Douglas Partners
Geotechnics / Environment / Groundwater

Integrated Practical Solutions

# Geotechnical Desktop Study

Proposed Mixed Use Commercial Development

Lots 8, 9 and 12 Australian Technology Park
Eveleigh

Prepared for
Mirvac Projects Pty Ltd

Project 84955.01
December 2015

Douglas Partners
Geotechnics | Environment | Groundwater

## Document History

Document details

| | | | |
|---|---|---|---|
| Project No. | 84955.01 | Document No. | 2 |
| Document title | Geotechnical Desktop Study<br>Mixed Use Commercial Development | | |
| Site address | Lots 8, 9 and 12 Australian Technology Park, Eveleigh | | |
| Report prepared for | Mirvac Construction | | |
| File name | P:\84955.01 - EVELEIGH ATP Geotechnical Support\8.0 Documents\8.2 Out\8.2.1 Current\84955.01.R.002.Rev4.doc | | |

Document status and review

| Revision | Prepared by | Reviewed by | Date issued |
|---|---|---|---|
| 0 | Josef Major | John Braybrooke | 20 November 2015 |
| 1 | Josef Major | John Braybrooke | 2 December 2015 |
| 2 | Josef Major | John Braybrooke | 3 December 2015 |
| 3 | Josef Major | John Braybrooke | 4 December 2015 |
| 4 | Josef Major | John Braybrooke | 9 December 2015 |

Distribution of copies

| Revision | Electronic | Paper | Issued to |
|---|---|---|---|
| 0 | 1 | 0 | Mirvac Projects |
| 1 | 1 | 0 | Mirvac Projects |
| 2 | 1 | 0 | Mirvac Projects |
| 3 | 1 | 0 | Mirvac Projects |
| 4 | 1 | 0 | Mirvac Projects |

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.

| | Signature | Date |
|---|---|---|
| Author | | 9 December 2015 |
| Reviewer | JBraybrooke | 9 December 2015 |

NCS INTERNATIONAL
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

# Geotechnical Report
# Proposed Mixed Use Commercial Development
# Lots 8, 9 and 12 ATP, Eveleigh

## 1. Introduction

This report supports a State Significant Development Application (SSDA) submitted to the Department of Planning and Environment pursuant to Part 4 of the Environmental Planning and Assessment Act 1979 (EP&A Act). Mirvac Projects Pty Ltd (Mirvac) is seeking to secure approval for the redevelopment of three car parking lots within the Australian Technology Park (ATP) for commercial, retail and community purposes along with extensive upgrade of existing public domain areas. Building heights of 4, 7 and 9 storeys are proposed across the three development lots. Australian Technology Park (ATP) has been continuously developed since its establishment in 1996, but has been underutilised as a technology and business precinct for quite some time. UrbanGrowth NSW Development Corporation (UGDC) has actively encouraged new development and employment opportunities at the Park for the past 15 years, and Mirvac intends to continue upon this and deliver on the precinct's full potential, with the development of circa 107,400 m$^2$ for employment uses, which will facilitate the employment homes of an extra 10,000 staff everyday within ATP by development completion.

Mirvac has been announced by UrbanGrowth NSW as the successful party in securing ownership and redevelopment rights for the ATP precinct, following an Expression of Interest (EOI) and an Invitation to Tender (ITT) process which commenced in 2014. Mirvac has also secured the Commonwealth Bank of Australia (CBA) as an anchor tenant for the development and intends to immediately commence the urban regeneration of this precinct through the lodgement of this SSDA. CBA's commitment to the precinct is in the form of one of the largest commercial leasing pre-commitments in Australian history, occupying circa 95,000 m$^2$ of commercial, retail community and childcare NLA, which will house circa 10,000 technology focused staff by 2019 and 2020. Mirvac's redevelopment goes well beyond the development on the three development lots, as it includes the regeneration of the public domain within ATP, the addition of retail to activate the precinct and also the provision of community facilities such as a community centre, a gym and two childcare facilities (90 child each).

*State Environmental Planning Policy (SEPP) Major Development 2005* is the principal environmental planning instrument applying to the ATP. Schedule 3, Part 5 of the Major Development SEPP sets out the zoning, land use and development controls that apply to development on the Site. As the development has a capital investment value of more than $10 million it is identified as State Significant Development under the *State Environmental Planning Policy (State and Regional Development) 2011*, with the Minister for Planning the consent authority for the project.

This report presents the results of a geotechnical desktop study carried out by Douglas Partners Pty Ltd (DP) for proposed mixed use developments on Lots 8, 9 and 12, The Australian Technology Park (ATP), Eveleigh. Mirvac commissioned the work, which was carried out in accordance with Douglas Partners Pty Ltd's (DP) proposal dated 5 November 2015 (Ref: SYD151242.P002.Rev1). The geotechnical report is required to provide information on the subsurface conditions that will allow selection of appropriate shoring systems and designs for the foundations. Potential interaction between the proposed development and the existing rail and transmission cable tunnels was also assessed.

The proposed development will comprise:

- a four storey Community Centre Building on Lot 8 in DP 1136859 – site area circa 1,937m$^2$– no basement levels
- a nine storey Building 1 on Lot 9 in DP 1136859 – site area circa 8,299m$^2$ – no basement levels
- a seven storey Building 2 on Lot 12 in DP 1136859 – site area circa 11,850m$^2$– with two lower ground/basement levels.

The works are proposed to include extensive landscaping and public domain improvements throughout the precinct for the benefit of the local community and extension and augmentation of physical infrastructure/utilities, as required.

The north and western sides of the lower ground levels for Building 2 will back onto a retained cut, requiring bulk excavation depths of up to 5 m below ground level. Potential has been identified for the retention of fill materials beneath Lot 12 (Building 2). This may include over excavation of natural soils, estimated to be up to 10 m below ground level. Further assessment is required, from a contamination perspective, to assess the feasibility/suitability of this option.

The geotechnical investigation comprised a review of available published data, collation and review of previous investigation data and review of survey and "as constructed" data acquired from RailCorp. Details of the findings of the investigation, together with comments on excavation conditions, shoring and foundations are described in this report, which should be read in conjunction with the notes "About this Report" included in Appendix A.

## 2. Site Description

The ATP site is strategically located approximately 5 km south of the Sydney CBD, 8 km north of Sydney Airport and within 200 m of Redfern Railway Station. The site, with an overall area of some 13.2 hectares, is located within the City of Sydney local government area (LGA). The ATP site comprises many Lots, of which Lots 8, 9 and 12 are the subject of this report. At the time of this assessment, all three sites were being used as open space car parking lots. Lot 8 has a street frontage to Central Avenue and Davey Road on the north and west. Lot 8 is bound to the north-east by an approximately seven storey commercial building and on the south-east by the Vice Chancellors Oval, which is an open green space. Lot 9 is on the western side of Davey Road and is bound to the south by Henderson Road and to the north by Central Avenue and the Channel 7 Building. Open space bounds Lot 9 to the west and there is a sports ground between Henderson Road and the actual site of the proposed building. Lot 12 is on the northern side of Central Avenue and east of the Channel 7 Building, on the southern side of Locomotive Street.

The surface elevation of Lot 12 is split between the upper car park at approximately RL 21 m Australian Height Datum (AHD) and the lower car park at RL 16 m to 17 m. Lots 8 and 9 are on a flat to very gently, south-west sloping area with surface elevations between approximately RL 16.5 m along Central Avenue and about RL15.5 in the south-eastern corner. The proposed lower ground level for Lot 12 is between RL 15.45 m and RL16.93.

The Illawarra tunnel corridor runs down the west side of Wyndham Street, crosses Garden Street, runs below the southern portion of the Vice Chancellors Oval and then runs parallel to and on the northern side of Henderson Road. It daylights at Park Street, about 620 m to the west of Lot 9. The south-eastern corner of the proposed Building 1 is about 30 m north from the mapped rail tunnel, while the south-western corner appears to be about 6 m to the north. The top of the rail tunnel is estimated to be at a depth of about 5 m at its nearest approach to the proposed Building 1.

The St Peters to Haymarket Transgrid Cable Tunnel crosses the area in a south-west to north-east direction, beneath the Vice Chancellors Oval and adjacent to, and to the east of Lot 12. “As constructed” drawings indicate that the cable tunnel is at a depth of about 30 m. The Redfern Cable tunnel runs roughly in a north – south direction from Garden Street, east of Lot 12 and about 3 m above the Transgrid tunnel. Main features of the site are shown on Drawing 1 in Appendix B.

## 3. Geology

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is underlain by Quaternary aged alluvial and estuarine sediments which typically comprise silty to peaty quartz sand, silt, and clay with ferruginous and humic cementation in places. The unconsolidated sediments are underlain by the Ashfield Shale of Triassic age comprising shale, laminite and carbonaceous shale and then Hawkesbury Sandstone. Extensive filling is known to occur in the area.

Available field investigation data confirmed the presence of filling in each lot, underlain by sandy and clayey alluvial/estuarine sediments, which in turn were underlain by clayey residual soil. Based on a limited number of boreholes, the top of weathered shale and laminite appears to be present at a depth of about 10 m in Lot 12 and below a depth of about 12 m in Lots 8 and 9.

## 4. Previous Investigations

Fragmental geotechnical and contaminated land data acquired through numerous drilling investigations were collated in a desktop study report for Lots 9 and 12 as part of the Due Diligence process (DP report 84572.00, dated October 2014. Subsurface conditions for Lot 12, using available borehole data, were summarised in a series of cross sections, and accompanied by notes (DP 84955.00 Memo 01, dated July 2015). Comments on contaminated soil management, groundwater and shoring options for Lot 12 were provided in Memo 02 dated July 2015 (Project No 84955.00).

## 5. Summary of Subsurface Conditions

Review of the available data in the previous reports, indicated that the subsurface profile can be summarised as follows:

- The filling is typically 4 m to 6 m thick in the north-western and north-eastern corners of Lot 12 and about 1.5 m to 2.5 m thick in the lower car park. Within Lots 8 and 9 the thickness of filling is typically between 2.5 m and 4 m. The fill appears to comprise a 1 m to 1.5 m thick upper sandy layer (± bricks, ash, slag, gravel, rubble) and a lower clayey fill (± bricks and metal) layer, with this sequence being repeated locally. Most of the filling has been identified as contaminated.
- *In situ* soils mainly comprise clay with a few lenses of sand in Lot 12. Below Lots 8 and 9 the filling is underlain by a 4 m to 6 m thick sand layer, which is underlain by a continuous thin (about 0.5 m thick) peat layer, which is in turn underlain by residual clay developed on the Ashfield Shale rocks.
- The surface of extremely to highly weathered rock was encountered between about RL12 m and RL14 m in Lot 12 with medium strength rock likely to occur below about RL10 m. The top of rock in Lot 9 is indicated at about RL 4 m, about 12 m below ground surface. While there is no site specific data available for Lot 8, the top of rock is estimated to be between RL 2 m and RL 6 m.
- Groundwater in Lot 12 appears to be perched within the fill and sand lenses rather than being continuous groundwater. In Lots 8 and 9 the groundwater was consistently encountered at depths of about 2 m to 3 m, indicating that it affects both the granular fill and the underlying *in situ* sand. The groundwater is expected to be contaminated and the levels are likely to change rapidly following rain events.

Estimated top of rock contours are shown on Drawing 84572/1 in Appendix B.

# 6. Comments

## 6.1 Proposed Development

The supplied information indicates that the development will comprise construction of low and medium rise mixed use commercial buildings with two levels of lower ground floor car park on Lot 12. Depending on the findings of further contaminated soil investigations, the excavation within the upper car park on Lot 12 may extend below ground level of the existing lower car park.

## 6.2 Excavation Conditions, Batter Slopes and Shoring

### 6.2.1 Excavation Conditions

Most of the excavation is envisaged to occur in the northern and western part of Lot 12. Excavation to depths of about 4 m to 6 m is anticipated to be in filling, *in situ* sand and stiff to very stiff clay, which should be readily achieved using conventional earthmoving equipment such as tracked hydraulic excavators. Should the excavation encounter extremely to highly weathered shale/laminite of extremely low to low strength, it can be excavated using the same equipment. However, concrete structures and construction rubble was reported to occur within the filling and local over-excavation or the use of heavy rock hammers may be required to break up the concrete and/or rubble.

It would be prudent to monitor and limit vibration where rock hammering is required near adjacent structures (closer than 10 m). Vibration trials are suggested during initial rock hammering to verify vibration levels. Based on the results of the vibration trials, performance criteria should be established with references to the NSW Department of Environment and Conservation (DECC) Assessing Vibration: a technical guideline (2006) level of adjacent structures is suggested for human comfort considerations.

Note, any saturated sand will be difficult to stockpile and may require low permeability bunding. Dewatering, using sumps and pumps or a vacuum spear point methodology is suggested prior to excavation to improve handling characteristics and reduce the risk of contamination by leachate from stockpiles. The likely methodology will be dependent upon the preference of the yet to be appointed construction contractor and actual groundwater conditions encountered during excavation. All spoil resultant from construction/remedial activities will need to be dealt with in accordance with procedures presented in the remedial action plan prepared for the site.

It should be noted that any off-site disposal of spoil, including the use of the Vice Chancellors Oval, will generally require assessment for use or classification in accordance with the current Waste Classification Guidelines (NSW EPA 2014).

### 6.2.2 Batter Slopes and Shoring

The excavation along the western, northern and eastern walls of Lot 12 to a depth of about 4 m to 6 m will be in fill and the lower portion in very stiff to hard residual clays, shaley clays and sandy clays with lenses of sand. Support of these excavations could be provided by installation of contiguous piles, sheet piles or soldier piles with shotcrete infill panels and soil tie back.

The most cost effective of the above three ground support systems is likely to be soldier piles with shotcrete panels and soil tie back, provided ground conditions are suitable for their application. Although sheet piles could be considered, as the fill is underlain by clayey soils of sufficient thickness to achieve adequate embedment, the presence of concrete and brick rubble and other hard obstructions within the fill indicate that penetration of the sheet-piles may be difficult through the fill layers. Contiguous piles, while effective retaining systems, are typically more costly than the soldier piles with shotcrete and their use would only be required if the soldier pile system would not be suitable for the prevailing ground conditions, such as saturated, potentially running sands and collapsible fill.

#### Soldier Piles with Shotcrete Panels and Soil Tie Back

The excavation support considered for Lot 12 comprises soldier piles installed at 2.0 - 3.0 m centres with shotcrete panels between the piles to retain the soil. The soldier piles are likely to be constructed using the CFA drilling method with 600 mm diameter piles embedded into the very stiff or hard residual clay. Two or three rows of soil tie back anchors will be required with one row near the crest of the excavation and one row near the centre with total working capacities of 260 kN/m – 400 kN/m, depending on the depth of the excavation.

The preliminary design of retaining walls and shoring with up to two rows of anchors can be based on a triangular earth pressure distribution using the earth pressure coefficients provided in Table 1. Active earth pressure coefficient ($K_a$) values may be used where some wall movement is acceptable.

At rest earth pressure coefficient ($K_o$) values should be used where wall movement needs to be limited.

Where more than two rows of anchors are required the walls will essentially be braced and can be designed for a rectangular earth pressure distribution of 4H kPa where some movement is acceptable or 6 - 8H kPa where movement is to be limited, H is the height in metres of the retained material.

**Table 1: Preliminary Design Parameters for Shoring and Retaining Systems**

| Material Type | Unit Weight ($kN/m^3$) | Earth Pressure Coefficient | |
|---|---|---|---|
| | | Active ($K_a$) | At Rest ($K_o$) |
| Filling | 20 | 0.3 (0.4) | (0.6) |
| Very stiff to hard clay | 20 | 0.25 (0.3) | (0.5) |
| Sand | 20 | 0.3 (0.3) | (0.5) |
| Extremely low and very low strength laminite/shale/sandstone | 22 | 0.25 (0.30) | (0.40) |

Note: () Permanent earth pressure coefficients shown in brackets.

Where soldiers are socketed, a factor of safety must be applied to the ultimate passive earth pressure coefficient values provided. The top 0.5 m of the foundation embedment should be ignored in calculations to account for tolerance and any shallow detailed excavation.

All surcharge loads should be allowed for in the retaining wall design including building footings, services, etc. Retaining walls should 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 the slope stabilisation structures. The base of the strip drains should extend out from the retaining wall to allow any seepage to flow into a perimeter toe drain.

The soldier pile system is anticipated to be suitable for most of the proposed excavations. However, sections with *in situ* sand, where groundwater inflow is anticipated, is likely to require contiguous pile support to reduce the risk of loss of support and collapse of the wall before the soil tie backs and the shotcrete can be installed. Sand lenses underlying the fill were previously identified in the north-western corner of Lot 12 (Table 2 and Drawing 84955.00/4 in Appendix B).

The two sand lenses identified in the western and north western wall sections are likely to be connected within the proposed upper carpark excavation. The bottom of clayey fill between the two sand lenses is at around RL 16.0 m and the presence of metal and brick fragments were reported from the boreholes. The area between the two sand lenses may also require installation of contiguous piles, if the proportion of metal in the fill is such that large volumes of fill are poorly compacted and therefore susceptible to collapse. An additional borehole or carefully logged bored pile in this area during construction could provide additional information that could be used to select the appropriate ground support system.

**Table 2: Estimated Distribution of Sand Lenses and Extent of Requirement for Contiguous Piles for Building 2**

| Section along perimeter wall | Approximate chainage (m) | Length of section (m) | Estimated bottom of sand lens (m, AHD) |
|---|---|---|---|
| Western wall | -50 to -25 | 25 | 18 |
| Northwest corner | -5 to 15 | 20 | 15 |
| North wall centre | 65 to 85 | 20 | 17 |

Passive resistance for soldier piles retaining the soil and extremely weathered rock may be based on a preliminary ultimate passive restraint shown in Table 3 below. A factor of safety must be applied to these ultimate values to limit the amount of wall movement that is required to mobilise the passive resistance. Alternatively the soldier piles can be restrained by tie back anchors at the toe.

For preliminary design of anchors, the ultimate bond stresses shown in Table 4 should be adopted. The parameters given in Table 4 assume that the drill holes are clean and adequately flushed. The anchors should be bonded behind a line drawn up at 45 degrees from the base of the shoring. Testing should be carried out to confirm the anchor capacities. It should be noted that permission will be required from adjacent property owners prior to installing bolts/anchors below their land.

**Table 3: Design Parameters for Shoring Piles**

| Material Description | Estimated Ultimate Passive Pressure (kPa) |
|---|---|
| Very low strength laminite/shale | 400 |
| Low to medium strength laminite/shale/sandstone | 2,000 |
| Medium strength sandstone | 6,000 |

**Table 4: Bond Stresses for Anchor Design**

| Material Description | Ultimate Bond Stress (kPa) |
|---|---|
| Clay | 50 |
| Sand | 80 |
| Very low strength rock | 100 |
| Low strength rock | 300 |
| Medium strength rock | 1,000 |

It is anticipated that the buildings will prop/support the shoring walls over the long term and therefore ground anchors are expected to be temporary only. The use of permanent anchors, if required, would require careful attention to corrosion protection and further geotechnical advice should be sought.

The excavation adjacent to the walls supported by the soldier piles will need to progress in short sections, together with the installation of the soil tie backs and shotcrete panels. Once the area is supported, the next section can be excavated.

Temporary batters in Lot 12 in soil and rock, away from adjacent structures, can be cut at temporary batter slopes of 1.5:1 (H:V).

## 6.3 Foundations

It is expected that all structures will be supported by piled footings bearing on rock. Typical maximum allowable pressures for the various anticipated foundation materials, based on the foundation classification methods of Pells et al. (1998), are shown in Table 5.

Foundations proportioned on the basis of the allowable parameters would be expected to experience total settlements of less than 1% of the minimum footing width (or pile diameter) under the applied working load, with differential settlement between adjacent columns expected to be less than half this value.

For detailed design of piles, a limit state analysis should be undertaken as a more economical design is often achieved. All piling excavation should be inspected by a geotechnical engineer to confirm that foundation conditions are in accordance with the design assumptions.

**Table 5: Design Parameters for Pad/Strip Footings**

| Rock type | Typical Allowable End Bearing Pressure (kPa) | Typical Allowable Shaft Adhesion (kPa) |
|---|---|---|
| Extremely low to very low strength shale/laminite | 700 | 70 |
| Medium strength shale/laminite/Sandstone | 3,500 | 350 |

Bored, fully cased or CFA piles should be considered for the foundations, as groundwater is likely to be present from a depth of 1.5 m to 3 m in Lots 8 and 9, together with saturated, collapsible sand. Concrete and rubble obstructions have been reported to be present on Lot 12, together with collapsible filling and sand and perched groundwater.

## 6.4 Excavations Adjacent to Existing Buildings

Foundation conditions of the buildings adjacent to the excavation will need to be confirmed prior to the excavation. The level of basement excavation and type of retaining structures of the Channel 7 Building, adjacent to Lot 12 on the western side needs to be confirmed to assess both potential requirements for underpinning, and potential implications on the design and installation of excavation support. Supplied drawings indicate that the basement/lower ground level of the Channel 7 building is at the approximate Central Avenue street level, same as proposed for Building 2 and no interference is expected. However, excavations below approximate RL15.5 m AHD, which may be required to allow placement and burial of contaminated soil could affect the existing retaining structures and foundations along the northern portion of the boundary between Channel 7 and Building 2. Shoring, supporting the excavations in this area needs to be designed with considerations for the existing structures. For detailed design of the excavation support, foundation details of the Channel 7 Building will be required.

## 6.5 Design for Earthquake Loading

In accordance with AS1170-2007 "Structural Design Actions, Part 4: Earthquake Actions in Australia" a hazard factor (Z) of 0.08 and a site subsoil Class $C_e$ is considered to be appropriate for the site, provided all structural support is founded on piles bearing on rock.

## 6.6 Groundwater

The available groundwater data indicates that the bulk excavation on the upper levels of Lot 12 will intercept discontinuous groundwater bearing layers. While initial inflow rates may be substantial, they are anticipated to be of short duration, except following rain events.

During construction, it is anticipated that groundwater inflow should be readily controlled by perimeter drains connected to a "sump-and-pump" dewatering system. For estimating purposes a relatively small and isolated aquifer was considered with an assumed head of 3 m and with hydraulic gradient of 0.03. For every 25 m wide section of the excavation with a maximum depth of 8.5 m (estimated 3.5 m below groundwater level) the total estimated flows are about 2.25 x $10^{-5}$ $m^3$/sec or about 2 $m^3$/day. At this rate, during an assumed 6 month excavation period, a total inflow of 350 $m^3$ could be expected. For groundwater extraction greater than 3 ML/year, a dewatering license will be required from the relevant government authority (NSW Department of Primary industries – Water, previously NSW Office of Water)

The lower ground floors, adjacent to Locomotive Street are expected to be designed as a drained basement. A drained basement will require permanent drainage behind the retaining structures as well as subfloor drainage below the basement floor slab connected to a sump, which regularly pumps out the water or to an outlet, if the Central Avenue levels allow for gravity feed. The disposal requirements of water collected on-site will be dependent on the chemical composition of the water. Normally, water is disposed of to a stormwater system subject to approval from the relevant government authority. However, a drained basement will act as a low point to which groundwater will flow. Therefore, if present, contamination within the surrounding groundwater system could flow into the basement and adversely affect the quality of the water collected on site. The groundwater is anticipated to flow roughly from north to south.

## 6.7 Rail Tunnel

Based on the supplied concept stage drawings for the proposed building and on the rail tunnel requirements, the following potential geotechnical constraints have been identified:

- Based on the estimated depth of the tunnel adjacent to Building 1, the 45 degree line drawn from the base of the existing tunnel is estimated to intercept the ground surface at the southern boundary of the proposed building. Consequently, all footings for the building will need to be founded below the current surface level. Since the building is proposed to be founded on rock, which is estimated to be below RL 6 m AHD, at an approximate depth of 10 m, the rail requirements do not pose a geotechnical constraint for the foundations of the proposed building. The proposed foundation method of bored piles socketed into rock or CFA piles are viable foundation methods;
- During the construction phase for Building 1 an approximately 5 m wide exclusion zone should be established parallel to the boundary with the rail easement above the tunnel. No large earthmoving machinery or crane should be allowed in this exclusion zone and weight restriction should be in place for material storage;
- A dilapidation survey will need to be completed for the section of the tunnel parallel to Building 1, i.e. from the corner of Davey and Henderson Roads to the south-western corner of Lot 9; and
- No bulk excavation is planned for Building 1, therefore in DP's opinion there should be no need to carry out a 3D finite element analysis.

### 6.8 Cable Tunnel

The St Peters to Haymarket Transgrid Cable Tunnel crosses the area in a south-west to north-east direction, beneath the Vice Chancellors Oval and adjacent to, and to the east of Lot 12. The available "as constructed" drawings indicate that the cable tunnel is at a depth of about 30 m. The existing Redfern Cable tunnel runs roughly in a north – south direction, east of Lot 12 and about 3 m above the Transgrid tunnel. The proposed development on Lot 12 or the stockpiling on the oval is not expected to affect the two existing cable tunnels. However, the owners of the tunnels should be notified regarding the proposed activities.

## 7. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for the proposed mixed use development at Lots 8, 9 and 12, Australian Technology Park, Eveleigh in accordance with DP's proposal dated 5 November 2015 (Ref: SYD151242.P.002-Rev1). This report is provided for the exclusive use of Mirvac Projects for this project only and for the purposes 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. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. 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 on the site only at the specific 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 human 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 affected by undetected variations in ground conditions across the site between and beyond the testing 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 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 stated 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 instructions for construction.

The contents of this report do not constitute formal design components such as are required, by Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction of all works (not just geotechnical components) and the controls required to mitigate risk. This report does however, identify hazards associated with the geotechnical aspects of development and presents the results of risk assessment associated with the management of these hazards.

## 8. References

Pells PGN, Mostyn G and Walker BF, 1998, Foundation on sandstone and shale in the Sydney region, Australian Geomechanics, December 1998, p 17-29.

**Douglas Partners Pty Ltd**

# Appendix A

About this Report

# 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.

## Copyright

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

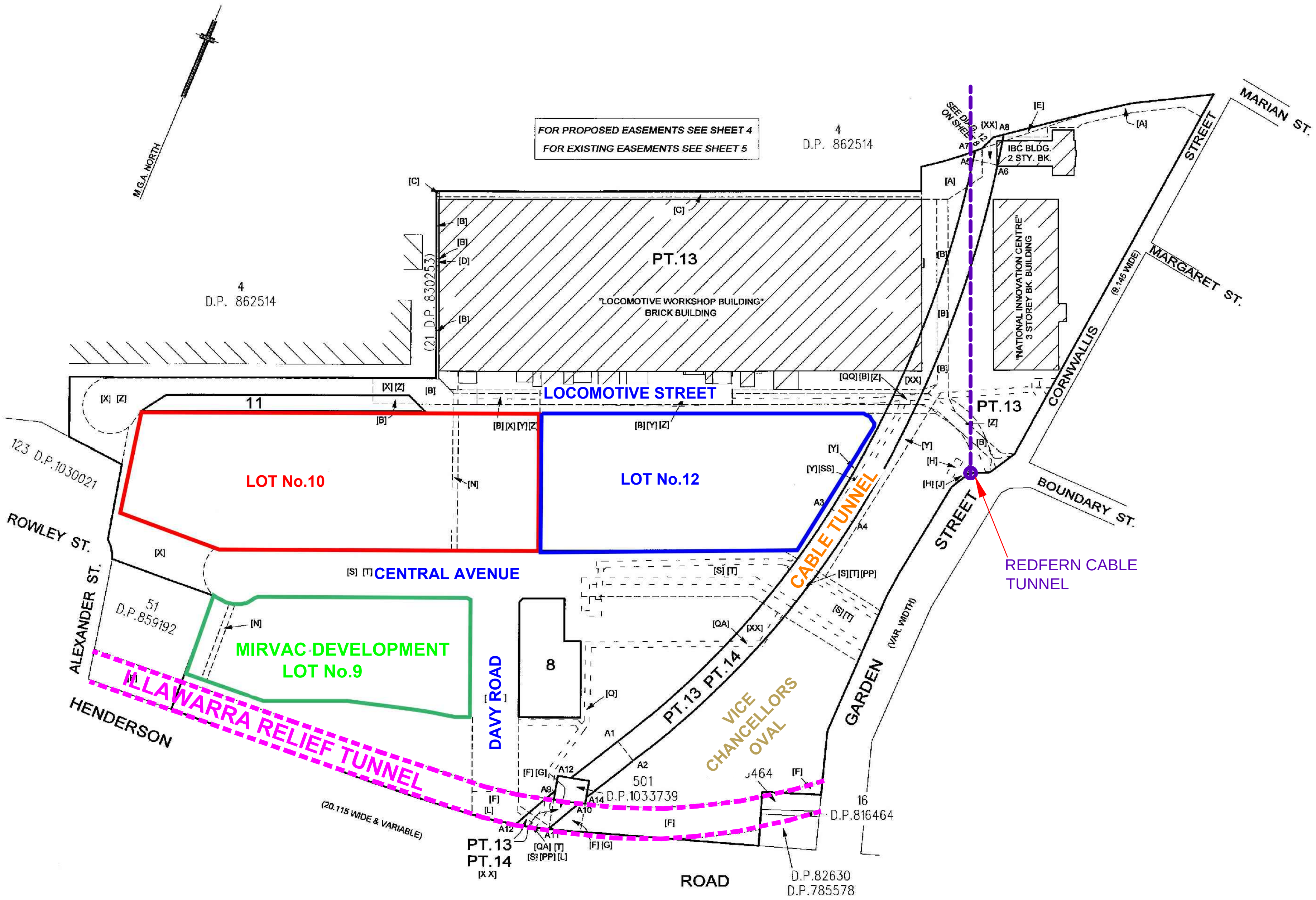
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate 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.

# Appendix B

Site Layout
Estimated Top of Rock Contour Map
Cross Section H-H’



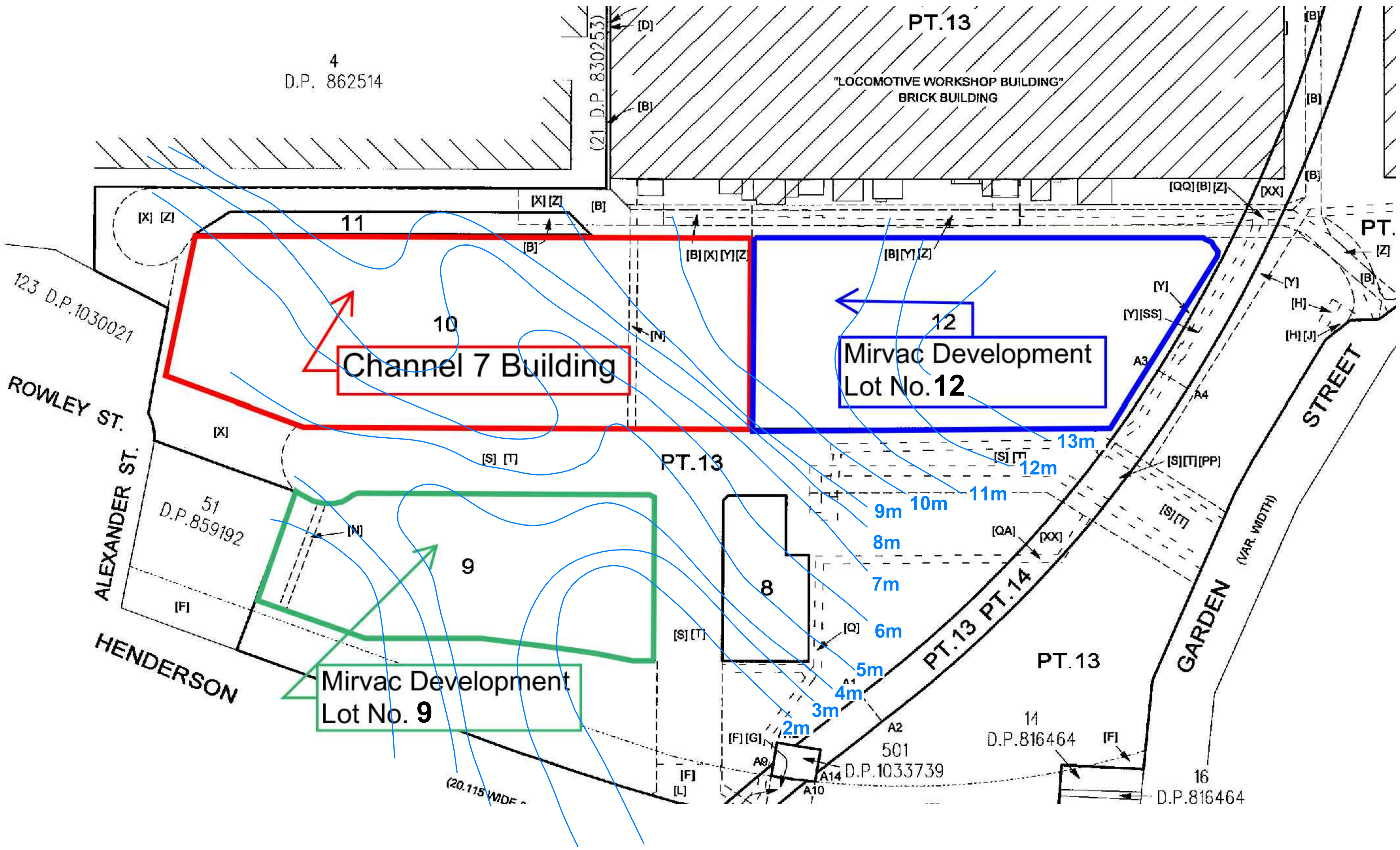


Locality Plan

NOTE: 1. Base drawing from Peter Raymond Salmon Surveyors (Ref 73068, dated 7.1.2009)

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

| | | | |
|---|---|---|---|
| CLIENT: Mirvac Construction | | TITLE: **Site Layout** | PROJECT No: 84955.01 |
| OFFICE: Sydney | DRAWN BY: PSCH | **Geotechical Suppport** | DRAWING No: 1 |
| SCALE: N.T.S. | DATE: 17.11.2015 | **ATP, EVELEIGH** | REVISION: 0 |




NOTE:
1. Base drawing from Peter Raymond Salmon (Dwg DP1136859, dated 7.1.2009)
2. Contours are an interpretation of levels between test locations and actual levels may vary on site.

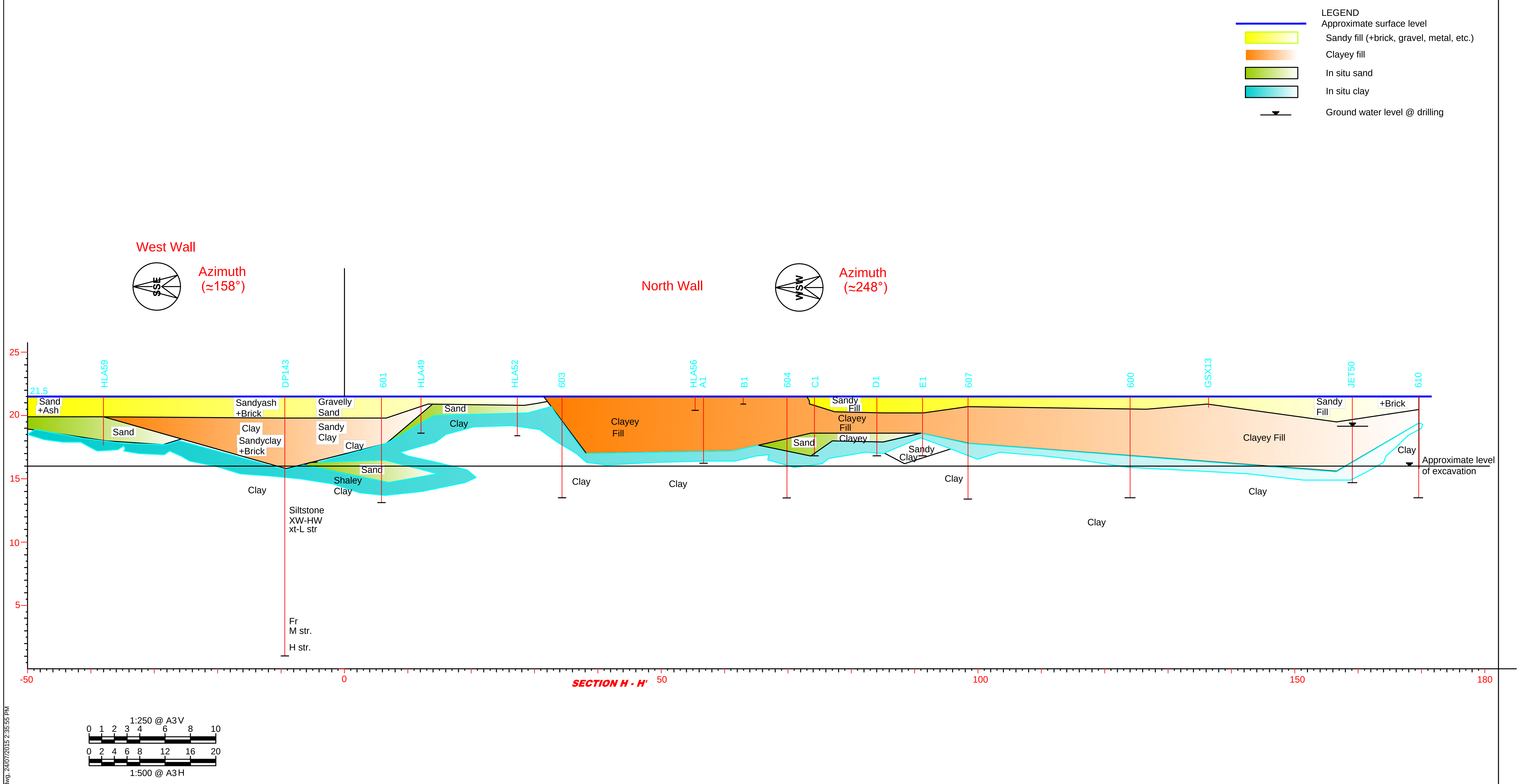
0 10 20 30 40 50 75 100m

1:1000 @ A3

**LEGEND**

RL (AHD) Top of rock contour

| | | | |
|---|---|---|---|
| **Douglas Partners** Geotechnics / Environment / Groundwater | CLIENT: Mirvac | TITLE: **Contour Plan of Top of Rock** | PROJECT No: 84572 |
| | OFFICE: Sydney — DRAWN BY: PSCH | **Australian Technology Park** | DRAWING No: 1 |
| | SCALE: 1:1000@ A3 approx. — DATE: 8.1.2015 | **EVELEIGH** | REVISION: 1 |




P:\84955.00 - EVELEIGH - Subsurface cross sections\V.0 Drawings\V.2 Out\84955-4.dwg, 24/07/2015 2:35:55 PM

| | | | |
|---|---|---|---|
| **Douglas Partners** Geotechnics / Environment / Groundwater | CLIENT: Mirvac Constructions Pty Ltd | TITLE: **Cross Sections H-H'** | PROJECT No: 84955.00 |
| | OFFICE: Sydney — DRAWN BY: Vojta | **Subsurface Cross Section** | DRAWING No: 4 |
| | SCALE: 1:250 @ A3 — DATE: 15.7.2015 | **13 Garden Street, ATP, EVELEIGH** | REVISION: 0 |