Aargus

Environmental - Remediation - Engineering - Laboratories - Drilling

GEOTECHNICAL INVESTIGATION REPORT

Timber Yard (Site 1)
Farr St, Mitchell St &Victoria Rd
Marrickville NSW

Prepared for

E & D Danias Pty Ltd

Report No. GS5610/1A

20th December 2013

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DOCUMENT HISTORY

Revision No.	Issue Date	Description
0	20/12/2013	Initial Issue

Issued By:

Mark Kiryakos

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1. Australian Standard – AS 1170.4-2007 Structural design actions - Earthquake actions in Australia.
2. Australian Standard – AS1726-1993 Geotechnical Site Investigation.
3. Australian Standard – AS 2159-2009 Piling - Design and installation.
4. Australian Standard – AS 2870-2011 Residential slabs and footings.
5. Australian Standard – AS3798-2007 Guidelines on Earthworks for Commercial and Residential Developments.
6. Department of Infrastructure, Planning and Natural Resources, "Salinity Potential in Western Sydney 2002", March 2003.
7. Pells, P.J.N, Mostyn, G. & Walker B.F., "Foundations on Sandstone and Shale in the Sydney Region", Australian Geomechanics Journal, 1998.

1. INTRODUCTION

Aargus Pty Ltd (Aargus) has been commissioned by E & D Danias Pty Ltd to carry out a geotechnical site investigation at the existing Timber Yard (Site 1) located in area bounded by Farr Street, Mitchell Street and Victoria Road, Marrickville, NSW 2204. The site investigation was carried out on the 23rd and 24th of September 2013 and was followed by laboratory testing, geotechnical interpretation, assessment and preparation of a geotechnical report.

The purpose of the investigation was to assess the ground conditions and general geotechnical design requirements of the site. The investigation included assessment of the site existing geotechnical conditions and providing recommendations for design and construction of future development at the site.

This report presents results of the geotechnical site investigation, laboratory testing, interpretation and assessment of the site existing geotechnical conditions, as a basis to provide general recommendations for design and construction of ground structures at the site. To assist in reading the report, reference should be made to the "Important Information About Your Geotechnical Report" attached as Appendix A.

2. AVAILABLE INFORMATION

At the time of preparation of this report, the following information was made available to Aargus by E & D Danias Pty Ltd:

- Surveyor's Report for the Storage Yard and Workshops at No. 191-197 Victoria Road, Marrickville prepared by S.J. Dixon & Associates Pty Ltd, referenced 30570 and dated 4th April 1997;
- Survey Report for a number of land parcels at Nos. 167-183 Victoria Road, Marrickville, prepared by T.J Gilbert & Associates – Land and Engineering Surveyors, referenced 4233 and dated 9th November 2004;
- Surveyor's Report for a number of properties and land parcels including Nos. 9-11 Farr Street, a 0.915m wide passage between Nos. 9 and 11 Farr Street, Nos. 13, 15, 17, 19, 21, 23, 25, 27, 29, 31 Farr Street, Nos. 6 and 8 Mitchell Street, Lot A in D.P. 166330, and the properties at Nos. 167, 169 and 173 Victoria Road,

Marrickville prepared by S.J. Dixon & Associates Pty Ltd, referenced 34723 and dated 29th June 1999;

- Survey Report for the property at No. 7 Farr Street, Marrickville prepared by W. Buxton Pty. Limited – Land and Engineering Surveyors, referenced 98234 and dated 18th September 1998;
- Survey Report for the property at No. 6-8 Mitchell Street, Marrickville prepared by T.J Gilbert & Associates – Land and Engineering Surveyors, referenced 2850 and dated 13th February 2004; and
- Survey Report for the property at No. 4 Mitchell Street, Marrickville prepared by John R. Holt Surveyors Pty. Ltd, referenced JH.21-206/4 and dated 22nd March 2007.

An Environmental Detailed Site Investigation (DSI) was carried out at the site by Aargus on 25th and 26th September 2013. The results are documented in a report referenced ES5610/2.

Soil sampling for Acid Sulphate Soils (ASS) Assessment at this site was carried out as part of this geotechnical investigation by Aargus. The results of the ASS assessment including laboratory testing are documented in a report referenced ES5610/3.

3. SCOPE OF WORK

In accordance with the brief, fieldwork for the geotechnical site investigation was carried out by an experienced Geotechnical Engineer from Aargus, following in general the guidelines provided in Australian Standard AS1726-1993 (Reference 2) and comprised the following:

- Collection and review of Dial-Before-You-Dig (DBYD) plans;
- A site walk-over inspection in order to determine the overall surface conditions and to identify any relevant site features;
- Service locating carried out by a specialist contractor using electromagnetic detection equipment to ensure that the investigation area is free from underground services;
- Machine drilling of four boreholes identified as BH1 to BH4 inclusive using solid flight auger techniques with V-bit and tungsten carbide (TC) bit. Drilling was

carried out using a truck mounted drilling rig owned and operated by Aargus. All boreholes were drilled to V bit refusal at depths varying from approximately 3.0m to 8.4m below ground level (bgl) and were terminated at TC bit refusal depths varying from approximately 3.2m to 9.4m bgl;

- Standard Penetration Tests (SPT) conducted within the boreholes to assess the in-situ strength of subsurface soil layers;
- Collection of soil samples during drilling; and
- Reinstatement of the boreholes with soil cuttings.

The approximate locations of the four boreholes drilled during the site investigation are shown on Figure 1, “Site Plan” attached as Appendix B.

Selected soil samples collected during the site investigation were tested by Eurofins laboratory for determination of aggressivity of the soils underlying the site to concrete and steel foundation elements.

Following completion of the site investigation and laboratory testing, Aargus carried out geotechnical interpretation of the results and assessment of the main potential geotechnical issues that may affect future development or may result from the future development. A geotechnical report was prepared to summarise the results of the geotechnical site investigation, interpretation and assessment.

The information provided in this report includes:

- Method of investigation;
- Site description, including surface conditions;
- Site plan indicating borehole locations and showing existing relevant site features;
- Subsurface conditions together with material characterisation;
- Borehole logs;
- Results of in-situ and laboratory tests;
- Assessment of potential geotechnical issues that may be associated with future development and potential effects on the surrounding buildings and infrastructure; and
- Site specific recommendations for design and construction of the generic development.

4. SITE DESCRIPTION

The site is located within the Marrickville Council area, at approximately six kilometres to the south-west of Sydney Central Business District.

The site is bounded by the following properties and infrastructure:

- Victoria Road road reserve and carriageway to the south-east;
- Farr Street road reserve and carriageway to the north-west;
- Mitchell Street carriageway and road reserve and the properties at No. 165 Victoria Road, No. 2 and nos. 10-14 Mitchell Street and No. 35 Farr Street to the north-east. These properties are occupied by one to two storey brick warehouse type buildings; and
- The property at nos. 1-5 Farr Street and No. 185 Victoria Road, which is occupied by a brick warehouse type building with metal roof; and the properties at No. 199 Victoria Road, nos. 109 and 111 Sydenham Road to the south, which are occupied by one to two storey buildings.

The provided survey reports do not provide ground surface elevations, however, our observation indicates the ground surface within the site appears to slightly slope towards the south.

The site is an irregular shaped land with an approximate area of 1.6 hectares, consisting of amalgamation of a number of adjoining properties being Danias Timber Yard, nos. 167-169, 171-177, 183 & 191 Victoria Road, nos. 4-8 Mitchell Street, No. 7, nos. 9-11, 13, 15-31 & 33 Farr Street, Marrickville.

The site was occupied by several buildings, consisting of brick and warehouse type buildings with the remainder of the site being covered with concrete hardstand areas.

A number of mature trees are present in the vicinity of the north-eastern, north-western and southern boundaries. Several cracks and signs of deterioration were observed on the concrete pavement within the site.

Selected site photographs recorded during the site investigation are provided in Figure 2, attached as Appendix C.

5. PROPOSED DEVELOPMENT

No drawings for any proposed future development for the site were available during the preparation of this report. However, email correspondence from Design Collaborative, the project architects, on 14th August 2013 indicates the development within this site may consist of construction of up to ten storey building with up to three basement levels for underground parking.

6. LOCAL GEOLOGY

Reference to the Sydney 1:100,000 Geological Series Sheet 9130 Edition 1, dated 1983, by the Geological Survey of New South Wales, Department of Mineral Resources, indicates the following:

- The northern portion of the site is located within an area underlain by Triassic Age Ashfield Shale of the Wianamatta Group. The Ashfield Shale is described as “black to dark grey shale and laminate” and denoted as (Rwa); and
- The southern portion of the site is located within an area underlain by alluvial deposits consisting of “peat, sandy peat and mud” and denoted as (Qhs).

In addition, the site is at approximately 150m to the north-east of the geological boundary of the Ashfield Shale and Alluvial Deposits with the Hawkesbury Sandstone, which is denotes as (Rh).

7. INVESTIGATION RESULTS

7.1 Surface Conditions

The majority of the site apart from the footprints of the existing buildings was covered with approximately 100mm thick concrete pavement.

7.2 Subsurface Conditions

The subsurface conditions encountered within the boreholes are detailed on the attached Engineering Borehole Logs presented in Appendix D.

Subsurface conditions encountered during drilling at the borehole locations consisted of the following:

- Fill consisting of dark grey/grey brown, low to medium plasticity, soft to firm and moist silty clay; overlying
- Residual soils, consisting of grey with reddish/orange brown mottling, medium to high plasticity, firm to stiff and moist silty clay to depths varying from approximately 1.0m to 2.0m bgl, becoming stiff to hard silty clay/clayey sand at depth; overlying
- Class V sandstone, grey and brown/dark brown and red mottling iron-stained, extremely weathered, extremely low to very low strength, with some sandy clay bands. The top of the bedrock within the northern portion of the site is inferred to be shallower than the bedrock within the southern portion. Horizons of stronger rock such as Class IV sandstone, which typically underlies Class V sandstone, may be present at this site.

Classification of the rock was carried out in accordance with the guidelines provided by Pells et al (Reference 7).

Table 1 presents a summary of the subsurface conditions encountered in the boreholes during the site investigation.

Table 1: Summary of Subsurface Conditions

Unit	Depth to Top of Layer (m bgl)	Thickness (m)	SPT N Values (blows/300mm)
Fill	0.1	0.1 to 0.9	Not tested
Firm to Stiff Residual Soils	0.2 to 1.0	0.6 to 1.8	8 to 9
Stiff to Very Stiff Residual Soils	1.0 to 2.0	0.5 to 4.0	12 to 20
Hard Residual Soils	2.5 to 5.0	0.5 to 3.4	+50 (SPT refusal)
Class V Sandstone	3.0 to 8.4	Unconfirmed	Not tested

7.3 Groundwater

During drilling, groundwater was encountered in the four boreholes at depths varying from approximately 2.6m to 4.2m bgl.

Three groundwater monitoring wells identified as GW1 to GW3 inclusive were installed at the site in three boreholes drilled as part of the DSI in 2013 by Aargus. Subsequent groundwater monitoring in the three wells indicated groundwater levels were at the following depth ranges:

- From approximately 1.4m to 2.2m bgl on 25th and 26th September 2013 respectively; and
- From approximately 1.53m to 1.83m bgl on 9th October 2013.

It should be noted groundwater levels may be subject to seasonal fluctuations influenced by rainfall, future development of the surrounding lands and other factors. Based on the site topography, groundwater flow is inferred to be in a north-east to south-west direction towards the Cook River, which is located approximately 1.7km to the south-west of the site.

7.4 Salinity and Aggressivity

Three soil samples were recovered during drilling in borehole BH1 at approximately 2.0m, 5.0m and 8.4m bgl and one sample recovered during drilling in borehole BH4 at approximately 3.0m bgl. These samples were tested by Eurofins Laboratories, a NATA accredited testing laboratory. The testing included determination of Saline, pH, Chloride and Sulphate contents. Results of the laboratory testing are attached in Appendix E of this report and are summarised in tables 2 and 3.

Table 2: Electrical Conductivity Test Results

Borehole	Depth(m)	Electrical Conductivity (dS/m) EC	Multiplication Factor [a]	Electrical Conductivity of Saturated Extract (dS/m) EC_e	Soil Type
BH1	2.0 – 2.5	0.076	7 – 8	0.53 – 0.61	Silty Clay
BH1	5.0 – 5.5	0.12	7 – 8	0.84 – 0.96	Silty Clay
BH1	8.4 – 9.0	0.12	17	2.04	Weathered Sandstone
BH4	3.0 – 3.2	0.11	17	1.87	Weathered Sandstone
Environmental Planning & Assessment Regulation 1994 **Dryland Salinity (1993)**				Saline at >4 dS/m Non-saline <2 dS/m Slightly saline 2-4 dS/m Moderately saline 4-8 dS/m Very saline 8-16 dS/m Highly saline >16 dS/m	

Table 3: Soil pH, Chloride and Sulphate Test Results

Borehole	Depth(m)	MC* %	pH	Chloride (mg/kg)	Sulphate as $S0_4$ (mg/kg)
BH1	2.0 – 2.5	15	5.5	23	123
BH1	5.0 – 5.5	17	6.0	64	180
BH1	8.4 – 9.0	17	6.3	110	114
BH4	3.0 – 3.2	23	6.2	49	195
AS2159-2009					
Piling - Design and Installation					
Reinforced Concrete Piles					
High Permeability Soils					
Mild			>5.5		<5000
Moderately aggressive			4.5 - 5.5		5000 – 10,000
Severely aggressive			4.0 - 4.5		10,000 – 20,000
Very severely			<4.0		>20,000
Low Permeability Soils					
Non-aggressive			> 5.5		<5000
Mild			4.5 - 5.5		5000 – 10,000
Moderately aggressive			4.0 - 4.5		10,000 – 20000
Severely aggressive			<4.0		>20,000
Steel Piles					
High Permeability Soils					
Non-aggressive			>5.0	<5000	
Mild			4.0 - 5.0	5000 – 20,000	
Moderately aggressive			3.0 - 4.0	20,000-50,000	
Severe			<3	>50,000	
Low Permeability Soils					
Non-aggressive			>5.0	<5000	
Non-aggressive			4.0 - 5.0	5000 – 20,000	
Mild			3.0 - 4.0	20,000-50,000	
Moderately aggressive			<3.0	>50,000	

Note: MC * = Moisture Content

8. GEOTECHNICAL ASSESSMENT

8.1 General

The main geotechnical aspects associated with the future development at this site are assessed to include the following:

- Basement Excavation
- Building Foundations
- Groundwater Management
- Retaining Walls
- Site Earthquake Classification
- Soil Salinity and Aggressivity

A summary for assessment of the geotechnical aspects above and recommendations for design and construction of the future development is presented in the following sections.

8.2 Excavation Conditions

The results of the borehole investigation indicated excavation for proposed future basement levels will be predominantly in fill, residual soils and Class V sandstone and possible Class IV sandstone.

Excavation in the soils and weathered sandstone materials should be typically feasible using conventional earthmoving equipment. However, excavation of less fractured Class V sandstone or low strength Class IV sandstone that may be encountered underlying the upper Class V, may require heavy ripping, high capacity or vibratory rock breaking equipment.

8.3 Vibration Control

To ensure vibration levels remain within acceptable levels and minimise the potential effects of vibration, excavation into low strength Class V and Class IV sandstone or stronger should be complemented with saw cutting or other appropriate methods prior to excavation. Rock saw cutting should be carried out using an excavator mounted rock saw, or the like, so as to minimise transmission of vibrations to any adjoining properties that may be affected. Hammering is not recommended and should be avoided. However, if necessary, hammering should be carried out horizontally along bedding planes of (pre-cut) broken rock blocks or boulders where possible with noise levels restricted to acceptable to comfortable limits to adjacent residents.

Induced vibrations in structures adjacent to the excavation should not exceed a Peak Particle Velocity (PPV) of 10mm/sec for brick or unreinforced structures in good condition, 5mm/sec for residential and low rise buildings or 2mm/sec for historical or structures in sensitive conditions. It is recommended that monitoring is carried out during excavation using a vibration monitoring instrument (seismograph) and alarm levels (being the appropriate PPV) selected in accordance with the type of structures present within the zone of influence of the excavation.

As vibrations are considered possible during the use of heavy ripping and rock hammers, it is recommended dilapidation survey of adjoining structures be undertaken prior to project excavation commencement including a minimum the adjoining roads and the existing buildings within the adjoining properties.

If vibrations in adjacent structures exceed the values recommended above or appear excessive during construction, excavation should cease and Aargus should be contacted immediately for appropriate reviews.

8.4 Stability for Basement Excavation

Temporary batter slopes may not be feasible for construction of basements at this site if sufficient setbacks between the basement perimeter walls and the site boundaries are not provided. Temporary batters slope are not recommended in general for deep excavations in areas surrounded by existing developments, especially where groundwater levels are relatively shallow. Excavation for basements should be retained by a shoring system prior to excavation along the perimeter walls. Suitable shoring option may consist of any of the two options below:

- Cast insitu reinforced concrete continuous Cutter Soil Mixing (CSM) diaphragm wall; or
- Cast insitu reinforced concrete semi contiguous, contiguous or secant pile wall, with reinforced concrete panels covering the gaps between semi-contiguous piles.

Other alternative shoring options may be considered subject to assessment by the project structural engineer in consultation with the project geotechnical engineer. If sufficiently embedded into the bedrock, the elements of the shoring wall can be designed to be incorporated into the building foundation system.

If not restrained, lateral movement in the shoring system due to the mobilisation of the active earth pressure will likely occur during construction prior to installation of floor slabs and beams of the ground level. Temporary anchorage or other temporary tie back system is expected to be required during construction to reduce the potential effects of wall movement on the adjoining properties. Anchor installation beyond the property boundaries will be subject to approval by owners of adjoining properties, roads and infrastructure.

If installation of temporary anchors is not feasible, consideration of other options to control wall lateral movement would be necessary. These options include the following:

- Temporary solutions such as installation of props associated with staged excavation;
- Staged excavation and creating temporary partial berms in front of walls; and
- Top-down construction where floor slabs and beams are constructed at top of shoring wall and at floor level for upper basement levels prior to excavation within the basement level underneath the floor slabs

With the recommended retention options above, construction of basement levels in the short and long terms is expected to have low effects on the adjacent buildings and infrastructure.

Vertical excavation for lift shafts may be feasible if temporary shoring is provided. Alternatively, temporary slope batters of 1V:2H to 1V:1H may be suitable for soils and rock respectively subject to availability of sufficient setback distances and confirmation by a geotechnical engineer during construction.

Shoring walls supporting the stormwater channel should be designed and constructed in a manner that would not result in movement in the existing or future diverted stormwater channel structure. The use of “at rest” coefficient of lateral earth pressure in the design of shoring wall supporting the channel is recommended.

Dilapidation survey will be required to be undertaken for the existing structures within the adjoining properties and adjoining carriageway and road reserve prior to excavation.

Earth retention structures and anchors can be designed using the recommended parameters provided in Section 8.5. Testing of anchors will be required following installation. Monitoring of lateral movement of the basement perimeter wall/ shoring system will be required to be carried out during construction under the supervision of the project geotechnical engineer. Monitoring and testing of anchors should constitute as “Hold Points”.

8.5 Retaining Walls

Earth retaining structures, including permanent perimeter walls and excavation shoring walls, should be designed to withstand lateral earth, hydrostatic and earthquake (if applicable) pressures and the applied surcharge in their zone of influence, including existing structures, traffic and construction related activities.

For the design of flexible retaining structures, where some lateral movement is acceptable, it is recommended the design should be based on active lateral earth pressure. Should it be critical to limit the horizontal deformation of a retaining structure, use of an earth pressure coefficient "at rest", should be considered such as the case when the shoring wall is in the final permanent state and is restrained by the concrete slab in its final state. Recommended preliminary parameters for the design of retaining structures are presented in Table 4.

Table 4: Preliminary Geotechnical Design Parameters for Retaining Walls

Units	Unit Weight (kN/m^3)	Effective Cohesion c' (kPa)	Angle of Effective Internal Friction ϕ' (degree)	Modulus of Elasticity $E_{s\,(v)}$ (MPa)	Poisson Ratio ν
Fill	17	0	26	10	0.35
Firm to Stiff Residual Soils	18	5	26	20	0.35
Stiff to Very Stiff Residual Soils	18	5	28	30	0.35
Hard Residual Soils	18	7	29	50	0.3
Class V Sandstone	22	15	35	100	0.3
Class IV sandstone	22	20	35	200	0.3

Table 5 provides preliminary coefficients of lateral earth pressure for the soil and rock horizons encountered during the geotechnical site investigation, or horizons inferred to be present underlying the site. The coefficients provided are based on horizontal ground surface and fully drained conditions.

Table 5: Preliminary Coefficients of Lateral Earth Pressure

Units	Coefficient of Active Lateral Earth Pressure Ka	Coefficient of Lateral Earth Pressure at Rest Ko
Fill	0.39	0.562
Firm to Stiff Residual Soils	0.39	0.562
Stiff to Very Stiff Residual Soils	0.361	0.531
Hard Residual Soils	0.347	0.52
Class V Sandstone	0.271	0.426
Class IV sandstone	0.271	0.426

- Coefficient of active and passive lateral earth pressure Ka and Kp, can be calculated using Coulomb's equations.
- Coefficient of lateral earth pressure at rest Ko, can be calculated using Jacky's equation.

The coefficients of lateral earth pressure should be verified by the project structural engineer prior to use in the design of retaining walls.

Simplified calculations of lateral active (or at rest) and passive earth pressures can be carried using the Rankine equations shown below:

$Pa = K\,\gamma\,H - 2c\sqrt{K}$ For calculation of Lateral Active or At Rest Earth Pressure

$Pp = K_p\,\gamma\,H + 2c\sqrt{K_p}$ For calculation of Passive Earth Pressure

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)

Kp = Coefficient of Passive Earth Pressure

H = Retained height (m)

c = Effective Cohesion (kN/m^2)

Temporary anchors will require embedment in Class V sandstone or stronger. An allowable bond stress of 100kPa may be adopted for temporary anchors within Class V sandstone and 250kPa may be adopted for temporary anchors within Class IV sandstone.

Anchors should undergo proof testing following installation. The anchors can be designed for the parameters recommended above providing:

- The bond (socket) length in Class V or Class IV sandstone is at least 3.0m; and
- Anchors are proof tested to 1.3 times the design working load specified by the structural engineer, before they are locked off at no higher than 75% of working load.

Depending on the magnitude of wall movement predicted, prestressing may be required in order to reduce the potential for any movement-induced damage to adjacent structures.

8.6 Foundations

The following foundation options are recommended for the proposed future buildings within the site to account for different ground conditions that may be encountered at different locations and different depths depending on actual depths of future basement levels at the site:

- Where the basement floor will be founded in Class V sandstone or better, shallow reinforced concrete foundations, such as pad or strip footings and/or raft slab on grade with thickened slab under columns and walls are assessed to be applicable. Installation of piles is expected to be required in cases of axial loads on columns and walls exceeding the allowable bearing pressure of the underlying strata. Other cases where piles may be required include the need to increase the stiffness of the founding rock, or increase the resistance against lateral seismic loads.

- For foundations at existing ground level or where the basement floor will be founded in soils cast insitu reinforced concrete bored piles or any similar rigid piling system would be suitable. Piles should be installed through all fills, alluvial residual soils and embedded into Class V sandstone or stronger.

For lightly to moderately loaded structures, fill and the alluvial soils are assessed to be unsuitable as bearing stratum unless improved or treated. Alternatively, installation of piles (reinforced concrete bored piles or similar rigid piling system) embedded in residual soils or Class V sandstone is expected to be required.

Table 6 provides geotechnical foundation design capacities and parameters recommended for the soil and rock strata encountered in the boreholes, or inferred to be present underlying the site that may be used for preliminary geotechnical foundation design.

Table 6: Preliminary Geotechnical Foundation Design Capacities and Parameters

Unit	Allowable End Bearing Capacity (kPa) (1)	Allowable Shaft Adhesion in Compression (2) (kPa)	Modulus of Elasticity (Vertical) (MPa)
Fill	NA(3)	10	10
Firm to Stiff Residual Soils	100 (shallow footings)	15	20
Stiff to Very Stiff Residual Soils	200 (shallow footings) 375 (piles)	20	30
Hard Residual Soils	300 (shallow footings) 500 (piles)	50	50
Class V sandstone	1000	100	100
Class IV sandstone (4)	2000	200	200

1 With a minimum embedment depth of 0.5m for deep foundations and 0.4m for shallow foundations.
2 Clean rock socket of roughness of at least grooves of depth 1mm to 4mm and width greater than 5mm at spacing of 50mm to 200mm. Shaft Adhesion in Tension is 50% of Compression.
3 N/A, Not Applicable, not recommended for the future buildings at this site.
4 The actual depth of the underlying Class IV sandstone should be confirmed by further investigation or during construction.

Shaft adhesion may be applied to socketed piles adopted for foundations provided socket shaft lengths conform to appropriate classes of sandstone and accepted levels of shaft sidewall cleanliness and roughness. The rock socket sidewalls should be free of soil and/or crushed rock to the extent that natural rock is exposed over at least 80% of the socket sidewall.

Shaft adhesion should not be applied to the upper 0.5m socket length within these bedrock sequences. Shaft adhesion should be reduced or ignored within socket lengths that are smeared and fail to satisfy cleanliness requirements. Additional attention to cleanliness of socket sidewalls may be required where presence of clay seams and extremely weathered rock bands is evident over socket lengths.

To minimise the effects of differential vertical rock deformation under the building loads, it is recommended all foundations should be founded on rock horizons of similar class.

Should groundwater flow, seepages or surface runoff be encountered within foundation excavations, the excavations should be dewatered prior to concrete placement or appropriate underwater placement techniques should be adopted. Any loose debris and wet soils should also be removed from excavations.

A geotechnical engineer should inspect foundation base excavations at the time of excavation to ensure the foundation bases have been taken to suitable materials of appropriate bearing capacity. The inspections should constitute as “Hold Points”.

8.7 Groundwater Management

Due to the potential for seepage to occur in the basement excavation below the natural groundwater level, monitoring of groundwater levels prior and during construction is recommended. Dewatering of basement excavation will likely be required. Typically, dewatering would involve excavation of a sump pit within the site to collect and remove intercepted water. Dewatering should be controlled in a manner that reduces the potential detrimental effects on existing structures and infrastructure within adjoining properties and roads. Installation of precautionary recharge spear well points or trenches around the excavations will likely to be required in order to maintain the groundwater levels within the surrounding areas and reduce the potential effects of dewatering induced settlement.

To ensure the long-term water tightness of the basement, the basement walls and floor below the natural groundwater level, plus a free-board estimated based on the predicted flood level for this site, should be constructed as impervious walls and floor with water-tight construction joints. The basement walls and slabs should be designed to withstand hydrostatic pressures taking into consideration the existing groundwater levels and predicted flood levels for this site.

With the recommended procedures and precautionary mitigation measures described above, the potential dewatering-induced effects on future development and surrounding properties and roads are expected to be low. Nevertheless, further assessment on the potential effects of dewatering should be carried out during the construction certificate stage based on the final detailed design drawings of the proposed development.

8.8 Site Earthquake Classification

The site is underlain by fill and natural soils with SPT 'N' values ranging from 8 to +50 blows/300mm extending to approximately up to 8.4m bgl underlain by sandstone bedrock. Therefore, in accordance with Australian Standards AS1170.4: 2007 (Reference 1) the site can be classified as a "Shallow Soil Site' (Class C_e).

8.9 Site Salinity and Aggressivity

Through introduction of a multiplying factor to the test results, as stipulated in the Department of Natural Resources (DNR) publication "Site Investigations for Urban Salinity" – 2002 (Reference 6), the resultant electrical conductivity of saturated extracts (ECe) ranges from approximately 0.53 dS/m to 0.96 dS/m for soil samples recovered frpm 2.0m to 5.5m bgl, as shown on Table 2, indicating the soils to be "Non-Saline". The resultant electrical conductivity for soils samples recovered from 3.0m bgl in borehole Bh1 and from 8.4m bgl in borehole Bh4, indicate the materials to be "Marginally to Slightly Saline". An appropriate saline soil management plan may be required to be implemented during exaction in the weathered sandstone materials.

Reference to AS2159-2009, "Piling – Design and Installation" (Reference 3), and the results of soil pH, Chloride, and Sulphate tests on three soil samples collected from boreholes BH1 and BH4, as presented in Table 3, indicate that the underlying soils at depths corresponding to the depths of the soil samples to have generally "Non-aggressivity" to reinforced concrete and steel foundation elements.

9. CONCLUSIONS AND RECOMMENDATIONS

The results of the geotechnical site investigation and assessment for this site indicate the ground conditions in general are suitable for the future development subject to adoption of the recommendations made in this report. The following is a summary of Aargus conclusions and recommendations:

9.1 The site is underlain by fill overlying residual soils to the top of horizons of very low to low strength weathered Class V sandstone bedrock, which is present at depths ranging from approximately 3.0m to 8.4m bgl. Stronger rock class, i.e. Class IV sandstone, which typically underlies Class V, may be present underlying the site. The actual depth of the underlying Class IV sandstone should be confirmed by further investigation or during construction.

9.2 Groundwater monitoring carried out for this site indicated the natural groundwater levels varies from approximately 1.4m to 2.2m bgl. Monitoring of groundwater levels prior and during construction is recommended. Dewatering of basement excavation will likely be required and should be controlled in a manner that reduces the potential detrimental effects on existing structures and infrastructure within adjoining properties and roads. Installation of precautionary recharge spear well points or trenches around the excavations will likely to be required. To ensure the long-term water tightness of the basement, the basement walls and floor below the natural groundwater level, plus a free-board estimated based on the predicted flood level for this site, should be constructed as impervious walls and floor with water-tight construction joints. The basement walls and slabs should be designed to withstand hydrostatic pressures taking into consideration the existing groundwater levels and predicted flood levels for this site.

9.3 Results of chemical laboratory testing indicate the upper horizons of the residual soils underlying the site are likely to be “Non-Saline”. The lower horizons and weathered rock may be “Marginally to Slightly Saline”. The soils and weathered rock materials underlying the site are assessed likely to have “Non- aggressivity” to reinforced concrete and steel foundation elements.

9.4 Earth retaining structures should be designed to withstand the lateral earth, hydrostatic and earthquake (if applicable) pressures, and the applied surcharge loads in their zone of influence, including existing structures, traffic and construction related activities. Recommended parameters for the design of earth retaining structures and anchors are provided.

9.5 Excavations for the proposed future basement should be retained prior to excavation along the perimeter walls using a shoring wall such as cast insitu reinforced concrete CSM diaphragm wall or semi-contiguous/ contiguous or secant pile wall. If sufficiently embedded into the underlying bedrock, the elements of the shoring wall can be designed to be incorporated into the building foundation system. Temporary anchorage will likely to be required in order to limit the magnitude of lateral movement in the shoring system. If installation of temporary anchors is not feasible, consideration of other options to control wall lateral movement would be necessary.

9.6 If the use of heavy ripping, high capacity or vibratory rock breaking equipment are requited, in order to reduce the induced vibrations in structures in the vicinity of the excavation, excavation into the less fractured Class V sandstone or low strength Class IV sandstone or stronger should be complemented with saw cutting or other appropriate method prior to excavation. A vibration monitoring programme should be planned and implemented to ensure Peak Particle Velocity (PPV) levels for all activities are within prescribed acceptable limits.

9.7 Recommended foundation systems for the proposed future building at this consist of:

- Shallow reinforced concrete footings and/or raft slab on grade with thickened slab under columns and walls for basement floors founded in sandstone bedrock. Piled foundations may be required in cases of axial loads on columns and walls exceeding the allowable bearing pressure of the underlying strata.
- Piled foundations for basement floors founded in soils.

9.8 Earth retaining structures should be designed to withstand the lateral earth, hydrostatic and earthquake (if applicable) pressures, and the applied surcharges in their zone of influence, including existing structures, traffic and construction related activities. Recommended parameters for the design of earth retaining structures are provided.

9.9 In accordance with Australian Standard AS1170.4: 2007, the site can be classified as a “Shallow Soil Site’ (Class Ce).

9.10 Dilapidation surveys for existing building and infrastructure within surrounding properties are recommended to be carried out prior to construction involving basement excavation. It is recommended the design drawings be provided to

Aargus for further assessment and confirmation of a suitable foundations and retaining walls. Inspections of the ground retention system, anchors and foundations, with possible anchor testing, should be carried out by a geotechnical engineer during construction. The inspections and testing should constitute as "Hold Points".

10. LIMITATIONS

The geotechnical assessment of the subsurface profile and geotechnical conditions within the proposed development area and the conclusions and recommendations presented in this report have been based on available information obtained during the work carried out by Aargus and in the provided documents listed in Section 2 of this report. Inferences about the nature and continuity of ground conditions away from and beyond the locations of field exploratory tests are made, but cannot be guaranteed.

It is recommended that should ground conditions including subsurface and groundwater conditions, encountered during construction and excavation vary substantially from those presented within this report, Aargus Pty Ltd be contacted immediately for further advice and any necessary review of recommendations. Aargus does not accept any liability for site conditions not observed or accessible during the time of the inspection.

This report and associated documentation and the information herein have been prepared solely for the use of **E & D Danias Pty Ltd** and any reliance assumed by third parties on this report shall be at such parties' own risk. Any ensuing liability resulting from use of the report by third parties cannot be transferred to Aargus Pty Ltd, directors or employees.

The conclusions and recommendations of this report should be read in conjunction with the entire report.

For and on behalf of

Aargus Pty Ltd

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