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Report on
Desktop Geotechnical Assessment

Macquarie Centre Redevelopment
Stage 1 Concept DA
Macquarie Park

Prepared for
AMP Capital

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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

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Proposed Macquarie Centre Redevelopment
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1. Introduction

This report presents the results of a desktop geotechnical assessment carried out by Douglas Partners Pty Ltd (DP) for the proposed Macquarie Centre Redevelopment, located between the corners of Waterloo Road, Herring Road, and Talavera Road, Macquarie Park. The assessment was commissioned on 23 October 2015 by Mr Derrick Burrows of Lend Lease on behalf of AMP Capital (AMPC), and was undertaken in accordance with DPs proposal dated 21 October 2015.

This report has been prepared on behalf of AMP Capital (AMPC) in support of a Stage 1 Development Application (DA) for the mixed use redevelopment of Macquarie Shopping Centre (Macquarie Centre). The Stage 1 DA seeks concept approval for the redevelopment of Macquarie Centre by establishing:

- Building envelopes and design parameters for future development on the site, including the proposed uses within the podium and tower components.
- The distribution of floor space across the site.
- Future pedestrian and vehicle connections to and within the site.

This report supports the proposed future redevelopment of the Macquarie Centre from a geotechnical perspective through a review of:

- Previous investigations undertaken on the site by DP over many years;
- Likely sub surface conditions across the site; and
- Potential geotechnical issues associated with the proposed development.

A preliminary contamination investigation (PSI) was undertaken by DP and has been reported separately (Project 85160.01).

2. Proposed Development

The Stage 1 DA seeks concept approval for the mixed use redevelopment of Macquarie Centre under s.83B of the *Environmental Planning & Assessment Act* 1979. The first stage will seek concept approval only for:

- Mixed use development to enable a range of land uses. The final mix of land uses will be subject to and determined under the relevant Stage 2 detailed DAs.
- Building envelopes for the proposed basement, expanded podium and tower forms.
- The four tower envelopes fronting Herring Road will have maximum heights ranging from 90m and 120m above existing ground level. The building envelope for Tower 1 is of sufficient dimensions to accommodate alternate tower forms.

- Maximum additional gross floor area (GFA) of 148,000sqm.
- The new retail podium along Herring Road will replace the existing structure. This will provide an active frontage with separate pedestrian entries to Herring Road and the creation of a vibrant atrium space.
- The creation of ‘Station Plaza’ between the train station and shopping centre, framed by active uses and a landmark building known as the “Shard”.
- The building envelopes for the proposed basement and upper levels of the expanded podium will accommodate a maximum of 2,175 additional car spaces.
- New vehicle and pedestrian access points.

The Stage 1 DA does not seek approval for

- Any works, including demolition, excavation, construction and public domain improvements.
- The final arrangement of land uses.
- Layout, mix and number of residential units.
- A specific number of car spaces (as this will be determined having regard to the final mix of land uses).
- The design of the building exteriors including facades and roofs.
- Public domain and landscape design.

Such approvals will be sought via subsequent development applications following receipt of development consent for the Stage 1 DA.

The overview of the indicative mix of land uses within the proposed building envelopes is identified in Table 1 below.

Table 1: Overview of Indicative Mix of Land Uses

Component	Proposed
Basement	Loading docks, car parking and associated vehicle circulation, waste rooms, utilities, future connection to existing train station (subject to consent from RailCorp) and retail premises.
Podium	Retail premises, commercial premises, food and drink premises, entertainment facilities, recreation facilities (indoor), recreation area, car parking and associated vehicle circulation, community uses (subject to further discussions with Council) and communal open space associated with the towers.
Tower 1	Mixed use development comprising commercial premises and/or residential accommodation and/or serviced apartments above a retail podium.
Towers 2, 3 and 4	Mixed use development comprising residential accommodation and/or serviced apartments above a retail podium.

3. Previous Investigations

DP has undertaken numerous investigations on the Macquarie Centre site between 1978 to 2014. Most of the previous rock cored borehole locations are shown on Drawing 1 in Appendix B. The borehole data and a geotechnical model are discussed in Section 6.

The following summarises the previous DP investigations;

- Geotechnical investigations between 1978-1980 for the original Macquarie Centre development (Project 6324). These investigations were commissioned by Concrete Constructions and included rock cored boreholes to depths of about 11 m;
- Geotechnical investigations between 1991 to 1995 for various extensions and alterations to the Macquarie Centre (Project 6324). These works were commissioned by Acer Wargon Chapman, consulting engineers at the time, and included thirteen rock cored boreholes to depths of about 5 m;
- Geotechnical investigations between 2010 to 2014 for various extensions and upgrades to the Macquarie Centre (Project 14121). These works were commissioned by Westfield Design & Construction and included thirty-two boreholes to depths of between 4.5 m to 10 m;
- Geotechnical inspections during construction for various stages to assess excavations and footings, mainly for the more recent extension on the north-eastern part of the site; and
- Various contamination assessments on the north-eastern portion of the site between 2011 to 2013 (further discussion is provided in the DP PSI contamination report).

Two boreholes (SH110 and SH111) were previously drilled by others near the Macquarie University railway station, near the south-western corner of the site. These boreholes were drilled for the Parramatta Rail Link and were taken to depths of approximately 30 m to 40 m.

4. Site Description

Macquarie Centre is approximately 11.25 hectares in area and is located at the corner of Waterloo Road, Herring Road and Talavera Road, Macquarie Park. The site is legally described as Lot 100 in DP 1190494.

The site is bound by Herring Road to the north west, Talavera Road to the north east, commercial uses to the south east and Waterloo Road to the south west. Located within the Macquarie Park Corridor, the site has excellent access to public transport, situated immediately adjacent the Macquarie University Railway Station and the Herring Road Bus Station. Located between the M2 Hills Motorway and Epping Road, the site also enjoys excellent vehicle connectivity.

Macquarie Centre was originally constructed in 1981. The centre has undergone various stages of redevelopment and extensions. A major refurbishment occurred in 2000, 2003 and most recently in 2014, creating a fresh food court, David Jones expansion, addition of second full line supermarket (Coles), a value supermarket (Aldi), with new speciality food and convenience stores. Today Macquarie Centre is the largest shopping centre in NSW and the 8th largest shopping centre in Australia and includes a wide range of retail, entertainment and service offerings.

The shopping centre currently spans five levels accommodating 368 stores, including major retailers such as David Jones, Myer, Target, Big W, Aldi, Coles and Woolworths. The centre also houses a large number of mini major international retails stores including H&M, Zara, Uniqlo, Forever 21, GAP and Sephora. A number of entertainment offerings exist in the centre including a cinema complex and ice skating rink. The site currently has a gross floor area of 170,850m2 and accommodates 4,755 car spaces.

The site boundaries are shown on Drawing 1 in Appendix B.

The Parramatta to Epping railway tunnels run below Waterloo Road adjacent to the southern site boundary. The Macquarie University railway station is located on Waterloo Road, near the south-western corner of the site. Drawings prepared by Allen Jack + Cottier Pty Ltd (for the Macquarie Centre Redevelopment) indicate that the station is set back approximately 7-8 m from the site boundary and the nearest tunnel is set back about 30 m from the site boundary. The bases of the railway station and the tunnels are about 20 m and 27 m below Waterloo Road, respectively.

5. Site Geology and Mapping

5.1 Geology

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Hawkesbury Sandstone. The Hawkesbury Sandstone typically comprises medium to coarse grained quartz sandstone with occasional shale or siltstone beds. The results of the previous investigations on the site confirmed the regional mapping with sandstone bedrock intersected at shallow depth.

Previous geotechnical investigations for the existing multi-storey carpark on the north-eastern part of the site encountered an igneous dyke about 0.5 m wide, running approximately north - south (see Drawing 1). Dykes are steeply dipping, often near-vertical igneous rock intrusions, which in Sydney, can vary from less than 1 m to about 6 m wide. Many dykes contain completely weathered basalt/dolerite (clay) near the surface but this grades to medium or high strength rock at depth. The host rock (i.e. sandstone) adjacent to dykes can be highly fractured and variable in strength due to the effects of the intrusion. Thermal alteration can sometimes also result in abnormally high strength host rock. Water seepage along the sides of the dyke can also occur due to the highly fractured host rock and deeply weathered igneous rock. Experience on the Macquarie Centre site has indicated that the dyke is about 0.5 m wide, near vertical, with fractured weaker sandstone about 1 m to 2 m wide on either side of the dyke.

Within the Sydney area the most common defects within the Hawkesbury Sandstone are widely spaced horizontal bedding planes, typically spaced at 1-3 m, and two orthogonal sets of steeply dipping joints. The joints typically have dips of 75 to 90 degrees from horizontal (i.e. close to vertical) and are oriented with strikes just east of north (about 010 degrees) and just south of east (about 110 degrees). Apart from these main defect sets there are likely to be other less common joints or faults with moderate dips of 20 – 30 degrees and 40 – 60 degrees.

5.2 Hydrogeology

A culvert associated with Shrimptons Creek runs in an approximate north-easterly direction below the eastern part of the site and feeds into the Lane Cove River which is located approximately 500 m to the north-east of the site.

A groundwater bore search of the NSW Office of Water database was conducted on 23 October 2015 for any bores registered within a 1 km radius of the site. Three bores were located at Macquarie University to the west of the site. The details provide indicate the bores are about 35 m to 46 m deep monitoring bores. The details are provided in Table 2 below.

Table 2: Summary of Groundwater Bores within 1km of site

Bore Identification	Standing Water Level (bgl)	Purpose
GW109694	Not supplied	Monitoring Bore
GW109696	Not supplied	Monitoring Bore
GW109695	Not supplied	Monitoring Bore

5.3 Acid Sulphate Soils

Reference to the 1:25000 Acid Sulphate Soil Risk Map (produced by the Department of Land and Water Conservation) indicates that the site and surrounding elevated areas are not underlain by known acid sulphate soils.

6. Geotechnical Model

Six geotechnical cross-sections (Sections 1 to 6) showing the interpreted subsurface profile between the boreholes on the site, are presented on Drawings 2 to 7 in Appendix B. The sections show interpreted geotechnical subdivisions of underlying soil and rock together with the approximate proposed basement outline on the western part of the site.

It should be noted that the interpreted boundaries shown on the sections are accurate at the borehole locations only and layers shown diagrammatically on this drawing are inferred only. Bands of lower and higher strength rock should be expected within the generalised layers. It is also noted that the surface levels on the earlier borehole logs were taken from site plans provided at the time and the reports do not confirm whether the levels are relative to AHD. Having said this, the more recent boreholes include surface levels relative to AHD and appear to be reasonably consistent with nearby older boreholes.

The general subsurface profile encountered within the previous boreholes, in increasing depth order, may be summarised as follows:

Filling: typically sand, clay and crushed sandstone filling to depths of 0.5 m to 2 m but up to depths of 3 m to 4 m in some areas. The deeper filling appears to be generally encountered near the Shrimptons Creek alignment;

Clay:	stiff residual clay and sandy clay, generally less than 1 m thick, and only encountered at some locations;
Sandstone:	underlying the filling and residual clay at depths of between 0.5 m to 4 m. The sandstone profile generally included extremely low to low strength rock about 1 m to 2 m thick over low to medium strength rock grading to medium to high strength rock, typically below depths of 3 m to 5 m. High strength, mostly slightly fractured and unbroken sandstone was encountered at depths of about 6 m to 8 m. Bands of very high strength rock were encountered at some locations; and
Dyke:	a near vertical, 0.5 m wide igneous dyke is present on the north-eastern part of the site (see Drawing 1). The sandstone to 1-2 m either side of the dyke, is highly fractured and of variable strength, with heavily iron stained joints

Groundwater seepage should be expected from along the top of clayey soils and the bedrock surface and are likely to occur within fractured zones and joints within the rock. Higher seepage flows may be encountered near Shrimptons Creek and the igneous dyke. Seepage flows are likely to increase following periods of extended wet weather.

7. Comments

The following preliminary comments are provided based on previous borehole data and preliminary details of the proposed development. Further investigations involving targeted cored boreholes will be required to confirm the subsurface profile for detailed design and planning.

Numerical modelling of the proposed excavation adjacent to the rail station and tunnels will be required to assess potential ground movements and to facilitate design of excavation support in this area. This modelling will also be required by TfNSW prior to approval of the proposed works adjacent to the rail infrastructure.

7.1 Excavations

The drawings by AJ+C indicate that the common basement below the proposed towers on the western end of the site will require excavation to depths of about 16 m below Waterloo Road level. The depth of excavation will reduce to the north to less than about 5 m below Talavera Road level. The lowest basement level is set back about 10 m from Waterloo Road and therefore excavation depths of about 13 m will be required on the Waterloo Road boundary.

The architectural drawings also indicate that the Macquarie University railway station is about 20 m below Waterloo Road and set back about 7-8 m from the site boundary. The rail tunnels are about 27 m below Waterloo Road and set back about 30 m from the site boundary. The proposed excavation is therefore expected to be “greater than 2 m depth and located within 25 m (measured horizontally) from the adjacent rail corridor” (as defined under State Environment Planning Policy (Infrastructure) 2007). More accurate details of the station and tunnels will be required for detailed design and planning purposes.

7.1.1 Excavation Conditions

It is expected that the proposed basement will require the excavation of soil and extremely low to low strength rock grading to medium to high and high strength, slightly fractured and unbroken sandstone.

Excavation of soil and extremely low to low strength rock should be achievable using conventional earthmoving equipment, however, the assistance of rock hammering or ripping will probably be required for effective removal of any medium to high strength ironstone bands within the weathered rock profile. It is anticipated that excavation of medium to high strength rock may require moderate to heavy ripping with a large bulldozer. High strength rock will probably require hydraulic rock breakers in conjunction with heavy ripping for effective removal of this material.

The detailed excavation for footings and services, or side trimming, within medium to high strength rock will generally require the use of a rotary rock saw or milling head, possibly in conjunction with a hydraulic rock hammer. Rock saws or milling heads may also be required to reduce vibrations near existing structures for human comfort and to reduce the potential for causing damage to such structures.

7.1.2 Vibrations

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits. The level of acceptable vibration is dependent on various factors including the type of 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 structure and the vibration transmitting medium.

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/s peak particle velocity (PPVi). This is generally much lower than the vibration levels required to cause structural damage to buildings. The Australian Standard AS2670.2-1990 “Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)” indicates an acceptable day time limit of 8 mm/s PPVi for human comfort.

Based on the experience of DP and with reference to AS2670, it is suggested that a maximum PPVi of 8 mm/s (applicable at the foundation level of existing buildings) be employed at this site for both architectural and human comfort considerations, although this vibration limit may need to be reduced if there are sensitive buildings or equipment in the area.

As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of rock excavation. The trial may indicate that smaller or different types of excavation equipment should be used for bulk (or detailed) excavation purposes.

7.1.3 Dilapidation Surveys

Dilapidation surveys should be carried out on adjacent buildings, pavements and rail infrastructure that may be affected by the excavation works. The dilapidation surveys should be undertaken before the commencement of any excavation work in order to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed.

7.1.4 Disposal of Excavated Material

All excavated materials will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the *Waste Classification Guidelines* (EPA, 2014). This includes filling and natural materials that may be removed from the site. Accordingly, environmental testing will need to be carried out to classify spoil prior to transport from the site. Reference should be made to DP PSI contamination report for comments on the potential contamination status of the soils.

7.2 Excavation Support

Vertical excavations within the filling, soils and extremely low to low strength rock will require both temporary and permanent lateral support during and after excavation. If there is sufficient room for temporary slopes to be used during construction then the maximum temporary batter slopes that should be adopted in the filling and soils are 1:1 (horizontal to vertical). Surcharge loads should be set well back from the crest of batter slopes.

The low to medium strength or stronger sandstone will generally be stable when cut vertically provided there are no adversely oriented joints or other defects present. All vertical faces in rock should be inspected by a geotechnical engineer at regular depth intervals to check for adversely inclined joints and to assess whether additional stabilisation measures (such as rock bolts or shotcrete) are required.

Given that the typical main joint sets within Hawkesbury Sandstone in the Sydney region are oriented at a slight angle to the proposed excavation faces, it is expected that there will be some narrow wedges formed where these near vertical joints intersect the excavation faces and some rock bolts may be required to stabilise these wedges.

On this site the design of the excavations and support systems will have to take into account the footings for the existing shopping centre, as well as any requirements from Transport of NSW (TfNSW) in relation to the rail station and easement. Generally TfNSW do not allow installation of anchors or bolts within the rail easement. In some cases TfNSW have allowed the use of fibreglass bolts within an easement where future tunnels are proposed. Further review and discussion with TfNSW will be required to assess excavation and support requirements adjacent to the rail easement and in particular Macquarie University station.

Retaining walls will be required to support the filling, soils and extremely low to low strength sandstone. Anchored soldier pile walls are often used to provide temporary retaining support to soils and weathered rock, but on this site anchors are unlikely to be permitted within the rail easement on Waterloo Road. It may be necessary to set the basement further back from the Waterloo Road boundary to allow some room for rock bolts and anchors.

Soldier pile shoring walls with either timber lagging or shotcrete infill panels are often used for temporary support of soils and weak rock. The soldier piles are usually spaced at approximately 2 m to 2.5 m centres, however, closer spaced piles may be required to reduce wall movements, or prevent collapse of infill materials, particularly where pavements, structures or services are located in close proximity to the excavation.

It is suggested that preliminary design of cantilevered shoring systems (or shoring with one row of anchors) be based on a triangular earth pressure distribution using the earth pressure coefficients provided in Table 3. 'Active' earth pressure coefficient (K_a) values may be used where some wall movement is acceptable, and 'at rest' earth pressure (K_o) values should be used where the wall movement needs to be limited.

Table 3: Recommended Design Parameters for Shoring Systems

Material	Unit Weight (kN/m^3)	Earth Pressure Coefficient		Effective Cohesion c' (kPa)	Effective Friction Angle (Degrees)
		Active (Ka)	At Rest (Ko)		
Filling	20	0.4	0.6	2	20
Clay and Sandy Clay	20	0.3	0.5	5	20
Extremely Low to Low Strength Sandstone	22	0.1	0.15	10	25
Medium Strength Sandstone	24	0	0	30	40

The design of the shoring should allow for all surcharge loads, including building footings, inclined slopes behind the wall, traffic and construction related activities.

Shoring walls should also be designed for full hydrostatic pressures unless drainage of the ground behind impermeable walls can be provided. Drainage could comprise 150 mm wide strip drains pinned to the face at 2 m centres behind shotcrete in-fill panels. The base of the strip drains should extend out from the shoring wall to allow any seepage to flow into a perimeter toe drain which is connected to the stormwater drainage system.

Passive resistance for piles founded in rock below the base of the bulk excavation (including allowance for services or footings) may be based on the ultimate passive restraint value provided in Table 4. This ultimate value represents the pressure mobilised at high displacements and therefore it will be necessary to incorporate a factor of safety to limit wall movement. The top 0.5 m of the socket should be ignored due to possible disturbance and over-excavation.

Table 4: Recommended Passive Resistance Values

Foundation Stratum	Ultimate Passive Pressure (kPa)
Low strength sandstone	2,000
Medium strength or stronger sandstone	4,000

7.3 Foundations

7.3.1 General

It is expected that the bulk excavation for the basement below the towers will expose medium strength or stronger sandstone. High strength sandstone is expected in the deeper parts of the excavation towards the Waterloo Road boundary.

Pad founded on the sandstone (Class III and Class II) may be designed using the values given in Table 5. Shaft adhesion values for uplift (tension) may be taken as being equal to 70% of the values for compression.

Table 5 : Recommended Design Parameters for Foundation Design

Foundation Stratum	Maximum Allowable Pressure (Serviceability)		Maximum Ultimate Pressure (Ultimate)		Young's Modulus, E (MPa)
	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	
Medium strength sandstone (Class III)	3,500	350	20,000	600	500
High strength sandstone (Class II)	6,000	600	40,000	1,000	1,000

Higher allowable bearing pressures of about 10,000 kPa could be adopted in high strength sandstone (Class I) with very minor fractures and defects. This would be subject to further review and investigation and also spoon testing in all footings.

Foundations proportioned on the basis of the allowable bearing pressure in Table 5 would be expected to experience total settlements of less than 1% of the footing width under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value.

All footings should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters. Spoon testing should be carried out in one third of all footings that are designed for an allowable end bearing capacity of greater than 3,500 kPa. Spoon testing involves drilling a 50 mm diameter hole below the base of the footing, to a depth of 1.5 times the footing width, followed by testing to check for the presence of weak/clay bands. If weak seams are detected then footings may need to be taken deeper to reach suitable foundation material.

7.3.2 Footings Adjacent to Rail Corridor

TfNSW standard requirements are usually that no footings can be located above a line which extends at 45 degrees up from the lowest excavation level for the proposed station or rail tunnel. However, these requirements have been developed to cover all sites, including those with deep soils. On this site, where the previous investigations in the vicinity of the railway station have indicated that the strata generally comprises low to medium strength or stronger sandstone with relatively few defects, it is likely that detailed numerical modelling would show that high level footings founded at shallower depths would have minimal impact on the rail infrastructure. This solution, if required, would require detailed modelling to be undertaken during the detailed design stage and would require negotiations and agreement with TfNSW.

It is noted that the current preliminary drawings indicate that the bulk excavation and footings adjacent to the rail corridor would probably be located below a line drawn up at 45 degrees from the rail station.

7.4 Seepage

Groundwater was not generally encountered during auger drilling of the previous boreholes. Water has been measured in some wells at depths of between about 5 m to 10 m, however this is likely to be associated with perched seepage. Some seepage flows should be expected along the interface of soil and bedrock together with minor seepage through fractures and joints in the rock, particularly following periods of wet weather.

During construction and in the long term, it is anticipated that seepage into the excavation will be relatively minor and should be readily controlled by perimeter drains connected to a "sump-and-pump" system. Permanent subfloor drainage should be provided below the basement floor slab to direct seepage to the stormwater drainage system. It is likely that iron oxide will precipitate from the groundwater seepage and may lead to a build-up of an iron-oxide sludge. Allowance for periodic cleaning of such sludge should be made in the long term maintenance requirements.

7.5 Excavation Induced Ground Movement and Adjacent Rail Infrastucture

A survey plan and section showing the rail corridor and infrastructure in relation to the site boundaries and proposed development will be required for detailed assessment and review by Transport for NSW. It is expected that the proposed excavations will be greater than 3 m depth within the second reserve boundary, as outlined in *ECRL Underground Infrastructure Protection Guidelines Report No. 20007300/PO-4532,* prepared by Transport Infrastructure (2008). Numerical modelling of the proposed excavation adjacent to the rail infrastructure will therefore be required to assess potential ground movements and to satisfy TfNSW requirements and assessment requirements outlined in the ECRL guidelines.

Horizontal movements due to stress relief will occur during the excavation works. Based on published literature and DP's experience, the lateral deflections associated with excavation in Hawkesbury Sandstone is likely to be in the order of 0.05% to 0.15% of the excavation height, which corresponds to approximately 8 - 24 mm for a 16 m depth of excavation. The predicted deflections would generally be greatest at the centre of the excavated faces and would reduce with distance from the excavation face/boundary.

It is expected that deflections at the shaft for the railway station, which is set back about 7-8 m from the boundary would be less than half the values indicated above due to the set back and also due to stress relief that would have occurred during the station and tunnel construction. It is expected that the proposed excavation would have negligible impact on the tunnels that are understood to be set back about 30 m from the boundary and located about 27 m below Waterloo Road.

It is unlikely to be practicable to provide restraint for the relatively high in-situ horizontal stresses associated with stress relief movements. Therefore it is recommended that appropriate allowance be made for movements of this order in construction and planning.

Precise survey monitoring of the existing footings and buildings/structures should be carried out to assess vertical and horizontal movements during the excavation. The survey should commence prior to excavation to provide a baseline and should continue every 1.5 m drop of the excavation. If surveyed deflections show a rapid increase in the rate of movement or exceed the predicted movements, then the structural engineer and geotechnical engineer should be contacted for immediate review.

8. Limitations

Douglas Partners (DP) has prepared this report for this project at the Macquarie Centre for AMP Capital (AMPC) and was undertaken in accordance with Douglas Partners' proposal dated 21 October 2015 and acceptance received on 23 October 2015 by Mr Derrick Burrows of Lend Lease on behalf of AMPC.

The work was carried out under a consultancy agreement between AMPC and DP. This report is provided for the exclusive use of AMPC for the specific project and purpose as described in the report. It should not be used by or be relied upon for other projects or purposes on the same or another site or by a third party. 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 encountered in previous investigations and 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 have occurred on this site since the previous investigations were undertaken.

DP's advice is based upon the conditions encountered during previous investigations. The accuracy of the advice provided by DP in this report may be limited by undetected variations in ground conditions between sampling locations.

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 from 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 instruction for construction.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires a risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the (geotechnical / environmental / groundwater) components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

Appendix A

About this Report

About this Report

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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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.

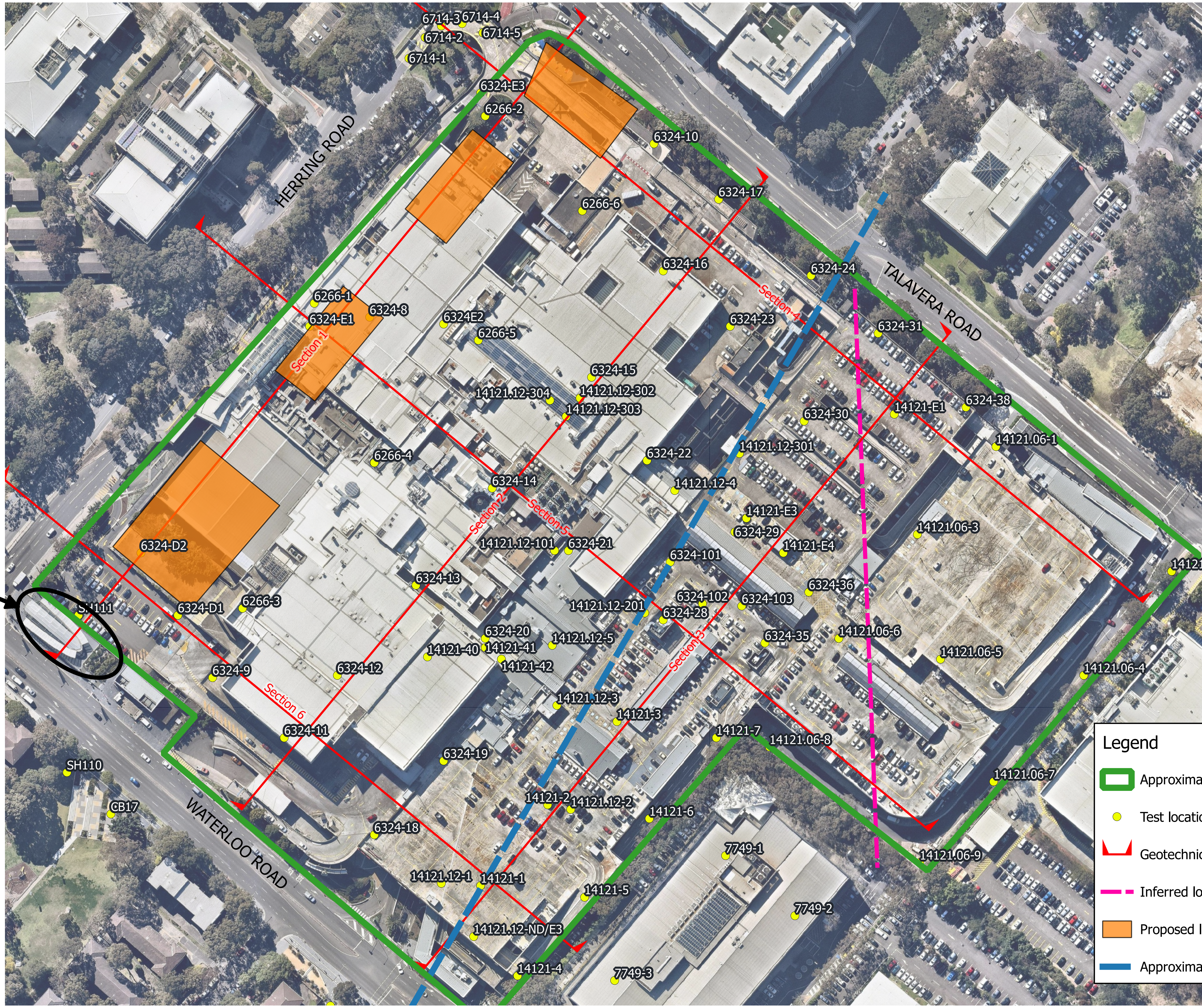
Site Inspection

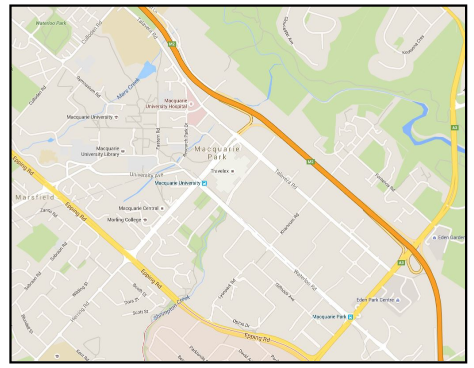
The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Appendix B

Drawings

Macquarie University Station



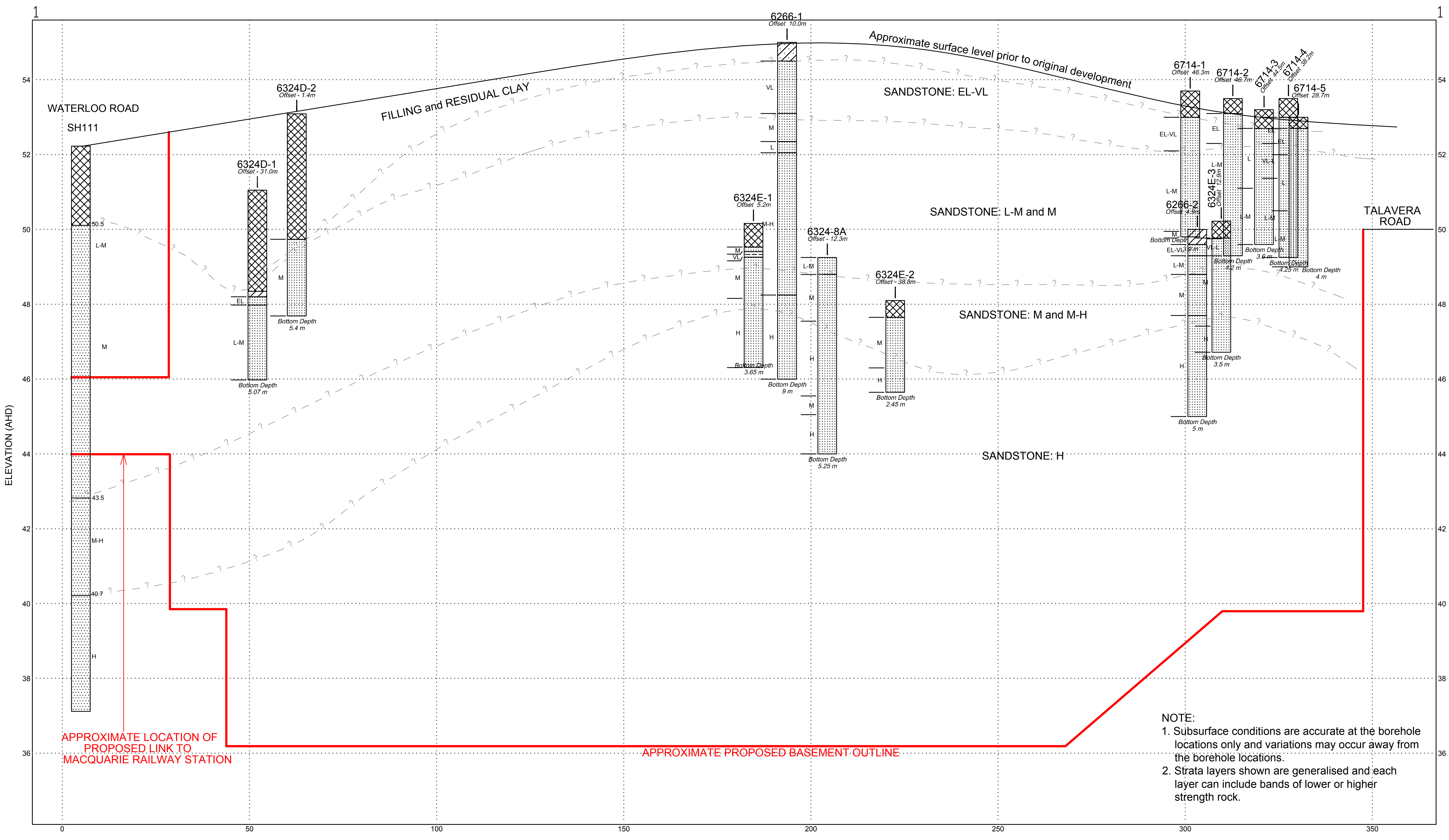


Site Locality

Legend

- Approximate site boundary
- Test locations
- Geotechnical cross sections
- Inferred location of igneous dyke
- Proposed location of towers
- Approximate location of culvert (Shrimptons Creek)

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: AMP Capital		TITLE: **Location of Previous Tests and Site Features**	PROJECT No: 85160.00
	OFFICE: Sydney	DRAWN BY: AP	**Macquarie Centre Redevelopment Desktop Study**	DRAWING No: 1
	SCALE: 1:1800 @ A3	DATE: 29.10.2015	**Corner Waterloo Road, Herring Road and Talavera Road, Macquarie Park**	REVISION: 0



NOTE:
1. Subsurface conditions are accurate at the borehole locations only and variations may occur away from the borehole locations.
2. Strata layers shown are generalised and each layer can include bands of lower or higher strength rock.

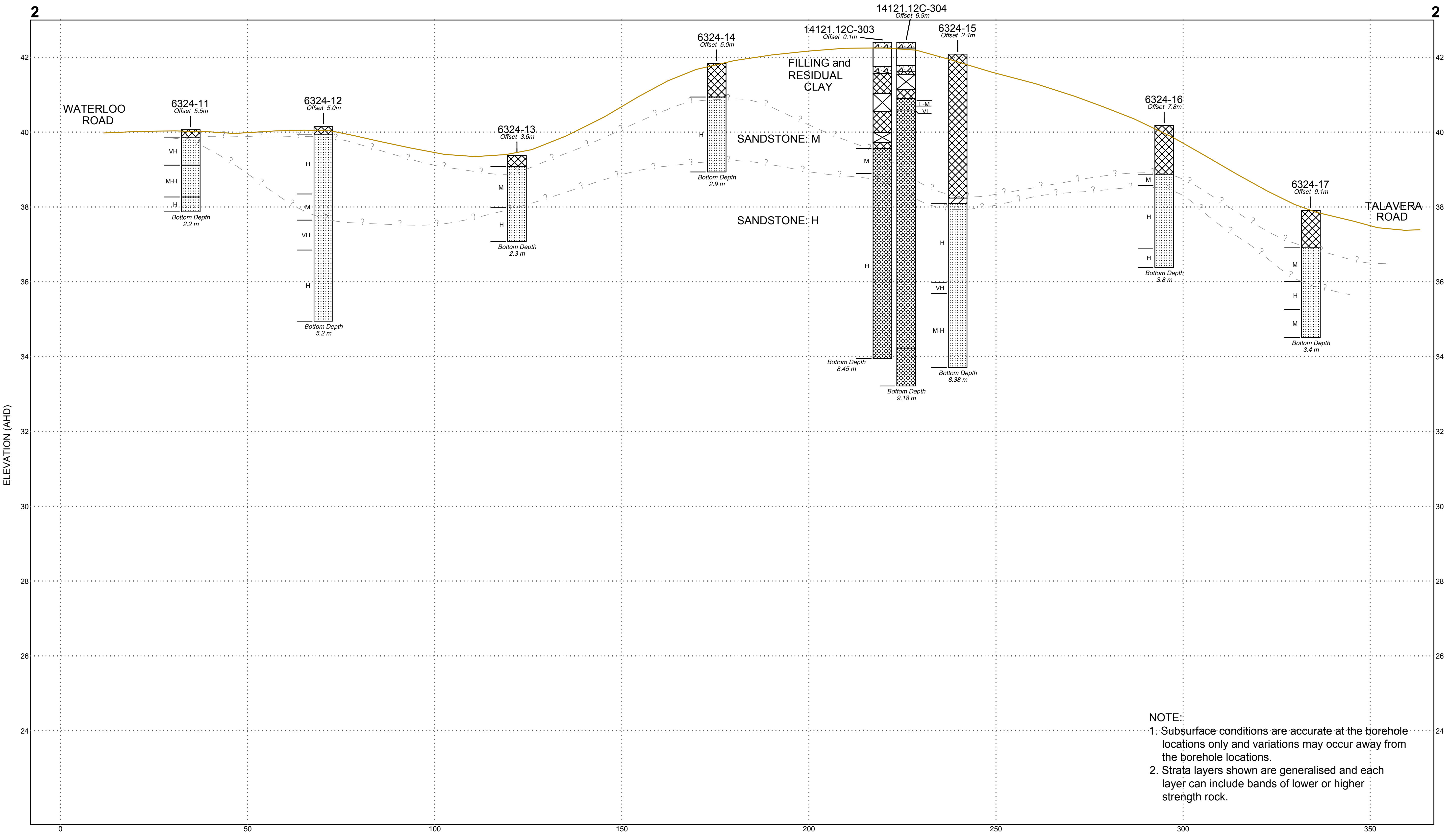
LEGEND
- Filling
- Clay
- Sandy Clay
- Shale
- Sandstone

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

TESTS/ OTHERS
— ? — — Interpreted geotechnical boundary

0 20
Horizontal Scale (metres)
Vertical Exaggeration = 10.0

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: AMP Capital	TITLE: **Interpreted Geotechnical Cross-Section 1**	PROJECT No: 85160.00
	OFFICE: Sydney — DRAWN BY: AP/ PSCH	**Macquarie Centre Redevelopment**	DRAWING No: 2
	SCALE: 1:1000 (H) 1:100 (V) @ A3 — DATE: 27.10.2015	**Macquarie Park**	REVISION: 0



NOTE:
1. Subsurface conditions are accurate at the borehole locations only and variations may occur away from the borehole locations.
2. Strata layers shown are generalised and each layer can include bands of lower or higher strength rock.

LEGEND

- Core Loss
- Clay
- Concrete
- Filling
- Sandstone
- Sandstone coarse grained

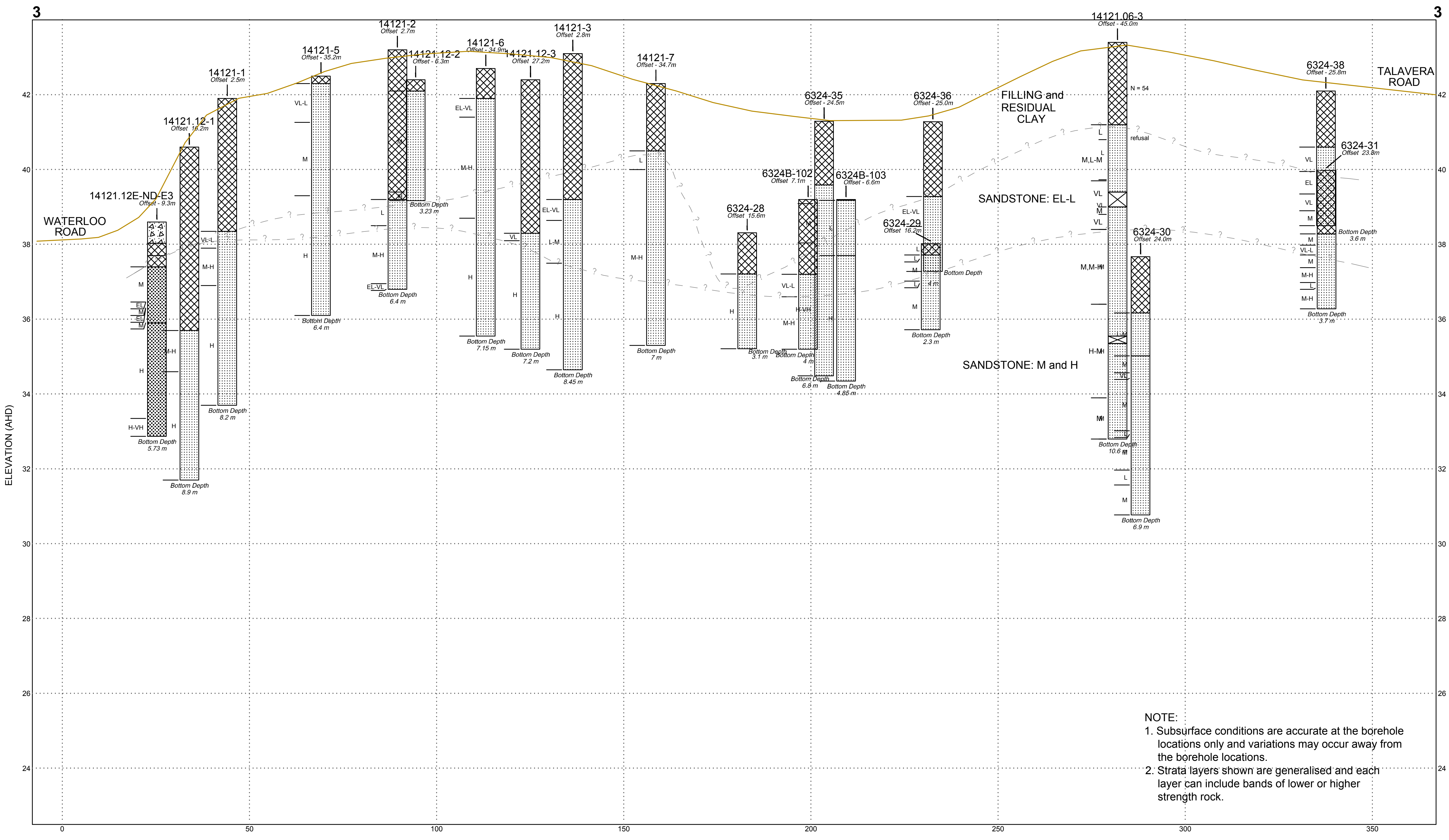
ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High
VH - Very High

TESTS / OTHER
N - Standard penetration test value
Water level
PP - Pocket penetrometer test value

Horizontal Scale (metres) 0 20

Vertical Exaggeration = 10.0

Douglas Partners — Geotechnics / Environment / Groundwater	CLIENT: AMP Capital	TITLE: **Interpreted Geotechnical Cross-Section 2**	PROJECT No: 85160.00
	OFFICE: Sydney / DRAWN BY: SE/PSCH	**Macquarie Centre Redevelopment**	DRAWING No: 3
	SCALE: 1:1000 (H) 1:100 (V) @ A3 / DATE: 27.10.2015	**Macquarie Park**	REVISION: 0



NOTE:
1. Subsurface conditions are accurate at the borehole locations only and variations may occur away from the borehole locations.
2. Strata layers shown are generalised and each layer can include bands of lower or higher strength rock.

LEGEND

- Core Loss
- Asphaltic Concrete
- Concrete
- Filling
- Roadbase
- Sandstone
- Sandstone coarse grained

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

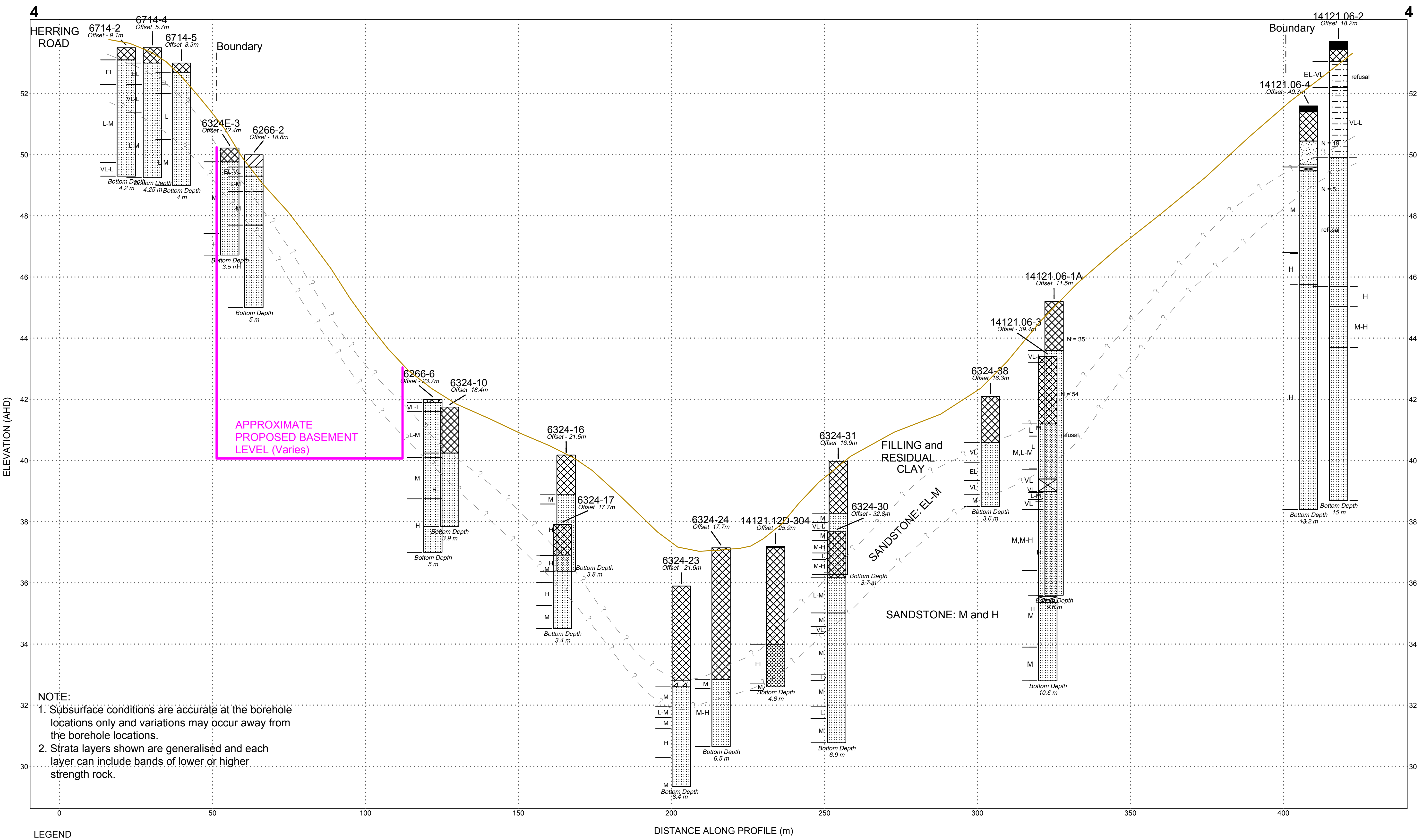
TESTS / OTHER
N - Standard penetration test value
▽ - Water level
PP - Pocket penetrometer test value

0 20
Horizontal Scale (metres)
Vertical Exaggeration = 10.0

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: AMP Capital	
	OFFICE: Sydney	DRAWN BY: SE/PSCH
	SCALE: 1:1000 (H) 1:100 (V) @ A3	DATE: 27.10.2015

TITLE: **Interpreted Geotechnical Cross-Section 3**
Macquarie Centre Redevelopment
Macquarie Park

PROJECT No:	85160.00
DRAWING No:	4
REVISION:	0



NOTE:
1. Subsurface conditions are accurate at the borehole locations only and variations may occur away from the borehole locations.
2. Strata layers shown are generalised and each layer can include bands of lower or higher strength rock.

LEGEND

Core Loss	Filling	Siltstone
Asphaltic Concrete	Sand	Topsoil
Clay	Sandstone	
Concrete	Sandstone coarse grained	

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

TESTS / OTHER
- N - Standard penetration test value
- Water level
- PP - Pocket penetrometer test value

0 24
Horizontal Scale (metres)
Vertical Exaggeration = 10.0

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: AMP Capital	TITLE: Interpreted Geotechnical Cross-Section 4	PROJECT No: 85160.00
	OFFICE: Sydney / DRAWN BY: SE/PSCH	Macquarie Centre Redevelopment	DRAWING No: 5
	SCALE: 1:1200 (H) 1:120 (V) @ A3 / DATE: 27.10.2015	Macquarie Park	REVISION: 0

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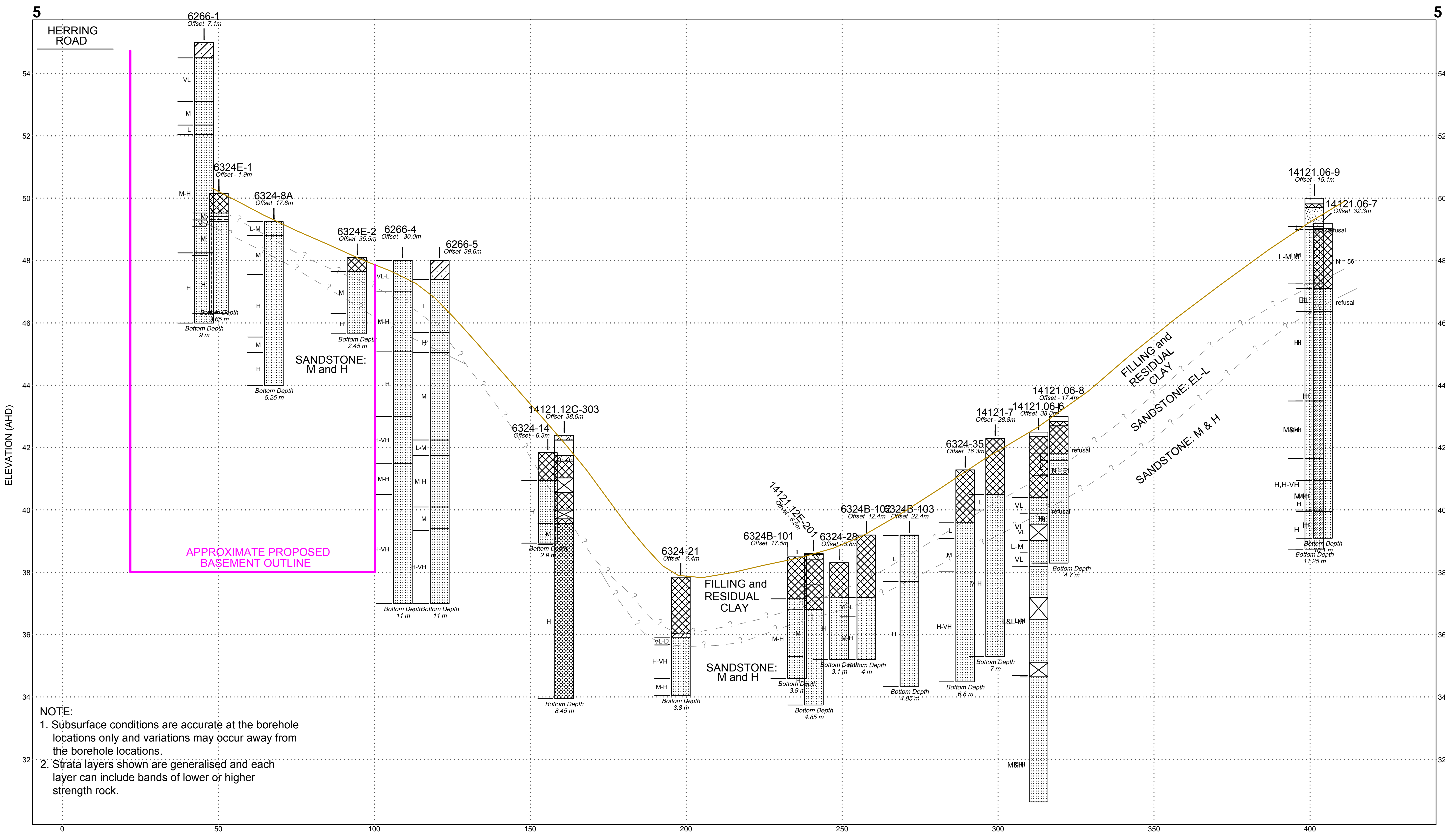

LEGEND

Core Loss	Filling	Sandstone
Asphaltic Concrete	Roadbase	Sandstone coarse grained
Concrete	Sand	Sandy Clay
		Shale

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

TESTS / OTHER
N - Standard penetration test value
Water level
PP - Pocket penetrometer test value

Horizontal Scale (metres)
Vertical Exaggeration = 10.0

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: AMP Capital	TITLE: **Interpreted Geotechnical Cross-Section 5**	PROJECT No: 85160.00
	OFFICE: Sydney / DRAWN BY: SE/PSCH	**Macquarie Centre Redevelopment**	DRAWING No: 6
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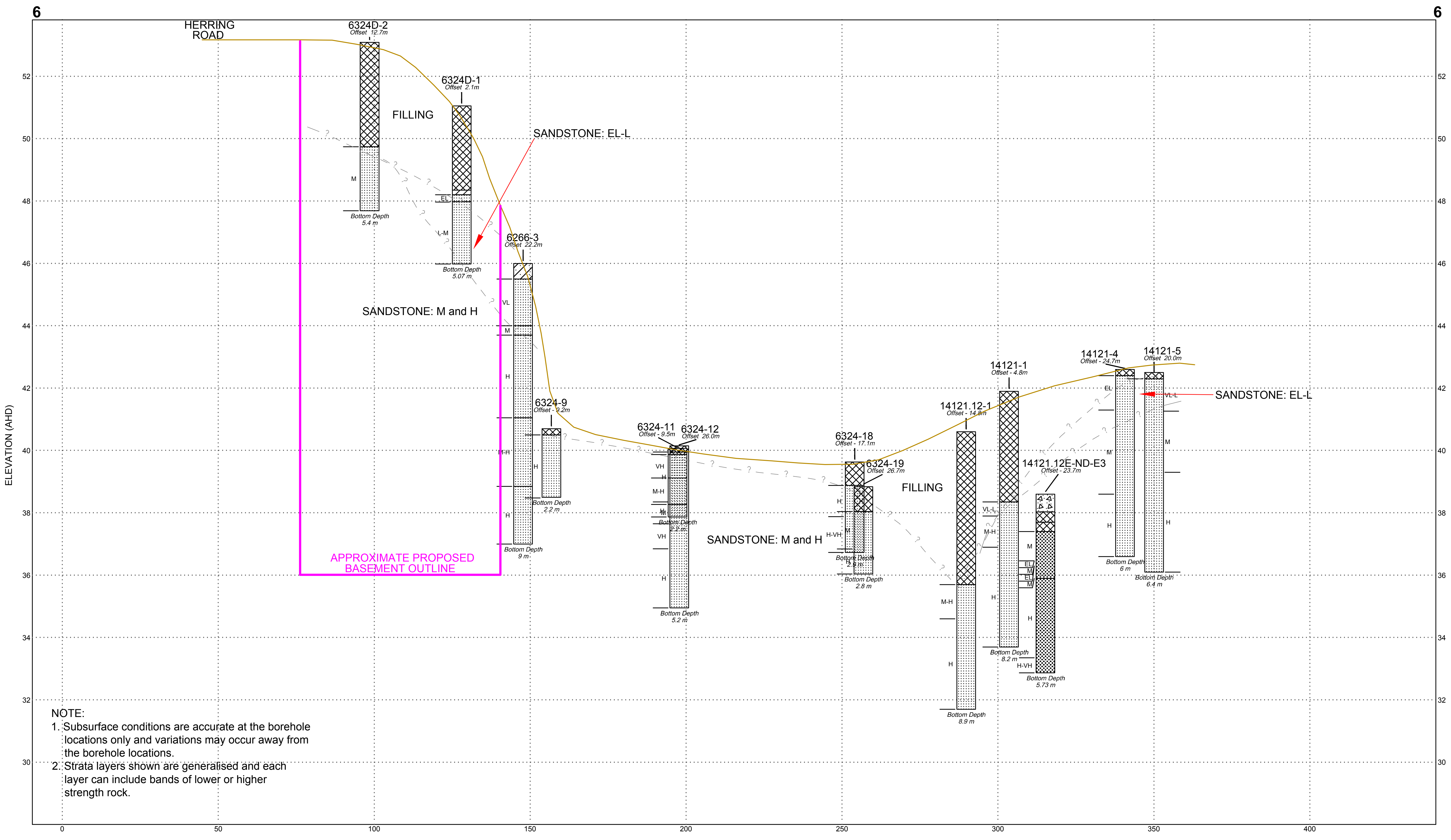

NOTE:
1. Subsurface conditions are accurate at the borehole locations only and variations may occur away from the borehole locations.
2. Strata layers shown are generalised and each layer can include bands of lower or higher strength rock.

LEGEND

- Filling
- Sandstone
- Concrete
- Sandstone coarse grained
- Sandy Clay
- Clay

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

TESTS / OTHER
N - Standard penetration test value
▽ - Water level
PP - Pocket penetrometer test value

0 24
Horizontal Scale (metres)
Vertical Exaggeration = 10.0

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: AMP Capital	TITLE: **Interpreted Geotechnical Cross-Section 6**	PROJECT No: 85160.00
	OFFICE: Sydney / DRAWN BY: SE/PSCH	**Macquarie Centre Redevelopment**	DRAWING No: 7
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