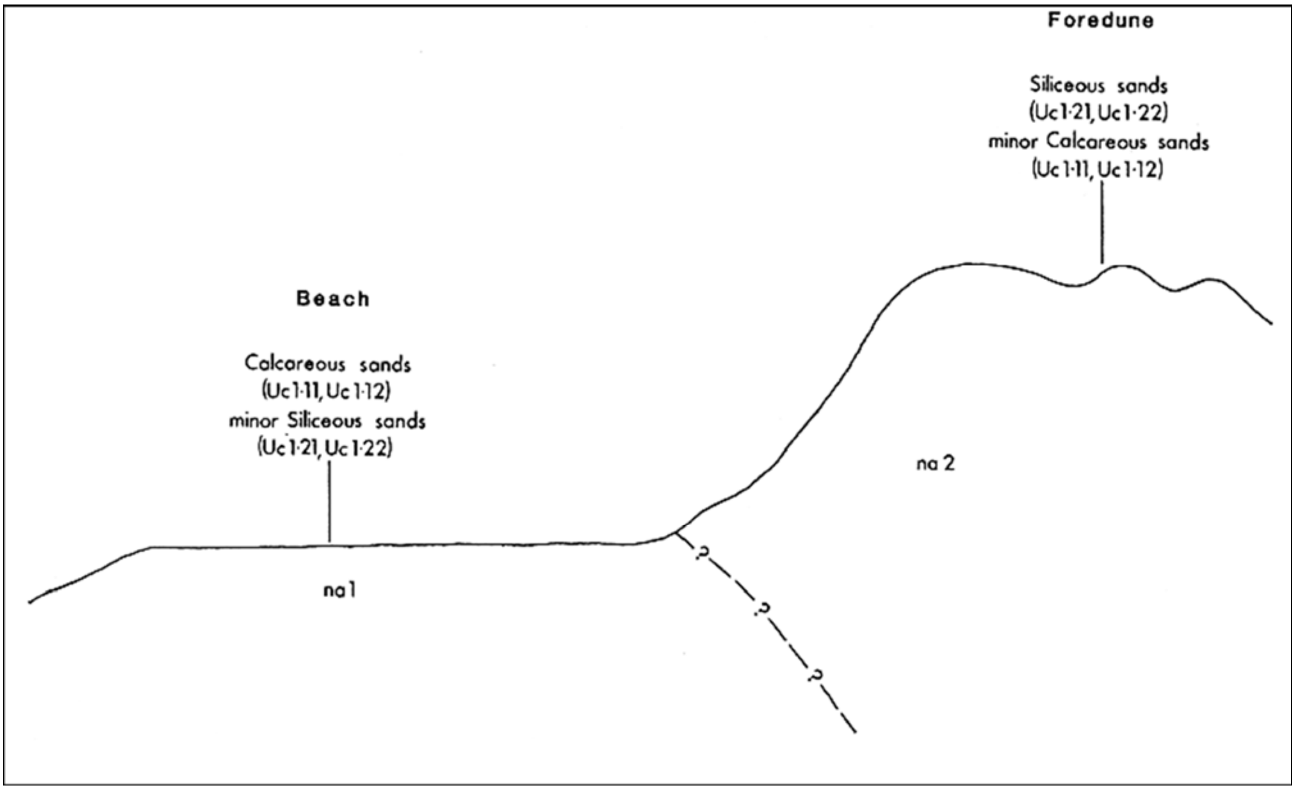
The soil materials have very low erodibility. They consist of well drained, loose, coarse sands.

Erosion Hazard

The erosion hazard for non-concentrated flows is low. Calculated loss for the first twelve months of urban development ranges up to 25 t/ha. Erosion hazard for concentrated flows, wind erosion and wave erosion is extreme.

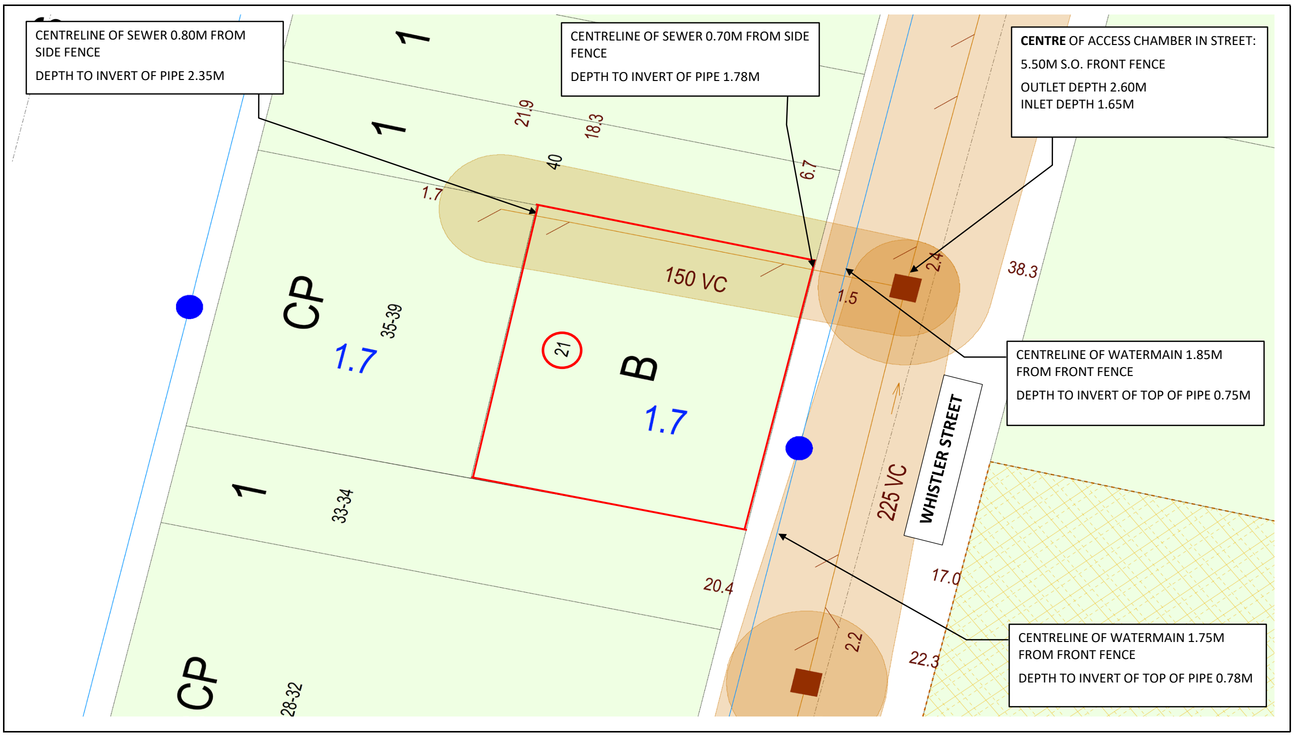
Surface Movement Potential

The sandy soil materials are stable.



Schematic cross-section of Narrabeen soil landscape illustrating the occurrence and relationship of the dominant soil materials.

APPENDIX J



SERVICE PROTECTION REPORT by OLSEN INFRASTRUCTURE PTY LTD

Legend: Approximate Borehole/DCP Testing Location

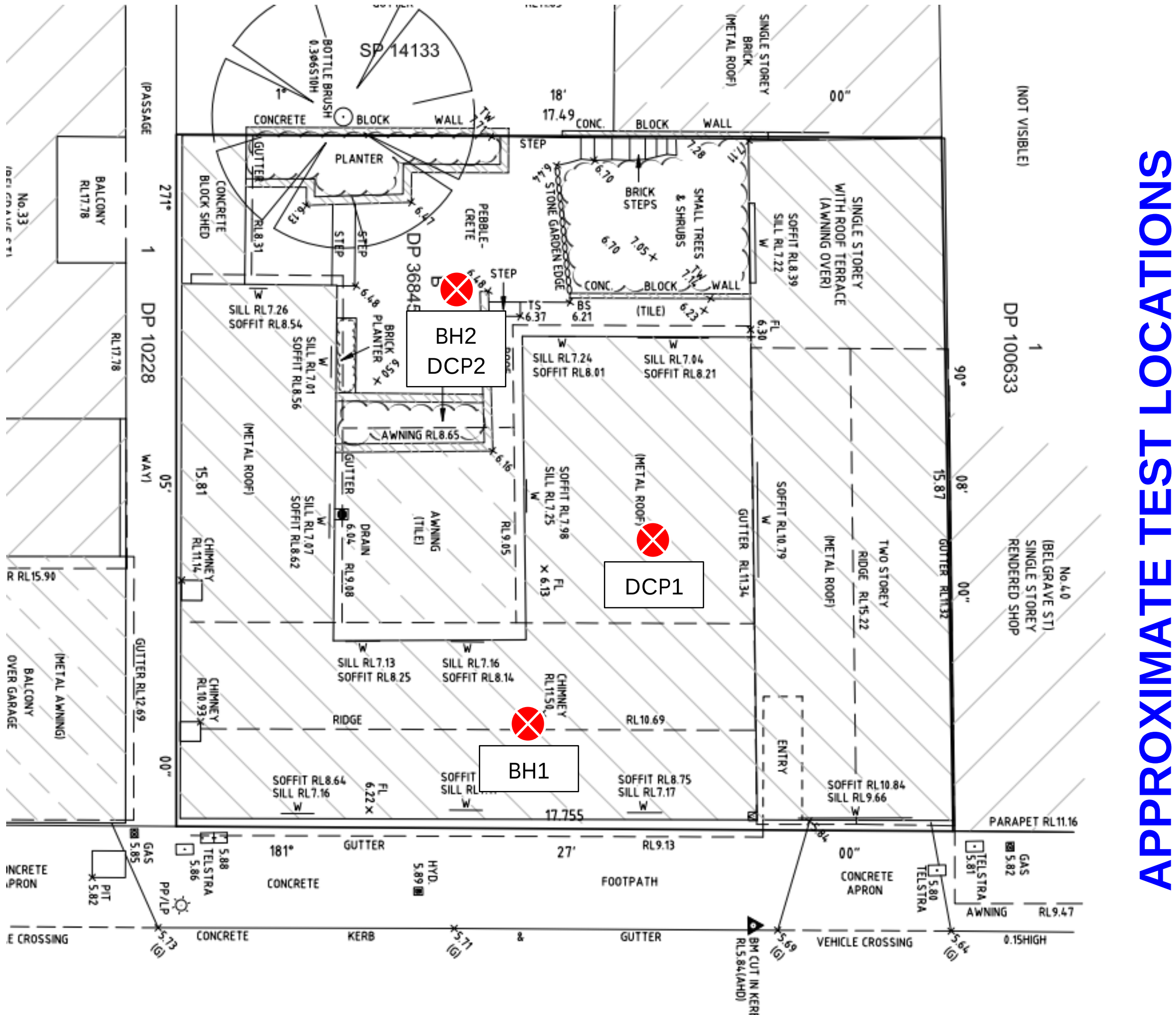


Figure 1 Site Plan	Geotechnical Investigation	Drawn: GN/GA
	J & CG Con NSW Pty Ltd	Date: 22/08/2022
Job No.: G21699-1-Rev A	21 Whistler Street Manly NSW 2095	Scale: NTS

Geotechnical Consultants Australia

Image source: Site survey plan prepared by Norton Survey Partners, titled "Plan Showing Selected Detail & Level Over No. 21 Whistler Street, Manly", referenced No. 53011, sheet 1 of 1 and dated 13th March 2018.

Table 2. Summary of Inferred Subsurface Conditions From DCP Testing

Unit	Unit Type	Estimated Consistency/ Strength[1]	Depth/Thickness of Unit (m bgl) DCP ID: DCP1	DCP2 (BH2)
1	Fill	N/A		0.0 – 0.5
2	Marine Sands[2]	Very loose	~~0.0 – 0.6?~~	0.5 – 2.5
		Loose	0.6 – 3.8	
		Medium Dense	3.8 – 4.3	2.5 – 4.0
		Dense	4.3 – 5.2	4.0 – 5.5
		Dense to Very Dense	5.2 – 5.3	5.5 – 5.8

[1]Estimated soil consistency/strength is based on DCP testing to the maximum practical terminated depths at the selected testing locations within the site. The potential for weak or softer layers throughout the unit should be considered, predominately below the practical DCP testing depths of approximately 5.3m to 5.8m bgl.
[2]Confirmation of the underlying soil composition and consistency/strength should be made by additional boreholes and CPT. Precaution should be made when considering these layers throughout the site.

Table 5. Preliminary Geotechnical Design Parameters

Material	Fill (Unit 1) 1	Marine Sands (Unit 2)[4]: Very Loose to Loose 2	Medium Dense 3	Dense to Very Dense 4
Unit Weight (kN/m³)[3]	16	16	17	18
Effective Cohesion **c' (kPa)**	0	0	0	0
Angle of Friction ϕ' (°)	26	28	30	34
Modulus of Elasticity E_{sh} (MPa)	3	8	15	20
Earth Pressure Coefficient At Rest Ko[1]	0.56	0.53	0.5	0.44
Earth Pressure Coefficient Active Ka[2]	0.39	0.36	0.33	0.28
Earth Pressure Coefficient Passive Kp[2]	2.56	2.77	3.0	3.54
Poisson Ratio v	0.4	0.35	0.35	0.35

[1]Earth pressure coefficient at rest (Ko) can be calculated using Jacky's equation.
[2]Earth pressure coefficient of active (Ka) and passive (Kp) can be calculated using Rankine's or Coulomb's equation.
[3]Above groundwater levels. GCA should be contacted for further advice on unit weight values of soils which are below the groundwater level.
[4]Subject to confirmation by a geotechnical engineer by additional borehole drilling and appropriate testing (CPT).

3.2 Groundwater

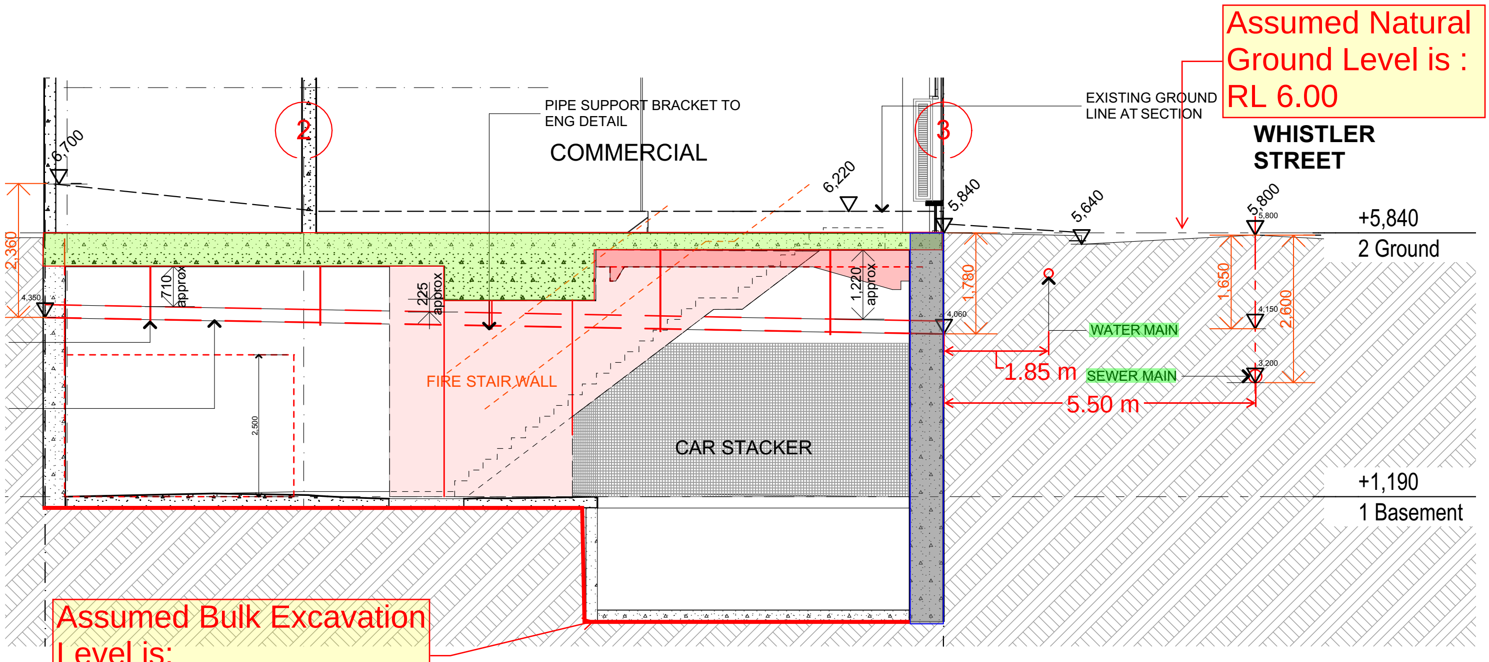
Groundwater was encountered drilling of boreholes BH1 and BH2 at depths of approximately 5.6m and 6.0m bgl, respectively. We note that soils were observed to be "moist to wet" from a depth of about 5.0m bgl in borehole BH2, which may indicate groundwater levels to be at shallower depths within the site.

Summary Subsurface Conditions From DCP Testing & Geotechnical Design Parameters

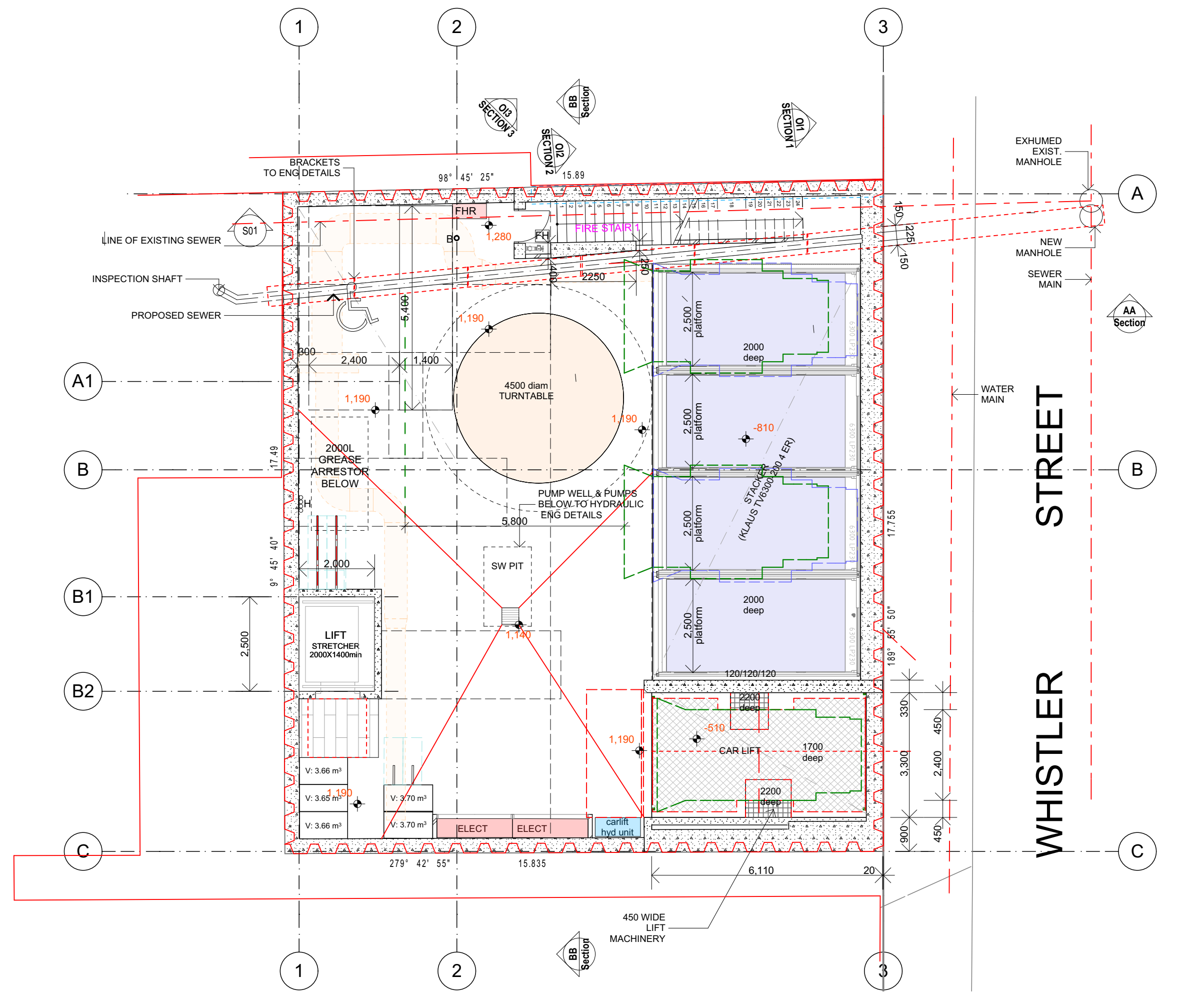
REFERENCE:

GCA report number: G21699-1-Rev A - 22nd August 2022
Table 2 - page 10 & Table 5 - page 18

Geotechnical Consultants Australia



SECTION AA



BASEMENT PLAN

ARCHITECTURAL SECTION AND PLAN

ARCHITECTURAL SECTION AND PLAN

REFERENCE:

ARCHITECT:

WOLSKI . COPPIN

A R C H I T E C T U R E

SUITE 3, LEVEL 1, 507 MILITARY ROAD MOSMAN NSW 2088
T: 9953 8477 E: info@wolskicoppin.com.au
DAVID WOLSKI NSW ARB No. 5297

PROJECT TITLE:

Multi Unit Residential Development

PROJECT ADDRESS:

21 Whistler St Manly

CLIENT:

URBAN PARTNERS

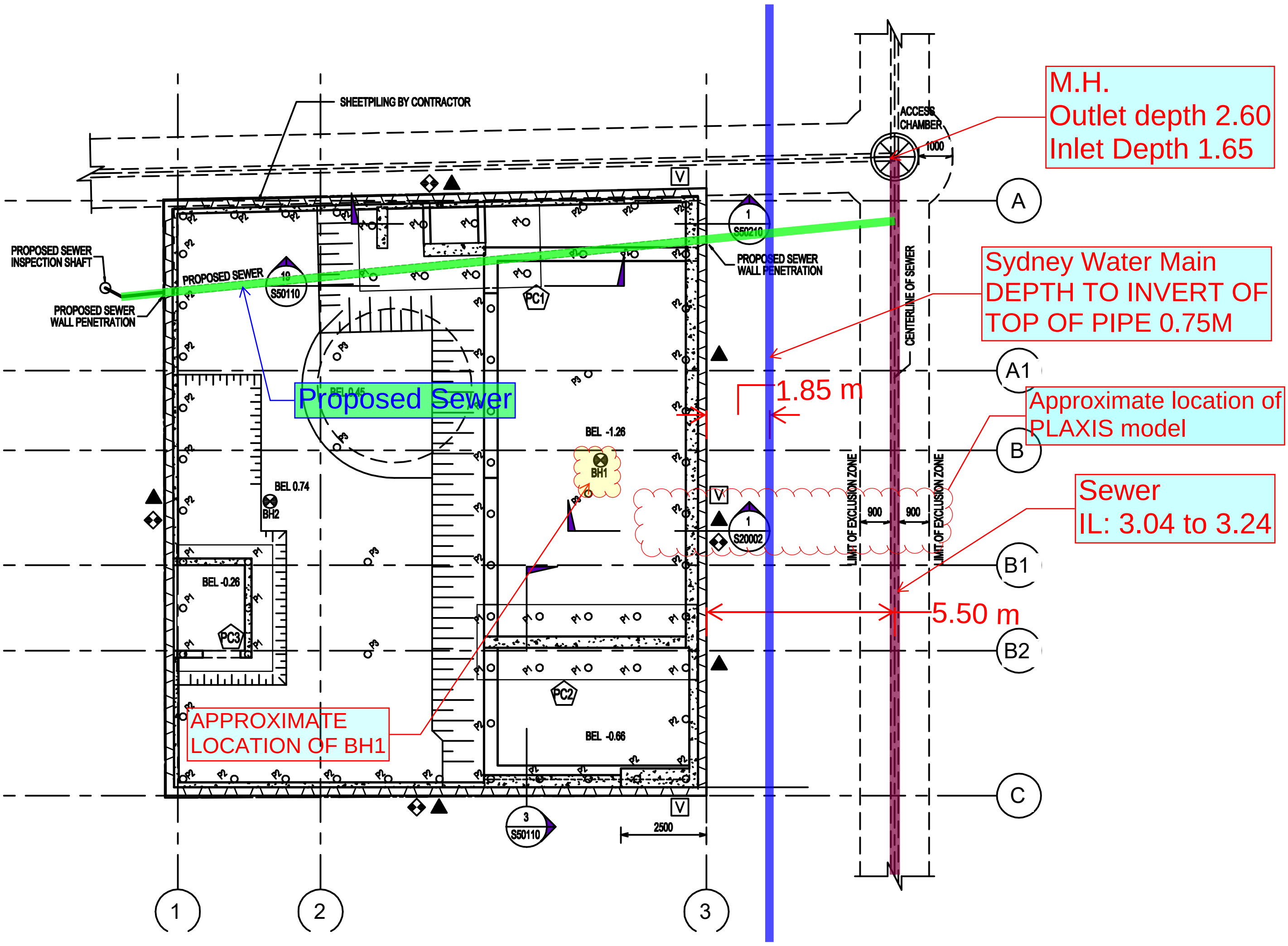
DRAWING TITLE:

SECTION AA

PROJECT No:	DRAWING No:
21806	CC 08
REVISION:	NORTH POINT:
B	
SCALE:	PLOT DATE:
1:100 @ A3	10/02/2023

FILE PATH:

BIM Server: BIMSRV01 - BIM Server 20/21806 21 Whistler Street Manly CC-CD DW Basement 090520

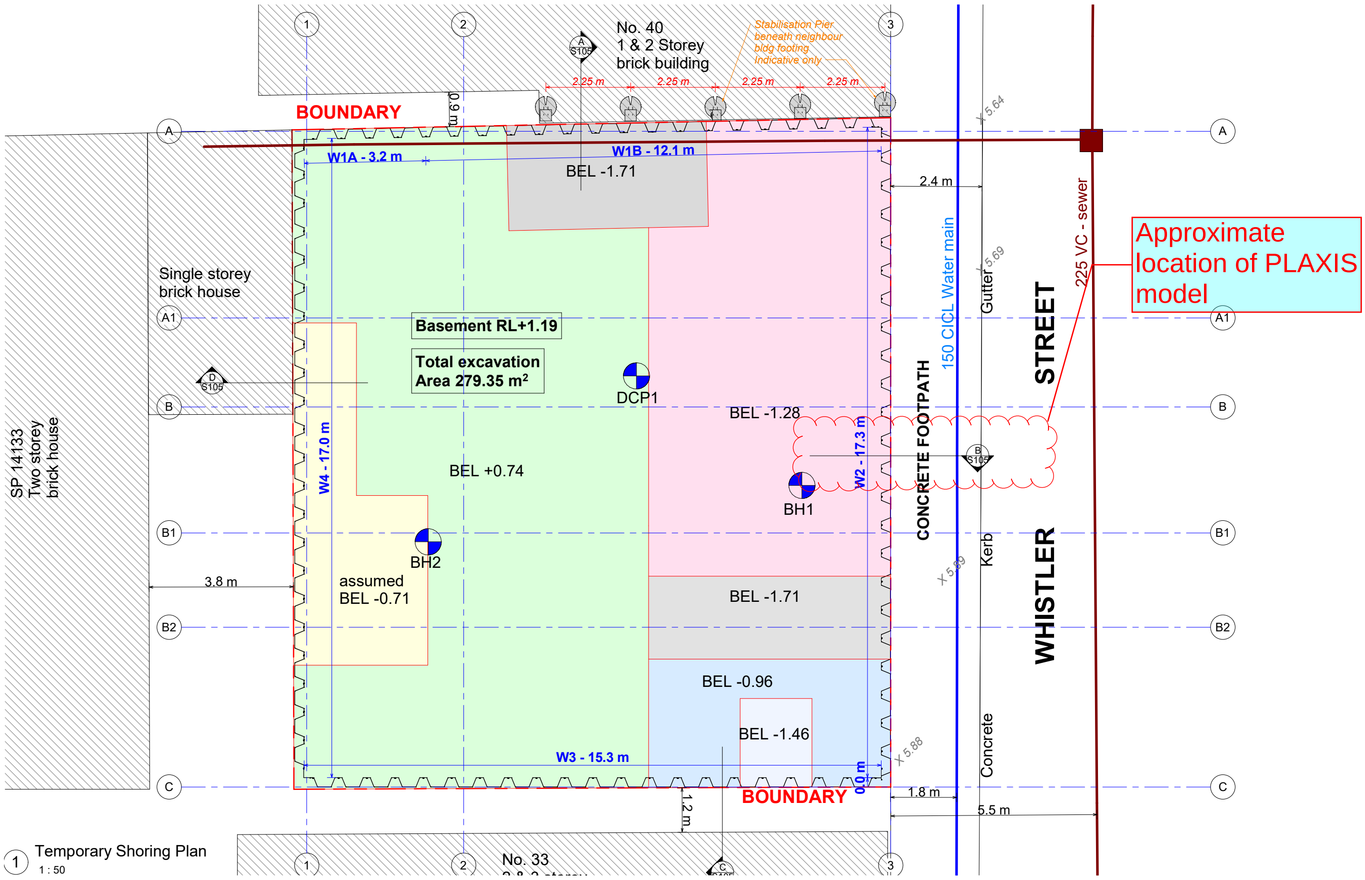


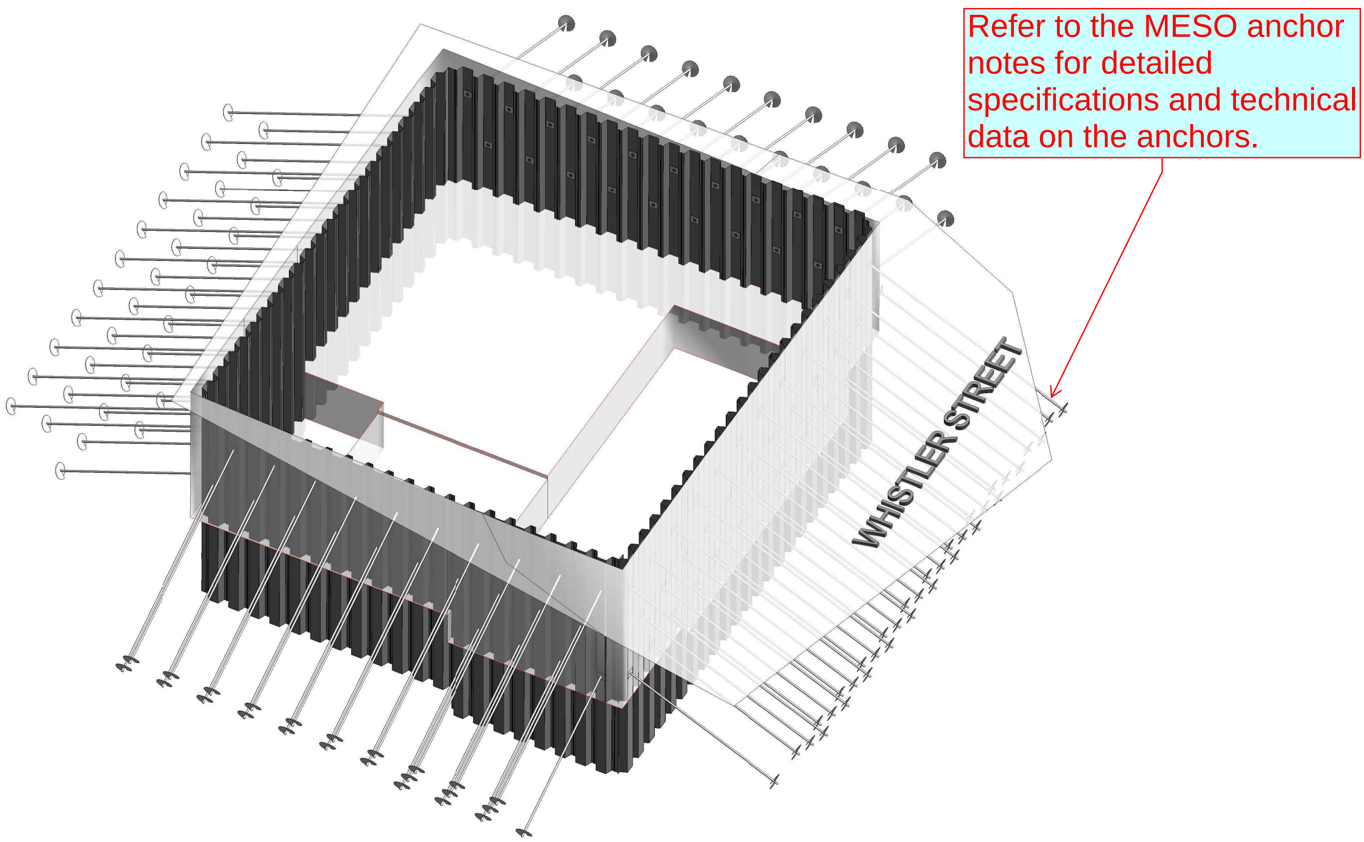
SHORING PLAN

PROJECT TITLE	MULTI UNIT RESIDENTIAL DEVELOPMENT 21 WHISTLER ST, MANLY
ARCHITECT	
CLIENT	URBAN PARTNERS
BUILDER	J + CG
	ZAIT Engineering Solutions Pty Ltd Suite 5, 5-7 Villiers Street, Parramatta, NSW 2150 Phone: 02 9630 3087 Email: info@zait.com.au ABN 40 608 862 899 ACN 608 862 899
DRAWING TITLE	SHORING AND FOOTING PLAN
DRAWING STATUS	FOR SYDNEY WATER APPROVAL

DESIGNED:	SN		
DRAWN:	AW	SCALE	SIZE
CHECKED:	DZ	As indicated	A1
JOB No.	21-2872	DRAWING No. S20000	REV. E

Structural Plan
REFERENCE:





TEMPORARY SHORING PLAN AND 3D VIEW

Temporary Shoring Plan

REFERENCE:

METAUS PTY LTD ACN: 644 319 595 18 Chemside St, Highgate Hill QLD 4101	MESO SOLUTIONS PTY LTD ABN: 28 611 157 478 53 General Holmes Drive NSW 2016 www.mesos.com.au 1300 10 MESO (6376)	Project 21 Whistler Street Manly	Job ID 21-140	Sheet No. S103
		Sheet Subject Temporary Shoring Plan	Designed by SG	Drawn by SG

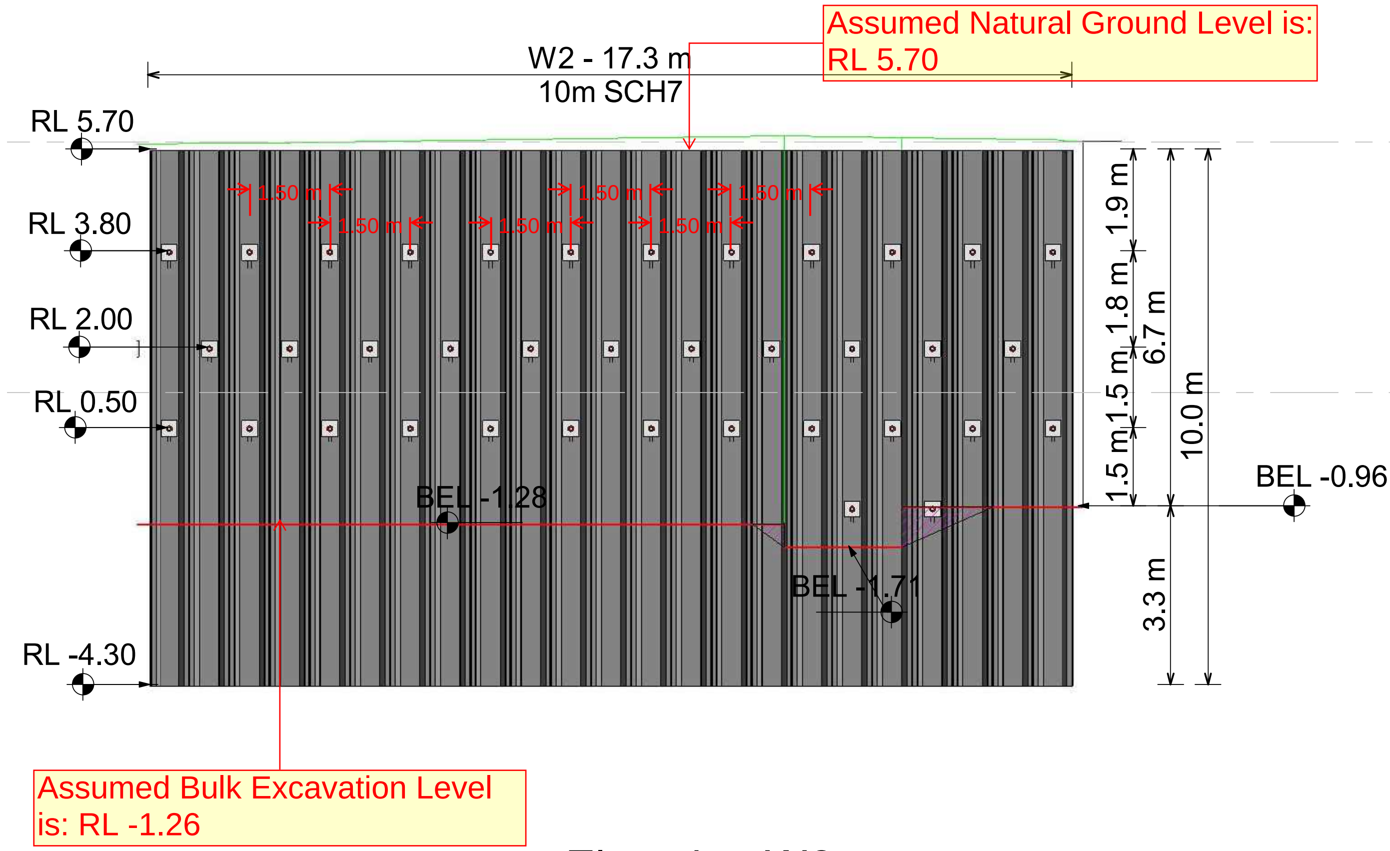
ANCHOR DATAILS

ANCHOR TABLE - BOQ BY BAR TYPE AND LENGTH		
Anchor Bar	Anchor Length	Count
R51B	3.00 m	5
R51B	4.00 m	10
R51B	4.50 m	10
R51B	5.00 m	36
R51B	5.50 m	10
R51B	6.00 m	33
R51B	7.00 m	32
Stabilisation Pier	13.00 m	6

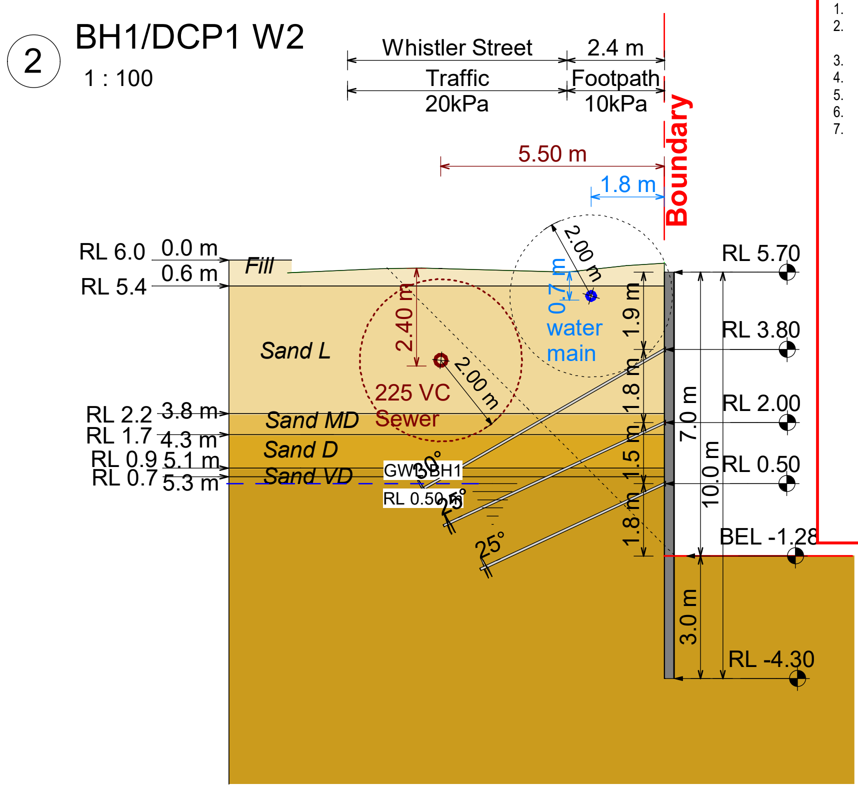
TECHNICAL DATA

	Nominal Thread Diameter	External Diameter	Average Internal Diameter	Cross Sectional Area	Ultimate Load	Yield Load	Weight	Thread Type	Elongation (A100mm)
	mm	mm	mm	mm^2	kN	kN	kg/m	L/R	≥%
TK-R51B	51	48.30	33	993	800	630	7.80	[illegible]	6 / 10

ANCHOR TABLE - BOQ BY WALL						
Wall ID	Anchor Row	Anchor Length	Anchor Angle	Lock Off	Proof Load	Yield Load
1	1	7.00 m	30.00°	120.00 kN	156.00 kN	630.00 kN
1	2	5.50 m	25.00°	150.00 kN	195.00 kN	630.00 kN
1	3	4.50 m	25.00°	180.00 kN	234.00 kN	630.00 kN
1	4	3.00 m	25.00°	210.00 kN	273.00 kN	630.00 kN
2	1	7.00 m	30.00°	120.00 kN	156.00 kN	630.00 kN
2	2	6.00 m	25.00°	150.00 kN	195.00 kN	630.00 kN
2	3	5.00 m	25.00°	170.00 kN	221.00 kN	630.00 kN
2	4	5.00 m	25.00°	170.00 kN	221.00 kN	630.00 kN
3	1	7.00 m	25.00°	120.00 kN	156.00 kN	630.00 kN
3	2	6.00 m	25.00°	150.00 kN	195.00 kN	630.00 kN
3	3	5.00 m	25.00°	170.00 kN	221.00 kN	630.00 kN
3	4	5.00 m	25.00°	170.00 kN	221.00 kN	630.00 kN
4	1	6.00 m	25.00°	120.00 kN	156.00 kN	630.00 kN
4	2	5.00 m	25.00°	150.00 kN	195.00 kN	630.00 kN
4	3	4.00 m	25.00°	170.00 kN	221.00 kN	630.00 kN



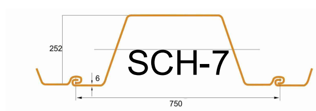
2 Elevation W2
1 : 100



B Section W2
1 : 100

SHEET PILE NOTES

1. The proposed sheet pile wall acts as a temporary earth-retaining wall.
2. Vibration monitoring is recommended to record the vibration level as and when required by the vibration consultant.
3. The sheet pile wall consists of SCH7-750 section.
4. All sheet piles shall be interlocking with each other.
5. The steel grade of the sheet pile is S355 according to DIN EN 10249.
6. The sheet pile shall installed in accordance to AS2159 - Piling.
7. The project architect and project structural engineer shall take into account the stipulated positional tolerance AS2159 - Piling and the estimated deflection of the sheet pile wall.



Weight /sq.m. (kg/m^2)	74.3
Weight /l.m. (kg/m)	55.7
Sectional Area (cm^2/m)	94.68
Section Modulus (cm^3/m)	750
Moment of Inertia (cm^4/m)	9732

SHEET PILE DETAILS

TEMPORARY SHORING WALL ELEVATION AND SECTION

Temporary Shoring Plan

REFERENCE:

METAUS
METAUS PTY LTD ACN: 644 319 595
18 Chemside St, Highgate Hill QLD 4101

MESO SOLUTIONS
MESO SOLUTIONS PTY LTD ABN: 28 611 157 478
53 General Holmes Drive NSW 2016
www.mesos.com.au 1300 10 MESO (6376)

Project	Job ID	Sheet No.
21 Whistler Street Manly	21-140	S103
Sheet Subject	**Designed by**	**Drawn by**
Temporary Shoring Plan	SG	SG

PROJECT TITLE
MULTI UNIT RESIDENTIAL DEVELOPMENT
21 WHISTLER ST, MANLY

ARCHITECT
WOLSKI COPPIN ARCHITECTURE

CLIENT
URBAN PARTNERS

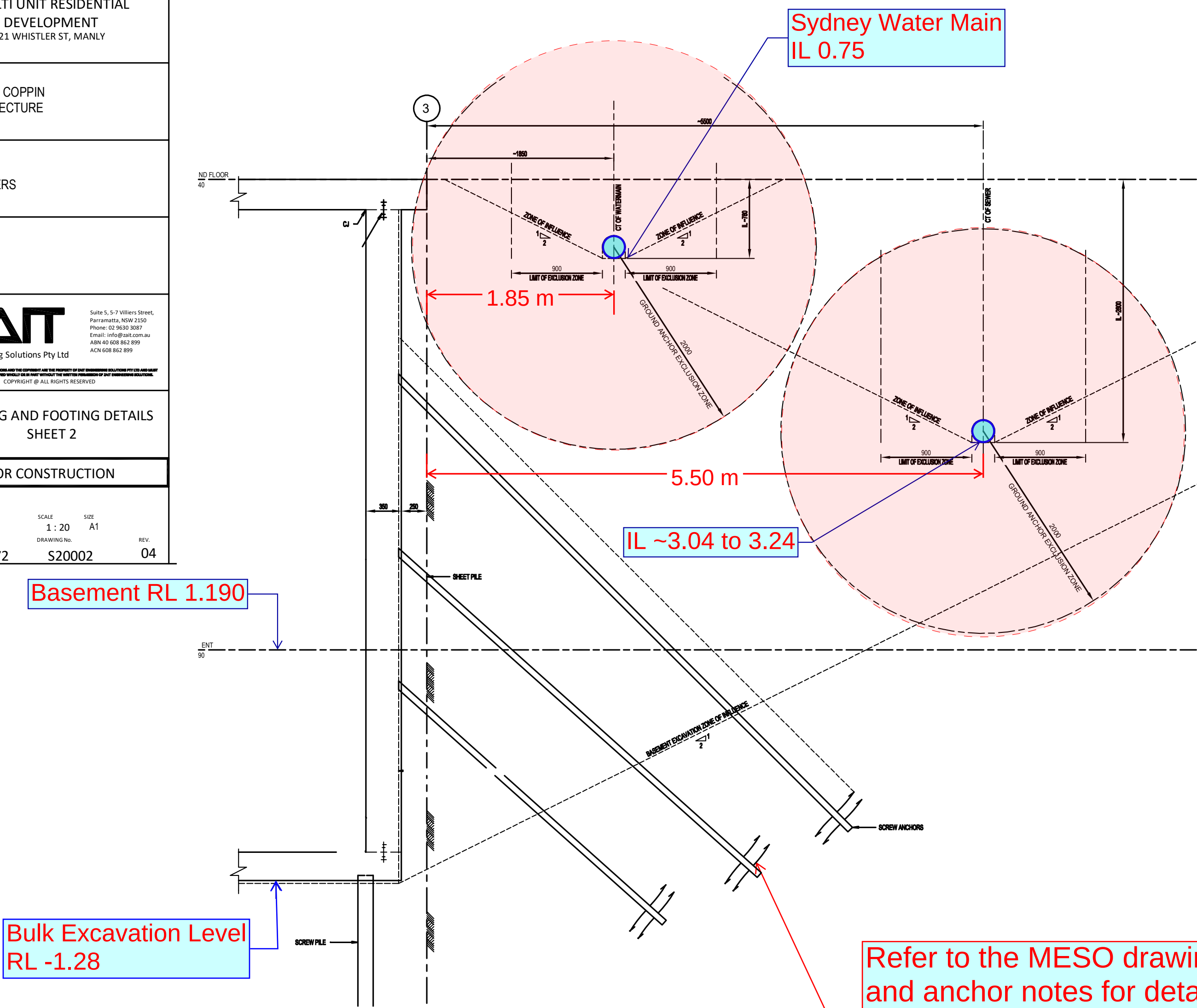
BUILDER
J + CG

ZAIT Engineering Solutions Pty Ltd
Suite 5, 5-7 Villiers Street, Parramatta, NSW 2150
Phone: 02 9630 3087
Email: info@zait.com.au
ABN 40 608 862 899
ACN 608 862 899

DRAWING TITLE
SHORING AND FOOTING DETAILS
SHEET 2

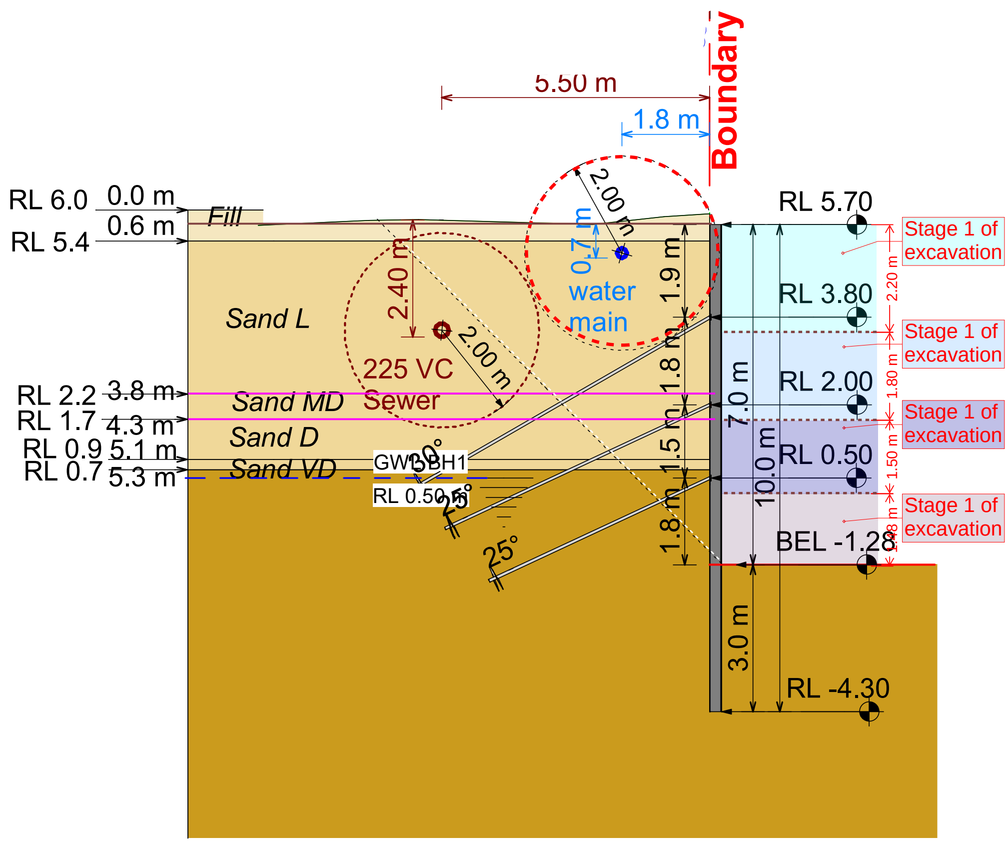
DRAWING STATUS
FOR CONSTRUCTION

DESIGNED: SN		
DRAWN: AW	SCALE 1 : 20	SIZE A1
CHECKED: BP		
JOB No. 21-2872	DRAWING No. S20002	REV. 04

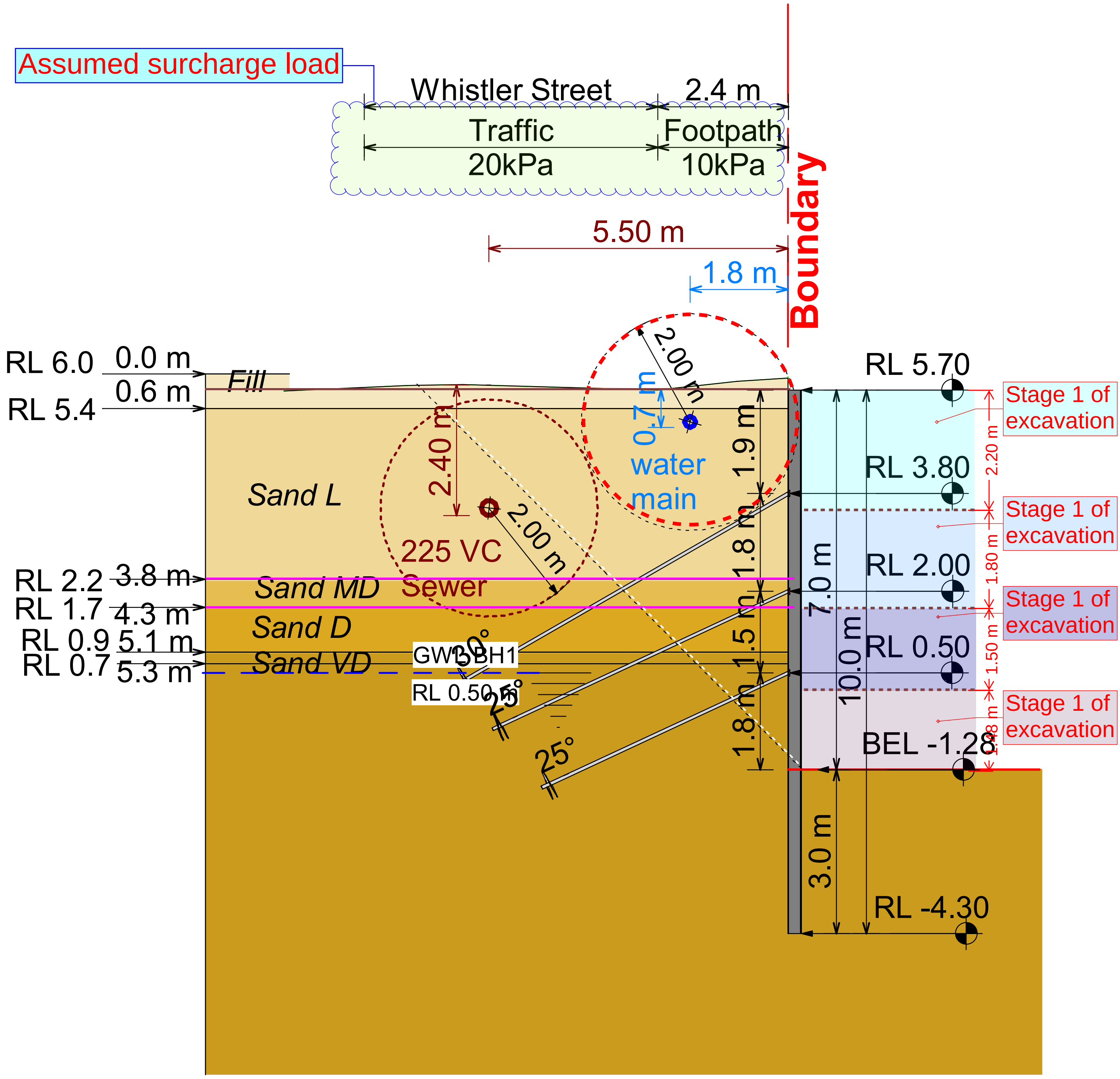


Refer to the MESO drawings and anchor notes for detailed specifications and technical data on the anchors.

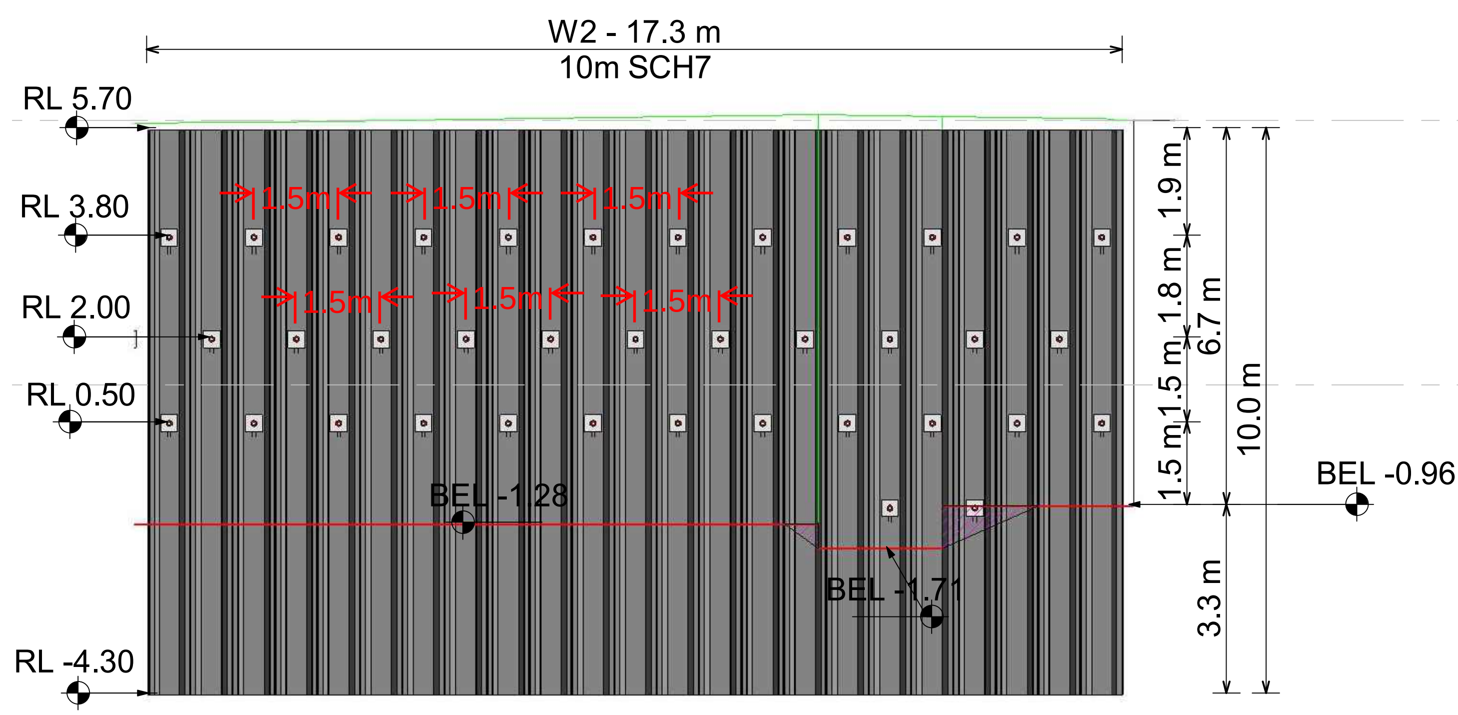
STRUCTURAL SECTION



EXCAVATION STAGING



Assumed surcharge load
Whistler Street
2.4 m
Traffic
20kPa
Footpath
10kPa
Boundary
5.50 m
1.8 m
2.00 m
0.7 m
water
main
RL 6.0
0.0 m
0.6 m
RL 5.4
Fill
Sand L
2.40 m
2.00 m
225 VC
Sewer
RL 2.2
3.8 m
RL 1.7
4.3 m
Sand MD
Sand D
RL 0.9
5.1 m
RL 0.7
5.3 m
Sand VD
GW1 BH1
RL 0.50 m
30°
25°
25°
1.9 m
1.8 m
1.5 m
1.8 m
7.0 m
10.0 m
3.0 m
RL 5.70
RL 3.80
RL 2.00
RL 0.50
BEL -1.28
RL -4.30
2.20 m
1.80 m
1.50 m
1.78 m
Stage 1 of
excavation
Stage 1 of
excavation
Stage 1 of
excavation
Stage 1 of
excavation



W2 - 17.3 m
10m SCH7
RL 5.70
RL 3.80
RL 2.00
RL 0.50
RL -4.30
1.5m
1.5m
1.5m
1.5m
1.5m
1.5m
BEL -1.28
BEL -1.71
BEL -0.96
1.9 m
1.8 m
1.5 m
1.5 m
6.7 m
10.0 m
3.3 m

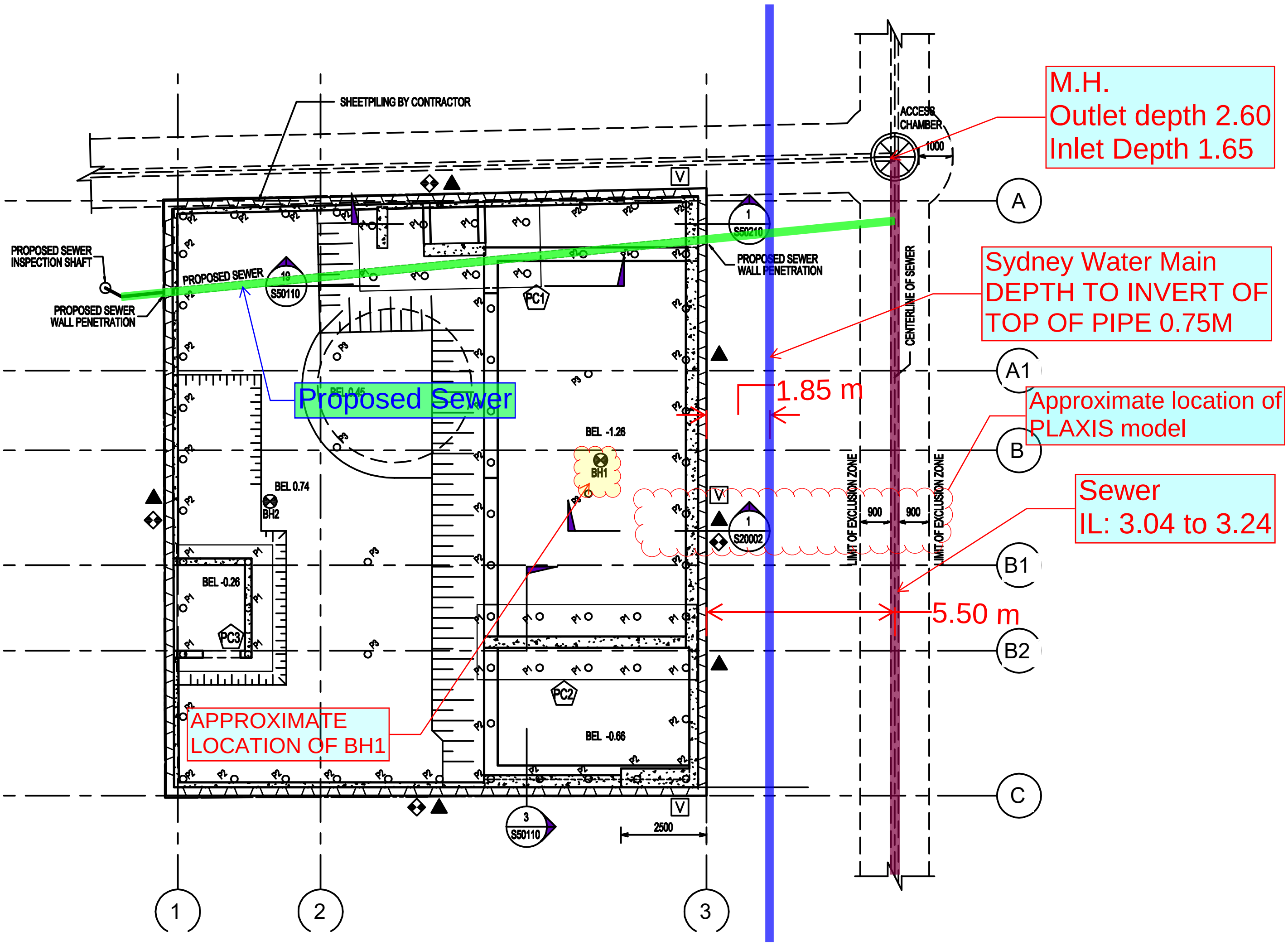


Figure1

Figure 1 presents approximate location of the PLAXIS models

Structural Plan

REFERENCE:

PROJECT TITLE
MULTI UNIT RESIDENTIAL DEVELOPMENT
21 WHISTLER ST, MANLY

ARCHITECT

CLIENT
URBAN PARTNERS

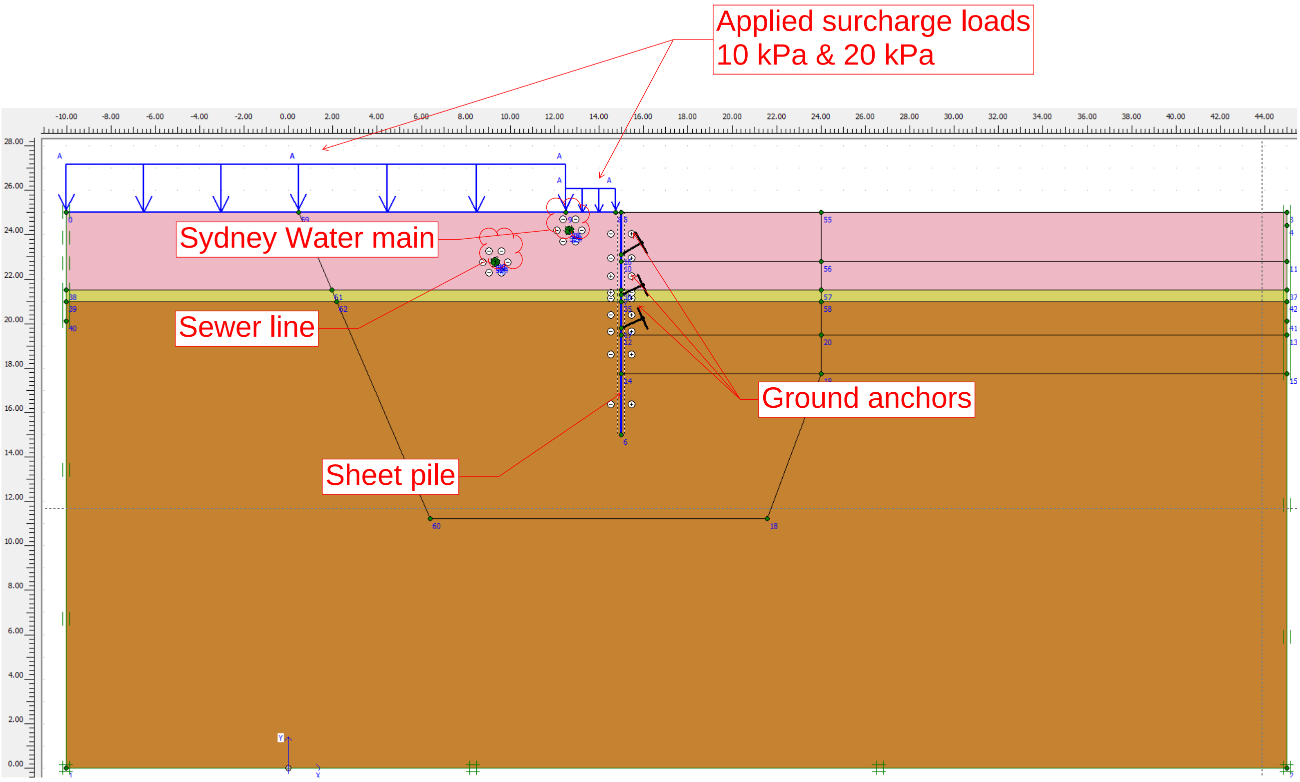
BUILDER
J + CG

ZAIT Engineering Solutions Pty Ltd
Suite 5, 5-7 Villiers Street, Parramatta, NSW 2150
Phone: 02 9630 3087
Email: info@zait.com.au
ABN 40 608 862 899
ACN 608 862 899

DRAWING TITLE
SHORING AND FOOTING PLAN

DRAWING STATUS
FOR SYDNEY WATER APPROVAL

DESIGNED:	SN	SCALE	SIZE
DRAWN:	AW	As indicated	A1
CHECKED:	DZ		
JOB No.	21-2872	DRAWING No. S20000	REV. E



Figure

Figure 2 presents the geotechnical model used in the numerical modeling. The location of proposed sewer line, watermain, sheet pile and structural elements are shown on the figure.

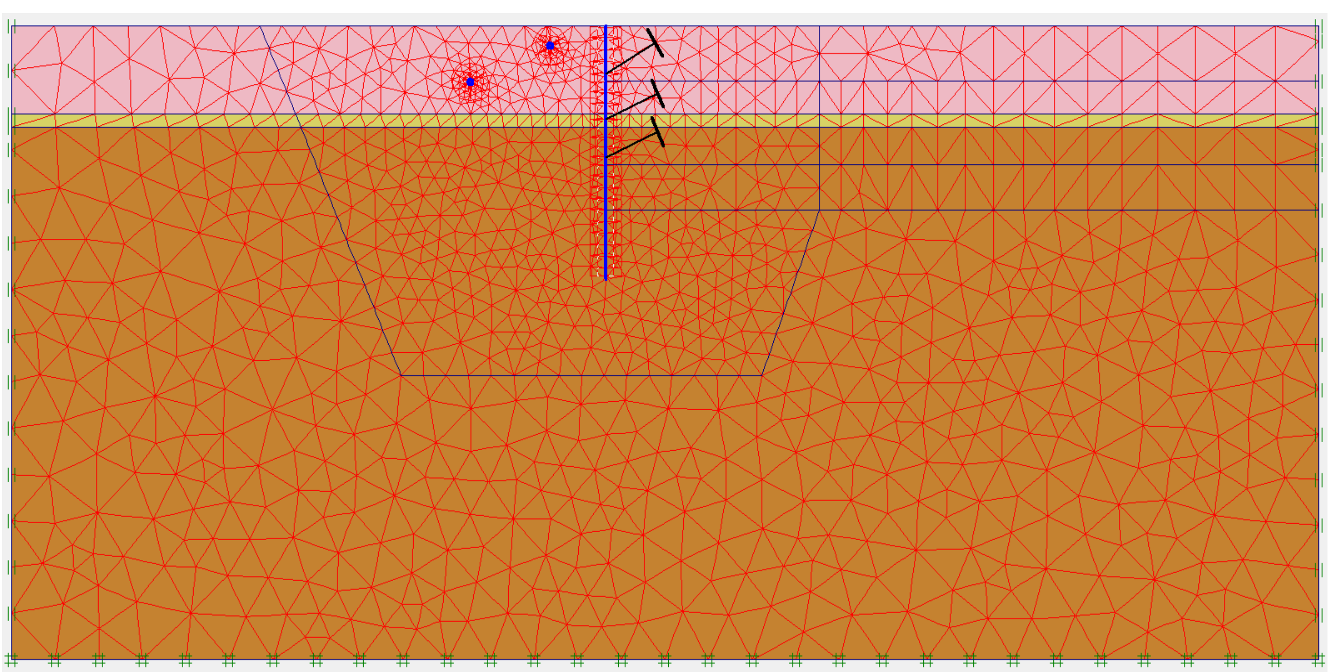


Figure 3

Figure 3 presents the geotechnical model used in the numerical modelling. The location of the pipes, structural elements, generated mesh and subsurface conditions are shown on the figure.

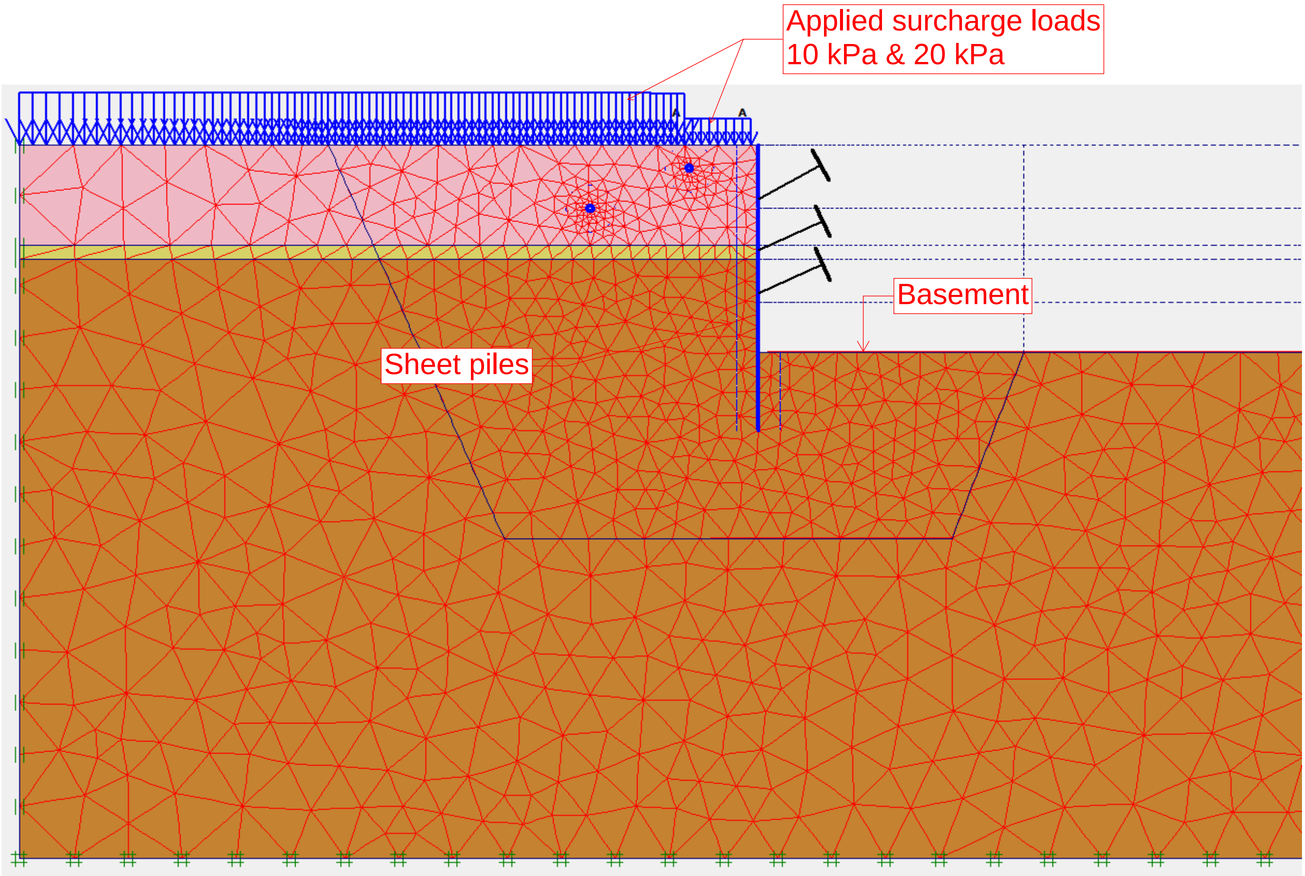


Figure 4

Figure 4 presents the geotechnical model used in the short-term final stage numerical modeling. The location of the pipe, wetermain, structural elements, generated mesh and subsurface conditions are shown on the figure.

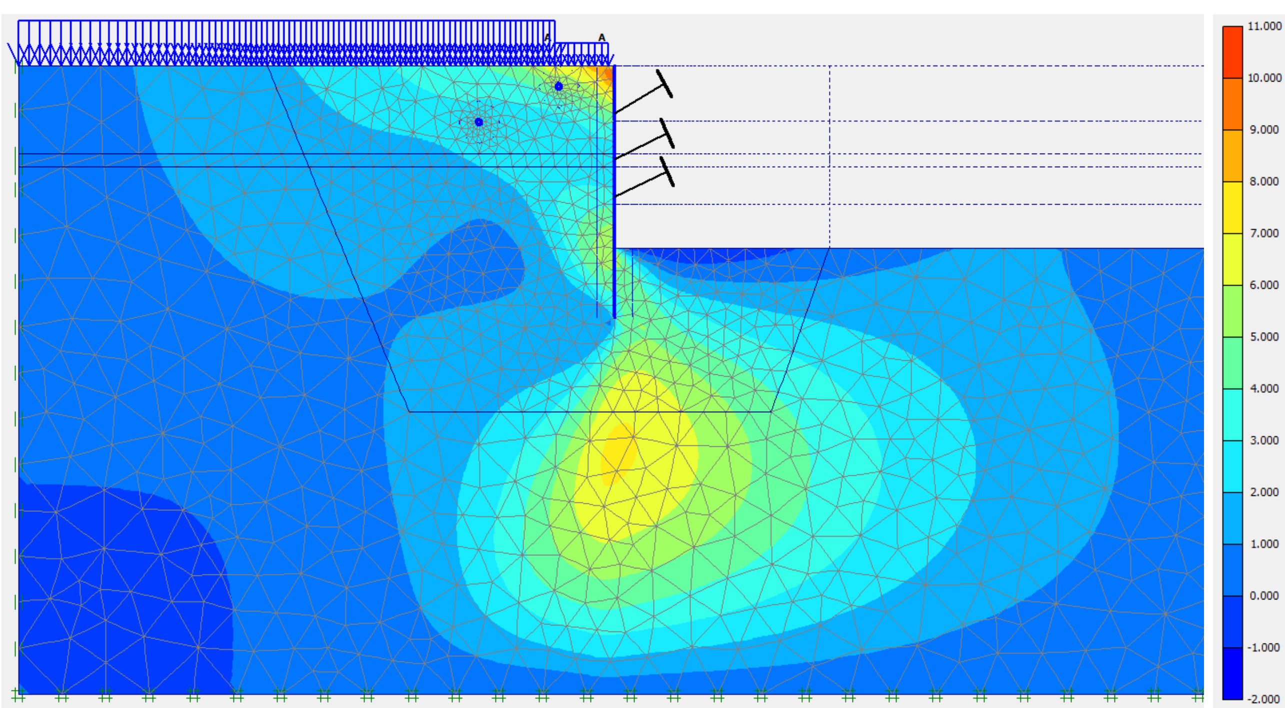


Figure 5

Figure 5 shows the deformed model and ground horizontal movement due to the proposed work. Assumed depth of the excavation is 7.26m.
Where controlling point is on top of the Water main a maximum of 6mm movement is estimated.
Where controlling point is on top of the Sewer a maximum of 5mm movement is estimated.

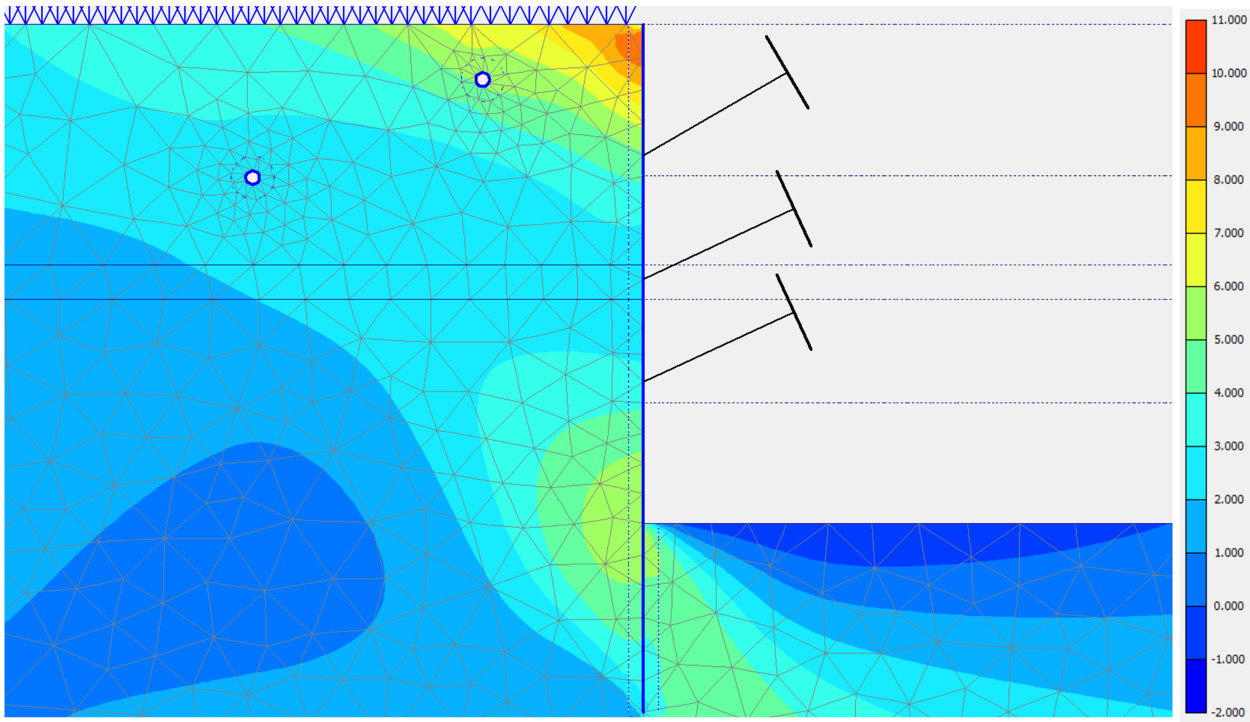


Figure 5

Figure 5 shows the deformed model and ground horizontal movement due to the proposed work. Assumed depth of the excavation is 7.26m.
Where controlling point is on top of the Water main a maximum of 6mm movement is estimated.
Where controlling point is on top of the Sewer a maximum of 5mm movement is estimated.

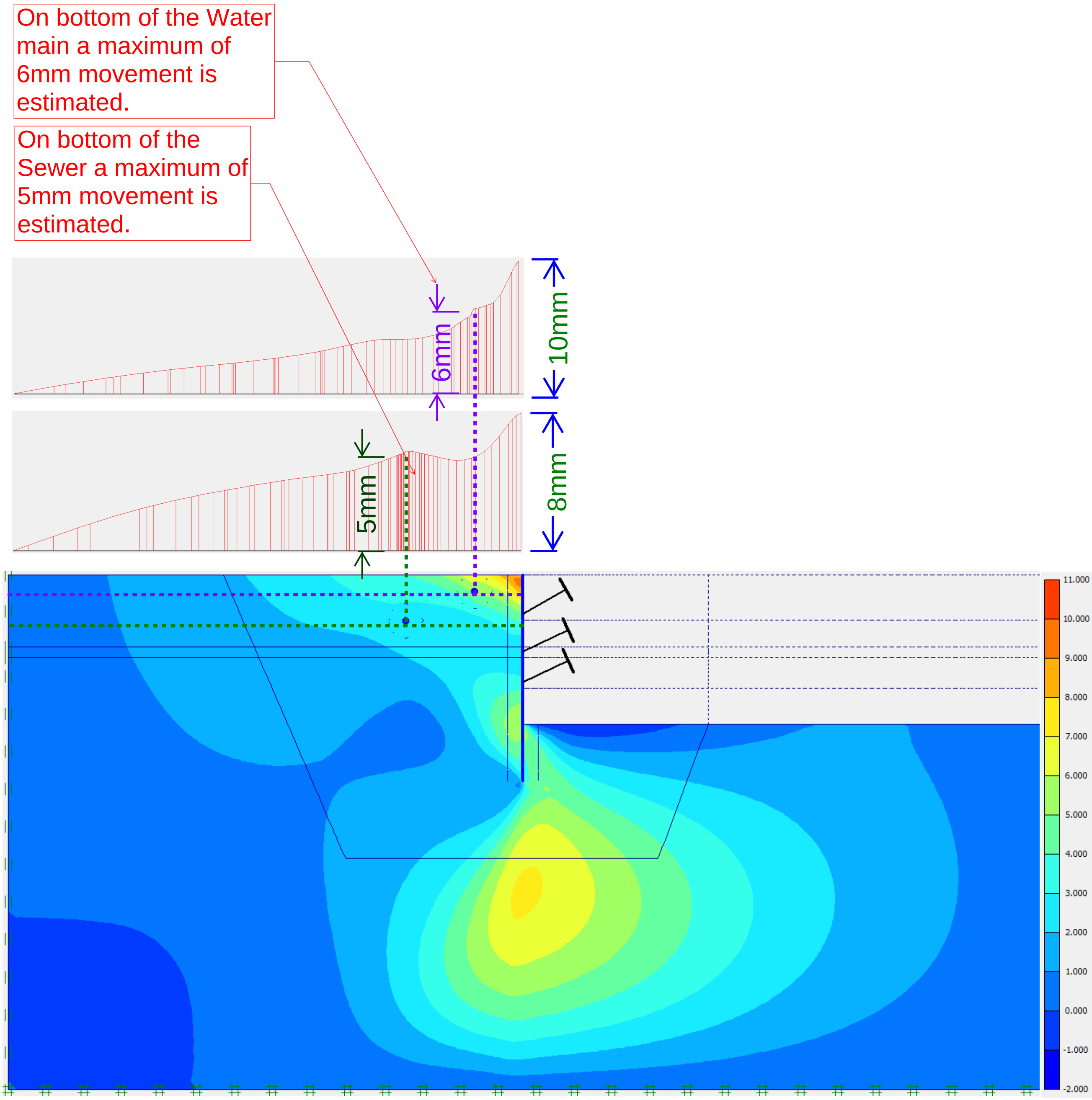


Figure 6

Figure 5 shows the deformed model and ground horizontal movement due to the proposed work.
Assumed depth of the excavation is 7.26m.
Where controlling point is on top of the Water main a maximum of 6mm movement is estimated.
Where controlling point is on top of the Sewer a maximum of 5mm movement is estimated.

Figure 7

Figure 7 shows Axial forces, Shear forces and Bending moment diagram along the pile in the short term.

195 kN/m Axial load, 72.56 kN/m shear load and 40.87 kN.m /m bending moment is estimated.

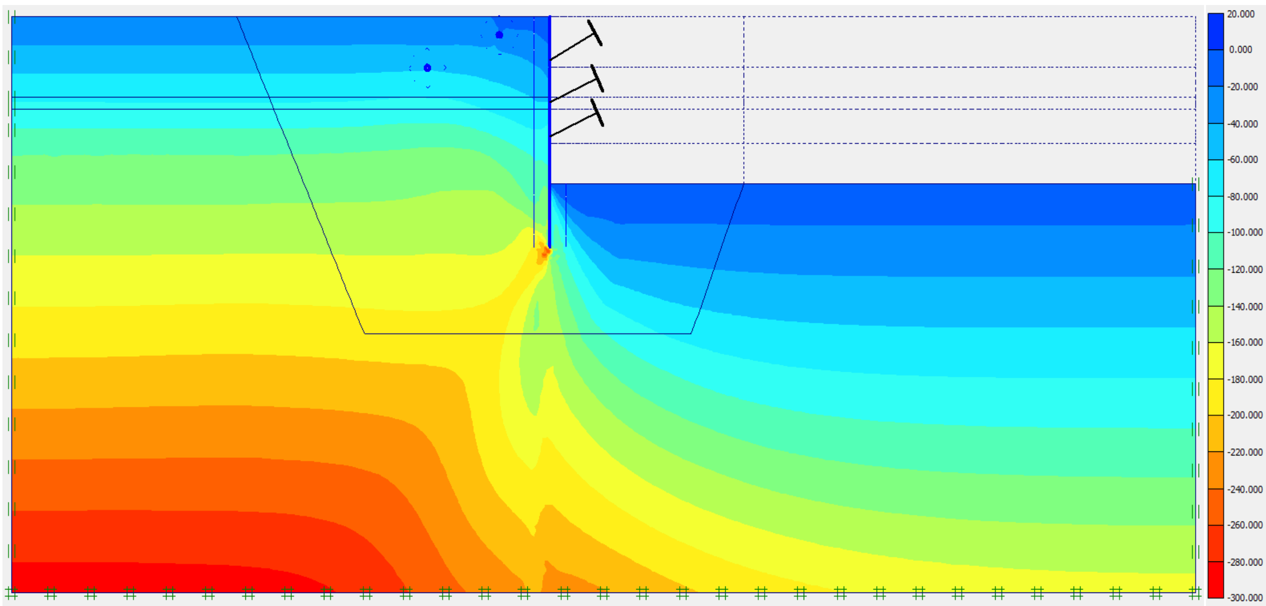


Figure 8

Figure 8 shows Cartesian effective stresses Y-Y.
Where controlling point is closer to the Water main a maximum of 30kN/m^2 is estimated.
Where controlling point is closer to the Sewer a maximum of 45kN/m2 is estimated.