Douglas Partners
Geotechnics / Environment / Groundwater

Integrated Practical Solutions

Report on
Geotechnical Investigation

Proposed Macquarie Village
110 - 114 Herring Road, Macquarie Park

Prepared for
Stamford Property Services Pty Ltd

Project 72138
January 2011

# Douglas Partners
Geotechnics / Environment / Groundwater

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CERTIFIED QUALITY MANAGEMENT SYSTEM

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
96 Hermitage Road
West Ryde NSW 2114
PO Box 472
West Ryde NSW 1685
Phone (02) 9809 0666
Fax (02) 9809 4095

## Table of Contents

# Report on Geotechnical Investigation
# Proposed Macquarie Village
# 110 - 114 Herring Road, Macquarie Park

## 1. Introduction

This report presents the results of a geotechnical investigation undertaken by Douglas Partners Pty Ltd (DP) for the proposed Macquarie Village at 110 - 114 Herring Road, Macquarie Park. The work was requested by Stamford Property Services Pty Ltd, the developers of the project.

It is understood that the development of the site will include the construction of seven new multi-storey buildings, ranging in height from four to twenty residential storeys, a common basement to depths ranging from 7 m to 13 m and access driveways. Site investigation was carried out to provide information on subsurface conditions for the design of earthworks, retaining walls, foundations and pavements.

The investigation included the site inspection, drilling of sixteen boreholes, the excavation of one test pit, geotechnical mapping of the existing rock outcrops and laboratory testing of selected samples recovered from the test bores and pit. The details of the field work are presented in this report, together with comments and recommendations on the issues listed above.

This investigation was carried out concurrently with a preliminary contamination assessment and waste classification assessment. The results of these assessments are documented separately.

## 2. Background

A review of the 1930, 1951, 1961, 1970 and 1986 historical aerial photographs indicates that the site was originally used for agricultural purposes and subsequently developed in the 1950s with several buildings on-site.

DP has previously carried out a geotechnical investigation, including four boreholes (Bores 1 - 4), of the extensions for the Stamford Hotel, at the north-western end of the site. The results of this investigation are presented in a report (Project 28604) dated 27 October 1999. The investigation encountered shallow filling overlying natural clays with low strength sandstone at depths ranging from 1.0 m – 1.7 m.

## 3. Site Description

The site is identified as 110 – 114 Herring Road (Lot 1 in D.P.780314), Macquarie Park. It is an approximate rectangular shaped area of 2.24 hectares. The ground surface generally falls to the north at slopes of approximately 1º to 3º, resulting in an elevation change of approximately 10 m (RL 65.2 – RL 75.1 relative to Australian Height Datum [AHD]).

The site is currently occupied by the Stamford Grand North Ryde Hotel which includes several buildings ranging in height from one to three storeys, a one level underground basement together with associated driveways, gardens and recreational facilities (eg pool, tennis court etc).

The site is bounded by the Epping Road to the southwest, Herring Road to the southeast, a residential development which includes single storey villas and a three storey unit block to the northeast and a Baptist Retirement Village with one to two storey buildings to the northwest.

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is underlain by Ashfield Shale but in close proximity to Hawkesbury Sandstone at lower elevations to the north. The field work is interpreted as intersecting both the Mittagong Formation, a thin (typically <6m) transitional unit between the Ashfield Shale and Hawkesbury Sandstone on the upper portions of the site, and the Hawkesbury Sandstone.

## 4. Field Work Methods

The current field work comprised:

- Site inspection by a Senior Geotechnical Engineer.
- Sixteen boreholes (Bores 101 - 116) drilled with an underpinning rig or a truck, track or bobcat-mounted auger/rotary drilling and sampling rig. The boreholes were generally drilled to depths ranging from 0.5 m to 2.3 m with 110 mm spiral flight augers, then cased using HW or NW casing and extended to final depths ranging from 10.0 m to 17.55 m by NMLC (52 mm core diameter) diamond coring techniques. Bores 101, 111 and 112 also required dia-coring to penetrate concrete layers at various stages during auger drilling. Standard penetration tests (SPTs) were carried out within soils at regular depth intervals. Disturbed soil samples retrieved from the cuttings returned by the auger blade were used for identification and classification purposes.
- One test pit (TP 117) excavated to 0.2 m depth using hand tools for the purpose of obtaining a sample for subsequent laboratory testing.
- Installation of standpipes to depths of 14.0 m, 16.0 m and 11.8 m in Bores 103, 110 and 116 respectively, for subsequent monitoring of the groundwater level. The standpipe wells were purged of water after the completion of drilling and then measured on two subsequent occasions.

All field work was carried out under the direction of a Senior Geotechnical Engineer.

The locations of the bores and pits are given in Drawing 1, Appendix A. The test bore and pit surface levels relative to AHD, shown on the borehole and test pit logs, were generally estimated from the survey plan of the site dated 13 October 2010 and prepared by Denny Linker & Co. Pty Ltd (Drawing No. 100915). Test Bore 113 was located within the basement and its level was not indicated on the survey plan and was levelled relative to an identifiable point on the survey plan, to determine its level relative to AHD.

## 5. Field Work Results

Details of the conditions encountered in the current boreholes and test pits are presented in Appendix B. The results of previous investigations are presented in Appendix C. Notes defining classification methods and descriptive terms used in logging the boreholes and test pit are also given in Appendix B. Summary geological cross sections (Sections A – A' to F – F') are included in Drawings 2 - 7 in Appendix A.

The material strata encountered in current and previous bores is described in generally increasing depth order below:

| | |
|---|---|
| **FILLING** | Ranging from 0.2 m to 2.9 m and comprising a surficial layer of asphaltic concrete, concrete or pavers in all but Bores 1 – 4, underlain by gravels, roadbase, crushed sandstone, gravelly silty sand and clayey silty sand. Additional concrete layer were intersected in Bores 101, 107, 109 – 111 and 116. |
| **CLAYS** | Clay and sandy clay to depths of 0.4 – 2.4 m in Bores 102, 105, 106 and 113 to 116. |
| **MITTAGONG FORMATION** | Initially a thin layer of weathered sandstone and laminite (sandstone interbedded with approximately 10% to 30% siltstone) overlying medium and high strength, highly weathered to fresh, fragmented to slightly fractured, grey and brown sandstone and laminite with ironstone banding (including medium, high and very high strength bands) and clay seams up to 200 mm thick to depths of 2.7 – 4.7 m in Bores 103 to 111. This unit contains joints dipping from 45° to 90°. A fault with 50 mm displacement was intersected in Bore 111. |
| **SANDSTONE** | Typically medium and high strength, moderately weathered to fresh, fractured to unbroken with some fractured zones, medium and coarse grained, grey and orange sandstone with distinct and indistinct siltstone laminations. This unit contained frequent joints dipping from 45° to 85° and some minor crushed zones. |

Outcrops of medium strength sandstone are exposed in the car park on the north-western boundary. The locations and the reduced levels of the top of the sandstone exposure are indicated on Drawing 1, Appendix A.

No free groundwater was observed in the test bores during auger drilling or the excavation of the test pit. The use of water as a drilling fluid during diamond coring, and the immediate backfilling of the test bores and pit, precluded long-term measurement of the groundwater levels.

The groundwater levels were measured by DP during December 2010 and January 2011 within the standpipes installed in Bores 103, 110 and 116. The results of these measurements are detailed in Table 1.

**Table 1: Results of Standpipe Measurements**

| Test Bore | Surface RL | Water Levels | | | | | |
|---|---|---|---|---|---|---|---|
| | | 20/12/2010 | | 22/12/2010 | | 11/1/2011 | |
| | | Depth (m) | RL | Depth (m) | RL | Depth (m) | RL |
| 103 | 72.3 | 4.3 | 68.0 | 4.7 | 67.6 | 4.6 | 67.7 |
| 110 | 74.0 | 11.5 | 62.5 | 11.7 | 62.3 | -[1] | - |
| 116 | 66.8 | 2.4 | 64.4 | 2.6 | 64.2 | 2.7 | 64.1 |

Note 1: Standpipe appeared to have been damaged– object stuck in pipe. Water level could not be measured

## 6. Laboratory Testing

Samples recovered from the field investigation were tested in the laboratory to determine compaction properties, California bearing ratio (CBR) value, moisture contents values and aggressivity (pH, chloride and sulphate content). The detailed results are given in Appendix D and are summarised in Table 2.

**Table 2: Summary of CBR and Aggressivity Laboratory Test Results**

| Test Loc | Depth (m) | Material | w (%) | OMC (%) | MDD ($t/m^3$) | CBR (%) | pH | $Cl^-$ | $SO_4^=$ |
|---|---|---|---|---|---|---|---|---|---|
| 102 | 1.0 - 1.1 | Filling | - | - | - | - | 5.5 | 27 | 31 |
| 103 | 1.0 - 1.1 | Sandstone | - | - | - | - | 8.6 | 15 | 45 |
| 116 | 1.0 - 1.1 | Clay | - | - | - | - | 5.2 | 45 | 40 |
| 117 | 0.1 - 0.2 | Filling | 11.1 | 16.5 | 1.73 | 7 | - | - | - |

*Where:* *w = Moisture content* *OMC = Optimum moisture content (standard compaction)*
*MDD = Maximum dry density* *CBR = California bearing ratio*
*$Cl^-$ = Chloride* *$SO_4^=$ = Sulphate*

The sample of filling was measured to be between 5.4% dry of OMC.

The results of aggressivity testing, when compared with Table 6.4.2(C) in AS2159-2009 “Piling: Design and Installation”, indicates that an exposure classification of ‘mildly aggressive’ is appropriate for subsurface concrete elements.

Point Load Strength Index ($Is_{50}$) testing was carried out on selected rock core specimens. The results of the tests are given on the test bore report sheets at the appropriate depth, indicating values of 0.3 – 1.0 MPa for medium strength bedrock, 1.0 – 3.0 MPa for high strength bedrock and 5.0 MPa in a very high strength ironstone band.

## 7. Proposed Development

The proposed Macquarie Village development will include the construction of seven multi-storey buildings. A common, split level basement ranging from 7 m to 13 m in depth. The lower level bulk excavation level (BEL), on the western side, will be at RL 58.3. The upper level BEL, on the eastern side, will be at RL 61.4. The excavation is generally located at least seven metres from the property boundary. In addition, an access pavement is proposed surrounding the perimeter of the building. Some minor filling may be required around the perimeter of the site beneath pavement formations.

Footing loads are assumed to range from 3 000 kN to 15 000 kN (working) for the various multi-storey buildings.

## 8. Comments

### 8.1 Geotechnical Model

The geological model comprises in increasing depth order:

- Unit 1 - A mostly thin (0.2 – 1 m) but locally deeper (to 2.9 m) filling layer.
- Unit 2 - Residual clay soils developed on rocks of both the Mittagong Formation and Hawkesbury Sandstone.
- Unit 3 - Sandstone and laminite (sandstone with approximately 10% to 30% siltstone laminations) of the Mittagong Formation.
- Unit 4 - Sandstone of the Hawkesbury Sandstone, the surface of which dips at approximately 2° – 3° northward across the site. In most borehole intersections, the medium and medium to coarse grained sandstone is of medium or high strength.

The groundwater levels were variable and appear to be either groundwater seepage flowing over or near the soil/rock interface or through fractures in the rock, particularly after wet weather.

### 8.2 Site Preparation

Filling is anticipated for pavement formation areas and possibly backfilling behind retaining walls.

The following subgrade preparation measures are recommended for filling operations:

- Remove all vegetation-affected filling materials (topsoil).
- Test roll the exposed surface using a minimum 12 tonne smooth drum roller in non-vibration mode. The surface should be rolled a minimum of six times with the last two passes observed by an experienced geotechnical engineer to detect any ‘soft spots’.
- Any heaving materials identified during test rolling should be removed as directed by the geotechnical engineer.

- Any new filling should be placed in layers of 250 mm maximum loose thickness except for backfilling behind retaining walls. Compaction equipment behind retaining walls should be limited to hand held “wacker packers”, plate compactors or small trench rollers and consequently the loose layer thickness should be limited to 150 mm.
- Each fill layer should be compacted to a dry density ratio between 100% and 103% relative to standard compaction with moisture contents maintained within 2% of the optimum moisture content for standard compaction. The select fill should be free of oversize particles (>100 mm) and deleterious material.
- Density testing of the filling should be carried out at Level 1 or Level 2 responsibility, as defined in AS3798-2007 “Guidelines for Earthworks for Commercial and Residential Developments”.

## 8.3 Excavation Conditions

Bulk excavation to RLs 58.3 to 61.4 will encounter all the geological units encountered in the investigation (Units 1 to 4).

Excavation within Units 1 and 2 should be readily achievable by bulldozer blade or hydraulic excavator. Some light to medium ripping assistance or the use of rock hammers may be required for layers of concrete within Unit 1 and medium to very high strength ironstone bands included in the mostly extremely low to low strength upper sections of Unit 3.

Any excavation within the medium and high strength sections of Unit 3 and Unit 4 will require medium to heavy rock breaking equipment. Medium strength rock is expected to have an unconfined compressive strength (UCS) of 6 – 20 MPa; high strength rock is expected to have a UCS of 20 – 60 MPa. Low productivity during excavation should be expected with such materials. Rock breaking equipment will generally cause noise and vibrations that could be disturbing to surrounding personnel.

All excavated materials will need to be disposed in accordance with current DECC policies. Under the Waste Avoidance and Resource Recovery Act (NSW EPA, 2001) a waste/fill receiving site must be satisfied that materials received meet the environmental criteria for proposed land use. This includes filling and virgin excavated natural materials (VENM), such as may be removed from site. Reference should be made to DP’s preliminary waste classification assessment for the site.

Based on the groundwater monitoring, it is anticipated that there may be some seepage of groundwater into the excavation. Such seepage will need to be collected during construction by the judicious placement of drainage sumps and by intermittent pumping. At this stage, it is not possible to estimate the likely extent and rate of seepage although it is anticipated that it will be low and that it should be readily handled by sump and pump measures. It is suggested that monitoring of flow during the early phases of excavation below the groundwater table be undertaken to assess long-term pumping requirements.

Noise and vibration will be associated with excavation within bedrock materials. Discussions of appropriate methodologies for management are in Section 8.5.

## 8.4 Excavation Support

### 8.4.1 General

The filling, residual clays and extremely weathered rock (Units 1 and 2), together with the very low strength rock of Units 3 and 4 will require either battering or temporary shoring support during excavation with permanent retaining wall support required as part of the final construction.

The medium and high strength sandstone and laminite of Unit 3 is not considered suitable to be cut vertically and left exposed due to the significant fracturing of this unit together with the presence of clay seams and faulting. There is a very high risk that unstable wedges could develop within this unit that would pose both an OH&S hazard to personnel on site and a sidewall stability problem. Therefore, it is recommended that battering or shoring walls extend below the base of Unit 3.

It is considered that a soldier pile/infill panel wall system would be a suitable retaining wall type for the retention of Units 1 to 3 on this site. A soldier pile wall typically involves bored piles or continuous flight auger (CFA) piles installed at 2 m to 3 m centres with associated reinforced shotcrete infill panels. The soldier piles at this site would involve the installation of bored piles followed by progressive vertical excavation in 2 m lifts. At the completion of each 2 m excavation lift the construction of reinforced shotcrete infill panels proceeds. Ground anchors, where required, are installed at the appropriate level. It is possible that adverse jointing in the rock may give rise to unstable wedges and thus cause localised or even major instability in the exposed material. Therefore, regular inspections should be carried out by a geotechnical engineer following each progressive lift in excavation to reduce the risk of instability developing.

Drilling through the medium and high strength rock together with very high strength ironstone bands will require a drill rig with a high torque capacity. The drilling contractor should confirm that the rig proposed for site works is able to drill through these layers.

Strip drains should be installed behind the shotcrete of the soldier pile/infill panel wall system to facilitate drainage and prevent build-up of water pressures behind the shoring.

The medium and high strength sandstone of Unit 4 is expected to be suitable for excavation of vertical faces. However, steeply dipping jointing was also observed in this unit and there is a risk that unstable wedges could develop within exposed faces. Inspections by a geotechnical professional should be carried out each 2 m vertical lift of excavation to determine if such wedges are present and whether stabilisation measures are required. This requirement should be explicitly stated on the drawings and a hold point instruction developed until inspection for each section of excavation is carried out.

### 8.4.2 Batters

The maximum recommended temporary batter slopes are given in Table 3.

**Table 3: Temporary Batter Slopes**

| Material Description | Batter Slope (H:V) |
|---|---|
| Filling, Residual Clays and Extremely Weathered Rock (Units 1 to 2) | 1.5:1 |
| Very low to low strength laminite and sandstone (Unit 3) | 1:1 |
| Medium and high strength laminite and sandstone (Unit 3) | 1:1 |
| Medium and high strength sandstone (Unit 4) | Vertical[1] |

Note: 1: Batter slopes subject to geotechnical inspection every 2 m lift of excavation to determine if flatter batters or stabilisation measures are required.

### 8.4.3 Design of Lateral Support

The design of retaining walls should take due account of both lateral earth pressures and surcharges acting on the walls.

The lateral earth pressure coefficients and bulk unit weights in Table 4 are suggested for the preliminary design of a single anchored/propped wall using a triangular pressure distribution.

**Table 4: Suggested Design Parameters for Retaining Structures**

| Strata | Lateral Earth Pressure Coefficients | | | | |
|---|---|---|---|---|---|
| | Bulk Unit Weight, ($kN/m^3$) | 'Active' Temporary $K_a$ (temp) | 'Active' Permanent $K_a$ (perm) | 'At Rest' Temporary $K_o$ (temp) | Passive* |
| Filling, residual clays and extremely weathered rock (Units 1 to 2) | 20 | 0.3 | 0.35 | 0.5 | NA |
| Very low to low strength sandstone and laminite (Unit 3) | 22 | 0.2 | 0.25 | 0.3 | $K_p$ =4.0 or 400kPa |
| Medium and high strength sandstone and laminite (Unit 3) | 22 | 0.0 | 0.0 | 0.0 | 2000kPa |
| Medium and high strength sandstone (Unit 4) | 23 | 0.0 | 0.0 | 0.0 | 6000kPa |

Notes: * Ultimate values requiring incorporation of a factor of safety

Wall design using these parameters assumes the following:

- a level surface behind the top of the excavation;
- retaining walls will need to allow for full hydrostatic pressures from the ground surface level if drainage measures behind walls are not provided;
- construction traffic and other surcharge loadings (eg stacked materials) are not applied at the crest of the retaining walls, for a distance of say 5 m behind the wall/shoring (otherwise the resultant additional lateral loads need to be considered);
- Passive resistance may be developed in Unit 4 from beneath one pile diameter below the bulk excavation level or below the base of any adjacent localised excavation. The passive pressures calculated are ultimate values to which an appropriate factor of safety (say 2.5) should be incorporated.

The lateral active earth pressure coefficient, $K_{a,}$ to be used for estimating soil pressures in Table 4 is for a flexible wall allowing some lateral or outward "tilting" movement. Where it is necessary to limit movement, it is suggested that the wall be designed for $K_0$ (lateral earth pressure coefficients "at rest") conditions in combination with an analytical approach that considers the excavation and propping or anchoring sequence.

Preliminary design for lateral earth pressures for a multi-anchored wall system may be based on a uniform rectangular earth pressure distribution. A uniform lateral earth pressure over Units 1 to 3 of 4H (h = height to be retained) or 6H (where lateral movements are to be limited) should be adopted. Additional lateral pressures due to surcharge loadings behind the wall and hydrostatic pressures (as appropriate) should be allowed for within the structural design.

The design of temporary and permanent support will also need to consider the possibility of 45° joints, as intersected by the boreholes, which may intersect the northeastern, eastern and southern walls of the excavation. These may lead to large wedges of rock, up to 4 m high, which will need to be supported by the temporary and permanent retaining structures. Sufficient rock bolting and anchoring should take place to prevent movement along 45° joints, even though there is a low probability that a joint would run the full length of the excavation.

It is suggested that the design be carried out such that the support system has a factor of safety of 1.1 against sliding on the most unfavourable joint. The support system would typically comprise rock bolts or anchors spaced at 2 – 3 m centres over the rock face. These anchors have their bond lengths behind the projected 45° line and should provide sufficient force to resist the movement of a wedge of rock projected at 45° from just below the bolt to the ground surface. The frictional resistance of the wedge along the joint may be calculated assuming an angle of friction of 25°.

The final or detailed design of retaining walls are normally undertaken using interactive computer programs such as WALLAP or FLAC, which can take due regard of soil-structure interaction during the progressive stages of wall construction, anchoring and bulk excavation.

### 8.4.4 Ground Anchors

Temporary ground anchors will be required for the lateral restraint of most boundary shoring walls, unless struts are used, until such time that the walls are permanently strutted by the building floor slabs. The anchors should have their bond length within medium and high strength rock of Units 3 or 4 and behind the projected 45° line of potential joints from the base of Unit 3.

Suggested allowable bond stresses for the design of temporary ground anchors for the support of piled wall systems are given in Table 5.

**Table 5: Bond Stresses for Temporary Anchor Design**

| Material Description | Allowable Bond Stress (kPa) |
|---|---|
| Medium and high strength laminite (Unit 4) | 800 |
| Medium and high strength sandstone (Unit 5) | 2000 |

Ground anchors should be designed to have a free length that extends beyond an imaginary line drawn upwards at an angle of 45° from the toe of the wall. The minimum free length should be 3 m. After installation, each anchor should be proof loaded to 125% of the design working load and locked-off at about 80% of the working load. Periodic checks should be carried out during the construction phase to ensure that the lock-off load is maintained and not lost due to creep effects or other causes. The above parameters are based on the assumption that the anchor holes are clean and thoroughly flushed, with grouting and other installation procedures carried out carefully and in accordance with normal good anchoring practice. The successful anchoring contractor should be required to demonstrate that either the above or his own design bond values are achievable with the proposed anchor construction methods.

There will generally be sufficient room on-site for rock bolts and rock anchors to be contained entirely within site boundaries. Approval should be sought from the adjacent property owners, where rock anchors extend below neighbouring properties, roads or public access areas. It is understood that the building structure will be used to provide long-term permanent support for all retaining walls on site, and hence all anchors are expected to be temporary.

### 8.4.5 Excavation Induced Ground Movements

For a major excavation, such as is proposed on this site, there is likelihood that there will be horizontal movement of the excavation face due to stress relief effects. Release of these stresses may cause horizontal movements along the rock bedding surfaces and defects. At the midpoint of the crest of a deep excavated face, stress relief may cause a horizontal movement of approximately 1 mm/m to 2 mm/m depth of excavation. The amount of horizontal movement would diminish along the crest away from the midpoint, and down the excavated face away from the crest. The movement would be expected to occur progressively during the excavation and should be completed shortly after excavation is completed. This may cause cracks within buildings adjacent to the excavation, particularly on the northeastern boundary. Appropriate allowance should be made for the repair of these structures where excavation is carried out in close proximity. Dilapidation surveys of adjacent buildings should be made both prior to and at the completion of bulk excavation.

## 8.5 Vibrations

During excavation it will be necessary to use appropriate methods and equipment to keep ground vibrations within acceptable limits. The standards detailed in the Appendix E are considered appropriate for management of ground vibrations.

**Provisional Allowed Vibration Limit**

From current information, it is considered that the structures adjacent to the site can withstand vibration levels higher than those required to maintain the comfort of their occupants. A human comfort criterion is therefore indicated and the peak particle velocity in any direction i (PPVi), is proposed as the control parameter. It is recommended that a Provisional Allowed Vibration Limit of 8.0 mm/sec PPVi be set during normal working hours, at foundation level of the potentially affected building/s.

**Excavation Plant**

DP maintains a database of vibration trial results which can provide guidance for the selection of plant. Trial data is dependent on site conditions and equipment, hence actual vibration levels may differ from predictions and a specific trial is recommended at the commencement of rock excavation. The database suggests that buffer distances within the ranges shown below in Table 6 should be maintained between excavation plant and adjacent buildings. These estimates should be examined in relation to the distances between adjacent buildings and the proposed excavation footprint, in order to select suitable plant.

**Table 6: Approximate Buffer Distances for Excavation Plant**

| Excavation Plant | Buffer Distance | |
|---|---|---|
| | (from trial maxima)[1] | (from trial averages) |
| Provisional Allowed Vibration Limit: | 8 mm/s PPVi | |
| Likely equivalent maximum Vector Sum PPV: | 11 mm/s VSPPV | |
| Rock Saw on Excavator[2] | 0.8 m | 0.4 m |
| Ripper on 20t Excavator | 2.5 m | 0.9 m |
| Rock Hammer < 500 kg operating weight | 5.6 m | 2.2 m |
| Rock Hammer 501 - 1000 kg operating weight | 6.3 m | 2.6 m |
| Rock Hammer 1001 - 2000 kg operating weight | 9.7 m | 4.3 m |
| Rock Hammer > 2000 kg operating weight | 6.2 m | 4.3 m |

1. Smaller distances can generally be determined from individual trials, as indicated by those from trial averages.
2. Loading effects from buildings may reduce vibration levels, to enable boundary saw cuts with few exceedances.

## 8.6 Foundations

Footing loads are assumed to range from 3 000 kN to 15 000 kN (working) for the various multi-storey buildings.

It is anticipated that the medium and high strength sandstone (Unit 4) will be exposed at the BEL. Therefore, shallow footings with an allowable bearing pressure of 6 000 kPa are recommended provided than spoon testing is carried out in a third of footings across the site. This bearing pressure may be increased to 10 000 kPa where spoon testing or core drilling is carried out in each footing. Settlements are not expected to exceed 1% of the footing width for footings loaded to the above recommended maximum values.

The foundation design parameters assume that the foundation excavations (eg pads) are clean and free of loose debris, with pile sockets free of smear and adequately roughened immediately prior to the placement of concrete.

It is recommended that all load bearing foundations be inspected and spoon tested (as indicated above) by an experienced geotechnical engineer or engineering geologist.

## 8.7 Seismic Design

In accordance with the Earthquake Loading Standard, AS1170.4 – 2007, the site is assessed to have a Site Sub-Soil Class of “Be”.

## 8.8 Pavements

Based on experience within the region and the results of CBR testing, a California bearing ratio of 3% is recommended for filling and clay subgrades of pavements to surround the new buildings on site. Subgrades should be prepared in accordance with Section 8.2.

## 8.9 Floor Slabs

The ground floor slab at the lowest level of the basement is expected to be used for carparking and hence will probably only be lightly loaded. The base of the excavation will have sandstone exposed which will provide adequate support for a slab-on-grade. The final surface should be trimmed and scraped clean of debris etc.

## 8.10 Drainage

Surface and subsurface drainage should be incorporated into the design to protect footings and pavements. All collected stormwater and roof runoff should discharge into the stormwater disposal system.

## 9. Limitations

Douglas Partners (DP) has prepared this report for a project at 110 – 114 Herring Road, Macquarie Park, NSW in accordance with DP's proposal dated 25 November 2010 and acceptance received from Mr Anthony Rice of Stamford Property Services Pty Ltd on 30 November 2010. The report is provided for the exclusive use of Stamford Property Services Pty Ltd for this project only and for the purpose(s) described in the report. It should not be used for other projects or by a third party. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions only at the specific sampling or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of anthropogenic influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be limited by undetected variations in ground conditions between sampling locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion given in this report.

This report, or sections of this report, should not be used as part of a specification for a project without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

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**Douglas Partners Pty Ltd**

# Appendix A

About this Report
Drawings 1 - 7

# About this Report

Douglas Partners

## Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations 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.

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This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

## Borehole and Test Pit Logs

The borehole and test pit logs presented in this report 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 or excavation. 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 and test pits 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 or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

## Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole 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; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements 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.

## Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP 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 conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

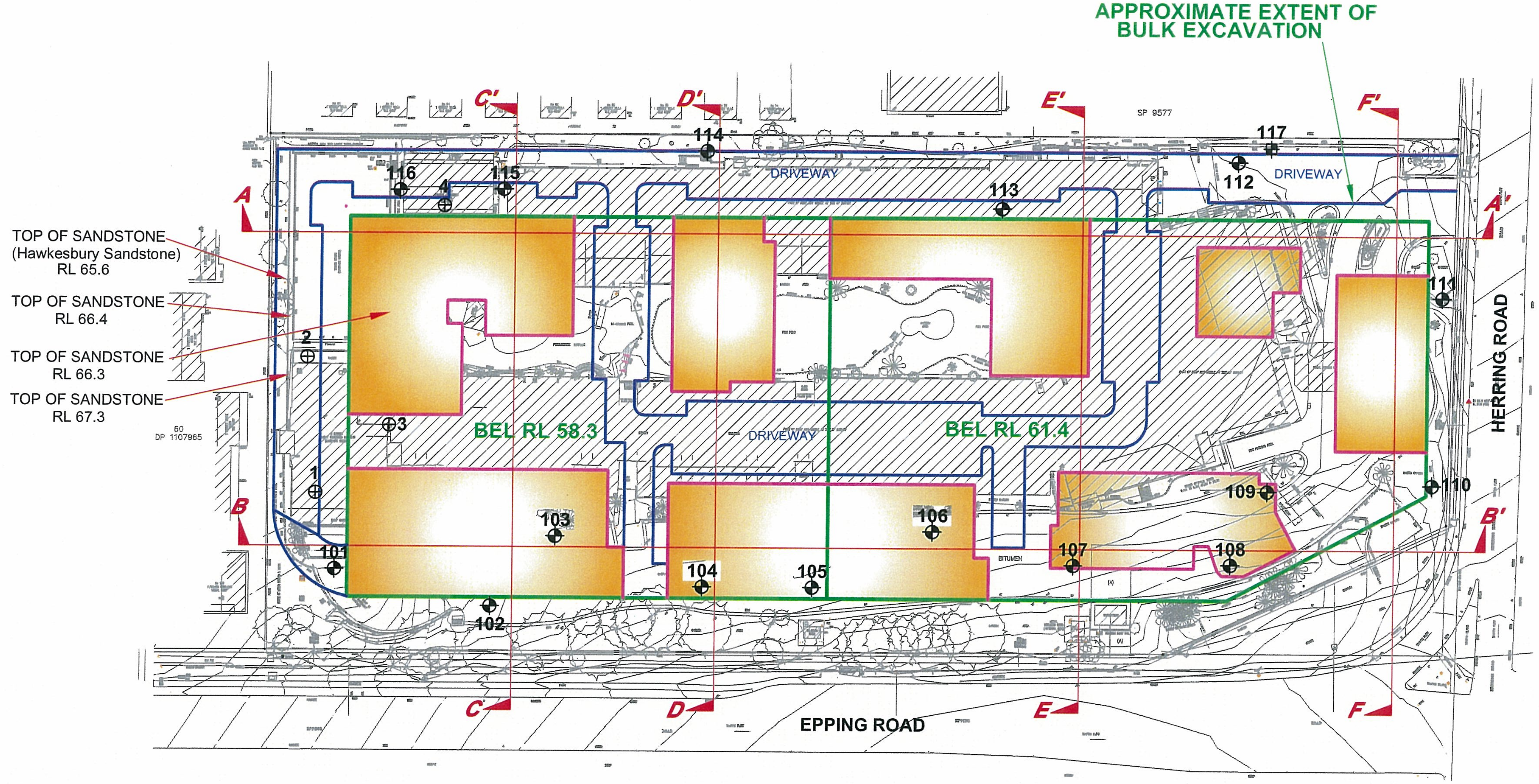
If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

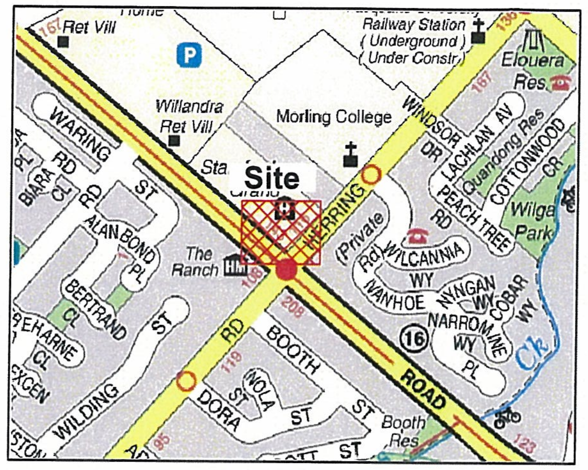
# About this Report

## 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, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

## Information for Contractual Purposes
Where information obtained from this report 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. DP 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 and environmental 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.




Locality Plan

NOTE:
Survey drawing from Denny Linker & Co
(Ref.100915, dated 13.10.2010)

0 10 20 30 40 50 75 100m

1:1000 @ A3

NOTE: For details of Sections A-A' to F-F' refer to Drawings 2-7

## LEGEND

- ⊕ Previous Test Bore ( October 2009)
- Current Test Bore (December 2010)
- Current Test Pit (December 2010)
- Proposed Multi-Storey Building Footprint
- BEL Bulk Excavation Level

**Douglas Partners**
*Geotechnics / Environment / Groundwater*

| | | |
|---|---|---|
| CLIENT: Stamford Property Services Pty Ltd | | |
| OFFICE: Sydney | DRAWN BY: PSCH | |
| SCALE: As shown | DATE: 14.1.2011 | |

TITLE: **Location of Test Bores**
**Proposed Macquarie Village**
**110 - 114 Herring Road, Macquarie Park**

MGA

| | |
|---|---|
| PROJECT No: | 72138 |
| DRAWING No: | 1 |
| REVISION: | B |

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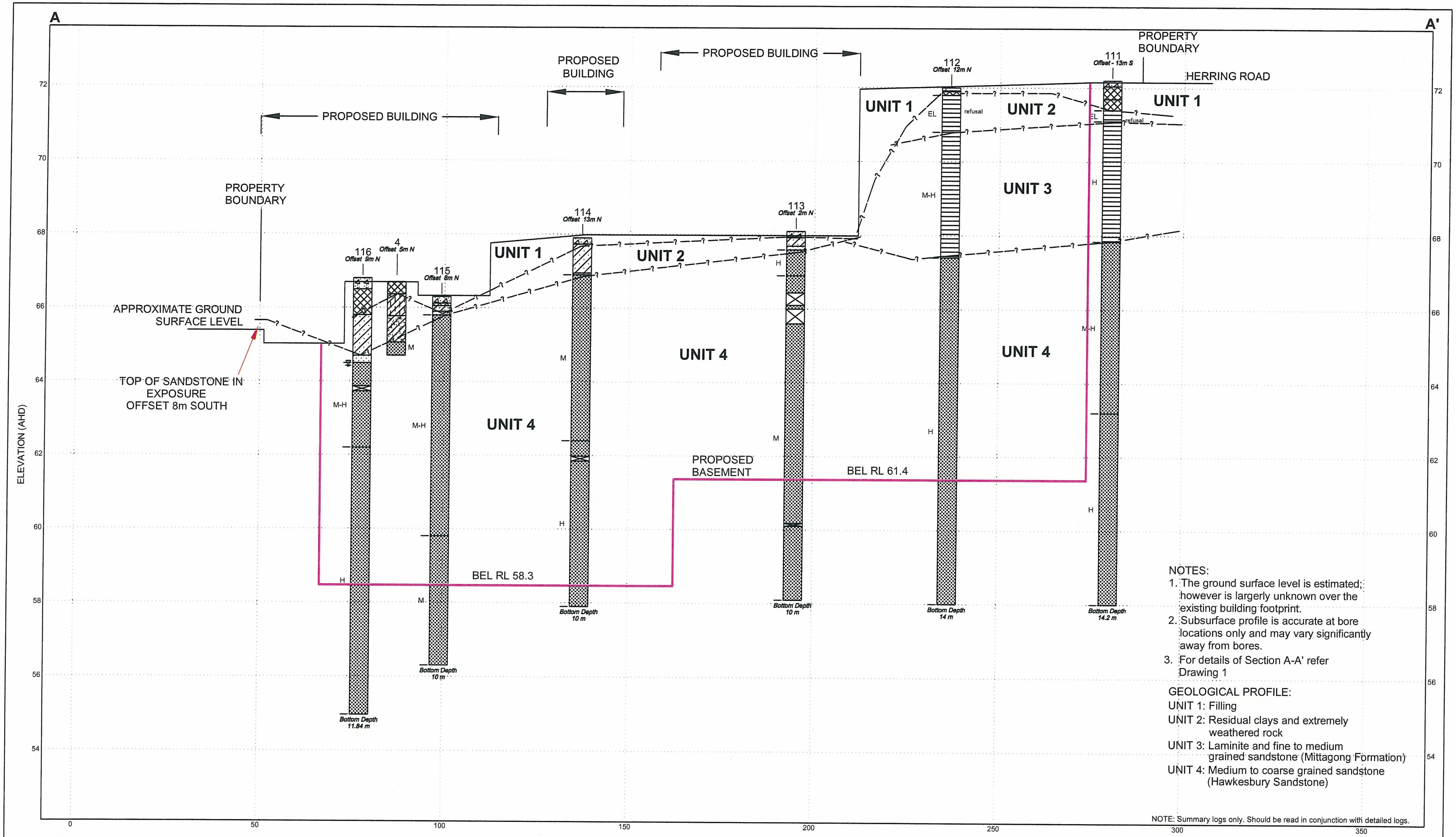



NOTES:
1. The ground surface level is estimated; however is largerly unknown over the existing building footprint.
2. Subsurface profile is accurate at bore locations only and may vary significantly away from bores.
3. For details of Section A-A' refer Drawing 1

GEOLOGICAL PROFILE:
UNIT 1: Filling
UNIT 2: Residual clays and extremely weathered rock
UNIT 3: Laminite and fine to medium grained sandstone (Mittagong Formation)
UNIT 4: Medium to coarse grained sandstone (Hawkesbury Sandstone)

LEGEND

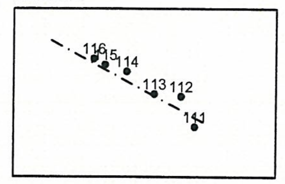
- Core Loss
- Concrete
- Filling
- Laminite
- Pavers
- Sandstone coarse grained
- Sandstone fine grained
- Sandy Clay
- Silty clay

ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High
VH - Very High

SOIL CONSISTENCY
vs - very soft
s - soft
f - firm
st - stiff
vst - very stiff
h - hard
vl - very loose
l - loose
md - medium dense
d - dense
vd - very dense

TESTS / OTHER
N - Standard penetration test value
Water level
BEL Bulk excavation level
Inferred boundary

SITE MAP




Douglas Partners
Geotechnics / Environment / Groundwater

| | |
|---|---|
| CLIENT: Stamford Property Services Pty Ltd | |
| OFFICE: Sydney | DRAWN BY: GRB/LD |
| SCALE: 1:1000 (H) 1:100 (V) @ A3 | DATE: 17.01.2011 |

TITLE: **INTERPRETED GEOLOGICAL SECTION A - A'**
**Proposed Macquarie Village**
**110-114 Herring Road, Macquarie Park**

| | |
|---|---|
| PROJECT No: | 72138 |
| DRAWING No: | 2 |
| REVISION: | 0 |

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