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REPORT
on
PRELIMINARY GEOTECHNICAL INVESTIGATION AND WASTE CLASSIFICATION ASSESSMENT

PROPOSED VEHICLE AND PEDESTRIAN SAFETY (VAPS) PROJECT
SYDNEY OPERA HOUSE
BENNELONG POINT

Prepared for
SYDNEY OPERA HOUSE TRUST

Project 71529
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EXECUTIVE SUMMARY

The construction of a new central, underground loading dock is proposed beneath the existing Opera House, together with a vehicle entry ramp and two service tunnels extending beneath the main building. A preliminary geotechnical investigation was carried out to supplement available subsurface information and to form a geotechnical model for the site.

Two boreholes were drilled to depths of up to 17 m with continuous sampling of the underlying sandstone bedrock undertaken, followed by the installation of a standpipe for groundwater sampling and measurement. Rock strength testing was conducted on the rock cores.

The current and previous borehole data were used to create an interpreted rock surface contour plan of the site. In general, rock depths are indicated to be between 1 m and 2 m over the central portion of the loading dock, increasing to about 5 m at the eastern side of the proposed excavation, in the area of the truck turning bay. Some irregularities are, however, anticipated due to previous land-uses of the site, which probably included quarrying the sandstone to build Fort Macquarie. The overburden materials typically included sand, gravel and rubble (rock) filling. Previous investigations have indicated the presence of minor amounts of organic clays and natural sands beneath the filling.

An interpreted geotechnical cross-section (A-A') presented in Appendix A shows that bulk of the proposed excavation will be within high strength, Class II and Class I Sandstone. Heavy ripping and rock hammering will generally be required for bulk excavation although noise and vibration constraints are likely to dictate that much of the rock is removed using rotary rock saws and milling heads, possibly in conjunction with line-drilling around the perimeter, so as to avoid excessive overbreak.

The presence of post-tensioned cables that act to brace the existing Monumental Steps will preclude the opportunity to carry out the excavation using conventional excavation and support methods for the main loading dock. However, where the excavation footprint extends beyond the Steps, it is expected that a conventional piled or diaphragm wall system will be required to support the deeper overburden materials and form a relatively impermeable barrier to groundwater.

A major constraint for the proposed service tunnels and also for the main loading dock excavations is the degree of stress-relief that will occur and is largely unavoidable. Lateral movements around the loading dock could be in the range of 8 – 30 mm and settlements above the proposed tunnel roofs could be in the order of 20 – 40 mm. Careful consideration should be given to the implications of stress relief movements for the existing Opera House and surrounding structures.

Although a “drained basement” system is obviously preferable, the proximity of the Harbour and possibility that the site is traversed by the GPO Fault Zone suggests that there would be a relatively high likelihood of experiencing large rates of saline groundwater inflows/upflows through the rock.

Further geotechnical investigation will generally be necessary to address the key issues of stress relief related ground movements and rock mass permeability. In particular, the possible presence of the GPO Fault Zone should be investigated using inclined boreholes. From a contamination viewpoint, additional chemical analyses of both the site soils and groundwater will be required to aid the design and costing of various basement construction and disposal schemes.

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Project 71529

19 February 2010

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1. INTRODUCTION

This report describes the results of a preliminary geotechnical investigation and waste classification assessment undertaken by Douglas Partners Pty Ltd (DP) for the proposed Vehicle and Pedestrian Safety (VAPS) Project at the Sydney Opera House, Bennelong Point. The work was commissioned by the Project Manager, Savills Australia Pty Ltd (Savills), on behalf of the client, the Sydney Opera House Trust (SOHT).

It is understood that the construction of a new central, underground loading dock is proposed beneath the existing Opera House together with a vehicle entry tunnel and two service tunnels. The purpose of the VAPS project is to restrict the use of the current roadway across the forecourt to taxis and other patrons' vehicles, thereby enhancing pedestrian safety. The present investigation did not address the new access tunnel or an associated major stormwater diversion, both of which will extend beneath the Opera House Forecourt, between the Monumental Steps and the Tarpeian Way. These two aspects of the project will require separate specific geotechnical investigation.

The preliminary geotechnical investigation was carried out to supplement available subsurface information, so as to form a geotechnical model of the site and provide information for preliminary design and planning purposes. In particular, the scope included a geological desk-top study of the broader site to provide an indication of bedrock levels under the proposed loading dock and main Opera House building. Preliminary comments on excavation conditions, vibrations, support/retention requirements, underpinning and groundwater seepage are included, together with advice on foundation design and construction. The investigation was carried out in conjunction with a preliminary waste classification assessment, which is incorporated within this report.

Field work comprised the drilling of two geotechnical boreholes with in-situ testing and sampling of the subsurface materials, together with the installation of a standpipe for groundwater sampling and monitoring. Laboratory testing of rock core samples was undertaken, followed by engineering analysis and reporting. Details of the field work are given in the report together with comments on design and construction practice.

Information supplied in the Brief by the Project Manager, Savills (email dated 19/11/09) includes an overview of the proposed works together with an overall plan diagram of the proposed underground loading dock (Drawing No. Sk 09 010 S01 V3). Additional information supplied by Savills included existing house floor plans (Drawing No. 10 BG 20904, 18 in total) and a draft report by Godden, Mackay & Logan – Heritage Consultants entitled: “Sydney Opera House (Loading Dock) – Archaeological Management Plan and Heritage Impact Assessment,” October 2009.

2. BACKGROUND

2.1 Overview of Previous Site Development

Prior to the construction of the Sydney Opera House at Bennelong Point, the headland had been used for a variety of purposes dating back to the early days of settlement around Sydney Cove (Circular Quay). Based on numerous historical reports and publications, including the archaeological report referenced above, the following periods of land use for the site are summarised below:

- **1788 – 1795** – Initially, the site was an important intermediary meeting place between settlers and the local Aborigines. It was also the site of Bennelong’s hut and then subsequently a windmill and salt works.

- **1795 – 1901** – The site was used as a defensive point housing a half-moon battery until the construction of Fort Macquarie in 1821. Based on paintings and photographs, it appears that some excavation of the sandstone was carried out to provide blocks for the Fort construction. Fort Macquarie remained a manned fort against potential invasions until 1901.
- **1901 – 1958** – Tram sheds were constructed after the demolition of Fort Macquarie, with a series of finger wharfs extending into Farm Cove on the eastern side of Bennelong Point.
- **1958 – 1973** – Construction of the Sydney Opera House (opened in 1973).

Stage 1 construction for the podium and foundations of the Opera House commenced in 1959 and was completed in 1963. The finished podium consists of a reinforced concrete monolith. A notable technical design feature is the design of the single-span concrete beams, which are visible under the main Opera House steps. Stage 2 involving construction of the roof shells commenced the same year (in 1963), with the challenges in construction demanding pioneering applications and many new materials as well as building and engineering practices. The Sydney Opera House was officially opened in 1973.

It is noted that the present shoreline of Bennelong Point which is contained by seawalls forming the perimeter boardwalk, represents entirely reclaimed land dating back to 1829 when the first area of the shoreline along the southeastern side of the point was reclaimed. Subsequent land reclaimations took place up until the 1880's and included an encircling tidal seawall around Fort Macquarie and a number of wharves, jetties and wharf buildings along the western shore.

Based on historical data, the surface ground levels at Bennelong Point have also been modified. The most significant modification to ground levels across Bennelong Point appears to be for the construction of the tram sheds and associated railway lines, where significant cut and fill earthworks operations occurred.

The Sydney Opera House has undergone many geotechnical site investigations over the years dating back to 1958 when Bennelong Point was used as a tram shed depot. Previous geotechnical site investigations were conducted by DP and others. The approximate locations of the available borehole information (known to DP) drilled at the site of the Sydney Opera House are presented in Drawing 1 in Appendix A, and available borehole information listed in Table 1 on Page 9. A summary of the previous investigations by DP and other companies is given in the following sections.

2.2 Previous Investigations Conducted by Douglas Partners

Geotechnical investigations and construction-phase geotechnical inspections conducted by DP are given in chronological order as follows:

- **1995** – Borehole investigation comprising 28 boreholes for the new boardwalk foundations along the eastern (denoted "DPBHE") and northern boardwalk (denoted "DPBHN") for contractors McConnell Dowell (DP Project 20619A). The boreholes were drilled from the boardwalk (deck) level (approximately 3.6 m AHD) to depths of between 7.75 and 11.45 m, below deck level. The subsuface profile encountered in most of the boreholes comprised sand and boulder filling directly overlying sandstone bedrock. The sandstone was generally medium or high strength and slightly fractured.
- **1998** – Borehole drilling for the installation of 6 mini-piles (denoted "MP") for the proposed boardwalk studio located on the western side of the Opera House for contractors Austin Australia (DP Project 24937). The mini-piles were core drilled within sandstone to depths between 8.0 and 9.0 m from the boardwalk (deck) level (approximately 3.6 m AHD).
- **2004** – Inspection of trenching work and reporting on settlement was undertaken during construction of the mechanical bollards for contractors Construction Building Design (DP Project 36814). No borehole information was associated with this project.

Historical field records of geological mapping along the western section of the Tarpeian Way by DP indicate that there are several sub-vertical joints dipping to the west at 80° to 90° and striking at 020° to 030°. The spacing of these joints typically varied between 5 m and 15 m.

The approximate locations of the previous DP boreholes (and mini-piles) are shown on Drawing 1, in Appendix A.

2.3 Previous Investigations Conducted by Others

Geotechnical investigations conducted by others are given in chronological order as follows:

- **MacDonald, Wagner and Priddle (1958)** - Twelve hand-drawn boreholes logs (denoted "TH") were obtained from a geotechnical investigation undertaken in 1958 for preliminary work on the Opera House, when tram sheds existed on the site. Reduced levels at the ground surface and at the top of rock (converted to Australian Height Datum, AHD) were able to be read from Drawing 7095/1 (1958) with some degree of confidence.
- **Jeffrey and Katauskas (1994)** – Initial borehole investigation comprising seven boreholes (denoted "JKBH") for the proposed upgrade to the northern and eastern boardwalk. The boreholes were drilled from the boardwalk (deck) level (approximately 3.6 m AHD) to depths of between 7.8 and 9.6 m.
- **ARUP Geotechnics (2004)** – Borehole investigation comprising four boreholes (denoted "ARUP") drilled to depths of 18.5 m from the boardwalk (deck) level or "Ground Floor Level" (approximately 3.6 m AHD), for the proposed Set Storage Area located within the central section of the eastern side of the Opera House. It is noted that the ARUP logs and report describe the surface level from which these bores were drilled as RL 0.0 m AHD. Email advice from the SOHT on 19/01/10 confirmed that these levels were in error and that the bores were drilled from Ground Floor Level (approx. RL 3.6 AHD).

The approximate location of previous boreholes drilled by other geotechnical firms is also shown on Drawing 1, in Appendix A.

3. GEOLOGICAL DESK STUDY

3.1 General

Reference to the 1:100,000 Geological Map Sheet for Sydney indicates that the site is underlain by filling and/or a soil layer overlain by Triassic-Aged Hawkesbury Sandstone. The Hawkesbury Sandstone typically comprises medium to coarse-grained quartz sandstone with very minor shale and laminite lenses. The formation normally has near horizontal bedding partings spaced from less than 1 m to well over 3 m in places and is typically cut by two sets of steeply dipping joints:

- **Primary Joint Set (i)** **Strike 020° - 035°/ Dip 70° - 90° W**

  These joints are generally spaced from 1 m to over 10 m except in joint swarms (strike-slip fault zones where they can be spaced 0.1 – 2 m, often with associated crushed zones). The joints are generally persistent, both laterally and vertically, with surfaces varying from undulating to smooth and stepped.

- **Secondary Joint Set (ii)** **Strike 110° - 130°/ Dip 70° - 90° N and S**

  These joints are generally more widely spaced and less persistent than those of Joint Set (i) and often being confined to individual beds. The joint interface surfaces also vary from undulating to smooth and stepped.

As indicated above, strike-slip fault zones are present and one such fault zone, the GPO Fault Zone, is inferred to possibly traverse the proposed development site. This fault zone comprises closely spaced joints (0.1 – 2 m) often showing slight vertical movement, as well as crushed zones varying from 10 mm to over 2 m wide. The fault zone itself varies from 1 m to 40 m in width, but is more likely to be in the form of 1 – 2 m of bad ground, comprising crushed and sheared rock between closely spaced joints at the Opera House site, if present.

Apart from near vertical strike-slip faults, there are also numerous low-angle (0° - 25°) thrust faults. These often manifest themselves as crushed or clayey bedding planes between individual sandstone beds or clayey zones ramping-up along cross beds. They generally strike east-west and dip either north or south.

Near vertical geological structural features in the Sydney CBD were described in the paper by Pells, Braybrooke & Och (2004). Figure 1 – Vertical Geological Structures in the Sydney CBD, is reproduced from the paper and shown below:

Figure 1 – Vertical Geological Structures in the Sydney CBD

3.2 Interpreted Rock Surface Contour Plan

Previous and current borehole information has been combined to produce an interpreted rock surface contour plan. In order to produce the interpreted rock surface contour plan, the boreholes were graded in relation to the amount of "useable" and "reliable" information that they each provided (see Table 1 below), particularly in respect of the confidence that can be placed on the surface level from which the boreholes (or piles) were drilled. A ranking system comprising grades or classes, A, B and C was developed; A being the most reliable and C being the least reliable. A summary description of the ranking system developed for use in the preparation of the rock surface contour plan is provided below:

- **A -** Borehole logs with Reduced Levels (RL's) and accurate locations.
- **B -** Borehole or drillers/piling logs without Reduced Levels (RL's) but with clearly traceable locations and surface levels. Based on information derived from old drawings or "local knowledge" (e.g. discussions with Sydney Opera House sub-contractors who have a reliable knowledge of previous construction work on the site) most of this old borehole information was able to be used with some confidence.
- **C -** Borehole logs or sketches without Reduced Levels (RL's) or accurate locations.

Using the data shown in Table 1 an interpreted bedrock surface contour plan, giving an indication of bedrock levels at the proposed new loading dock and main Opera House building was developed using the Discover/Surfaces module within the MapInfo® GIS computer program. The Minimum Curvature method/application within the Surfaces module was used to fit a surface to the data presented in Table 1, except for eight of the MacDonald, Wagner & Priddle, (1958) borehole data (denoted TH), and is generally an effective and suitable method for a wider range of smoothly varying data.

The eight TH boreholes denoted by a "*" in Table 1 were not used in the development of the interpreted bedrock surface contour plan, as the values did not fit with the other input data and the resultant surface did not reflect actual Opera House forecourt heights. This could possibly be due to previous excavation of the sandstone bedrock around the time of the tram shed and finger wharf construction. The interpreted rock surface contour plan (Drawing 2) is presented in Appendix A.

Table 1 – Combined Borehole Data Ranking Table

Borehole Number	Reduced Level at Ground Surface (AHD m)	Reduced Level at Top of Rock (AHD m)	Data Ranking(1)		Borehole Number	Reduced Level at Ground Surface (AHD m)	Reduced Level at Top of Rock (AHD m)	Data Ranking(1)
DPBHE1	3.6	-1.8	B		MP1	3.6	-0.9	B
DPBHE4	3.6	-2.1	B		MP2	3.6	-0.8	B
DPBHE7	3.6	-3.8	B		MP3	3.6	-0.6	B
DPBHE9	3.6	-4.2	B		MP4	3.6	-0.2	B
DPBHE11	3.6	-3.7	B		MP5	3.6	-0.5	B
DPBHE14	3.6	-3.7	B		MP6	3.6	-0.4	B
DPBHE17	3.6	-3.5	B		TH1	3.4	1.5	C
DPBHE20	3.6	-3.8	B		*TH2	3.4	-2.6	C
DPBHE23	3.6	-3.8	B		*TH3	3.2	-4.0	C
DPBHE26	3.6	-3.6	B		*TH4	3.1	-2.7	C
DPBHE28	3.6	-5.0	B		*TH5	4.7	3.9	C
DPBHE32	3.6	-4.9	B		TH6	4.7	2.6	C
DPBHE35	3.6	-5.0	B		*TH7	3.6	1.4	C
DPBHE38	3.6	-5.0	B		*TH8	4.0	0.9	C
DPBHN1	3.6	-2.3	B		TH9	4.6	2.7	C
DPBHN4	3.6	-3.8	B		*TH10	3.2	-0.5	C
DPBHN7	3.6	-3.0	B		TH11	3.2	-0.7	C
DPBHN11	3.6	-2.6	B		*TH12	3.2	-2.5	C
DPBHN13	3.6	-2.5	B		JKBH1	3.6	-3.8	B
DPBHN16	3.6	-2.3	B		JKBH2	3.6	-4.3	B
DPBHN19	3.6	-2.5	B		JKBH3	3.6	-3.6	B
DPBHN22	3.6	-2.1	B		JKBH4	3.6	-2.7	B
DPBHN25	3.6	-3.1	B		JKBH5	3.6	-2.7	B
DPBHN28	3.6	-3.6	B		JKBH6	3.6	-3.6	B
DPBHN31	3.6	-2.7	B		JKBH7	3.6	-2.8	B
DPBHN34	3.6	-3.0	B		ARUP1	3.6	0.3	B
DPBHN37	3.6	-3.8	B		ARUP2	3.6	0.8	B
DPBHN40	3.6	-4.8	B		ARUP3	3.6	1.8	B
					ARUP4	3.6	2.8	B
					DPBH101	3.6	-1.4	A
					DPBH102	3.6	1.8	A

Notes: (1) Data ranking refers to reliability of surface level and location information for boreholes/mini-piles (see text for description of A, B & C).

(2)* denotes borehole data not used to develop the interpreted rock surface contour plan due to a conflict with adjacent borehole information.

4. SITE DESCRIPTION

The Sydney Opera House is located on Bennelong Point on Sydney Harbour. Bennelong Point is bounded by Circular Quay to the west and Farm Cove to the east. The Sydney Opera House and the adjoining forecourt occupy an area of approximately 30,000 m^2. It extends from the vertical rock cutting to the south known as the Tarpeian Way to the northern tip of the Bennelong Point, a distance of approximately 250 m. The width of the site is approximately 120 m in an east-west direction.

The site is level with the Opera House forecourt and surrounding boardwalks at approximately 3 m to 5 m above the harbour seawater level, at approximately RL 3.6 m AHD. The Opera House itself comprises a complex of terraced theatres and halls linked together beneath a roof comprising sets of interlocking vaulted shells surrounded by terrace areas that function as pedestrian concourses.

An underground car-park comprising two concentric cylindrical excavations to depths of approximately 40 m is located to the south of the Opera House and the Tarpeian Way cliffline. The Sydney Harbour Tunnel is located within about 80 m of western seawall of the Sydney Opera House and strikes in an approximately north-north-west orientation.

5. CURRENT INVESTIGATION

5.1 Field Work Methods

The Stage 1 field investigation comprised the drilling of two test bores (BH101 and BH102) to depths of between 13.5 m and 17.1 m. The borehole locations were set out relative to existing surface features (e.g. walls, staircases and gutters) by tape measurement. The locations of test bores BH101 and BH102 are shown in Drawing 1 within Appendix A.

The test bores were initially pre-drilled through the surface concrete and asphalt (AC) layers using a 150 mm diameter diatube corer. The bores were then drilled at night-time using a Multi-drill, track-mounted drilling rig. Each bore was drilled using solid, spiral flight augers and rotary washbore drilling techniques in the overburden materials, with Standard Penetration Testing (SPT) carried out where possible, in the sub-surface materials.

Standard Penetration Tests (SPT's) were carried below depths of 0.5 m to sample the sub-surface materials and to assess the in-situ strength of the materials. Disturbed soil samples were retrieved from the cuttings returned by the auger blade and used for identification and classification purposes as well as for waste classification testing purposes.

The bores were advanced into the underlying bedrock using diamond coring equipment to obtain continuous, NMLC sized (51 mm diameter) core samples for lengths of between 8.5 m and 14.9 m. Bore BH101 was terminated at 13.5 m depth due to the significant loss of drilling water and sediment through the adjacent seawall. A plume of drilling sediment was observed in the harbour adjacent to the drilling work, obviously due to some form of hydraulic connection between the borehole and the harbour, and it was decided to stop drilling to prevent further disturbance.

A slotted standpipe piezometer was installed in borehole BH101 to allow water table measurement, after purging the standpipe to remove excess drilling fluids. The standpipe was slotted over the bottom 6 m length with gravel backfill to 2 m above the slotted zone.

The groundwater level was measured in the borehole BH101 during drilling and subsequent water level measurements were taken in the standpipe installed, after the field investigation was completed.

The approximate ground surface level at boreholes BH101 and BH102 was determined by interpolation between survey makers shown on the drawing by Hard & Forester Consulting Surveyors, 2005 entitled: Sydney Opera House Survey Control Plan, Ground Floor +12 External, in particular, Sydney Opera House Bench Mark P6-01 (SOHBM – P6-01). SOHBM P6-01 was located at the base of the foyer stairs adjacent to the eastern broadwalk, a distance of between 7 m and 17 m from the borehole locations. The Reduced Level (RL) shown on SOHBM – P6-01 is understood to be relative to AHD. The location of Sydney Opera House Bench Marks (Ground Floor; + 12' External) are shown in Drawing 1 within Appendix A.

It is noted that the SOHT have "in-house" levels that have remained in Imperial format (feet) since the construction of the Opera House. The levels are relative to an original site datum and are commonly used on site by sub-contractors and employees. The Imperial levels, the corresponding metric levels and the Opera House floor level descriptions are summarised in Table 2.

Table 2 – Sydney Opera House Levels (2)

Reduced Level (RL) (1)	Imperial Level (feet)	Opera House Floor Level
-10.972	- 36	Sub-Basement Level (to be constructed)
-10.0584	- 33	Sub-Basement Level (to be constructed)
-4.053	- 13.3	Dock Level (to be constructed)
-2.438	- 8	Sub-basement level (to be constructed)
0.3048	+ 1	Basement Level
3.6576	+ 12	Ground Level/Pedestrian Concourse (and Boardwalk/Deck Level)
6.4008	+ 21	Mezzanine Level
9.1440	+ 30	First Floor Level
12.8016	+ 42	Second Floor Level

Notes: (1) All levels relative to original site datum (standard datum not AHD)
(2) This table has been reproduced from VAPS draft sketch SK-003 rev.000 (23.11.09)

5.2 Field Work Results

Details of the subsurface conditions encountered in the boreholes are given in the borehole logs included in Appendix B, together with notes defining classification methods and descriptive terms. The SPT results are shown on the logs at the appropriate depth. Photographs of the rock cores were taken and are presented opposite the relevant logs in Appendix B.

The boreholes generally encountered soil and rock filling material over sandstone bedrock. The general sequence of materials encountered in the boreholes is described below:

PAVEMENTS: typically comprised asphaltic concrete (AC) also referred to as bituminous concrete over concrete where present over roadbase gravel with a combined pavement thickness of between 0.2 m to 0.4 m; overlying,

FILLING: encountered in boreholes BH101 and BH102 to depths of between 1.8 m and 4.95 m, respectively. The filling generally comprised sand with inclusions of sandstone gravel overlying ballast ("blue metal" gravels and cobbles); and was overlying,

BEDROCK: the bedrock generally comprised medium and high strength, fresh sandstone down to approximately 8.5 m to 15.3 m below the bedrock surface. The bedrock was typically slightly fractured or unbroken, medium to coarse grained sandstone with occasional thin bands of siltstone or laminite of less than 250 mm thickness. A few clay seams of up to 80 mm thickness were also noted, mainly associated with possible shear zones along bedding planes.

Groundwater was recorded during auger drilling at borehole BH101 at a depth of 4.1 m (RL-0.6). Groundwater was not observed during augering in borehole BH102, over the depth of augering and the use of water as a drilling fluid within the rock precluded further measurement.

The standing water levels measured in the standpipe piezometer installed in borehole BH101 on 4 January 2009 and 12 January 2009 are summarised in Table 3 below. It is noted that the variable water levels measured in the standpipe may be related to the seawater level. Based on the nature of the filling material (i.e. ballast and sand) and the proximity of the borehole BH101 to the eastern sea-wall (approximately 6 m) together with evidence of lost drilling water into the harbour during diamond core drilling, it is assumed that the measured water levels are tidal in nature and represent the sea-water level (at the time of measurements).

Table 3 – Standing Water Levels in BH101 Piezometer

Date	Water Level Measurement in Standpipe		Depth to Sea Water at Adjacent Sea-Wall (1) (m)
	Depth (m)	RL (m)	
4 January 2009	2.48	1.0	2.9
12 January 2009	3.15	0.4	3.4

Notes: (1) Depth is relative to general Forecourt and Ground Level, at approximately RL 3.6

6. LABORATORY TESTING

6.1 Engineering Tests

Point Load Strength Index (Is_{50}) testing was carried out on selected rock core specimens from BHs 101 and 102. The results of the tests are given on the borehole logs at the appropriate depth and indicate (Is_{50}) values mainly in the range 1.1 – 3.2 MPa corresponding to a high strength classification. Using the typical published correlations between Uniaxial Compressive Strength (UCS) and Is_{50} 15:1 of 20:1, the results indicate UCS in the approximate range, 15 – 65 MPa.

UCS testing on two saturated samples of rock core was carried out in a NATA accredited laboratory. The results of the tests are given in Appendix C and summarised in Table 4.

Table 4 – Unconfined Compressive Strength Test Results

Borehole	Sample Depth (m)	Rock Type	UCS (Mpa)
BH101	7.86 – 8.00	Sandstone	17.1
BH102	14.63 – 14.76	Sandstone	18.8

Comparison between the above UCS results and the nearest Is_{50} results indicted a correlation ration of between 10 and 13, somewhat lower than the typical figure.

7. GEOTECHNICAL MODEL

A geotechnical model of the site is presented in the form of an interpreted rock surface contour plan of the site and an interpreted geotechnical cross-section (Section A-A') in Drawings 2 and 3, respectively, within Appendix A. The cross-section shows the depth of filling noted at the relevant test bore locations, together with the interpreted geotechnical boundaries for the underlying rock. The extent of the proposed eastern service corridor (i.e. tunnel) is also indicated, together with a notional roof at approximately RL -4.5m (AHD). The level of the roof or tunnel crown was based on verbal advice from Arup (ref: meeting on 8/2/10), where a minimum clearance requirement of 6.5 m was indicated, to allow the movement of large semi-trailer freight trucks.

The results of the field work were reasonably consistent with the published mapping and results of previous geotechnical investigations at the site.

The profile at the locations of BH101 and BH102 comprised filling overlying generally fresh sandstone bedrock. The filling typically comprised sand with some sandstone gravels overlying bluemetal gravels and cobbles (ballast). Based on previous investigations, boulder-sized rocks are likely to be present, particularly in the areas closer to the harbour and seawalls. The depth of filling increased from 1.8 m at BH102, below the Monumental Steps, to about 5 m, adjacent the eastern seawall (at BH101). This trend of an increasing depth to the bedrock surface moving from the central part of the Opera House towards the existing eastern shoreline is indicated by the interpreted rock surface contour plan presented in Drawing 2 (of Appendix A).

Based on Drawing 2, the Bennelong Point peninsula appears to be underlain by a central ridge of rock at relatively shallow depth beneath present surface level, extending out to approximately the northern extent of the Central Passageway. Based on the available borehole information, rock levels generally fall on either side of the ridgeline (i.e. to the east and west) and also towards the main harbour, to the north.

Based on the two current boreholes, the sandstone bedrock underlying the proposed loading dock is characterised by an apparently dipping bedrock surface that falls towards the east at a slope of approximately 7 – 10°. Based on previous experience around the foreshores of Sydney Harbour, the bedrock surface is likely to be stepped in a series of benches formed by previous stress relief and erosion along joints and bedding surfaces.

The underlying sandstone was generally fresh and of medium and high strength, with some thin intermittent bands of high strength shale and laminite. A few clay seams and crushed or sheared zones of highly weathered sandstone were noted in the rock core from BH102. The 80 mm thick clay seam at 12.6 m depth is inclined at approximately 20° to the horizontal and is inferred to be a result of low-angle thrust faulting causing movement along cross beddings. It is not known whether this apparent thrust faulting is associated with the broader strike-slip, GPO Fault Zone.

The sandstone encountered in the current test bores was classified in accordance with the procedures given in References 1 and 2. The interpreted depth and Reduced Level (RL), to the Australian Height Datum (AHD), at the upper surface of the various sandstone classes is shown in Table 5. It should be noted that the profiles are accurate at the test bore locations only and that variations must be expected away from the bores. Thus, the strata units or layers have been shown on the cross-sections by interpreted strata boundaries only. It is noted that in some cases the classification of relatively strong sandstone was down-rated due to the presence of significant fracturing and other defects, such as clay seams.

Table 5 – Summary of Interpreted Geotechnical Model

Borehole No.	Surface RL at Borehole Location (m AHD)	Depth (Reduced Level) of Top of Various Sandstone Classes (2)				
		Class V	Class IV	Class III	Class II	Class I
BH101	3.5	-	-	-	5.0 (-1.5)	9.7 (-6.2)
BH102	3.6	-	-	1.8 (1.8)	5.0 (-1.4) 8.5 (-4.9)	6.0 (-2.4) (3) 13.5 (-9.9)

Notes:

(1) Bracketed numbers are the Reduced Level (to AHD) for the top of the stratum.
(2) Rock classification based on References 1 & 2.
(3) BH102 encountered a 2.5 m thick layer of Class I sandstone at RL -2.4 m which was underlain by Class II sandstone.

The in-situ sandstone rock is assessed to be Hawkesbury Sandstone, which typically has quartz content in the order of 70% within a clay matrix.

8. COMMENTS

8.1 Proposed Development

The proposed VAPS development involves the construction of a new underground loading and delivery dock below the existing driveway entrance and Monumental Steps. The purpose of the development is to restrict the use of the existing Forecourt area to taxis and VIP vehicles only, thereby enhancing pedestrian safety and improving the aesthetics of the Opera House for patrons arriving and departing. It is understood that two service corridors are to be constructed as tunnels below the main Opera House building, extending to the north from the loading dock area. These service tunnels are to provide storage areas together with access to new internal lifts.

It is understood that main part of the loading dock will be located underneath the Monumental Steps. The base of the new loading dock will be at RL -10.97 m (AHD) {Level -36 foot} and will be approximately 14.6 m below the Ground Floor Level at RL +3.66 m (AHD) {Level +12 foot}.

The loading dock will be accessed via a new vehicle entry access tunnel located beneath the forecourt area, starting from near the current main gate house and striking in a north-easterly direction towards the Opera House. The architectural drawings indicate that the width of the tunnel will be about 11 m. The southern section of the access tunnel will be located adjacent to the Tarpeian Way cliffline and the existing Sydney Harbour Tunnel. (The proposed access tunnel was not within the scope of this preliminary geotechnical investigation).

The dimensions of the main loading dock are about 45 m x 35 m in plan. The main loading dock area will also include a turning bay to accommodate large semi-trailer trucks, extending 20 – 25 m eastwards, towards the Man-O-War Steps. The two service corridors (eastern and western) will extend as tunnels from the base of the loading dock for a length of between 45 – 55 m beneath the main building, towards the central part of the Opera House. The eastern tunnel is shown as approximately 11 m in width, but will now apparently be reduced to 8 m and will extend to a proposed new temporary scenery lift located below the set storage area. This corridor may also provide a storage area for containers. The western tunnel is approximately 6 – 7 m in width and will link-up with the existing "Lift 12". A new goods lift will also be located midway along the western corridor. All three lift pits are shown to extend locally down to approximately RL -15 m (AHD), about 3 m lower than the proposed floor level of the service tunnels.

8.2 Site Preparation and Earthworks

8.2.1 Excavation Conditions

Construction of the proposed main loading dock will entail around 15 m depth of excavation. As shown on Drawing No. 3 the proposed excavation for the main loading dock area (and the associated turning bay) will involve excavation of filling materials and medium and high strength Hawkesbury Sandstone. Typically, the depth of the filling is expected to be around 2 m towards the central part of the loading dock, increasing to about 5 m towards the eastern extent of the turning bay. Deeper filling is also expected to be present locally along the alignment of the existing large stormwater culvert which traverses the proposed loading dock footprint.

Based on the available borehole information, the filling material generally overlies 2 to 3 m of Class III Sandstone in the central area of the proposed excavation. Although the strength of the upper few metres of rock is generally medium to high strength, the presence of significant fracturing and some weathered and weaker seams resulted in a down-rating of the rock cores in accordance with the rock classification system described in References 1 and 2. Borehole BH101, however, indicated the presence of medium to high strength (Class II) sandstone that was slightly fractured or unbroken from the bedrock surface at 5 m depth.

Below the upper few metres, the sandstone cores from both the current and previous investigations typically graded to fresh, high strength, unbroken sandstone, which was classified as either Class II or Class I Sandstone.

Excavation of the filling should be readily achieved using conventional earthmoving equipment.

The rippability of rock depends primarily on the rock material strength and the degree of jointing. Moderate to heavy ripping with large excavators and bulldozers (e.g. Caterpillar D11) and the use of medium to large sized hydraulic rock hammers will generally be required to remove the medium to high strength (Class III) sandstone. The Class I and II Sandstone is generally of high strength and only slightly fractured or unbroken. The use of large hydraulic rock hammers (e.g. Krupp 900kg), in conjunction with heavy ripping, will generally be required to remove this material. Ripping the high strength rock with a large dozer could be difficult on the site due to the unbroken condition of the lower bedrock sequence.

Excavation productivity within the Hawkesbury Sandstone is likely to be low, particularly with the excavation techniques likely to be necessary to reduce noise and vibration to acceptable levels. It is suggested that intending excavation (and tunnelling) contractors inspect the core samples before submitting a tender for bulk excavation (or tunnelling) works.

Vertical rock excavation should employ diamond-tipped rotary rock saws or milling heads around the perimeter of the basement to reduce vibrations and minimise over-break. The adoption of rock sawing or line drilling techniques may also be appropriate to construct aesthetic cut faces and to avoid excessive overbreak. Line drilling would involve percussion drilling of nominal 75mm diameter holes at 150 – 200 mm spacing along the proposed excavation alignment.

All rock faces should be progressively inspected for every 1.5 – 2.0 m depth of excavation, by an experienced geotechnical engineer, who should assess the need for rock-bolting to stabilise any potentially unstable wedges or blocks of rock.

Excavation for footings and trenches in high strength rock will also require the use of large hydraulic rock hammers, together with rotary rock saws or milling heads.

It should be noted that the existing system of concrete slabs and beams beneath the pavement in the existing driveway area (encountered at BH102) is understood to extend under the full extent of the Steps. It is further understood that the slab comprises post-tensioned cables that act to brace the Monumental Steps above and based on discussion with Arup (meeting on 8/2/10), the cable beams cannot be disturbed. Therefore, this will preclude the opportunity to carry out the excavation of the loading dock basement using conventional excavation and support methods. Arup advised that a “top-down” excavation methodology will be adopted, where the ground around the excavation will be braced at the ground surface with a series of additional beams and rock pillars. Conventional “open-cut” excavation methods may, however, be necessary and possible around the proposed turning bay extension to the dock, where piling plant would not be limited by the head-room constraints that would apply to working beneath the Steps.

As shown in Drawing 3, the tunnel excavations to form the proposed service corridors beneath the main building are expected to be wholly within medium to high and high strength sandstone.

Conventional tunnelling methods, using road-header machines with the provision for rock-bolting the tunnel roof and sidewalls, as appropriate, are expected to be feasible.

Excavation for the proposed loading dock will require both the provision of satisfactory lateral support, particularly where existing structures are located adjacent to the excavation, and adequate drainage measures to control groundwater entering the excavation. These aspects are discussed in more detail in the following sections of this report.

Materials that will be derived from the planned excavation works include significant amounts of filling and relatively fresh sandstone. It should be noted that any off-site disposal will require assessment for re-use or classification of the soil in accordance with the Department of Environment and Climate Change NSW (DECC), "*Waste Classification Guidelines*" (2008); updated July 2009. A preliminary waste classification assessment, based on sampling from only the two current boreholes (BH101 and BH102) is included within this report, in Section 8.3.

8.2.2 Vibration with Excavation

Dilapidation surveys should be carried out on the internal and external parts of the Opera House (including the Forecourt and Monumental Steps) within the immediate vicinity of the proposed excavations. The surveys should be conducted prior to the commencement of all excavation or tunnelling work so as to allow appropriate assessment of the causes of any damage arising from construction activities. It would also be prudent to conduct a pre-construction dilapidation survey of the Opera House carpark structure, particularly in the areas closest to the proposed VAPS development.

Noise and vibration will be caused by excavation work on the site and precautions therefore will be required when excavating close to the main building foundations and structures. The sandstone bedrock underlying the proposed main dock excavation (and tunnels) is likely to transmit vibrations generated by the excavation process. Consequently, it will be necessary to adopt appropriate construction methodologies and equipment to limit the vibration (and noise) within the Opera House and adjacent areas, to acceptable levels.

The level of acceptable vibration will be dependent on various factors including the type of building structure (e.g. reinforced concrete, brick, etc.), its structural condition, the frequency range of vibrations produced by the construction equipment, the natural frequency of the building and the vibration transmitting medium. Also, given the useage of the Opera House, it is anticipated that there would be additional operational constraints that will apply, particularly when rehearsals and performances are scheduled.

The Australian Standard AS 2187.2 – 1993 (Explosives Code) recommends the maximum peak particle velocity (PPV) of 25 mm/sec for commercial and industrial buildings or structures of reinforced concrete or steel construction subjected to vibration. A PPV limit of 10 mm/sec is suggested for houses and low-rise, residential or commercial buildings and a lower vibration limit is typically adopted for sensitive or historic structures. Ground vibration arising from excavation plant is of a continuous nature, as opposed to transient nature such as with blasting events. Therefore, more stringent vibration limits than given for blasting should generally apply. It is likely that all existing foundations for the Opera House are founded on the upper rock profile. Given the heritage-listed status of the Opera House, it is therefore suggested that peak particle velocity (PPV) be initially limited to 5 mm/sec at the building line for preliminary planning purposes. The structural engineer should also advise on whether specific limits on vibrations will apply to particular structures or existing foundations. For example, a lower vibration limit may be appropriate adjacent to the footing for the Monumental Steps, the post-tensioned cable beams or the existing sandstone block seawall adjacent to the proposed excavation for the turning bay area.

Again, it is noted that vibration levels above 5 mm/sec may be disturbing to the daily activities involved with the SOH and that some reassurance, possibly via vibration monitoring, may be necessary.

Vibration monitoring carried out by DP at various excavation sites within the Sydney area has indicated that to limit vibrations (PPV) to 5 mm/s, a Krupp 600 kg or 900 kg (or equivalent) hydraulic rock hammer should not be used within 8 m and 18 m, respectively from the building or structure in question. The use of rock sawing and/or rock milling methods of rock excavation in conjunction with smaller excavation plant will generally be required close to the main building and Forecourt, so as to reduce vibrations.

The preliminary vibration limits suggested above and the approach distances should be modified on the basis of a planned vibration monitoring trial prior to the start of bulk excavation. Also, it should be noted that humans are very sensitive to vibrations, even at levels that are considered inconsequential for buildings and utilities. It would therefore be beneficial to give ample notice to the occupants of the Opera House of any construction work likely to produce significant vibrations or noise.

8.2.3 Unsupported Excavation Batters

Based on the expected space constraints for the development, it is anticipated that broad, free-standing batter slopes within the filling material will not be appropriate. Shoring systems comprising secant pile walls or grouted/stabilised soils will probably be required, as discussed in Section 8.5.

Where localised temporary batter slopes can be adopted during bulk excavation, a maximum batter slope of 1.5 (H):1(V) may be adopted within the filling and overburden materials, provided that the batter is unaffected by groundwater or seepage.

Excavation within the medium strength (Class III) or better sandstone should generally be self supporting and may be cut vertically, provided that progressive geological inspection confirms the absence of adverse jointing that could give rise to potential instabilities such as sliding wedges or block fallout.

All vertical rock faces should be progressively inspected by a geotechnical engineer, at maximum 2 m and final excavation depth intervals, to check for adversely inclined joints and to assess whether additional stabilisation measures are required. Stabilisation of vertical rock faces may include shotcrete of fractured or highly weathered zones or rock-bolts and tensioned ground anchors where adverse joints form potentially unstable wedges of rock.

8.3 Preliminary Waste Classification Assessment

Off-site disposal of spoil generated during excavation works will generally require assessment for classification in accordance with current Department of Environment & Climate Change NSW (DECC) *Waste Classification Guidelines* (2008), updated July 2009. The guidelines outline the following six-step process for waste classification:

- Establish if the waste is ‘special waste’.

- Establish if the waste is 'liquid waste'.
- Establish if the waste is 'pre-classified' by the EPA.
- Establish if the waste possesses hazardous characteristics.
- Determine the contaminant concentrations of the waste.
- Establish if the waste is putrescible.

Samples of the filling were taken during the drilling of the boreholes for chemical analyses as a part of the preliminary waste classification assessment. Over the depth of environmental sampling, the drilling augers and SPT equipment were decontaminated using a 3% solution of phosphate-free detergent (Decon 90) and distilled water prior to collecting each sample. Samples were stored in a cooled, insulated and sealed container prior to dispatch to an analytical laboratory.

The soils samples did not contain clinical waste or tyres and therefore the soils on the site need not be classified as special waste.

The samples analysed were not in liquid form and therefore are not liquid waste.

The DECC has pre-classified glass, plastic, rubber, bricks, concrete, building and demolition waste, and asphalt waste as general soil waste (non-putrescible). The soil samples did not contain any of the above listed materials and therefore the soils on the site need not be classified as pre-classified waste.

The samples analysed did not possess any obvious hazardous characteristics and could not be described as hazardous waste prior to chemical analysis. All samples analysed were assessed on a visual and tactile basis as being incapable of significant biological transformation and are therefore considered to be non-putrescible.

In addition to the six-step waste classification process, replicate soil samples collected in sealed zip-lock bags, were allowed to equilibrate under ambient temperatures before screening for Total Photoionisable Compounds (TOPIC) using a calibrated photoionisation detector (PID).

The PID provides an indication of the presence of volatile organic compounds in the soil. All samples recorded PID results of less than 1 ppm.

Four selected soil samples obtained during the field work, plus one replicate sample, were tested for a range of potential chemical contaminants with the aim of providing advice on waste classification. Detailed results of the laboratory analyses (by Envirolab Services Pty Ltd) are included in Appendix D. Summaries of the results for the basic screening analyses to determine specific contaminant concentrations are provided in Tables 6 to 8. Significant contaminant concentrations according to the Waste Classification Guidelines (criteria thresholds indicated for General and Restricted Solid Waste) are shaded.

Table 6 – Summary of TPH/BTEX Results

Sample/ Depth	TPH (mg/kg)	Benzene (mg/kg)	Toluene (mg/kg)	Ethylbenzene (mg/kg)	Xylene (mg/kg)
BH101/0.2	<275	<0.5	<0.5	<1	<2
BH101/1.5	120	<0.5	<0.5	<1	<2
BH102/0.45	140	<0.5	<0.5	<1	<2
BH102/1.0	<275	<0.5	<0.5	<1	<2
BD1*	<275	<0.5	<0.5	<1	<2
General Solid Waste [1]	10000	10	288	600	1000
Restricted Solid Waste [1]	40000	40	1152	2400	4000

Note: TPH = Total petroleum hydrocarbons ($C_{10} – C_{36}$)
* = replicate sample from BH102/1.0
[1] = thresholds without TCLP

Table 7 – Summary of Organic Compound Results

Sample/ Depth	Total PAHs (mg/kg)	B(a)p (mg/kg)	OCP (mg/kg)
BH101/0.2	<1.55	<0.05	<0.1
BH101/1.5	35.4	**3.5**	<0.1
BH102/0.45	41.1	**4.2**	<0.1
BH102/1.0	14.7	**1.3**	<0.1
BD1*	14.9	**1.4**	0.7
General Solid Waste [1]	200	0.8	50
Restricted Solid Waste [1]	800	3.2	50

Note: PAHs = Polycyclic aromatic hydrocarbons
B(a)p = Benzo(a)pyrene
OCP = Organochlorine pesticides (those designated as Scheduled Chemicals)
* = replicate sample from BH102/1.0
[1] = thresholds without TCLP

Table 8 – Summary of Heavy Metal Results

Sample/ Depth	Arsenic (mg/kg)	Cadmium (mg/kg)	Chromium (mg/kg)	Copper (mg/kg)	Lead (mg/kg)	Mercury (mg/kg)	Nickel (mg/kg)	Zinc (mg/kg)
BH101/0.2	<4	<0.5	9	81	4	<0.1	**77**	41
BH101/1.5	<4	<0.5	25	63	54	<0.1	37	82
BH102/0.45	4	<0.5	10	41	70	1.6	11	43
BH102/1.0	<4	<0.5	13	22	25	0.8	7	17
BD1*	<4	<0.5	12	19	32	0.9	8	18
General [1]	100	20	100	NA	100	4	40	NA
Restricted [1]	400	80	400	NA	400	16	160	NA

Notes: * = replicate sample from BH102/1.0
1 = thresholds without TCLP
NA = Not applicable

On the basis of the above results, all four of the samples were selected for analyses by Toxicity Characteristics Leaching Procedure (TCLP). TCLP results are also included in Appendix D and are summarised in Table 9.

Table 9 – Summary of Leachability (TCLP) Results

Sample/ Depth	Nickel (mg/L)	Total PAH (mg/L)	B(a)P (mg/L)
BH101/0.2	0.1	-	-
BH101/1.5	-	0.004	<0.001
BH102/0.45	-	0.004	<0.001
BH102/1.0	-	0.005	<0.001
General Solid Waste [2]	2	NA	0.04
Restricted Solid Waste [2]	8	NA	0.16

Notes: PAHs = Polycyclic aromatic hydrocarbons;
B(a)p = Benzo(a)pyrene
[2] = threshold with TCLP
NA = Not applicable

Although it is noted that holding times for analysis of the B(a)P leachate were slightly exceeded, given that the results for total concentrations and that all results for B(a)P in leachate were below detection limits (Table 9), B(a)P concentrations are considered to be low.

The laboratory analysis recorded results for asbestos below detection limits and did not detect respirable fibres in any of the samples analysed. This is consistent with field observations.

The specific contaminant and leachable concentrations summarised in Tables 6 to 9 were compared to the contaminant threshold criteria provided in the Waste Classification Guidelines (2008). All results where within the general solid waste thresholds. Based on the visual observations and chemical analysis the results indicate that the filling material could be classified as *General Solid Waste (non-putrescible).*

Importantly it should be noted that given the limited nature of the assessment and that the classification of excavated materials could vary, further assessment should be undertaken during excavation to ensure an appropriate classification is provided for all materials requiring off-site disposal.

Based on the field observations of the deeper filling and visual inspection of cores of the underlying natural sandstone at this site, it is likely that the sandstone bedrock material could be classified as Virgin Excavated Natural Material (VENM). Confirmation of this classification however, should be undertaken through inspection and possible chemical analysis (depending on field observations) once the natural sandstone is exposed during bulk excavation works.

Reference should be made to both Section 9 of this report and the Notes Relating to this Report included in Appendix B, which provide important information about the limitations of this report and how it may be used.

8.4 Groundwater and Site Dewatering

The proposed excavation will extend to at least 10 m below the sea level in the adjacent harbour. Most of the planned excavation, however, will be wholly within sandstone bedrock, with the only area where soils (or filing) extend significantly below the sea level likely to be the proposed turning bay.

It is expected that the preferred design scheme for the loading dock is a "drained basement" with a system of sub-floor drains feeding to sumps, serviced by activated pumps. Without any specific building weight to hold the loading dock structure down, a "fully-tanked", watertight basement is unlikely to be feasible. A tanked basement would generally require a comprehensive system of tension piles or anchors to hold down the basement floor slab and overall structure.

The viability of a "drained basement" system will largely depend on the rate of groundwater flow up through the floor of the excavation, through any defects in the vertical rock cuts and below the toe (and through) of any "cut-off" walls around the perimeter (e.g. at the turning bay). If the rate of seepage inflow (or upflow) is too great and cannot be reliably sealed via pressure grouting or other method, it may be necessary to adopt a "fully-tanked" basement system.

Most of the basement excavation projects around the foreshores of Sydney Harbour have experienced seepage upflows of between 3 and 30 litres/m^2/day, but in some instances upflows in the order of 100 litres/m^2/day have been recorded. The rate of groundwater inflow up through the floor of the excavation will depend on the presence of major jointing, faulting and other defects, particularly where such defects provide a hydraulic connection to the harbour.

Although there are no clear indications of major faulting or through-going defects based on the current and previous borehole logs, the low-angle thrust faulting inferred from the rock core at BH102 may be associated with a strike-slip fault, such as the GPO Fault Zone. Elsewhere, the

GPO Fault has been associated with highly broken and relatively permeable rock. Therefore, given the possible presence of faulting and that there will be a hydraulic head of about 10 m, in close proximity to the harbour, the likelihood of experiencing large rates of inflow/upflow is considered to relatively high.

The estimation of likely groundwater inflows would generally be aided by a comprehensive programme of rock mass permeability testing. Also, the drilling of some inclined boreholes would be appropriate, to assess if the GPO Fault Zone is likely to traverse the proposed excavation.

The other aspect to the seepage inflow is the water quality and composition. It is likely to be highly saline (essentially sea water) with potential problems associated with salt intrusion into the concrete (as experienced at Bennelong Apartments) and is likely to have relatively high dissolved iron and manganese content. The later two elements will lead to significant precipitation of red-brown iron-oxide sludge in the drainage system and a requirement to address the water quality being discharged. If too much soluble iron remains within the discharged water, it can cause discolouration of the receiving water body with resulting possible heavy fines from the EPA. Aeration of the water is often the cheapest option for removal of the iron, but it is expected that the space constraints of the Opera House precinct would make this type of treatment a difficult proposition.

8.5 Excavation Support

8.5.1 General

It is understood that a "top-down" construction sequence is proposed for the main loading dock excavation due to both the presence of the post-tensioned cable beam system below the driveway pavement surface and height constraints in this area. The height limitations beneath the Monumental Steps (about 3.6 m) will preclude the use of most conventional piling rigs that could be used to build a perimeter shoring wall for the main excavation.

To the east of the Monumental Steps, at the proposed truck turning bay, the above constraints do not apply and a conventional secant pile or diaphragm wall shoring system could be used. The deeper filling encountered at borehole BH101 extended to a few metres below the sea level in the harbour and the inferred groundwater level. The shoring wall should be taken through the filling and any other overburden materials and keyed into medium strength, or better sandstone. This should limit the amount of groundwater seepage into the basement excavation to manageable levels. Given the water loss and the small plume of drilling water in the harbour, adjacent to borehole BH101, it may be necessary to extend the pile/diaphragm wall a few metres into the sandstone to reduce the amount of seepage directly below the toe of the wall.

A secant pile system involves the use of intersecting or overlapping piles constructed using continuous flight auger (CFA), concrete-injection techniques. Piles may be either all of full strength ("hard-hard") or alternating full strength piles with low strength bentonite-cement or sand-cement piles in between ("hard-soft"). In the later case the "soft" piles are typically of 5 to 10 MPa strength to enable easier drilling adjacent piles. Usually only the alternate, "primary" piles will have steel reinforcement and the "soft" piles should only be considered as temporary as they typically allow some leakage through the wall over the longer-term. Several recent projects involving basement construction around Sydney Harbour foreshore have used jet-grouting to form soft secondary "piles" in conjunction with CFA, concrete-injected (primary) piles.

Piling rigs used to construct shoring walls should be capable of penetrating medium and high strength sandstone bedrock and large boulders or blocks of rock, as indicated by some of the bore logs along the eastern side of the Opera House.

Internal propping using struts or one or more rows of ground anchors tied into waling beams will generally be required, particularly where the control of ground movement behind the wall is required.

Beneath the Monumental Steps, Arup have proposed a "top-down" construction method, after a system of strutting or bracing beams is installed across the main excavation footprint (ref. Arup meeting 8/2/10). It is understood that the bracing beams will be of reinforced concrete construction in the order of 1.6 m deep and supported on a system of rock pillars and piles. Further, it is understood that while the overburden materials will require support, Arup envisage that the underlying sandstone will be able to be cut vertically, with only localised support necessary. Specific geotechnical assessment of the integrity of any rock pillars will be required and depending on the dimensions and loading acting on the pillars, rock-cored boreholes to the full depth of the excavation may be required at or close to each one, to allow proper planning and design.

The above scheme is considered feasible although it will be necessary to stabilise any filling, soil overburden or weak rock that could collapse if excavated vertically. Stabilisation of the existing filling materials, which is described as predominantly sands and gravels, could be achieved using jet-grouting or possibly permeation grouting methods. Jet-grouting involves the use of very high pressures to mix cement grout into the soil matrix to form a composite, cement-stabilised soil columns or panels of 0.6 – 1 m width. Permeation grouting involves the use of microfine cements pumped into the soil matrix at relatively low pressures. There are often concerns about the capacity of grout-stabilised soils and therefore some form of temporary lateral restraint may be required (e.g. passive “tie-back” dowel bars or nominally tensioned anchors).

It may be necessary to underpin any existing foundations supported on marginal rock or on soil materials. Also, there are some key footings that will require detailed consideration, such as those supporting the Monumental Steps. Some options for underpinning are described in the following sections.

8.5.2 Design Earth-Pressures

Excavations braced either temporarily or permanently will be subjected to earth pressures from the ground surface down to the top of the Class III Sandstone. Below this level the sandstone was generally medium to and high and high strength and should be largely self-supporting, except for localised rockbolting of unstable rock wedges.

The following active earth pressures and bulk unit wights are recommended for the design of multi-anchored or propped walls and also for temporary mass-grouted gravity walls formed by jet-grouting or similar:

Table 10 – Recommended Active Earth Pressure Coefficients and Bulk Unit Weights

Material	Active Earth Pressure Coefficient		Bulk Unit Weight (kN/m^3)
	Short Term/Temporary	Long Term/Temporary	
Filling or alluvial soils	0.3	0.4	20
Class IV Sandstone	0.1	0.15	22
Class III (or better) Sandstone	0.0	0.0	24

Preliminary design of anchored shoring walls or basement walls rigidly braced by floor slabs to support soils or weak rock material may be based on a uniform rectangular pressure distribution, extending from the ground surface down to the top of the Class III Sandstone, of:-

$$\mathbf{P_z = 0.8K_a\gamma H}$$

where: P_z = active earth pressure at depth z below ground surface
K_a = active earth pressure coefficient (refer Table 10)
γ = unit weight of soil or weak rock (refer Table 10)
H = depth below surface to top of Class III Sandstone

The above pressure coefficients assume a level ground surface behind the top of the wall.

Note that a triangular pressure distribution would generally apply to any gravity-type wall or cantilevered system.

Additional allowance should be made for the effects of building or structure loads on the wall, as well as any short-term surcharges such as construction plant or vehicles operating behind the top of the wall. Where the footings for the adjacent building are located directly behind the retained height, such as the Monumental Steps along the southern side of the excavation, the coefficient of earth pressure at rest (K_0) should be adopted for wall design, instead of the above active earth pressures. K_0 values may be taken as 50% greater than the above coefficients.

On the basis of the limited groundwater monitoring carried out it is recommended that design of the shoring walls should allow for near full hydrostatic pressure behind the walls. That is, the wall designer should assume that groundwater will rise to say, RL +3.0 m (AHD). Wall design for lateral earth pressure should be based on the submerged unit weight of the retained materials (generally 8 – 10 kN/m^3 less than the values shown in Table 10) and the water pressures should be calculated separately and added to the earth pressures.

8.5.3 Ground Anchors

The use of "tie-back" ground anchors for the project is expected to be limited to the piled shoring wall around the proposed truck turning bay and possibly for jet-grout stabilised soils at the top of the main loading dock excavation.

Rock anchors should be inclined below the horizontal, as steeply as possible, to allow anchorage into the stronger sandstone materials at depth. The design of temporary or permanent ground anchors for the support of piles or jet-grouted wall systems may be carried out on the basis of the maximum allowable average bond stresses given in Table 11.

Table 11 – Allowable Average Bond Stresses for Anchor Design

Material Description	Maximum Allowable Average Bond Stress (kPa)
Class V & IV Sandstone	100
Class III Sandstone	500
Class II Sandstone (or better)	800

Ground anchors should be designed to have a free length equal to their height above the base of the excavation (minimum 3 m bond length) and after installation they should be proof loaded to 125% of the design Working Load and locked-off at no higher than 60% of the Working Load. For ground anchors retaining wall systems adjacent to existing structures, lock-off values should be at least 90% of the design Working Load. Periodic checks should also be carried out throughout the construction phase to ensure that the lock-off load is maintained and not lost due to creep effects or other causes.

The parameters given above assume that anchor holes are clean and adequately flushed, with grouting and other installation procedures carried out carefully and in accordance with normal good anchoring practice.

In normal circumstances the basement structure, including the ground floor slab, will restrain the excavation over the long-term and therefore ground anchors are expected to be temporary only. The use of permanent anchors or rock-bolts (at the toes of the shoring piles) would generally require careful attention to corrosion protection. Further advice on design and specification should be sought if permanent anchors are to be employed at this site.

8.5.4 Ground Movement Due to Stress Relief

For relatively major excavations in rock, there is a possibility that there will be some slight horizontal movement due to stress relief effects. It is unlikely to be practicable to provide restraint for the relatively high in-situ horizontal stresses anticipated within the Hawkesbury Sandstone. Release of these stresses due to the excavation will generally cause horizontal movement along the rock bedding surfaces and partings.

Based on monitoring experience for excavations in the Sydney region, vertical cuts in Hawkesbury Sandstone may give rise to lateral movements in the order of 0.5 to 2.0 mm per metre depth of excavation on the adjoining ground surface (i.e. behind the top of the mid point along an excavation face). Empirical data suggest that most of the movement occurs during or shortly after the bulk excavation phase.

Given the proposed excavation depth of 15 m, lateral movements at the surface due to stress relief alone, could be in the range 8 – 30 mm.

Stress relief related movements could cause damage to the existing Opera House building where it is supported on footings founded at shallow depth, behind the proposed excavation. Some minor damage due to stress relief is generally unavoidable. It is recommended that appropriate allowance be made for the repair of minor cracking pavements and public utilities, where excavation is carried out close to such structures. Again, with respect to existing buildings and surrounding structures, it is recommended that a dilapidation survey be carried out prior to excavation works so that an appropriate response may be made to damage claims.

Due consideration should also be given to the impact of possible broader stress relief effects on the existing Opera House carpark and its' associated network of access and service tunnels.

8.6 Tunnelling

It is understood that construction of the two service corridors under the main Opera House building will be carried out by tunnelling from the loading dock excavation. The interpreted subsurface profile for the eastern tunnel is shown on Cross-Section A-A' in Drawing 3 (Appendix A). The available borehole information indicates that the tunnel excavation will be predominantly within high strength (Class I and II) sandstone. Also, the profile above the proposed tunnels is expected to comprise 5 m to 6 m of Class II and III Sandstone, below between 1 m and 3 m of overburden materials (mainly soil filling) directly beneath the Ground Floor Level. As shown in Drawing 3 the roof of the two tunnels will be about 5 m below the groundwater table and the water level in the adjacent harbour.

In normal circumstances, tunnelling is only undertaken where the rock cover thickness is at least half the proposed tunnel width. The rock above the tunnel roof acts as a beam and as such, it must be relatively competent. The degree of settlement that occurs at the ground surface largely depends of the intact strength of the rock and the amount of defects present, such as joints and bedding partings.

Based on the proposed tunnel dimensions (height = 6.5 m and width = 6 to 8 m) and the thickness and (reasonable) quality of sandstone expected to form the roof beams, surface settlements in the order of 20 – 40 mm would be expected for open ground. The additional dead loading of the existing Opera House building is expected to significantly increase the expected settlements caused by the tunnelling work. This level of ground movement is likely to be unacceptable for the existing building. Specific numerical modelling and analyses of expected deformations above and around the tunnel will be required once the details of the proposed service corridor tunnels are confirmed. It would be prudent to undertake further cored borehole drilling along the alignment of the proposed tunnels, so as to improve the geotechnical model to be used for the tunnel analysis and design.

With respect to construction, it is anticipated that either a full-face or "top-heading" (i.e. benched) excavation methodology will be employed. This will largely depend on the size of the road-header tunnelling plant to be used. If a "top-heading" approach is adopted, then the excavation of the tunnel would probably involve leaving a small bench of say, 1.5 m height at the bottom of the advancing tunnel face. This bench would subsequently be removed, after the installation of the required roof support.

Preliminary estimates of the roof support requirements include patterned bolting of the roof of the tunnels with tensioned rock bolts at 1.5 – 3 m spacings. The rock bolts would be at least 3 m in length and would include corrosion protection to provide permanent support, with a reinforced shotcrete coverage over the full roof span.

As for the areas of open excavation, the rate of groundwater inflow to the tunnels will be of critical importance. Further investigation of this aspect will be necessary to better assess dewatering and construction options of the proposed tunnels.

8.7 Underpinning of Existing Structures

It may be necessary to carry out underpinning to support existing foundations, utilities and structures located close to the proposed excavation. Options for underpinning will generally depend on the stratum beneath the structure requiring support. For example, where the material is predominantly granular soils or filling, underpinning could be achieved using jet-grouting techniques. This method is particularly useful where the material to be stabilised extends below the groundwater table.

Where the stratum supporting the existing structure is weathered rock, underpinning could be carried out using a system of “hit-and-miss” panels. The panel depths, to reach the competent sandstone, would generally be limited to 1.5 – 2 m. Panel widths would normally be limited to about 1 – 1.5 m, depending on the existing footing loads, width and the strength of the founding stratum. This style of approach may be necessary to underpin the existing footings supporting the Monumental Steps and the main Opera House building, in the area of the Stage Door entrance. It will depend on the condition of the sandstone upon which the existing footings are supported.

The extent of underpinning that is likely to be required should be the subject of further investigations, via test pits excavated to expose some of the key existing footings around the proposed loading dock excavation. Given the extent of excavation proposed, some amount of damage to the existing Opera House building is considered unavoidable, due to the effects of stress relief, vibrations associated with rock excavation, tunnelling works and possibly some settlement behind the shoring/and bracing systems proposed. Therefore, some provision should be made for repair work to the existing structure, most of which should hopefully be relatively minor.

8.8 Foundations

Based on the architectural drawings there is no system of footings shown to support any form of basement or building superstructure within the proposed loading dock or service tunnels. There are, however, likely to be some footings associated with the proposed lift pits, stair wells and any internal walls.

The borehole logs and the inferred geotechnical cross-section presented in Drawing 3 (Appendix A) indicate that high strength (Class I or II) sandstone will be exposed at the bulk excavation level (BEL). It is therefore expected that shallow footings (e.g. pads or strips) would be appropriate for the support of footings associated with the above structures or any possible column/piles for the proposed bracing scheme proposed by Arup (ref. meeting 8/2/10).

Recommended maximum allowable parameters for the design of shallow footings (e.g. pads or strips) or rock socketed bored piles are presented in Table 12.

Table 12 – Recommended Maximum Parameters for Foundation Design

Material Description[1]	Maximum Allowable Pressures (kPa)	
	End Bearing	Shaft Adhesion[2]
Class II Sandstone	6000	600
Class I Sandstone	8000	800

Notes:
(1) Classification based on References 1 & 2.
(2) Shaft adhesion applies only for the design of rock socketed piles and depends on the degree of sidewall roughening achieved.

The foundation design parameters presented in Table 12 assume that the foundation excavations are clean and free of loose debris, with pile sockets (i.e. shafts) free of smear and adequately rough prior to concrete placement.

All load bearing foundations (e.g. pads, piles) should be assessed by an experienced geotechnical engineer or engineering geologist. Spoon hole tests will be required in at least one-third of the shallow footings, where they are proportioned for an allowable end bearing pressure of between 3500 kPa and 6000 kPa, increasing to 100% of the footings where a bearing pressure of greater than 6000 kPa is used. The purpose of spoon hole testing is to check that no significant weak seams exist within a depth of 1.5 times the least footing dimension below the foundation level.

8.9 Further Geotechnical Investigation

Further investigation will be required to provide more information on the geotechnical model, so as to address the key aspects of the VAPS project. Some of the main components of further geotechnical investigation are summarised as follows:

- Borehole investigations in the Forecourt area to provide information on the subsurface profile along the alignment of the proposed vehicle access tunnel and for the planned stormwater diversion.
- Additional boreholes in the area of the main loading dock excavation and the proposed service corridor tunnels. The purpose of these boreholes is to provide a better geological model of the subsurface profile, for design and planning purposes. At least some of the boreholes should be inclined, so as to maximise the chances of encountering the (GPO) strike-slip fault, reported to possibly traverse Bennelong Point in the vicinity of the site.
- Rock-mass permeability (i.e. “packer”) testing in the sandstone bedrock, to provide a basis for better estimating likely inflow rates for both the open-cut and tunnel excavations.
- Laboratory chemical analysis of the water quality, with particular emphasis on the soluble iron and manganese content of the groundwater, together with the total dissolved solids (TDS) content. Also, testing for pH, chloride and sulphate ion content, and electrical conductivity (EC) testing should be carried out to provide a basis for an assessment of the aggressivity of the groundwater at the site (likely to be highly saline and equivalent to sea water).

- Test pits to expose the existing footings and founding conditions for the existing building, including the Monumental Steps. Also, it would be beneficial to expose the post-tensioned cable beams beneath the Ground Floor driveway pavement. This would provide the dimensions and locations of the cable beams and also confirm annedotal evidence of a continuous ground slab either in between or on top of the beams.
- Numerical modelling of the overall VAPS project to assess the likely impact of the proposed excavations on existing openings and tunnels in the vicinity (e.g. Opera House Carpark and associated tunnels, Sydney Harbour Tunnel).

9. LIMITATIONS OF THIS REPORT

DP has prepared this report for this project at the Sydney Opera House, Bennelong Point as per DP's proposal dated 2 December 2009 and acceptance from Savills on behalf of the Sydney Opera House Trust (SOHT) dated 9 December 2009. This report is provided for the exclusive use of Savills and SOHT for the specific project and purpose outlined. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party.

The testing methods adopted are indicative of the site's sub-surface conditions to the depths penetrated at the specific sampling and/or testing locations in this investigation, and only at the time the work was carried out. The accuracy of geotechnical engineering advice provided in this report may be limited by unobserved variations in ground conditions across the site in areas between and beyond test locations and by any restrictions in the sampling and testing which was able to be carried out, as well as by the amount of data that could be collected given the project and site constraints. These factors may lead to the possibility that actual ground conditions and materials behaviour observed at the test locations may differ from those which may be encountered elsewhere on the site. Should such variations in subsurface conditions subsequently be encountered, then additional advice should be sought from DP.

This report must be read in conjunction with the attached "Notes Relating to This Report" and any other attached explanatory notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions from others' review of this report or test data, which are not otherwise supported by an expressed statement, interpretation, outcome or conclusion stated in this report. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

DOUGLAS PARTNERS PTY LTD

Reviewed by

for

Torben Poirot
Engineering Geologist

Dr T J Wiesner
Principal

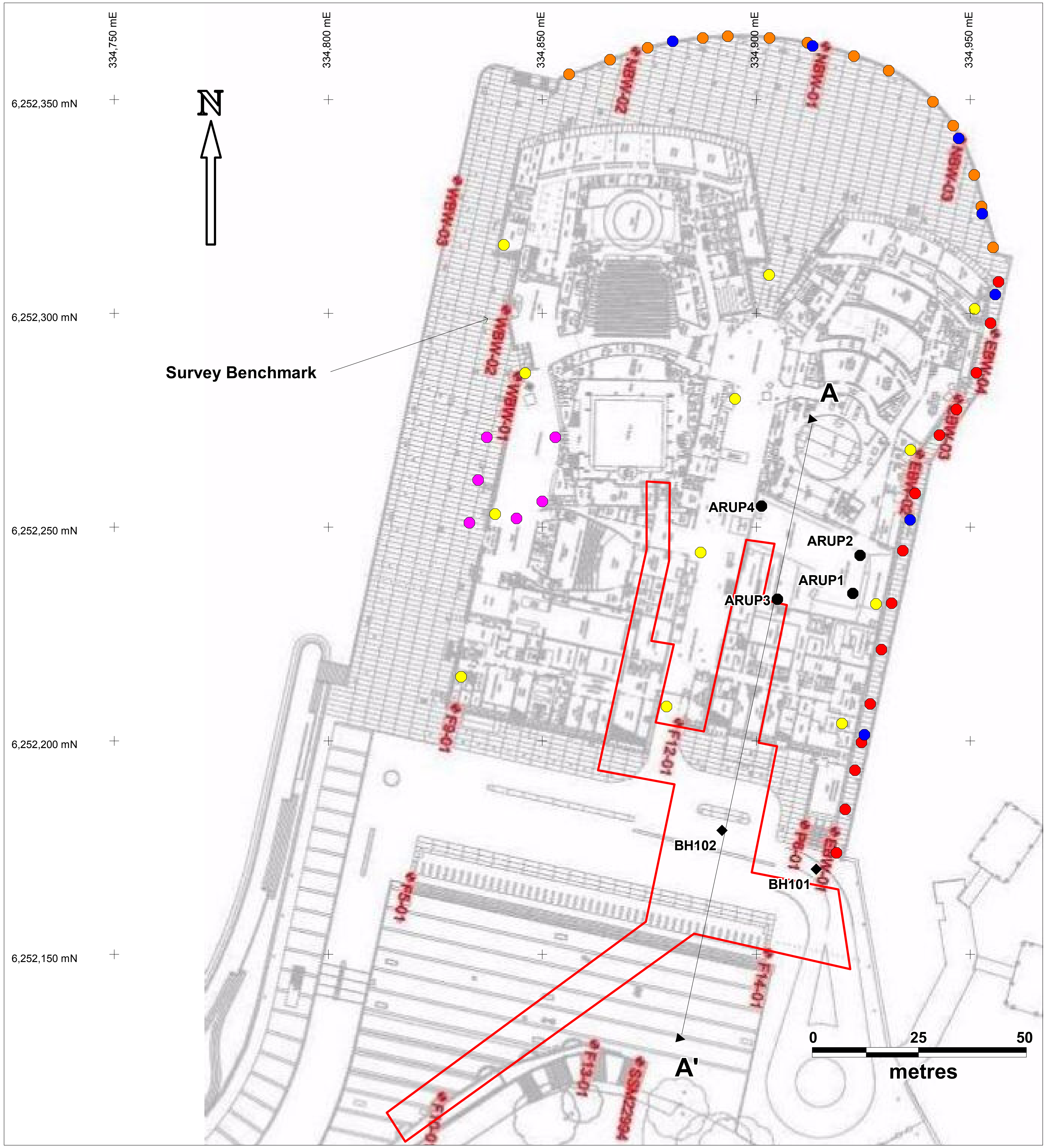
Bruce McPherson
Principal

References:

1. Pells, P. J., Mostyn, G. and Walker, B. F. *"Foundations on Sandstone and Shale in the Sydney Region"*. Australian Geomechanics Journal, Vol. No. 33 Part 3, Dec. 1998.
2. Pells, P. J. Douglas, D. J., Rodway, B., Thorne, C. and McMahon, B. K. *"Design Loadings for Foundations on Shale and Sandstone in the Sydney Region"*. Australian Geomechanics Journal, Vol. 3, 1978.
3. Braybrooke, J. C. *"The State of the Art of Rock Cuttability and Rippability Prediction"*. Fifth Australia – New Zealand Conference on Geomechanics. Sydney, 22-23 August, 1988.
4. AS 2187.2 – 1993 – *"Explosives Code"*.
5. Pells, P. J., Braybrooke, J. C. and Och, D. J. *"Map and Selected Details of Vertical Structural Features in the Sydney CBD"*. Australian Geomechanics Society, 2004.

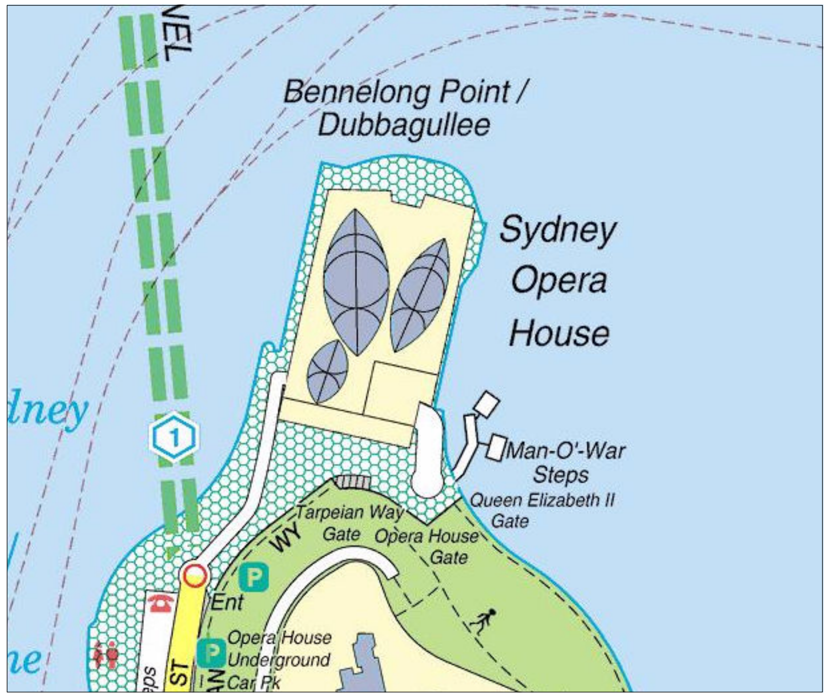
APPENDIX A

Drawing 1 – Borehole Location Plan
Drawing 2 – Interpreted Rock Surface Contour Plan
Drawing 3 – Inferred Geotechnical Cross-Section A-A’



Note 1: Drawing based on Drawing No. 110219034 by Hard & Forester Consulting Surveyors, dated 5/9/2005.

LOCALITY PLAN



LEGEND

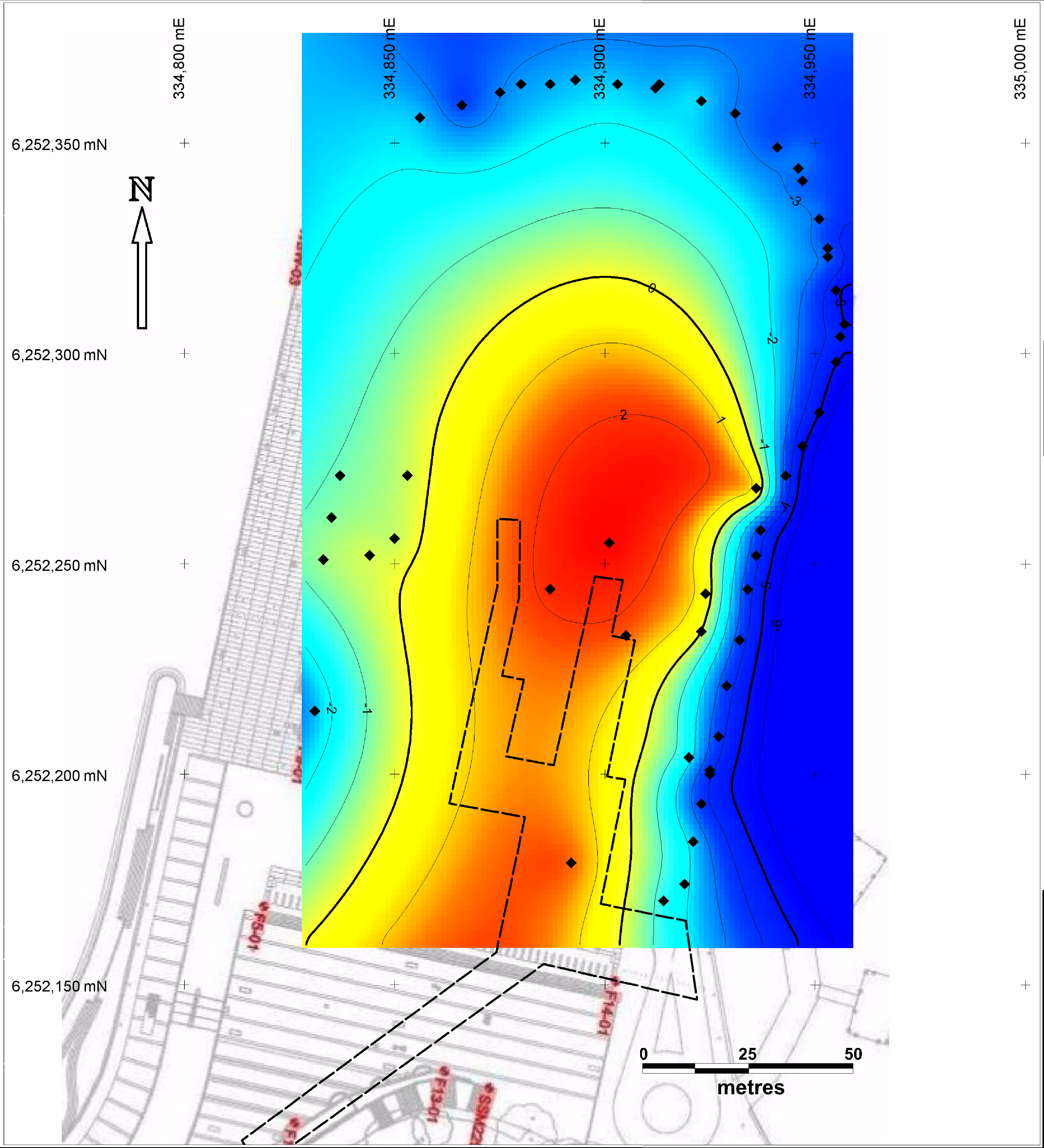
- ◆ Current Borehole by Douglas Partners
- ● Borehole by Douglas Partners (Northern Boardwalk 1995
- ● Borehole by Douglas Partners (Eastern Boardwalk 1995)
- ● Borehole by ARUP Geotechnics (2004)
- ● Borehole by Jeffery & Katauskas (1994)
- ● Borehole by MacDonald, Wagner & Priddle (1958)
- ● Mini piles by Douglas Partners (1998)
- Proposed VAPS Excavation Footprint
- Cross Section A-A'

Douglas Partners
Geotechnics . Environment . Groundwater

Sydney, Newcastle, Brisbane, Melbourne, Perth, Darwin, Wyong, Campbelltown, Canberra Townsville, Cairns, Wollongong

Borehole Location Plan (Current & Previous)
Vehicle & Pedestrian Safety (VAPS) Project
Sydney Opera House, Bennelong Point

CLIENT: Sydney Opera House Trust			
DRAWN BY: TP	SCALE: 1:1000	PROJECT No: 71529	OFFICE: Sydney
APPROVED BY: BJM		DATE: January 2010	DRAWING No: 1



Note 1: Drawing based on Drawing No. 110219034 by Hard & Forester Consulting Surveyors, date 5/9/2005.

Note 2: Rock surface contours are based on the Minimum Curvature function in the MapInfo/Discover Surfaces module and should be treated as approximate and accurate only at borehole locations.

Note 3: Contour intervals is 1 m and contours are Reduced Levels (RL) relative to Australian Height Datum (AHD).

LEGEND

Proposed VAPS Excavation Footprint

Contour Denoting Interpreted Top of Rock

Interpreted Top of Rock (RL;mAHD)

3
1.85
1.48
1.18
0.768
0.309 — 0
-0.136
-0.544
-0.926
-1.33
-1.77
-2.19
-2.59
-3.15
-3.88
-6.36
-6

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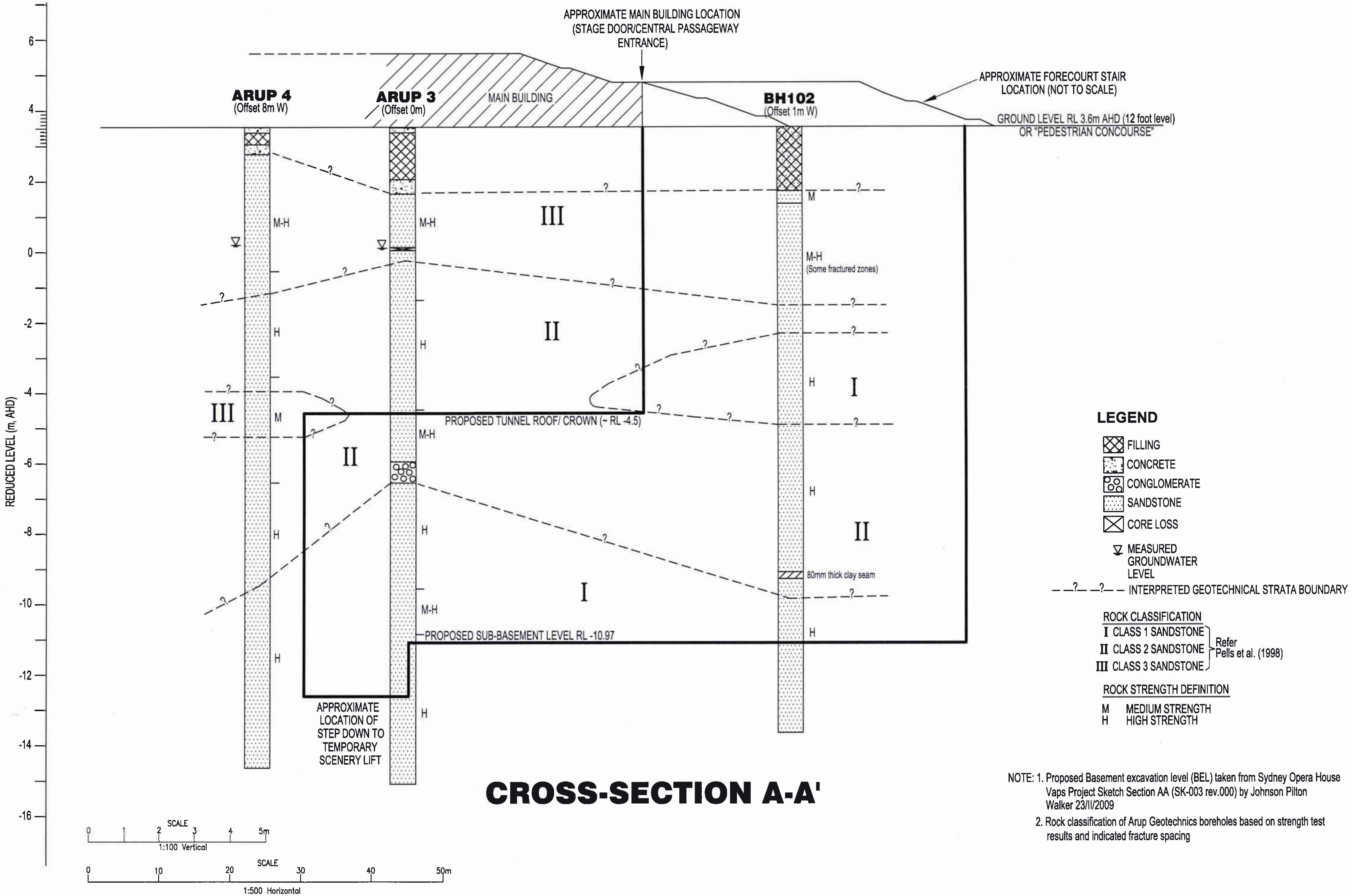
Sydney, Newcastle, Brisbane, Melbourne, Perth, Darwin, Wyong, Campbelltown, Canberra, Townsville, Cairns, Wollongong

Interpreted Rock Surface Contour Plan
Vehicle & Pedestrian Safety (VAPS) Project
Sydney Opera House, Bennelong Point

CLIENT: Sydney Opera House Trust			
DRAWN BY: TP	SCALE: 1:1000	PROJECT No: 71529	OFFICE: Sydney
APPROVED BY: BJM		DATE: January 2010	DRAWING No: 2



CROSS-SECTION A-A'

NOTE: 1. Proposed Basement excavation level (BEL) taken from Sydney Opera House Vaps Project Sketch Section AA (SK-003 rev.000) by Johnson Pilton Walker 23/II/2009

2. Rock classification of Arup Geotechnics boreholes based on strength test results and indicated fracture spacing

Douglas Partners *Geotechnics · Environment · Groundwater*	CLIENT: Sydney Opera House Trust	TITLE: **Interpreted Geotechnical Cross Section A-A'**	PROJECT No: 71529
	DRAWN BY: PSCH / SCALE: As shown / OFFICE: Sydney	**Vehicle & Pedestrian Safety (VAPS) Project**	DRAWING No: 3
	APPROVED BY: BJ McPherson / DATE: 19.2.2010	**Sydney Opera House, BENNELONG POINT**	REVISION: A

APPENDIX B

Notes Relating to this Report

Results of Field Testing

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NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00 mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300 mm)	CPT Cone Value (q_c — MPa)
Very loose	less than 5	less than 2
Loose	5—10	2—5
Medium dense	10—30	5—15
Dense	30—50	15—25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling are given in the report.

Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Test Pits — these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling — the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" — Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm

  as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil.

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6)\ N \text{ (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18)\ c_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms.

Bore Logs

The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than 'straight line' variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions — the potential for this will depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

Reproduction of Information for Contractual Purposes

Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

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DESCRIPTION AND CLASSIFICATION OF ROCKS FOR ENGINEERING PURPOSES

DEGREE OF WEATHERING

Term	Symbol	Definition
Extremely Weathered	EW	Rock substance affected by weathering to the extent that the rock exhibits soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.
Highly Weathered	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decreased compared to the fresh rock usually as a result of iron leaching or deposition. The colour and strength of the original fresh rock substance is no longer recognisable.
Moderately Weathered	MW	Rock substance affected by weathering to the extent that staining or discolouration of the rock substance usually by limonite has taken place. The colour of the fresh rock is no longer recognisable.
Slightly Weathered	SW	Rock substance affected by weathering to the extent that partial staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable.
Fresh Stained	Fs	Rock substance unaffected by weathering, but showing limonite staining along joints.
Fresh	Fr	Rock substance unaffected by weathering.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index ($I_{s(50)}$) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by Australian Standard 4133.4.1 - 1993.

Term	Symbol	Field Guide*	Point Load Index $I_{s(50)}$ MPa	Approx Unconfined Compressive Strength q_u ** MPa
Extremely low	EL	Easily remoulded by hand to a material with soil properties	<0.03	< 0.6
Very low	VL	Material crumbles under firm blows with sharp end of pick; can be peeled with a knife; too hard to cut a triaxial sample by hand. SPT will refuse. Pieces up to 3 cm thick can be broken by finger pressure.	0.03-0.1	0.6-2
Low	L	Easily scored with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150 mm long 40 mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.	0.1-0.3	2-6
Medium	M	Readily scored with a knife; a piece of core 150 mm long by 50 mm diameter can be broken by hand with difficulty.	0.3-1.0	6-20
High	H	Can be slightly scratched with a knife. A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken with pick with a single firm blow, rock rings under hammer.	1 - 3	20-60
Very high	VH	Cannot be scratched with a knife. Hand specimen breaks with pick after more than one blow, rock rings under hammer.	3 - 10	60-200
Extremely high	EH	Specimen requires many blows with geological pick to break through intact material, rock rings under hammer.	>10	> 200

Note that these terms refer to strength of rock material and not to the strength of the rock mass, which may be considerably weaker due to rock defects.

* The field guide assessment of rock strength may be used for preliminary assessment or when point load testing is not able to be done.

** The approximate unconfined compressive strength (q_u) shown in the table is based on an assumed ratio to the point load index of 20:1. This ratio may vary widely.

STRATIFICATION SPACING

Term	Separation of Stratification Planes
Thinly laminated	<6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	>2 m

DEGREE OF FRACTURING

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude known artificial fractures such as drilling breaks. The orientation of rock defects is measured as an angle relative to a plane perpendicular to the core axis. Note that where possible, recordings of the actual defect spacing or range of spacings is preferred to the general terms given below.

Term	Description
Fragmented	The core consists mainly of fragments with dimensions less than 20 mm.
Highly Fractured	Core lengths are generally less than 20 mm - 40 mm with occasional fragments.
Fractured	Core lengths are mainly 40 mm - 200 mm with occasional shorter and longer sections.
Slightly Fractured	Core lengths are generally 200 mm - 1000 mm with occasional shorter and longer sections.
Unbroken	The core does not contain any fracture.

ROCK QUALITY DESIGNATION (RQD)

This is defined as the ratio of sound (i.e. low strength or better) core in lengths of greater than 100 mm to the total length of the core, expressed in percent. If the core is broken by handling or by the drilling process (i.e. the fracture surfaces are fresh, irregular breaks rather than joint surfaces) the fresh broken pieces are fitted together and counted as one piece.

SEDIMENTARY ROCK TYPES

This classification system provides a standardised terminology for the engineering description of sandstone and shales, particularly in the Sydney area, but the terms and definitions may be used elsewhere when applicable.

Rock Type	Definition
Conglomerate	More than 50% of the rock consists of gravel-sized (greater than 2 mm) fragments
Sandstone:	More than 50% of the rock consists of sand-sized (0.06 to 2 mm) grains
Siltstone:	More than 50% of the rock consists of silt-sized (less than 0.06 mm) granular particles and the rock is not laminated.
Claystone:	More than 50% of the rock consists of clay or sericitic material and the rock is not laminated.
Shale:	More than 50% of the rock consists of silt or clay-sized particles and the rock is laminated.

Rocks possessing characteristics of two groups are described by their predominant particle size with reference also to the minor constituents, eg. clayey sandstone, sandy shale.

GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL

- BITUMINOUS CONCRETE
- CONCRETE
- TOPSOIL
- FILLING
- PEAT
- CLAY
- SILTY CLAY
- SILT
- SANDY CLAY
- GRAVELLY CLAY
- SHALY CLAY
- CLAYEY SILT
- SANDY SILT
- SAND
- CLAYEY SAND
- SILTY SAND
- GRAVEL
- SANDY GRAVEL
- COBBLES/BOULDER
- TALUS

SEDIMENTARY ROCK

- BOULDER CONGLOMERATE
- CONGLOMERATE
- CONGLOMERATIC SANDSTONE
- SANDSTONE FINE GRAINED
- SANDSTONE COARSE GRAINED
- SILTSTONE
- LAMINITE
- MUDSTONE, CLAYSTONE, SHALE
- COAL
- LIMESTONE

SEAMS

- SEAM >10mm
- SEAM <10mm

METAMORPHIC ROCK

- SLATE, PHYLLITE, SCHIST
- GNEISS
- QUARTZITE

IGNEOUS ROCK

- GRANITE
- DOLERITE, BASALT
- TUFF
- PORPHYRY

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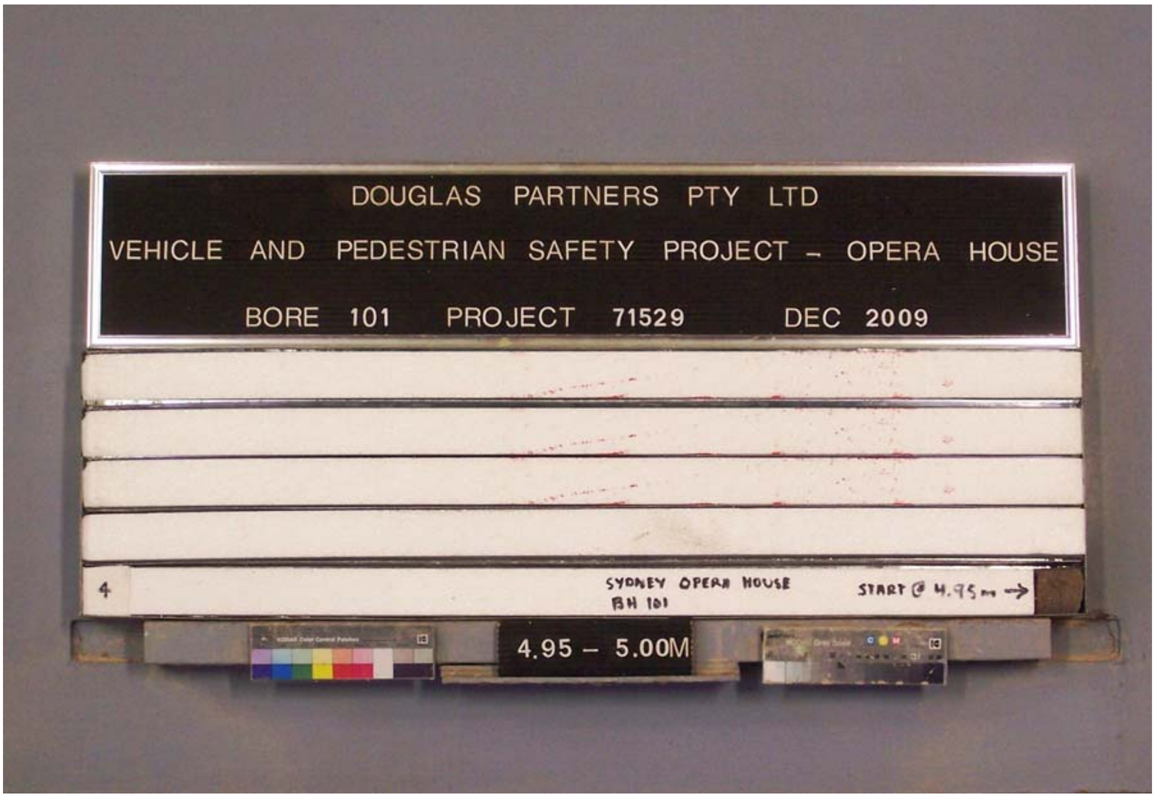
DOUGLAS PARTNERS PTY LTD
VEHICLE AND PEDESTRIAN SAFETY PROJECT - OPERA HOUSE
BORE 101 PROJECT 71529 DEC 2009
4
SYDNEY OPERA HOUSE
BH 101
START @ 4.95m
4.95 - 5.00M


DOUGLAS PARTNERS PTY LTD
VEHICLE AND PEDESTRIAN SAFETY PROJECT - OPERA HOUSE
BORE 101 PROJECT 71529 DEC 2009
5
6
7
8
9
5.00 - 10.00M

BOREHOLE LOG

CLIENT: Sydney Opera House Trust
PROJECT: Vehicle & Pedestrian Safety (VAPS) Project
LOCATION: Bennelong Point

SURFACE LEVEL: 3.5 m AHD*
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 71529
DATE: 17 Dec 09
SHEET 1 OF 2

Depth (m)	Description of Strata	Water	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
0.13	ASPHALT - 130mm thick						
0.2	ROADBASE - blue metal gravel and some sand			A			
	FILLING - grey sand filling, with some sandstone gravel, dry			A			PID<1ppm
				S			3,3,3 N = 6 30/150mm refusal
1.5	FILLING - grey sand filling, with some sandstone gravel and blue metal gravel and cobbles (ballast)			A			PID<1ppm
2.0	FILLING - bluemetal gravel and cobbles (ballast)						Water level measured on 9/1/10 at 2.48m; Water level measured on 12/1/10 at 3.15m
4.1	FILLING - loose, black, medium grained sand filling with some clayey silt, wet	17-12-09		A			PID<1ppm
4.95	SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey with yellow coating, medium to coarse grained sandstone		Note: Unless otherwise stated, bedding planes are planar and rough; 5.61m: B2°, clay veneer	C	100	100	PL(A) = 1.2MPa
5.8	5.8-7.63m: fine to medium grained sandstone		6.15m: B2°; 6.38m: B2°; 6.55m: D; 6.82m: D; 7.15m: B2°; 7.25m: B2°; 7.49m: B3°; 7.62m: B2°	C	100	100	PL(A) = 1.4MPa; PL(A) = 1.3MPa UCS=17.1MPa
7.63			9m: B4°; 9.28m: B2°	C	100	100	PL(A) = 1.4MPa; PL(A) = 3.2MPa
9.6 / 9.68	9.6-9.68m: very high strength siltstone band		9.68m: B4°, ironstained; 9.75m: B3°, clay smear				PL(A) = 1.2MPa

RIG: Multi-Drill **DRILLER:** Traccess **LOGGED:** PGH **CASING:** HW to 5.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 4.5m; Rotary to 4.95m; NMLC-Coring to 13.48m

WATER OBSERVATIONS: Free groundwater observed at 4.1m whilst augering (possibly sea water level). 80% water loss from approx 6.0m depth

REMARKS: Standpipe installed: Solid PVC 0.0-7.5m; Screen 7.5-13.5m. *Borehole surface level (approximate only) measured from SOBM-P601 and interpolated from survey plan (Sydney Opera House Survey Control Plan, Ground Floor + 12' External) by Hard & Forester

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: BJM
Date: 18/2/10

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VEHICLE AND PEDESTRIAN SAFETY PROJECT - OPERA HOUSE
BORE 101 PROJECT 71529 DEC 2009
10
11
12
13
14
10.00 - 13.45M

BOREHOLE LOG

CLIENT: Sydney Opera House Trust
PROJECT: Vehicle & Pedestrian Safety (VAPS) Project
LOCATION: Bennelong Point

SURFACE LEVEL: 3.5 m AHD*
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 71529
DATE: 17 Dec 09
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01 0.05 0.10 0.50 1.00)	Discontinuities (B - Bedding J - Joint S - Shear D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
-7		SANDSTONE - high strength, fresh, slightly fractured and unbroken, pale grey with yellow coating, medium to coarse grained sandstone *(continued)*	FR		High			10.37m: D				PL(A) = 1.5MPa
	11								C	100	100	
-8												
	12							11.95m: D				PL(A) = 1.5MPa
		- indistinct cross-beds at 12.20m										
-9									C	100	100	
	13							12.94m: D				PL(A) = 1.3MPa
		- low strength siltstone band			Low							
-10	13.48	Bore discontinued at 13.48m										PL(A) = 1.3MPa
	14											
-11												
	15											
-12												
	16											
-13												
	17											
-14												
	18											
-15												
	19											
-16												

RIG: Multi-Drill **DRILLER:** Traccess **LOGGED:** PGH **CASING:** HW to 5.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 4.5m; Rotary to 4.95m; NMLC-Coring to 13.48m

WATER OBSERVATIONS: Free groundwater observed at 4.1m whilst augering (possibly sea water level). 80% water loss from approx 6.0m depth

REMARKS: Standpipe installed: Solid PVC 0.0-7.5m; Screen 7.5-13.5m. *Borehole surface level (approximate only) measured from SOBM-P601 and interpolated from survey plan (Sydney Opera House Survey Control Plan, Ground Floor + 12' External) by Hard & Forester

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: BJM
Date: 18/2/10

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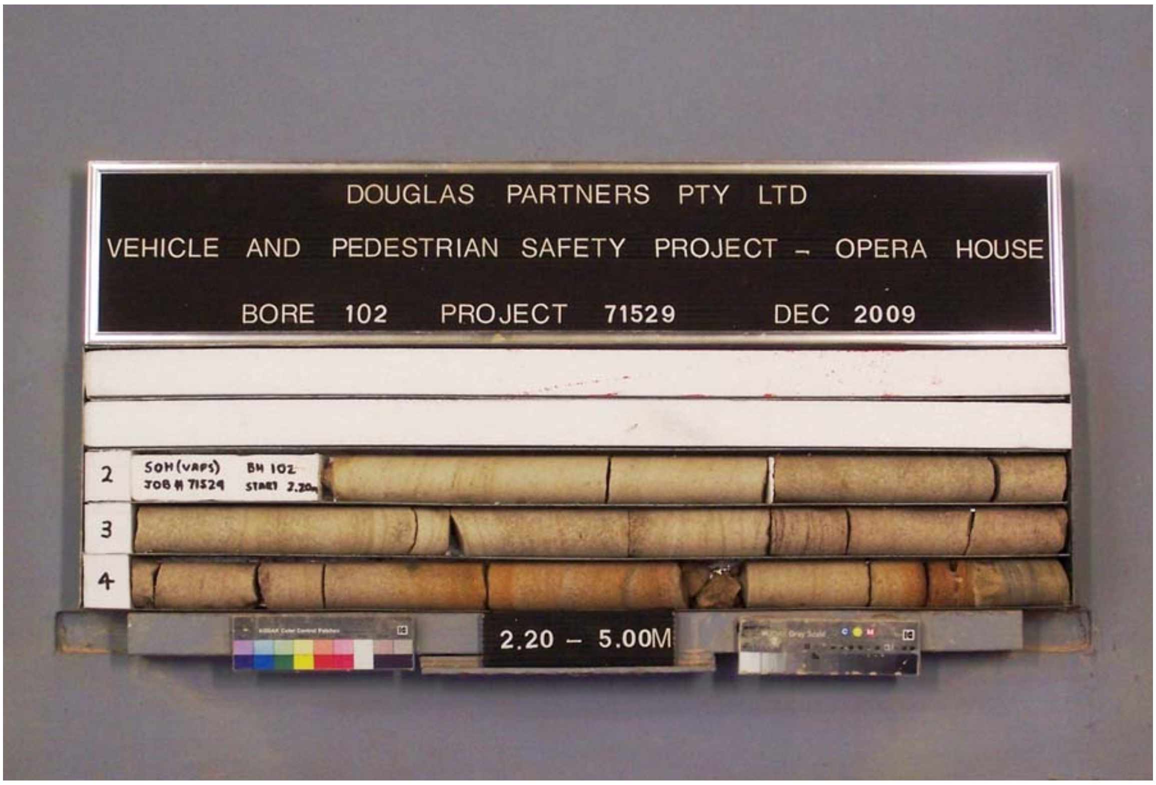
DOUGLAS PARTNERS PTY LTD
VEHICLE AND PEDESTRIAN SAFETY PROJECT - OPERA HOUSE
BORE 102 PROJECT 71529 DEC 2009
2.20 – 5.00M


DOUGLAS PARTNERS PTY LTD
VEHICLE AND PEDESTRIAN SAFETY PROJECT - OPERA HOUSE
BORE 102 PROJECT 71529 DEC 2009
5.00 – 10.00M

BOREHOLE LOG

CLIENT: Sydney Opera House Trust
PROJECT: Vehicle & Pedestrian Safety (VAPS) Project
LOCATION: Bennelong Point

SURFACE LEVEL: 3.6 m AHD*
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 71529
DATE: 20 Dec 09
SHEET 1 OF 2

Depth (m)	Description of Strata
0.075	ASPHALT - 70mm thick
	CONCRETE - 370mm thick
0.44	FILLING - sand filling with sandstone and blue metal gravel (ballast), dry
1.0	FILLING - sandstone filling, dry
1.4	FILLING - blue metal gravel filling (ballast)
1.8	SANDSTONE - medium strength, slightly to moderately weathered, white grey, medium grained sandstone
2.2	SANDSTONE - high strength, fresh then slightly weathered, slightly fractured and unbroken, light grey, medium to coarse grained sandstone
	4.25-4.95m: moderately weathered, fractured zone, 700mm
4.95	4.95-5.2m: high strength, laminite band, fresh
5.2	SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey with yellow coating, medium grained sandstone, medium bedded
	7.0-9.1m: possible 'yellow block' sandstone
	9.45-9.55m: carbonaceous laminations

Degree of Weathering: EW HW MW SW FS FR
Rock Strength: Ex Low, Very Low, Low, Medium, High, Very High, Ex High
Water
Fracture Spacing (m): 0.01 0.05 0.10 0.50 1.00

Discontinuities

B - Bedding J - Joint
S - Shear D - Drill Break

Note: Unless otherwise stated, all bedding planes are planar and rough

2.5m: D
2.9m: D
3.3m: B10°
3.34m: B10°
3.52m: B10°
3.76m: B2°, clay
4.03m: B5°
4.19m: D
4.2m: B2°
4.38m: B2°
4.59m: crushed zone, 2°, 60mm thick
4.78m: B2°
4.84m: B, ironstained
5.07m: B2°, 2mm clay
5.17m: B2°
5.41m: B0°
5.5m: B0°, carbonaceous laminations
5.85m: B6°
6.05m: B0°
6.75m: D
6.92m: D
7.15m: B1°
8.55m: B10°
8.94m: B1°
8.95m: B1°, smooth
9.3m: B1°
9.44m: B2°
9.52m: B2°
9.73m: B3°, carbonaceous infill

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
A			PID<1ppm
S			17,15,15 N = 30
A			PID<1ppm
(S)			10/40mm refusal
C	100	100	PL(A) = 1.7MPa
C	100	91	PL(A) = 2MPa PL(A) = 1.2MPa PL(A) = 1.1MPa
C	100	100	PL(A) = 1.9MPa PL(A) = 1.5MPa
C	100	96	PL(A) = 1.6MPa PL(A) = 1.9MPa
C	100	96	

RIG: Multi-Drill **DRILLER:** Traccess **LOGGED:** PGH **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger to 2.2m; NMLC-Coring to 17.11m
WATER OBSERVATIONS: No free groundwtaer observed whilst augering
REMARKS: (S) Indicates no SPT sample recovered. *Borehole surface level (approximate only) measured from SOBM-P601 and interpolated from survey plan (Sydney Opera House Survey Control Plan, Ground Floor + 12' External) by Hard & Forester

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: BJM
Date: 18/2/10

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VEHICLE AND PEDESTRIAN SAFETY PROJECT - OPERA HOUSE
BORE 102 PROJECT 71529 DEC 2009
10
11
12
13
14
10.00 - 15.00M


DOUGLAS PARTNERS PTY LTD
VEHICLE AND PEDESTRIAN SAFETY PROJECT - OPERA HOUSE
BORE 102 PROJECT 71529 DEC 2009
15
16
17
15.00 - 17.11M

BOREHOLE LOG

CLIENT: Sydney Opera House Trust
PROJECT: Vehicle & Pedestrian Safety (VAPS) Project
LOCATION: Bennelong Point

SURFACE LEVEL: 3.6 m AHD*
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 71529
DATE: 20 Dec 09
SHEET 2 OF 2

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
	SANDSTONE - high strength, fresh, slightly fractured and unbroken, light grey with yellow coating, medium grained sandstone, medium bedded *(continued)*		C	100	96	
11		10.41m: B5°, clay infill, 30mm thick 10.5m: D 10.74m: B2° 10.87m: B2° 11.1m: B2° 11.28m: D 11.45m: B5°	C	100	97	PL(A) = 1.6MPa PL(A) = 1.3MPa
12–13		11.62m: B2° 11.7m: D1° 11.8-11.90m: (x3) B1° 12.15m: B1° 12.62-12.70m: clay 80mm thick on possible thrust fault, 20°	C	100	87	PL(A) = 1.2MPa
14		13.52m: B5°, clay veneer 13.57m: B5°, clay veneer 14.05m: D 14.31m: D 14.58m: D	C	100	97	PL(A) = 1.1MPa PL(A) = 1.8MPa UCS=18.8MPa
15	15.28-17.11m: core undersized with numerous drill breaks (possible bent drill rod)	14.85m: D 15.15m: D 15.28-17.11m: D (x33)	C	100	100	PL(A) = 1.6MPa
16–17		16.58m: B4°	C	100	100	PL(A) = 1.3MPa
17.11	Bore discontinued at 17.11m					

RIG: Multi-Drill **DRILLER:** Traccess **LOGGED:** PGH **CASING:** HW to 2.5m

TYPE OF BORING: Solid flight auger to 2.2m; NMLC-Coring to 17.11m

WATER OBSERVATIONS: No free groundwtaer observed whilst augering

REMARKS: (S) Indicates no SPT sample recovered. *Borehole surface level (approximate only) measured from SOBM-P601 and interpolated from survey plan (Sydney Opera House Survey Control Plan, Ground Floor + 12' External) by Hard & Forester

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U,	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials: BJM
Date: 18/2/10

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APPENDIX C
Results of Laboratory Engineering Testing

Douglas Partners
Geotechnics · Environment · Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
Box 324
Hunter Region Mail Centre NSW 2310
Australia

15 Callistemon Close
Warabrook NSW 2304
Phone (02) 4960 9600
Fax: (02) 4960 9601
newcastle@douglaspartners.com.au

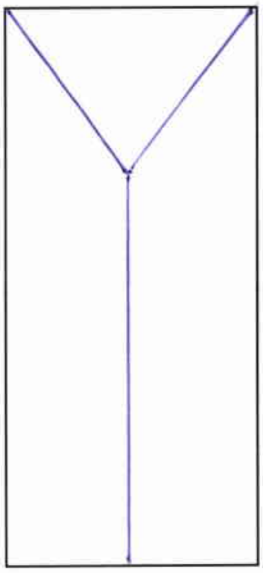
RESULTS OF UNIAXIAL COMPRESSIVE STRENGTH OF ROCK CORES

Client:	**Sydney Opera House Trust**	**Project No:**	71529
Project:	Vehicle and Pedestrian Safety Point	**Report No:**	N10-015
		Report Date:	22/1/2010
		Date Sampled:	-
Location:	Bennelong Point	**Page:**	1 of 1

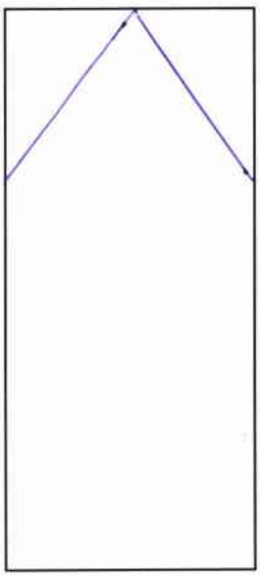
Unconfined Compressive Strength
Test Method AS 4133.4.2-1993

Bore Location		BH 101	BH 102		
Bore Depth	m	7.86-8.00	14.63-14.76		
Rock Description		SANDSTONE	SANDSTONE		
Storage History and Environment		Tested as Received			
Date of Testing		20.1.10	20.1.10		
Specimen Diameter	mm	51.5	51.3		
Specimen Height	mm	125	123		
Moisture Content (AS 4133.1.1.1-1993)	%	6.1	8.0		
Dry Mass Per Unit Volume	t/m³	2.32	2.22		
UNIAXIAL COMPRESSIVE STRENGTH	**MPa**	**17.1**	**18.8**		
Remarks					

Compression Machine: Autocon (1500kN) – Model CL10320



BH 101 (7.86-8.00m)



BH 101 (14.63-14.76m)

Approved Signatory:

Tested:	BB
Checked:	DM

D Millard
Laboratory Manager

APPENDIX D
Results of Laboratory Chemical Testing

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
enquiries@envirolabservices.com.au
www.envirolabservices.com.au

CERTIFICATE OF ANALYSIS 36506

Client:
Douglas Partners
96 Hermitage Rd
West Ryde
NSW 2114

Attention: Peter Hartcliff

Sample log in details:

Your Reference:	**71529, Sydney Opera House (VAPS)**
No. of samples:	5 Soils
Date samples received:	22/12/09
Date completed instructions received:	22/12/09

Analysis Details:
Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by:	6/01/10
Date of Preliminary Report:	Not Issued
Issue Date:	30/12/09

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This document is issued in accordance with NATA's accreditation requirements.
Accredited for compliance with ISO/IEC 17025.
Tests not covered by NATA are denoted with *.

Results Approved By:

Jacinta Hurst
Operations Manager

Joshua Lim
Chemist

vTPH & BTEX in Soil						
Our Reference:	UNITS	36506-1	36506-2	36506-3	36506-4	36506-5
Your Reference	-------------	BH101/0.2	BH101/1.5	BH102/0.45	BH102/1.0	BD/201 209
Date Sampled	------------	17/12/2009	17/12/2009	20/12/2009	20/12/2009	17/12/2009
Type of sample		Soil	Soil	Soil	Soil	Soil
Date extracted	-	23/12/2009	23/12/2009	23/12/2009	23/12/2009	23/12/2009
Date analysed	-	23/12/2009	23/12/2009	23/12/2009	23/12/2009	23/12/2009
vTPH C_6 - C_9	mg/kg	<25	<25	<25	<25	<25
Benzene	mg/kg	<0.5	<0.5	<0.5	<0.5	<0.5
Toluene	mg/kg	<0.5	<0.5	<0.5	<0.5	<0.5
Ethylbenzene	mg/kg	<1.0	<1.0	<1.0	<1.0	<1.0
m+p-xylene	mg/kg	<2.0	<2.0	<2.0	<2.0	<2.0
o-Xylene	mg/kg	<1.0	<1.0	<1.0	<1.0	<1.0
Surrogate aaa-Trifluorotoluene	%	88	89	81	89	86

sTPH in Soil (C10-C36)						
Our Reference:	UNITS	36506-1	36506-2	36506-3	36506-4	36506-5
Your Reference	-------------	BH101/0.2	BH101/1.5	BH102/0.45	BH102/1.0	BD/201 209
Date Sampled	------------	17/12/2009	17/12/2009	20/12/2009	20/12/2009	17/12/2009
Type of sample		Soil	Soil	Soil	Soil	Soil
Date extracted	-	23/12/2009	23/12/2009	23/12/2009	23/12/2009	23/12/2009
Date analysed	-	24/12/2009	24/12/2009	24/12/2009	24/12/2009	24/12/2009
TPH C_{10} - C_{14}	mg/kg	<50	<50	<50	<50	<50
TPH C_{15} - C_{28}	mg/kg	<100	120	140	<100	<100
TPH C_{29} - C_{36}	mg/kg	<100	<100	<100	<100	<100
Surrogate o-Terphenyl	%	90	97	96	93	92

PAHs in Soil Our Reference: Your Reference Date Sampled Type of sample	UNITS ------------- ------------	36506-1 BH101/0.2 17/12/2009 Soil	36506-2 BH101/1.5 17/12/2009 Soil	36506-3 BH102/0.45 20/12/2009 Soil	36506-4 BH102/1.0 20/12/2009 Soil	36506-5 BD/201 209 17/12/2009 Soil
Date extracted	-	23/12/2009	23/12/2009	23/12/2009	23/12/2009	23/12/2009
Date analysed	-	23/12/2009	23/12/2009	23/12/2009	24/12/2009	24/12/2009
Naphthalene	mg/kg	<0.1	0.1	0.1	0.1	0.1
Acenaphthylene	mg/kg	<0.1	0.3	0.4	0.1	0.1
Acenaphthene	mg/kg	<0.1	0.1	<0.1	<0.1	<0.1
Fluorene	mg/kg	<0.1	0.2	0.1	0.1	0.1
Phenanthrene	mg/kg	<0.1	3.5	4.0	2.1	2.1
Anthracene	mg/kg	<0.1	0.9	0.9	0.4	0.4
Fluoranthene	mg/kg	<0.1	6.1	7.0	2.5	2.6
Pyrene	mg/kg	<0.1	6.1	7.3	2.7	2.7
Benzo(a)anthracene	mg/kg	<0.1	2.9	3.3	1.1	1.2
Chrysene	mg/kg	<0.1	3.0	3.3	1.2	1.2
Benzo(b+k)fluoranthene	mg/kg	<0.2	4.7	5.4	1.7	1.8
Benzo(a)pyrene	mg/kg	<0.05	3.5	4.2	1.3	1.4
Indeno(1,2,3-c,d)pyrene	mg/kg	<0.1	2.0	2.6	0.7	0.7
Dibenzo(a,h)anthracene	mg/kg	<0.1	0.2	0.2	<0.1	<0.1
Benzo(g,h,i)perylene	mg/kg	<0.1	1.8	2.3	0.7	0.7
Surrogate p-Terphenyl-d_{14}	%	90	88	91	92	90

Organochlorine Pesticides in soil Our Reference: Your Reference Date Sampled Type of sample	UNITS ------------- ------------	36506-1 BH101/0.2 17/12/2009 Soil	36506-2 BH101/1.5 17/12/2009 Soil	36506-3 BH102/0.45 20/12/2009 Soil	36506-4 BH102/1.0 20/12/2009 Soil	36506-5 BD/201 209 17/12/2009 Soil
Date extracted	-	23/12/2009	23/12/2009	23/12/2009	23/12/2009	23/12/2009
Date analysed	-	23/12/2009	23/12/2009	23/12/2009	23/12/2009	23/12/2009
HCB	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
alpha-BHC	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
gamma-BHC	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
beta-BHC	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Heptachlor	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
delta-BHC	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Aldrin	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Heptachlor Epoxide	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
gamma-Chlordane	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
alpha-chlordane	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Endosulfan I	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
pp-DDE	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Dieldrin	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Endrin	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
pp-DDD	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Endosulfan II	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
pp-DDT	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Endrin Aldehyde	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Endosulfan Sulphate	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Methoxychlor	mg/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Surrogate TCLMX	%	97	99	101	101	102

Acid Extractable metals in soil Our Reference: Your Reference Date Sampled Type of sample	UNITS ------------- ------------	36506-1 BH101/0.2 17/12/2009 Soil	36506-2 BH101/1.5 17/12/2009 Soil	36506-3 BH102/0.45 20/12/2009 Soil	36506-4 BH102/1.0 20/12/2009 Soil	36506-5 BD/201 209 17/12/2009 Soil
Date digested	-	23/12/2009	23/12/2009	23/12/2009	23/12/2009	23/12/2009
Date analysed	-	29/12/2009	29/12/2009	29/12/2009	29/12/2009	29/12/2009
Arsenic	mg/kg	<4	<4	4	<4	<4
Cadmium	mg/kg	<0.5	<0.5	<0.5	<0.5	<0.5
Chromium	mg/kg	9	25	10	13	12
Copper	mg/kg	81	63	41	22	19
Lead	mg/kg	4	54	70	25	32
Mercury	mg/kg	<0.1	<0.1	1.6	0.8	0.9
Nickel	mg/kg	77	37	11	7	8
Zinc	mg/kg	41	82	43	17	18

Moisture Our Reference: Your Reference Date Sampled Type of sample	UNITS ------------- ------------	36506-1 BH101/0.2 17/12/2009 Soil	36506-2 BH101/1.5 17/12/2009 Soil	36506-3 BH102/0.45 20/12/2009 Soil	36506-4 BH102/1.0 20/12/2009 Soil	36506-5 BD/201 209 17/12/2009 Soil
Date prepared	-	23/12/2009	23/12/2009	23/12/2009	23/12/2009	23/12/2009
Date analysed	-	23/12/2009	23/12/2009	23/12/2009	23/12/2009	23/12/2009
Moisture	%	7.3	6.9	16	5.3	5.5

Asbestos ID - soils Our Reference: Your Reference Date Sampled Type of sample	UNITS ------------- ------------	36506-1 BH101/0.2 17/12/2009 Soil	36506-2 BH101/1.5 17/12/2009 Soil	36506-3 BH102/0.45 20/12/2009 Soil	36506-4 BH102/1.0 20/12/2009 Soil	36506-5 BD/201 209 17/12/2009 Soil
Date analysed	-	24/12/2009	24/12/2009	24/12/2009	24/12/2009	24/12/2009
Sample Description	-	Approx 30g Soil	Approx 30g Soil	Approx 30g Soil	Approx 30g Soil	Approx 40g Soil
Asbestos ID in soil	-	No asbestos found at reporting limit of 0.1g/kg	No asbestos found at reporting limit of 0.1g/kg	No asbestos found at reporting limit of 0.1g/kg	No asbestos found at reporting limit of 0.1g/kg	No asbestos found at reporting limit of 0.1g/kg
Trace Analysis	-	Respirable fibres not detected	Respirable fibres not detected	Respirable fibres not detected	Respirable fibres not detected	Respirable fibres not detected

Method ID	Methodology Summary
GC.16	Soil samples are extracted with methanol and spiked into water prior to analysing by purge and trap GC-MS. Water samples are analysed directly by purge and trap GC-MS.
GC.3	Soil samples are extracted with Dichloromethane/Acetone and waters with Dichloromethane and analysed by GC-FID.
GC.12 subset	Soil samples are extracted with Dichloromethane/Acetone and waters with Dichloromethane and analysed by GC-MS.
GC-5	Soil samples are extracted with dichloromethane/acetone and waters with dichloromethane and analysed by GC with dual ECD's.
Metals.20 ICP-AES	Determination of various metals by ICP-AES.
Metals.21 CV-AAS	Determination of Mercury by Cold Vapour AAS.
LAB.8	Moisture content determined by heating at 105 deg C for a minimum of 4 hours.
ASB.1	Qualitative identification of asbestos type fibres in bulk using Polarised Light Microscopy and Dispersion Staining Techniques.

QUALITY CONTROL vTPH & BTEX in Soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date extracted	-			23/12/09	36506-4	23/12/2009 || 23/12/2009	LCS-3	23/12/09
Date analysed	-			23/12/09	36506-4	23/12/2009 || 23/12/2009	LCS-3	23/12/09
vTPH C_6 - C_9	mg/kg	25	GC.16	<25	36506-4	<25 || <25	LCS-3	114%
Benzene	mg/kg	0.5	GC.16	<0.5	36506-4	<0.5 || <0.5	LCS-3	85%
Toluene	mg/kg	0.5	GC.16	<0.5	36506-4	<0.5 || <0.5	LCS-3	108%
Ethylbenzene	mg/kg	1	GC.16	<1.0	36506-4	<1.0 || <1.0	LCS-3	122%
m+p-xylene	mg/kg	2	GC.16	<2.0	36506-4	<2.0 || <2.0	LCS-3	127%
o-Xylene	mg/kg	1	GC.16	<1.0	36506-4	<1.0 || <1.0	LCS-3	132%
Surrogate aaa-Trifluorotoluene	%		GC.16	90	36506-4	89 || 93 || RPD: 4	LCS-3	94%

QUALITY CONTROL sTPH in Soil (C10-C36)	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date extracted	-			23/12/09	36506-4	23/12/2009 || 23/12/2009	LCS-3	23/12/09
Date analysed	-			24/12/09	36506-4	24/12/2009 || 24/12/2009	LCS-3	24/12/09
TPH C_{10} - C_{14}	mg/kg	50	GC.3	<50	36506-4	<50 || <50	LCS-3	104%
TPH C_{15} - C_{28}	mg/kg	100	GC.3	<100	36506-4	<100 || <100	LCS-3	123%
TPH C_{29} - C_{36}	mg/kg	100	GC.3	<100	36506-4	<100 || <100	LCS-3	126%
Surrogate o-Terphenyl	%		GC.3	98	36506-4	93 || 92 || RPD: 1	LCS-3	95%

QUALITY CONTROL PAHs in Soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date extracted	-			23/12/09	36506-4	23/12/2009 || 23/12/2009	LCS-3	23/12/09
Date analysed	-			23/12/09	36506-4	24/12/2009 || 24/12/2009	LCS-3	23/12/09
Naphthalene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	0.1 || 0.1 || RPD: 0	LCS-3	93%
Acenaphthylene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	0.1 || <0.1	[NR]	[NR]
Acenaphthene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
Fluorene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	0.1 || 0.1 || RPD: 0	LCS-3	95%
Phenanthrene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	2.1 || 1.9 || RPD: 10	LCS-3	94%
Anthracene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	0.4 || 0.4 || RPD: 0	[NR]	[NR]
Fluoranthene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	2.5 || 2.2 || RPD: 13	LCS-3	84%
Pyrene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	2.7 || 2.3 || RPD: 16	LCS-3	96%

QUALITY CONTROL PAHs in Soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Benzo(a)anthracene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	1.1 || 1.0 || RPD: 10	[NR]	[NR]
Chrysene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	1.2 || 1.0 || RPD: 18	LCS-3	101%
Benzo(b+k)fluoranthene	mg/kg	0.2	GC.12 subset	<0.2	36506-4	1.7 || 1.5 || RPD: 12	[NR]	[NR]
Benzo(a)pyrene	mg/kg	0.05	GC.12 subset	<0.05	36506-4	1.3 || 1.2 || RPD: 8	LCS-3	104%
Indeno(1,2,3-c,d)pyrene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	0.7 || 0.6 || RPD: 15	[NR]	[NR]
Dibenzo(a,h)anthracene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
Benzo(g,h,i)perylene	mg/kg	0.1	GC.12 subset	<0.1	36506-4	0.7 || 0.6 || RPD: 15	[NR]	[NR]
Surrogate p-Terphenyl-d_{14}	%		GC.12 subset	93	36506-4	92 || 89 || RPD: 3	LCS-3	93%

QUALITY CONTROL Organochlorine Pesticides in soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date extracted	-			23/12/09	36506-4	23/12/2009 || 23/12/2009	LCS-1	23/12/09
Date analysed	-			23/12/09	36506-4	23/12/2009 || 23/12/2009	LCS-1	23/12/09
HCB	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
alpha-BHC	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	LCS-1	103%
gamma-BHC	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
beta-BHC	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	LCS-1	120%
Heptachlor	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	LCS-1	98%
delta-BHC	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
Aldrin	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	LCS-1	100%
Heptachlor Epoxide	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	LCS-1	93%
gamma-Chlordane	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
alpha-chlordane	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
Endosulfan I	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
pp-DDE	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	LCS-1	119%
Dieldrin	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	LCS-1	104%
Endrin	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	LCS-1	109%
pp-DDD	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	LCS-1	114%
Endosulfan II	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
pp-DDT	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
Endrin Aldehyde	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
Endosulfan Sulphate	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	LCS-1	110%
Methoxychlor	mg/kg	0.1	GC-5	<0.1	36506-4	<0.1 || <0.1	[NR]	[NR]
Surrogate TCLMX	%		GC-5	93	36506-4	101 || 97 || RPD: 4	LCS-1	94%

QUALITY CONTROL Acid Extractable metals in soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date digested	-			23/12/09	36506-4	23/12/2009 || 23/12/2009	LCS-6	23/12/09
Date analysed	-			29/12/09	36506-4	29/12/2009 || 29/12/2009	LCS-6	29/12/09
Arsenic	mg/kg	4	Metals.20 ICP-AES	<4	36506-4	<4 || <4	LCS-6	105%
Cadmium	mg/kg	0.5	Metals.20 ICP-AES	<0.5	36506-4	<0.5 || <0.5	LCS-6	107%
Chromium	mg/kg	1	Metals.20 ICP-AES	<1	36506-4	13 || 16 || RPD: 21	LCS-6	108%
Copper	mg/kg	1	Metals.20 ICP-AES	<1	36506-4	22 || 22 || RPD: 0	LCS-6	111%
Lead	mg/kg	1	Metals.20 ICP-AES	<1	36506-4	25 || 22 || RPD: 13	LCS-6	106%
Mercury	mg/kg	0.1	Metals.21 CV-AAS	<0.1	36506-4	0.8 || 0.7 || RPD: 13	LCS-6	100%
Nickel	mg/kg	1	Metals.20 ICP-AES	<1	36506-4	7 || 9 || RPD: 25	LCS-6	110%
Zinc	mg/kg	1	Metals.20 ICP-AES	<1	36506-4	17 || 13 || RPD: 27	LCS-6	107%

QUALITY CONTROL Moisture	UNITS	PQL	METHOD	Blank
Date prepared	-			23/12/09
Date analysed	-			23/12/09
Moisture	%	0.1	LAB.8	<0.10

QUALITY CONTROL Asbestos ID - soils	UNITS	PQL	METHOD	Blank
Date analysed	-			[NT]

Report Comments:

Asbestos: A portion of the supplied sample was sub-sampled for asbestos according to Envirolab procedures. We cannot guarantee that this sub-sample is indicative of the entire sample.
Envirolab recommends supplying 30-40g of sample in it's own container.

Asbestos was analysed by Approved Identifier: Joshua Lim

INS: Insufficient sample for this test NT: Not tested PQL: Practical Quantitation Limit <: Less than >: Greater than
RPD: Relative Percent Difference NA: Test not required LCS: Laboratory Control Sample NR: Not requested

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike: A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample): This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria:

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the sample batch were within laboratory acceptance criteria.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes and LCS: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC and speciated phenols is acceptable. Surrogates: 60-140% is acceptable for general organics and 10-140% for SVOC and speciated phenols.

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CERTIFICATE OF ANALYSIS 36506-A

Client:
Douglas Partners
96 Hermitage Rd
West Ryde
NSW 2114

Attention: Peter Hartcliff

Sample log in details:

Your Reference:	**71529, Sydney Opera House (VAPS)**
No. of samples:	Additional Testing on 4 Soils
Date samples received:	22/12/09
Date completed instructions received:	08/01/10

Analysis Details:
Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by:	11/01/10
Date of Preliminary Report:	Not Issued
Issue Date:	11/01/10

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Accredited for compliance with ISO/IEC 17025.
Tests not covered by NATA are denoted with *.

Results Approved By:

Joshua Lim
Chemist

Metals in TCLP USEPA1311 Our Reference:	UNITS	36506-A-1	36506-A-2	36506-A-3	36506-A-4
Your Reference	-------------	BH101/0.2	BH101/1.5	BH102/0.45	BH102/1.0
Date Sampled	------------	17/12/2009	17/12/2009	20/12/2009	20/12/2009
Type of sample		Soil	Soil	Soil	Soil
Date extracted	-	8/01/2010	8/01/2010	8/01/2010	8/01/2010
Date analysed	-	11/01/2010	[NA]	[NA]	[NA]
pH of soil for fluid# determ.	pH units	9.70	9.50	9.90	9.60
pH of soil for fluid # determ. (acid)	pH units	0.900	0.900	1.00	0.900
Extraction fluid used	-	1	1	1	1
pH of final Leachate	pH units	5.20	5.30	6.30	5.10
Nickel in TCLP	mg/L	0.1	[NA]	[NA]	[NA]

PAHs in TCLP (USEPA 1311) Our Reference: Your Reference Date Sampled Type of sample	UNITS ------------- ------------	36506-A-2 BH101/1.5 17/12/2009 Soil	36506-A-3 BH102/0.45 20/12/2009 Soil	36506-A-4 BH102/1.0 20/12/2009 Soil
Date extracted	-	11/01/2010	11/01/2010	11/01/2010
Date analysed	-	11/01/2010	11/01/2010	11/01/2010
Naphthalene in TCLP	mg/L	0.001	<0.001	0.002
Acenaphthylene in TCLP	mg/L	<0.001	<0.001	<0.001
Acenaphthene in TCLP	mg/L	<0.001	<0.001	<0.001
Fluorene in TCLP	mg/L	<0.001	<0.001	<0.001
Phenanthrene in TCLP	mg/L	0.003	0.004	0.003
Anthracene in TCLP	mg/L	<0.001	<0.001	<0.001
Fluoranthene in TCLP	mg/L	<0.001	<0.001	<0.001
Pyrene in TCLP	mg/L	<0.001	<0.001	<0.001
Benzo(a)anthracene in TCLP	mg/L	<0.001	<0.001	<0.001
Chrysene in TCLP	mg/L	<0.001	<0.001	<0.001
Benzo(b+k)fluoranthene in TCLP	mg/L	<0.002	<0.002	<0.002
Benzo(a)pyrene in TCLP	mg/L	<0.001	<0.001	<0.001
Indeno(1,2,3-c,d)pyrene - TCLP	mg/L	<0.001	<0.001	<0.001
Dibenzo(a,h)anthracene in TCLP	mg/L	<0.001	<0.001	<0.001
Benzo(g,h,i)perylene in TCLP	mg/L	<0.001	<0.001	<0.001
Surrogate p-Terphenyl-d_{14}	%	116	122	119

Method ID	Methodology Summary
LAB.4	Toxicity Characteristic Leaching Procedure (TCLP).
EXTRACT.7	Toxicity Characteristic Leaching Procedure (TCLP).
LAB.1	pH - Measured using pH meter and electrode in accordance with APHA 20th ED, 4500-H+.
Metals.20 ICP-AES	Determination of various metals by ICP-AES.
GC.12 subset	Leachates are extracted with Dichloromethane and analysed by GC-MS.
GC.12 subset	Soil samples are extracted with Dichloromethane/Acetone and waters with Dichloromethane and analysed by GC-MS.
GC.12	Soil samples are extracted with Dichloromethane/Acetone and waters with Dichloromethane and analysed by GC-MS.

QUALITY CONTROL Metals in TCLP USEPA1311	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date extracted	-			08/01/10	[NT]	[NT]	LCS-W1	08/01/10
Date analysed	-			11/01/10	[NT]	[NT]	LCS-W1	11/01/10
Nickel in TCLP	mg/L	0.02	Metals.20 ICP-AES	<0.02	[NT]	[NT]	LCS-W1	93%

QUALITY CONTROL PAHs in TCLP (USEPA 1311)	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date extracted	-			11/01/2010	[NT]	[NT]	LCS-W1	11/01/2010
Date analysed	-			11/01/2010	[NT]	[NT]	LCS-W1	11/01/2010
Naphthalene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	LCS-W1	98%
Acenaphthylene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	[NR]	[NR]
Acenaphthene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	[NR]	[NR]
Fluorene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	LCS-W1	94%
Phenanthrene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	LCS-W1	101%
Anthracene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	[NR]	[NR]
Fluoranthene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	LCS-W1	90%
Pyrene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	LCS-W1	94%
Benzo(a)anthracene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	[NR]	[NR]
Chrysene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	LCS-W1	109%
Benzo(b+k)fluoranthene in TCLP	mg/L	0.002	GC.12 subset	<0.002	[NT]	[NT]	[NR]	[NR]
Benzo(a)pyrene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	LCS-W1	117%
Indeno(1,2,3-c,d)pyrene - TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	[NR]	[NR]
Dibenzo(a,h)anthracene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	[NR]	[NR]
Benzo(g,h,i)perylene in TCLP	mg/L	0.001	GC.12 subset	<0.001	[NT]	[NT]	[NR]	[NR]
Surrogate p-Terphenyl-d_{14}	%		GC.12	117	[NT]	[NT]	LCS-W1	113%

Report Comments:

Asbestos was analysed by Approved Identifier: Not applicable for this job
INS: Insufficient sample for this test NT: Not tested PQL: Practical Quantitation Limit <: Less than >: Greater than
RPD: Relative Percent Difference NA: Test not required LCS: Laboratory Control Sample NR: Not requested

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike: A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample): This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria:

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the sample batch were within laboratory acceptance criteria.
Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes and LCS: Generally 70-130% for inorganics/metals; 60-140% for organics and 10-140% for SVOC and speciated phenols is acceptable. Surrogates: 60-140% is acceptable for general organics and 10-140% for SVOC and speciated phenols.