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Moorebank Precinct East - Stage 2 Proposal

Geotechnical Interpretive Report

SIMTA

SYDNEY INTERMODAL TERMINAL ALLIANCE

Part 4, Division 4.1, State Significant Development

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REPORT

11 November 2016

MPE STAGE 2 - MOOREBANK PRECINCT, MOOREBANK

Geotechnical Interpretive Report

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Executive Summary

This report provides geotechnical advice for the Moorebank Precinct East (MPE) Stage 2 Proposal (the Proposal), which is the subject of an Environmental Impact Statement (EIS) for a State Significant Development (SSD) application (SSD16-7628). The results of geotechnical investigation, completed by Golder in 2014, 2015 and 2016 have been used to develop a geotechnical model for the site. Analysis of the geotechnical model has been used to provide recommendations for the redevelopment of the site and forms the basis of this Geotechnical Interpretive Report. The MPE Stage 2 development area is a rectangular shape approximately 1,200 m long (north to south) and approximately 600 m wide at its widest point (east to west) and is situated east of Moorebank Avenue approximately 1km south of the junction of Moorebank Avenue and the M5 Motorway.

The geology of the site generally comprises a thin layer of fill material at the existing ground surface, generally 0.5 m thick (but more than 1m locally) over alluvium comprising stiff to very stiff clays or dense to very dense sands. Depth to rock varied significantly being most shallow within the elevated central eastern area (e.g. rock encountered within 2m of ground surface) compared with the western extent of the site (e.g. rock encountered at depths of more than 20m). Bedrock is typically shale underlain by sandstone, although the shale was absent from the profile in some locations. Depth to groundwater was also observed to vary but was typically in the range of 4m to 7m below ground surface (although groundwater was encountered within 1.5m towards the south eastern corner of the site in the vicinity of Anzac Creek).

Most aspects of the development will involve relatively routine geotechnical design and construction procedures. Given that the earthworks dominantly comprise filling with relatively shallow cut proposed (e.g. less than 2m in the basement parking area proposed for the Freight Village), the variability of depths to rock and groundwater will likely have the greatest impact on foundation system selection (e.g. shallow or deep foundations) and installation requirements (e.g. potential need for casing of piled foundations). Where space permits, the sides of excavations can be battered and recommended batter slopes are provided in the report. If space is limited, excavations may need to be laterally supported and recommendations are provided for the design of retaining systems. We would expect that conventional earthworks equipment could be used for the majority of excavation, with a provision for breaking-out occasional ironstone layers and shallow rock where encountered (e.g. within the elevated central eastern portion of the site) with rock breakers and/or large dozers equipped with ripping hooks.

Appropriate subgrade preparation below areas of filling will require consideration of the performance requirements of the specific overlying design elements and the nature of the new fill material to be placed and should be addressed in a site specific Earthworks Specification. Some over-excavation and replacement with new fill may be required. Dependent on the particular performance criteria adopted and new fill characteristics, depths of over-excavation and replacement would likely typically be limited to a depth of 1m below stripped surface level (i.e. after removal of existing surface features and topsoil). Re-use of existing site won material is possible, subject to the particular performance requirements to be met. However, based on the current earthworks design, volumes of site won material are likely to be relatively small. Furthermore, cohesive (clayey) material encountered during investigation, returned high plasticity and low California Bearing Ratio (CBR) test results (e.g. minimum CBR of 0.5%) with high swells (e.g. up to 5.5%) constraining its re-use potential to general filling outside of the zones of influence of pavement and movement sensitive structures. Granular (sandy) strata was also encountered within both existing fill and the tertiary alluvium. However, the vertical and lateral distribution of cohesive and granular material is likely to be variable. Thus selective winning of more granular material (with lower plasticity, higher CBR test values and corresponding better re-use potential) during earthworks may not be feasible.

We expect that conventional shallow level foundations would be suitable to support lightly loaded structures on the site, such as single-storey buildings and possibly some warehouse columns. Deep foundation options will likely be required for heavily loaded structures or structures sensitive to differential settlement. Settlements under warehouse floor slab loads, for the current cut/fill strategy (Arcadis, 2016a), are about 20 mm to 40 mm, which is within the typical tolerance limits for industrial structures.

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1.0 INTRODUCTION

Concept Plan Approval (MP 10_0193) for an intermodal terminal (IMT) facility at Moorebank, NSW (the Moorebank Precinct East Project (MPE Project) (formerly the SIMTA Project)) was received on 29 September 2014 from the NSW Department of Planning and Environment (DP&E). The Concept Plan for the MPE Project involves the development of an IMT, including a rail link to the Southern Sydney Freight Line (SSFL) within the Rail Corridor, warehouse and distribution facilities with ancillary offices, a freight village (ancillary site and operational services), stormwater, landscaping, servicing, associated works on the eastern side of Moorebank Avenue, Moorebank, and construction or operation of any part of the project, which is subject to separate approval(s) under the Environmental Planning and Assessment Act 1979 (EP&A Act).

The Environmental Impact Statement (EIS) is seeking approval, under Part 4, Division 4.1 of the EP&A Act, for the construction and operation of Stage 2 of the MPE Project (herein referred to as the Proposal) under the Concept Plan Approval for the MPE Project, being the construction and operation of warehouse and distribution facilities.

The EIS has been prepared to address:

- The Secretary's Environmental Assessment Requirements (SEARs) (SSD 16-7628) for the Proposal, issued by NSW DP&E on 27 May 2016 (Appendix A).
- The relevant requirements of the Concept Plan Approval MP 10_0913 dated 29 September 2014 (as modified) (Appendix A).
- The relevant requirements of the approval under the Environment Protection and Biodiversity Conservation Act 1999 (EPBC Act) (No. 2011/6229, granted in March 2014 by the Commonwealth Department of the Environment (DoE)) (as relevant) (Appendix A).

The EIS also gives consideration to the MPE Stage 1 Project (SSD 14-6766) including the mitigation measures and conditions of consent as relevant to this Proposal.

The EIS has been prepared to provide a complete assessment of the potential environmental impacts associated with the construction and operation of the Proposal. The EIS proposes measures to mitigate these issues and reduce any unreasonable impacts on the environment and surrounding community.

1.1 Overview of the Proposal

The Proposal involves the construction and operation of Stage 2 of the MPE Project, comprising warehousing and distribution facilities on the MPE site and upgrades to approximately 1.4 kilometres of Moorebank Avenue between the northern MPE site boundary and 120 metres south of the southern MPE site boundary.

Key components of the Proposal include:

- Warehousing comprising approximately 300,000m^2 GFA, additional ancillary offices and the ancillary freight village
- Establishment of an internal road network, and connection of the Proposal to the surrounding public road network
- Ancillary supporting infrastructure within the Proposal site, including:
  - Stormwater, drainage and flooding infrastructure
  - Utilities relocation and installation
  - Vegetation clearing, remediation, earthworks, signage and landscaping
- Subdivision of the MPE Stage 2 site

- The Moorebank Avenue upgrade would be comprised of the following key components:
  - Modifications to the existing lane configuration, including some widening
  - Earthworks, including construction of embankments and tie-ins to existing Moorebank Avenue road level at the Proposal's southern and northern extents
  - Raking of the existing pavement and installation of new road pavement
  - Establishment of temporary drainage infrastructure, including temporary basins and / or swales
  - Raising the vertical alignment by about two metres from the existing levels, including kerbs, gutters and a sealed shoulder
  - Signalling and intersection works
- Upgrading existing intersections along Moorebank Avenue, including:
  - Moorebank Avenue / MPE Stage 2 access
  - Moorebank Avenue / MPE Stage 1 northern access
  - Moorebank Avenue / MPE Stage 2 central access
  - MPW Northern Access / MPE Stage 2 southern emergency access

The Proposal would interact with the MPE Stage 1 Project (SSD_6766) via the transfer of containers between the MPE Stage 1 IMT and the Proposal's warehousing and distribution facilities. This transfer of freight would be via a fleet of heavy vehicles capable of being loaded with containers and owned by SIMTA. The fleet of vehicles would be stored and used on the MPE Stage 2 site, but registered and suitable for on-road use. The Proposal is expected to operate 24 hours a day, seven days per week.

An overview of the Proposal is shown in Figure 1: Overview of MPE Site. To facilitate operation of the Proposal, the following construction activities would be carried out across and surrounding the Proposal site (area on which the Proposal is to be developed):

- Vegetation clearance
- Remediation works
- Demolition of existing buildings and infrastructure on the Proposal site
- Earthworks and levelling of the Proposal site, including within the terminal hardstand
- Drainage and utilities installation
- Establishment of hardstand across the Proposal site, including the terminal hardstand
- Construction of a temporary diversion road to allow for traffic management along the Moorebank Avenue site during construction (including temporary signalised intersections adjacent to the existing intersections) (the Moorebank Avenue Diversion Road)
- Construction of warehouses and distribution facilities, ancillary offices and the ancillary freight village
- Construction works associated with signage, landscaping, stormwater and drainage works.

The Proposal would operate 24 hours a day, 7 days a week.

More information relating to the construction and operation of the Proposal is provided in Figure 1: Overview of MPE Site and Chapter 4 of the MPE Stage 2 EIS.

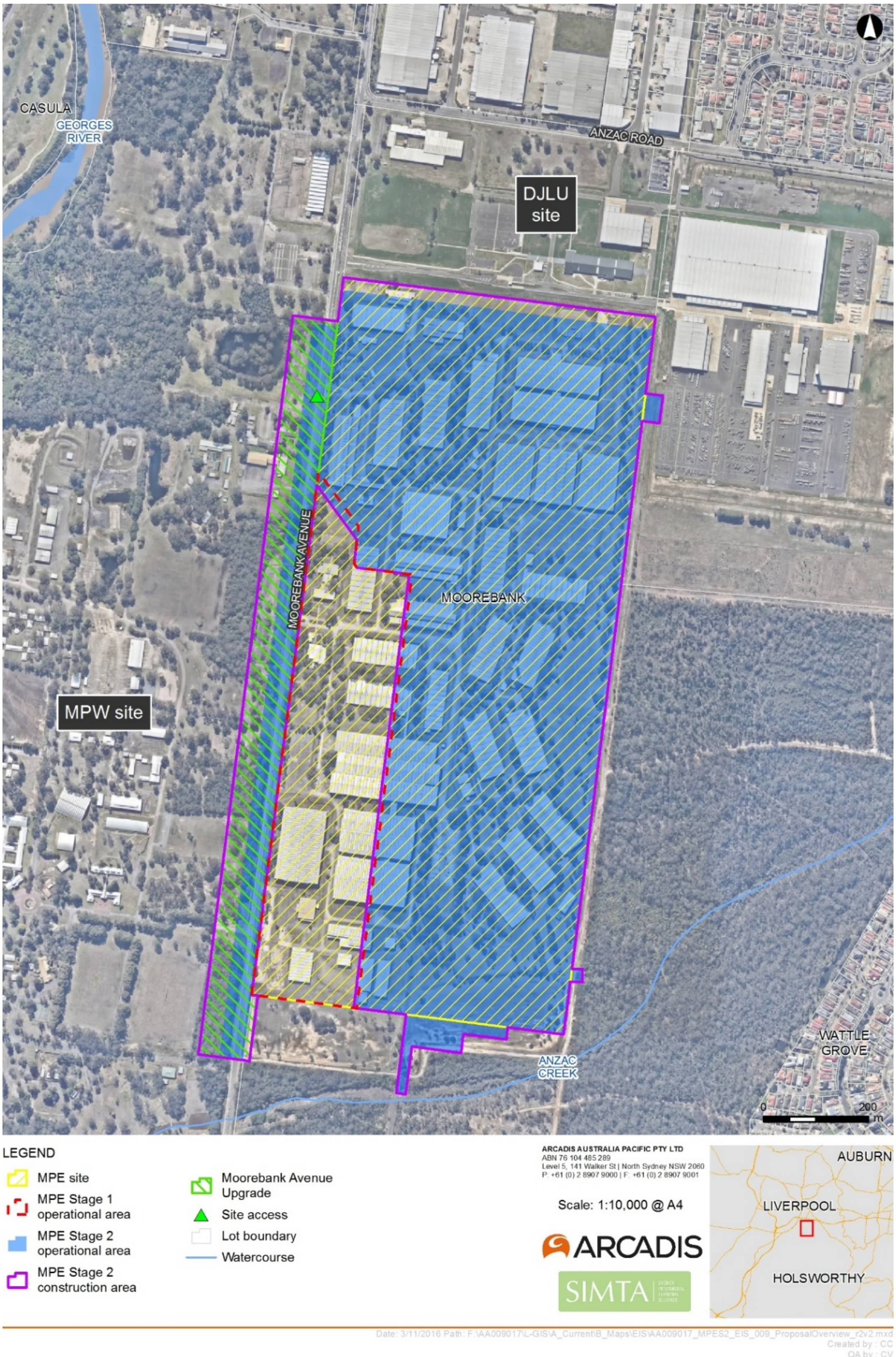


Figure 1: Overview of MPE Site

Table 1 provides a summary of the key terms relevant to the Proposal, which are included throughout this report.

Table 1: Summary of key terms used throughout this document

General terms	
The Moorebank Precinct	Refers to the whole Moorebank intermodal precinct, i.e. the MPE site and the MPW site.
Moorebank Precinct West (MPW) Project (formerly the MIC Project)	The MPW Intermodal Terminal Facility as approved under the MPW Concept Plan Approval (SSD_5066) and the MPW EPBC Approval (No. 2011/6086).
Moorebank Precinct West (MPW) site (formerly the MIC site)	The site which is the subject of the MPW Concept Plan Approval, MPW EPBC Approval and MPW Planning Proposal. The MPW site does not include the rail link as referenced in the MPW Concept Plan Approval or MPE Concept Plan Approval.
Moorebank Precinct East (MPE) Concept Plan Approval (formerly the SIMTA Concept Plan Approval)	MPE Concept Plan Approval (SSD_0193) granted by the NSW Department of Planning and Environment on 29 September 2014 for the development of former defence land at Moorebank to be developed in three stages; a rail link connecting the site to the Southern Sydney Freight Line, an intermodal terminal, warehousing and distribution facilities and a freight village.
Moorebank Precinct East (MPE) Project (formerly the SIMTA Project)	The MPE Intermodal Terminal Facility, including a rail link and warehouse and distribution facilities at Moorebank (eastern side of Moorebank Avenue) as approved by the Concept Plan Approval (MP 10_0913) and the MPE Stage 1 Approval (14_6766).
Moorebank Precinct East (MPE) Site (formerly the SIMTA Site)	Including the former DSNDC site and the land owned by SIMTA which is subject to the Concept Plan Approval. The MPE site does not include the rail corridor, which relates to the land on which the rail link is to be constructed.
Statement of Commitments (SoC)	Recommendations provided in the specialist consultant reports prepared as part of the MPE Concept Plan application to mitigate environmental impacts, monitor environmental performance and/or achieve a positive environmentally sustainable outcome in respect of the MPE Project. The Statement of Commitments have been proposed by SIMTA as the Proponent of the MPE Concept Plan Approval.
MPE Stage 1 Project-specific terms	
Rail Corridor	Area defined as the 'Rail Corridor' within the MPE Concept Plan Approval.
Rail Link	The rail link from the South Sydney Freight Line to the MPE IMEX Terminal, including the area on either side to be impacted by the construction works included in MPE Stage 1.
MPE Stage 1	Stage 1 (14-6766) of the MPE Concept Plan Approval for the development of the MPE Intermodal Terminal Facility, including the rail link at Moorebank. This reference also includes associated conditions of approval and environmental management measures which form part of the documentation for the approval.
MPE Stage 1 site	Includes the MPE Stage 1 site and the Rail Corridor, i.e. the area for which approval (construction and operation) was sought within the MPE Stage 1 Proposal EIS.

MPE Stage 2 specific terms	
MPE Stage 2 Proposal/ the Proposal	The subject of this EIS; being Stage 2 of the MPE Concept Plan Approval including the construction and operation of 300,000m^2 of warehousing and distribution facilities on the MPE site and the Moorebank Avenue upgrade within the Moorebank Precinct.
MPE Stage 2 site	The area within the MPE site which would be disturbed by the MPE Stage 2 Proposal (including the operational area and construction area). The MPE Stage 2 site includes the former DSNDC site and the land owned by SIMTA which is subject to the MPE Concept Plan Approval. The MPE site does not include the rail corridor, which relates to the land on which the rail link is to be constructed.
The Moorebank Avenue site	The extent of construction works to facilitate the construction of the Moorebank Avenue upgrade.
The Moorebank Avenue upgrade	Raising of the vertical alignment of Moorebank Avenue for 1.5 kilometres of its length by about two metres, from the northern boundary of the MPE site to approximately 120 metres south of the MPE site. The Moorebank Avenue upgrade also includes upgrades to intersections, ancillary works and the construction of an on-site detention basin to the west of Moorebank Avenue within the MPW site.
Construction area	Extent of construction works, namely areas to be disturbed during the construction of the MPE Stage 2 Proposal (the Proposal).
Operational area	Extent of operational activities for the operation of the MPE Stage 2 Proposal (the Proposal).

1.2 Purpose of this report

1.2.1 This Report

This report presents our interpretation of information obtained during geotechnical investigations carried out by Golder Associates Pty Ltd (Golder) for the Stage 2 State Significant Development (SSD) application of the proposed Moorebank Precinct East (MPE) transport terminal development.

The factual data on which this report is based is presented in the SIMTA Intermodal Terminal Stage 2 (DNSDC) Geotechnical Data Report (Golder, 2016a). Factual data relating to a 2014/2015 investigation campaign is presented in the SIMTA Intermodal Terminal, Geotechnical Data Report (Golder 2015a) and has been referenced where considered pertinent to this report.

The entire construction area for the Moorebank Precinct East (MPE) Stage 1 and Stage 2 development is approximately 50 ha. The site was previously occupied by the Defence National Storage and Distribution Centre (DNSDC).

The MPE Stage 2 area investigated abuts the Stage 1 area, adjoining its northern and eastern boundaries, forming a battle-axe approximately 1,200 m long (north to south) and approximately 600 m wide at its widest point (east to west).

It is understood from Tactical that the design surface levels across the site will be in the order of RL 17.5 to 19.0 m. The design surface levels will require cut and fill operations, with fill materials potentially imported or obtained from excavation such as that required within the elevated area on the eastern portion of the site, towards an area known as “Moorebank Hill”. It is anticipated that cut and fill operations of the scale of 2m high or deep may be required.

The site, including investigation undertaken by Golder Associates to date is shown on the attached Figure A001 of Appendix A.

This report supports the Environmental Impact Statement (EIS) for the Proposal (refer to above Section 1.1 for an overview of the Proposal) and has been prepared as part of a State Significant Development (SSD) Application for which approval is sought under Part 4, Division 4.1 of the EP&A Act.

This report has been prepared to address:

- The Secretary's Environmental Assessment Requirements (SEARs) (SSD 16-7628) for the Proposal, issued by NSW DP&E on 27 May 2016.
- The relevant requirements of Concept Plan Approval MP 10_0913 dated 29 September 2014 (as modified).
- The relevant requirements of the approval under the *Environment Protection and Biodiversity Conservation Act 1999* (EPBC Act) (No. 2011/6229, granted in March 2014 by the Commonwealth Department of the Environment (DoE)) (as relevant).

The SEARs and the Concept Plan Conditions of Approval and Statement of Commitments relevant to this study, and the section of this report where they have been addressed are provided in Table 2 and Table 3 respectively.

Table 2: Secretary's Environmental Assessment Requirements relevant to this study

Section	Environmental Assessment Requirement	Where addressed in this report
Plans and Documents	Geotechnical and structural report	This report

Table 3: Concept Plan conditions of approval and Statement of Commitments relevant to this study

Section	Environmental Assessment Requirement	Where addressed in this report
There are no Concept Plan conditions of approval and Statements of Commitment applicable to this report		

1.2.2 Objectives of the MPE Stage 2 Geotechnical Investigations

This report provides preliminary geotechnical recommendations specific to various components of the Proposal, namely earthworks, foundations, pavements and ground retention across the Proposal site.

Key considerations include:

- Viability of reusing existing fill on the Proposal site as General Fill;
- Extent, thickness and characteristics of topsoil across the Proposal site and consideration to its potential reuse as fill;
- Foundation options and founding levels for typical structures such as warehouses;
- Sub-grade conditions as they may relate to pavement, hardstand and storage areas;
- Surface water and groundwater considerations for concept design; and

- Suitability and characteristics of natural material from the elevated central eastern area of the site for excavation and re-use on site.

2.0 SCOPE OF WORK

The scope of work for the geotechnical investigations undertaken during the 2016 (MPE Stage 2) geotechnical investigations are summarised below. Figure A001 in APPENDIX A shows both the MPE Stage 1 and MPE Stage 2 test locations completed for context. Geotechnical investigations undertaken included:

- Three boreholes with recovery of rock core up to approximately 30 m deep;
- Fourteen large diameter boreholes with recovery of bulk disturbed samples to depths of about 1 m at areas of proposed pavements and hardstand areas;
- Three excavator pits within the local rise on the eastern side of the MPE site to depths of about 5 m;
- Cone penetration tests (CPT) at twenty-two locations to depths up to about 21 m;
- Twenty-nine hand augers to depths of about 0.5 m;
- Nine Dynamic Cone Penetrometer (DCP) tests at selected hand auger locations; and
- Laboratory testing of soil and rock samples for geotechnical purposes.

This Geotechnical Interpretive Report (GIR) addresses the following aspects of the Proposal:

- Geological and geotechnical interpretation and assessment of the site investigation results;
- Recommended geotechnical engineering design parameters with regards to:
  - Earthworks advice: bulking factors, material excavation and re-use of site-won materials, handling and placement considerations, batter slopes for permanent and temporary cut and fill batters, soil stiffness;
  - Foundation concepts for structures (warehouses, buildings, hardstands): alternative footing types (piles/ shallow pads and rafts); soil stiffness parameters; allowable bearing pressures and expected settlements; construction considerations; durability;
  - Retention: advice for potential retaining walls including retention options, design pressure profiles and lateral earth pressure coefficients;
- Advice about additional works that could be carried out to assist in development of the engineering design and to limit ground-related risks for the Proposal.

Sampling and testing to assess the potential for soil and groundwater contamination and acid sulphate soils has been carried out separately by others. No further details of contamination or acid sulfate soils are provided in this GIR.

3.0 SITE DESCRIPTION

3.1 Topography and Terrain

The site is relatively flat, with the natural ground surface rising gently to the east at less than 2 degrees to a locally elevated area within the Proposal site, known as “Moorebank Hill”. The Proposal site also rises gently, at similar shallow grades, from the south (near Anzac Creek) towards a relative high point through its central portion, then falling gently towards Anzac Road to the north.

Moorebank Avenue runs along the western boundary of the Proposal site. The Georges River is located approximately 1km west of the Proposal and generally flows in an approximately south to north direction.

Defence Joint Logistics Unit (DJLU) land borders the Proposal site to the north of Anzac Road. Heavily vegetated bushland borders the site to the east and south (the Boot Land), with Anzac Creek flowing generally west to east along the site’s southern and eastern boundary.

At the time of the investigations for the Proposal, the MPE site was typically occupied by low rise warehouses and administration buildings, with some isolated larger and taller structures (e.g. a tower structure estimated to be up to approximately 25m in height). Apart from the structures described above, the site is typically covered with a mixture of pavement, grasses with scattered trees and hardstand paved areas, with a greater majority of hardstand areas in the northern half of the Proposal site. There is a refuelling facility occupying the southern portion of the MPE site.

3.2 Drainage

The dominant water features of the area are the Georges River (which is not tidally influenced in the vicinity of the Proposal) to the west of the Proposal, and Anzac Creek, which lies to the south of the MPE site and drains away from the Georges River (generally to the north east). Other water systems in the area include ponds and creeks, some of which run towards the Georges River to the west of the Proposal site.

3.3 Climate and Meteorology

The Moorebank area experiences relatively mild temperatures and moderate rainfall. Meteorological data obtained from 1968 to 2016 from the nearest weather station site at Bankstown Airport (Ref, BOM, 2016), shown in Figure 2 presents the mean rainfall, mean maximum temperatures, mean minimum temperatures and rainfall from the 2015 meteorological records.

3.3.1 Overview

February is typically the wettest month and September is the driest. It is noted that in 2015, the wettest month was April whereas in 2014 it was March. Of the 48 years recorded, the annual mean minimum temperature sits at 12.0°C and the mean maximum temperature sits at 23.2°C. The hottest month is typically January, recording a mean maximum of 28.2°C and the coldest month is typically July recording a mean minimum of 5.1°C.

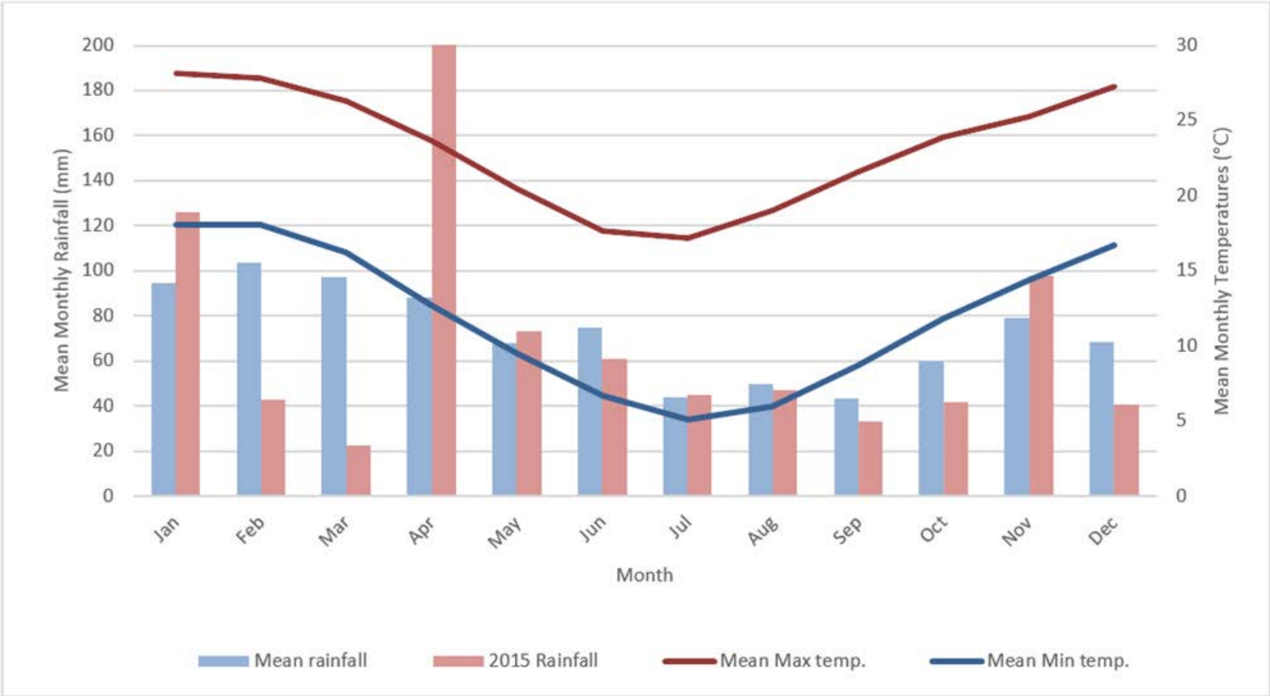


Figure 2: Monthly Rainfall and Temperature - Bankstown Airport (6.5 km from MPE Site)

3.3.2 MPE Stage 2 Investigation Rainfall Conditions

A plot showing daily rainfall data during the period of MPE Stage 2 investigation is presented as Figure 3. The rainfall conditions around the time of the site investigation may impact groundwater levels measured as part of the investigations.

The rainfall data shows that the cumulative rainfall over the two months (January and February) preceding the investigations for the Proposal (undertaken at the beginning of March 2016) was greater than the long term average. However, almost all of the combined 272 mm fell in January (262 mm), making February relatively drier than the previous year and significantly drier than the average. This dryness extended into the period of the Proposal investigations, with only 1 mm of rainfall being recorded during the fieldwork (01 to 09 March 2016). The remainder of the month of March was relatively wet, resulting in similar rainfall to March of the previous year (although less than the historic average).

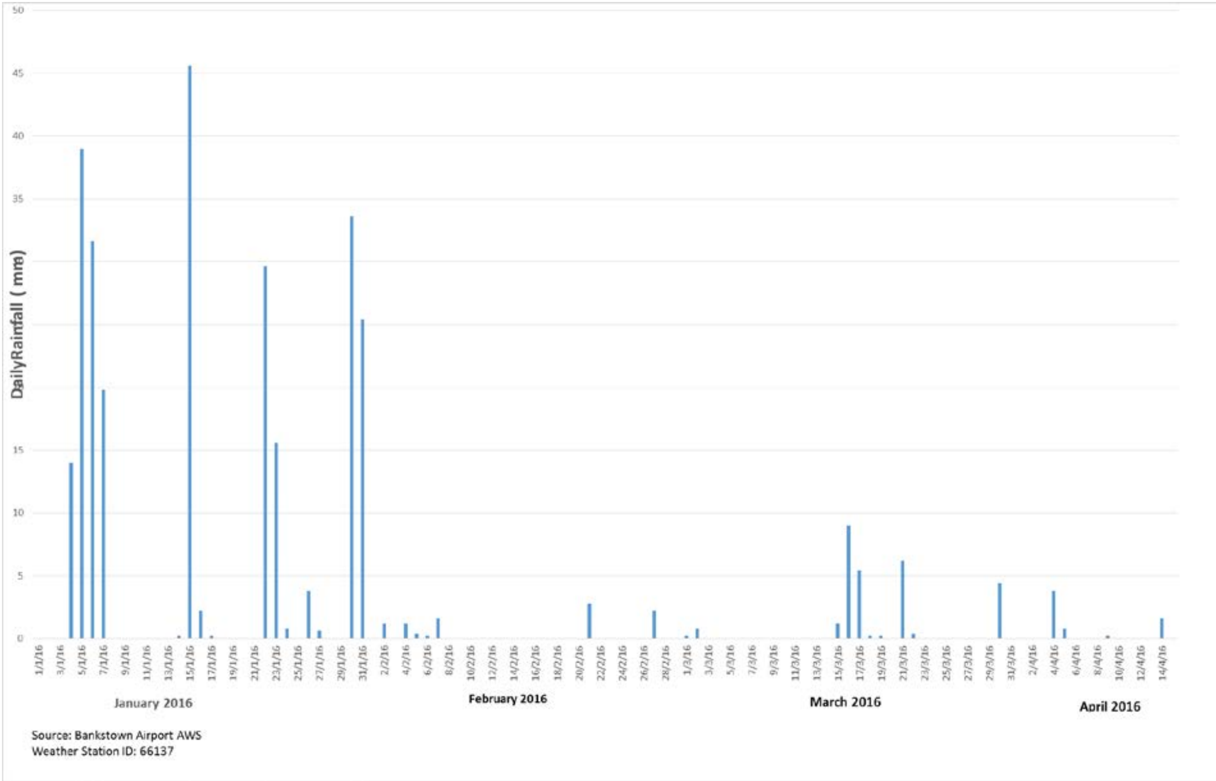


Figure 3: Daily Rainfall - Bankstown Airport (6.5 km from MPE Site) 1/1/2016-14/4/2016

4.0 GEOLOGY AND SOILS

4.1 Regional Geology

The published 1:100,000 Penrith Geological Map (NSW Department of Minerals, 1991), shows the area on which the site lies is characterised by Tertiary age (Ta) alluvial and fluvial deposits. The alluvial and fluvial deposits are described as being of the Pliocene epoch and consisting of clayey quartzose sand and clay. Published mapping indicates that younger Quaternary (Holocene) age (<10,000 years) alluvial deposits (Qha) are confined to the immediate proximity of the George's River, west of Moorebank Avenue (outside of the Proposal site). The Qha soils are therefore not expected to be present on the MPE site; however, there may be some more recently transported alluvial deposits in the vicinity of Anzac Creek, to the south of the Proposal site.

The Penrith Soils Landscape Map (Soil Conservation Service of NSW, 1989) indicates that the Proposal site lies in an area characterised by soils of the Berkshire Park Group (bp). This group is produced on alluvial soils, commonly on elevated Tertiary terraces. The soils comprise shallow clayey sand soils, with frequent ironstone nodules. The soils have a very high wind erosion potential if stripped of vegetation. Gully, sheet and rill erosion can occur on dissected areas.

The published geological mapping indicates nearby surface rock outcrops of Triassic Period Ashfield Shale (Rwa) and also the lower stratigraphic sequence unit Hawkesbury Sandstone (Rh). A transitional unit, known as the Mittagong formation (Rm) is often present between the Ashfield Shale and the underlying Hawkesbury Sandstone and has the potential to be intersected within the subsurface profile of the site. In view of the published mapping, Ashfield Shale is expected to be present, immediately below the overlying soils, across the majority of the Proposal site. However it is likely that over some portions of the Proposal site, particularly toward the south east, that the Ashfield Shale may be absent from the subsurface profile and Mittagong Formation or Hawkesbury Sandstone might be the first rock encountered beneath soils.

A summary of the regional geological stratigraphic column is provided in Table 4 which places the site's inferred position within the regional stratigraphy (as highlighted in green).

Table 4: Regional Geology of Sydney

Group	Formation	Member	Recorded Thickness (m)
Wianamatta Group (Triassic)	Bringelly Shale (youngest)		0 to 256
	Minchinbury Sandstone		0 to 6
	Ashfield Shale	Mulgoa Laminite	0 to 61
		Regentville Siltstone	
		Kellyville Laminite	
		Rouse Hill Siltstone	
Mittagong Formation (Triassic)			0 to 10
Hawkesbury Sandstone (Triassic)			0 to 290
Narrabeen Group (Triassic)	Newport Formation		0 to 50
	Garie Formation		0 to 8

The resolution of published mapping may not capture local variations in stratigraphic boundaries within and near the Proposal site. While the majority of the Proposal site lies entirely within an area mapped as having surface coverage of Tertiary Alluvium, the Proposal site has a long history of intensive military usage with associated cut and fill activities. In some areas it is likely that some of the alluvial soils have historically been removed or reused as fill in parts of the site.

It is also possible that parts of the Proposal site may only encounter locally limited alluvial soil cover and more extensive residual soil strata may be present. This is particularly likely within the central to eastern portions of the Proposal site (i.e. in the vicinity of “Moorebank Hill”) where the landform rises above the surrounding terrain.

Further information on typical bedrock characteristics reported in the literature is provided below.

4.2 Rock Formations

Ashfield Shale is typically a dark grey to black sideritic claystone and siltstone, which grades upwards into a laminite of fine sandstone and siltstone. Bedding within Ashfield Shale is typically close to horizontal, although small scale cross-bedding has been reported as occurring in sandier sub-units.

The Mittagong formation forms a marker band between the Hawkesbury Sandstone and the overlying Ashfield Shale. Pells (1993) makes reference to it being “the passage beds” between the two aforementioned rock units. The formation represents the transition from the fluvial or terrestrial environment of the Hawkesbury Sandstone deposition to the marine delta deposition of the Ashfield Shale, with boundaries often not being clearly distinguishable.

The Mittagong Formation comprises an upper, thin, very fine grained brown sandstone unit (typically 0.5 to 1.5 metres thick) over a lower unit of fine grained sandstone and siltstone (typically 1 to 3 metres thick, but can be up to 10 metres thick).

The Hawkesbury Sandstone is typically a medium to coarse-grained sandstone. Three main sedimentary facies are apparent within the Hawkesbury Sandstone, as follows:

- Massive Facies: Typically internally homogeneous in particle size and massive, with poorly defined undulose bedding. The sandstone is generally fine to medium grained with small flecks of siltstone scattered throughout. Shale breccia (angular siltstone fragments and rounded quartz gravel in a sandy matrix) commonly occurs within the troughs above the erosional surface.
- Sheet Facies: Typically well-developed cross bedding bounded by sub-horizontal bedding surfaces. Cross beds are from a few centimetres to more than 5 m in thickness and commonly dip towards the northeast. The sheet facies sandstone is coarser grained compared with the massive facies. Bedding thickness is generally between 1 m to 3 m. Lenses of conglomeritic sandstone may also occur.
- Mudstone Facies: Laterally discontinuous layers between 0.3 m and 3 m thick, composed of grey fissile mudstone (“Shale”) often laminated with fine sandstone (“Laminite”). These layers have significantly different engineering properties to the sandstone. Clay minerals consist of illite and kaolinite with quartz being the most abundant mineral. Slaking occurs on exposure to wetting and drying effects.

The massive facies sandstones generally have a lower proportion of quartz and higher clay content compared with the sheet facies sandstone. Iron cementation is common in the upper weathered areas and can occur as very high strength bands (generally less than 200 mm thick), which have been referred to as “ironstone”.

The Hawkesbury Sandstone has a shallow weathering profile (typically <3 m) with variable and often discontinuous residual soil cover of sandy clays and clayey sands.

Nearby outcrops of sandstone close to the road bridge at Cambridge Avenue (south-west of the Proposal site) are known to be present. Such outcrops broadly corroborates regional structure contours provided on the Penrith 1:100,000 Sheet 9030. The sheet indicates a structure contour for the base of the Wianamatta Group (top of Hawkesbury Sandstone) of approximate RL10m south of the site (towards Anzac Creek) and approximately RL-10m north of the site. This indicates a stratigraphic boundary between the Ashfield Shale and Hawkesbury Sandstone (or Mittagong Formation) as dipping to the north-north-west across the site. Regionally the bedding of the sedimentary sequences generally dips between 0° and 15° to the west, although localised variations can occur with steeper bedding planes often associated with cross-beds.

4.3 Structural Features, Faults and Dykes

As indicated above, the mapped structure contours for the base of the Wianamatta group dips towards the north-north-west, towards the mapped axis of syncline associated with the Fairfield basin (which is shown to be approximately 5km north of the Proposal site).

The Coastal Lineament is indicated approximately 1km west of the site (i.e. to the west of the Georges River), whilst numerous linear features with a general north-south orientation are depicted on the eastern and western fringes of the MPE site.

Intrusive volcanic features form a minor part of the geology of the Sydney Basin, mainly in the form of diatremes and dykes. There are no dykes or faults shown on the geological plan in the vicinity of the Proposal; however, linear features shown on the geological map to the west and east of the site may imply that faults are present in the bedrock. These features may have had some impact on the present route of the Georges River, as the course of a river can be affected by the presence of weaker zones in the bedrock.

It is possible that localised unmapped faults and dykes occur. Lower strength zones of crushed rock are often associated with faults and dykes are often deeply weathered to clay. Small scale faulting occurs within shales of the Wianamatta Group, but they are usually of limited continuity (i.e. less than 10 m long).

4.4 Hydrogeology

Based on our experience in the area, we expect that there are two main aquifer systems across the MPE site; a perched system within alluvial soils, and a deeper aquifer within the bedrock. Groundwater in the shallow alluvial aquifer is expected to flow towards the Georges River.

Ashfield Shale has a very low rock mass permeability and may act as an aquitard (barrier to groundwater flow). This unit may well reduce the infiltration of groundwater into the underlying sandstone, although some groundwater may flow within this unit through joints or faults. Groundwater in the Shale unit is typically saline and hard, with salinity levels up to 3,100 mg/l having been recorded in the region.

Hawkesbury Sandstone (and rock of the Mittagong Formation) usually has a low rock mass permeability with groundwater flow generally controlled by joints, faults and bedding partings. High permeability is also likely along near-vertical dykes, sheared zones or open joints at relatively low cover below valleys and/or paleo channels.

Groundwater in sandstone is generally of reasonable quality (salinity: typically 200 to 2000 mg/L), mildly acidic and typically with high iron content. Oxidation of iron carbonates on exposure to the atmosphere results in the characteristic red brown staining.

4.5 Erodibility of Soils

In general, soils adjacent to creeks and rivers and on areas of the Proposal site with steeper slopes will be at higher risk of erosion.

Based on the method presented in the Blue Book (Landcom, 2004), the potential erosion hazard for soils in the site locality is that areas with slope gradients of > 10% have a high erosion hazard. The potential for both erodibility and dispersion should be borne in mind when managing these soils.

Material loss or scour, such as from embankment batter slopes or exposed surfaces, can result from erosion of material (i.e. liberation and transportation of material particles or ‘chunks’ of material) or dispersion of reactive clays (or a combination of the two) with water as the liberating agent.

Granular materials such as sands and gravels may be prone to bulk erosion but will require some energy from a water source (e.g. surface slope gradient, or exit velocity from a piped water source directed onto the material) to disturb and transport the material. Dispersion of reactive clays in contrast can occur with relatively minimal energy input (i.e. the clay material can essentially dissolve with sufficient hydraulic gradient only required to transport the turbid solution away).

The Emerson Class Number provides an indication of the dispersion potential of a soil. As part of the Proposal investigations, testing was carried out on numerous soils sampled across the Proposal site and the results are discussed in Chapters 5.0, 7.0 and 8.0 below. In general the soils have Emerson Class Numbers of 4 or 5.

Typically Emerson Class Numbers of 1 to 4 are taken to indicate soils with a dispersive potential requiring particular care with respect to their placement and the management of surface water flows and below surface groundwater gradients (such as may exist within stormwater detention basin earth bund walls), with Class 1 soils being of the highest dispersive potential.

The test results indicated that while the tested soils in their natural state are non-dispersive (i.e. no results in the range of Emerson Class Number 1 to 3 were recorded in the samples tested), the remoulding and breaking down of soil bonds can result in dispersive behaviour. These soils can break down by water turbulence or concentrated rapid water flow. Hence, soils which are exposed will need to be protected to mitigate the erosion potential, for example by planting vegetation, or use of geosynthetic products.

5.0 SITE GEOTECHNICAL MODEL

For the purpose of geotechnical characterisation of the subsurface conditions within the Proposal site, soil and rock types at the site have been generalised into a system of geotechnical units, as illustrated in Table 5.

Table 5: Geotechnical Model

Unit		Sub-unit	
1	Surficial Soils and Pavement	1A	Topsoil/Fill
		1B	Anthropogenic Fill
		1C	Granular Fill
		1D	Cohesive Fill
		1E	Existing Pavement
2	Recent Alluvium	2A	Sand – not observed in Stage 2 investigations
		2B	Clay – not observed in Stage 2 investigations
3	Older Alluvium	3A	Sand
		3B	Clay
4	Shale	4A	Residual Shale Soil
		4B	Extremely Low to Low Strength Shale
		4C	Shale of medium strength or higher
5	Sandstone	5A	Residual Sandstone Soil – not observed in Stage 2 investigations
		5B	Very Low to Low Strength Sandstone
		5C	Sandstone of medium strength or higher

A generalised description of the ground conditions encountered is provided in Chapter 5.1 below followed by a more detailed description of the geotechnical characteristics of each unit (Chapters 5.2 to 5.5. Geotechnical cross-sections are presented in APPENDIX B, which were developed using available existing investigation results and published regional geology.

5.1 Encountered Ground Conditions

Away from paved areas, the materials that were generally encountered across the Proposal site comprised a layer of topsoil overlying fill, below which tertiary alluvial soils are underlain by residual soil.

Alluvial soils are present extensively across the Proposal site and have their greatest depth at the northern, southern and western flanks of the site, where significant thicknesses of residual soil are not identified. Thicker residual soil layers are encountered to the east and over the central portion of the Proposal site (i.e. towards the elevated central eastern portion of the site).

The greater extents of alluvial soils are typically found over the lower lying portions of the Proposal site, which are in closer proximity to the Georges River and its tributaries (e.g. Anzac Creek at the south of the site). The more elevated portions of the MPE Stage 2 site are typically underlain by residual soil with alluvium absent from the profile in some places.

Bedrock is typically shale, which is underlain by sandstone; however, towards the south of the Proposal site, sandstone was encountered immediately below the soil (i.e. at borehole GA-BH-03). This corroborates structural contour mapping of the base of the Wianamatta Group presented in the 1:100,000 Penrith Geological Series Sheet 9030.

A typical characteristic of weathered sandstone and shale is planar weathered seams running parallel with bedding. These defects are a significant factor in the engineering classification of rocks in the Sydney Basin (i.e. Pells et al, 1998). The weathered seams are variable in thickness (usually less than 100 mm thick), generally sub-horizontal and usually contain a combination of sand, silt and high plasticity clay, depending on the parent rock. They generally decrease in frequency with depth and degree of weathering of the parent rock. More specific detail on rock defects observed within the rock core obtained as part of Proposal investigations is provided in Chapters 5.4 and 5.5.

Groundwater observations were obscured by the addition of drilling water in boreholes, except at borehole GA-BH-01 where minor inflow was observed during SPT testing at 11m below ground surface (approximate RL of 2.7m). Although affected by the subsequent addition of drill water, a standing water depth of 7.9m below surface level (approximate RL of 5.8m) was recorded within borehole GA-BH-01 on completion of drilling.

The water level within Georges River weir has previously been noted as approximately RL 5m (Golder, 2015b).

All excavator pits and hand auger investigation locations were dry during and on completion of excavation. Cone Penetrometer Testing (CPT) indicated water levels between 1.5m (GA-CPT-14, southernmost and closest to Anzac Creek) and 7m (GA-CPT-03, north eastern corner of site) depth below ground surface level, equivalent to a range in level of approximately RL 9m to RL14.5m. Groundwater depths observed during the Stage 2 investigation have been used to interpret groundwater levels shown on the attached Cross-Sections in APPENDIX B.

5.2 Surficial Soils and Pavement – Unit 1

5.2.1 Unit 1A – Topsoil

A topsoil layer is present across most areas of the Proposal site where pavements or structures are not present. Topsoil was recorded at all but one location (GA-HA-14). The topsoil has a recorded thickness varying from 0 to 0.4 m but was typically 0.1 m thick. The topsoil is typically underlain by fill but in some locations has developed naturally above alluvial or residual soils.

The topsoil encountered was typically dry, fine to medium grained silty sand with fine to medium sub-angular igneous gravel. Some topsoil of a dominantly clay composition was encountered (i.e. at 3 of 29 hand auger locations, GA-HA-05, GA-HA-25 and GA-HA-29). At one location (GA-HA-19) the topsoil was essentially sand. Variations in gravel and organic contents were observed between samples.

Isolated occurrences of man-made waste materials, such as plastic bags and brick, were identified during the Stage 2 investigations.

A total of 22 organic content tests were undertaken on topsoil samples with results ranging from 0.1% to 1.2% (GA-HA-25) and an average value of 0.5%. These are considered relatively low values for a topsoil and not outside a range which might be accepted within general filling, dependent on the particular application of the fill and its performance requirements.

The colour of the topsoil varied from pale to dark grey and brown, with darker material typically, but not always, containing greater organic content. Figure 4 below shows a colour contrast of material representative of the range of topsoil and fill encountered. Note that the rightmost sample, although the darkest in colour, was not assessed to contain a significant proportion of organics, and is inferred to be a dark grey fill with high silt content.

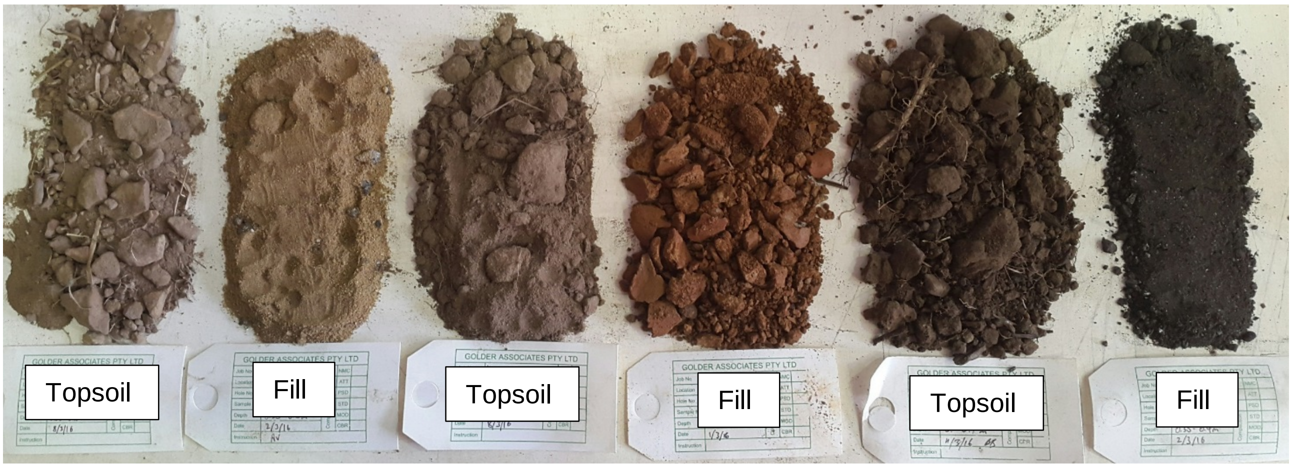


Figure 4: Colour contrast of topsoil and/or fill samples from Stage 2 MPE investigation

The assessment of what constitutes “topsoil” can be subjective and the term is best applied in a project specific sense which takes due consideration of the desired end usage of the material and project specific criteria for the definition of material as topsoil. We recommend that considerations and definitions associated with this should be addressed in a site specific Earthworks Specification.

5.2.2 Units 1B – Anthropogenic Fill

Anthropogenic fill was not encountered to any significant extent in the Proposal site, however isolated pockets of anthropogenic material were encountered at two locations during the Proposal investigations at GA-HA-01 (a brick and a nail) and GA-HA-07 (a plastic bag).

5.2.3 Unit 1C and 1D – Granular and Cohesive Fill

The majority of fill encountered beneath the topsoil was granular (19 out of 29 hand auger locations for example) and was typically a dry, silty sand. Where cohesive fill was encountered it was typically a medium to high plasticity clay, dry of the plastic limit and is inferred to have likely been re-worked site won material. The fill layer typically extends to depths of 0.3m to 0.5m. However, numerous hand auger locations terminated at their target depth of 0.5m within the fill and thus the fill may have extended further at those locations.

Dynamic Cone Penetration (DCP) test results often showed a reduction in blow count (i.e. number of blows per 100 mm penetration) at about 0.5m depth. This may be indicative of poor foundation preparation at the time of fill placement or may reflect the extent of desiccation prevalent at the time of investigation. The maximum depth of fill encountered over the Stage 2 MPE site during the investigations was 1.25 m at test pit TP1205.

Based on our field observations and the results of DCP testing, the existing fill on site is inferred to have some degree of consolidation, be it natural or from compactive effort. However, there are some localised areas that displayed low DCP blow counts, indicating that the fill may not be of consistent stiffness or strength across the site.

5.2.4 Unit 1E – Pavement

Existing pavement was encountered at 7 locations (borehole GA-BH-04, 05, 10, 12, 14, 16 and 17).

Two of these locations (boreholes GA-BH-14 and GA-BH-17) comprised concrete pavement, between 90mm and 130mm thickness. At these two locations, granular base-course materials extended to depths of 700mm and 560mm below existing ground surface, respectively.

The remaining five pavement boreholes encountered a mixture of asphaltic concrete (30mm to 60mm thickness) and asphaltic spray seal over a cement modified base. Two of these locations (boreholes GA-BH-12 and GA-BH-16) were in areas of pavement overgrown by topsoil/grass cover.

Total observed pavement thicknesses (including wearing course and base layers) varied between 110mm and 700mm and existing conditions varied from poor/over-grown to good. Sub-grade materials below pavements were typically high plasticity clay, inferred to be Unit 1D material (refer to section 5.2.3). The exception to this was at borehole GA-BH-12, where the pavement had been constructed above fill comprising shale, sandstone, brick and charcoal gravel.

DCP testing was undertaken within the sub-grade below pavement layers and recorded variable results. Generally, moderate blow counts (per 100 mm depth interval) of 5 or more were recorded in the sub-grade materials, except at boreholes GA-BH-12, GA-BH-14 and GA-BH-17, where blow counts were lower.

- Borehole GA-BH-12 displayed variable DCP blow counts (typically 3 on average) within the subgrade fill from 0.6m to 1.3m depth below pavement surface level (below which blow counts of greater than 5 per 100mm were recorded).
- Borehole GA-BH-17 displayed low DCP blow counts (typically 1 or 2 per 100 mm depth interval) within the sub-grade fill from 0.7m to 1.7m depth below ground pavement surface level (below which blow counts of greater than 7 blows per 100mm were recorded).
- Borehole GA-BH-14, the DCP blow counts within the sub-grade fills were typically 5 (or more) blows per 100 mm (between 0.8 m and 1.4 m depths). Below 1.4 m depth, the DCP blow count reduced significantly and remained low (typically 2 blows per 100mm) to completion of the test at 2.5m depth.

It is likely that some localised hollows and depressions on the MPE site have been filled prior to pavement construction. Where ‘poor’ fill subgrade is present within the depth of influence of pavement loading, some surface expression of the poor ground, such as cracking, rutting or deflection may be observable . This, along with the adequacy and design versus operational life of the pavement may be the case at location GA-BH-17, where the observed condition of the pavement was poor and low blow counts where recorded within the underlying material.

Where the “poor” fill material is deeper than the zone of influence of past pavement loading, surface failures may be absent. Thus it cannot be assumed that a lack of distress observable at the pavement surface is indicative of ‘sound’ ground conditions for the depths which might influence the performance of particular design elements (such as structural footings or more heavily loaded hardstands/pavements).

5.3 Older Alluvium – Unit 3

Where investigations extended beneath the Unit 1 fills, older alluvial soils were typically encountered. The thickness of alluvium recorded varied significantly between locations, with the deepest layers occurring at the northern, western and southern flanks of the MPE site. At the northern and southern extent of the MPE site, the thickness of alluvium was approximately 20 m based on borehole logs and CPT log interpretation, with a maximum depth of up to 23 m at borehole GA-BH-03. The depth of alluvium recorded reduced to approximately 5 m within the central portion of the MPE site (i.e. in the vicinity of GA-BH-02) and less than 1 m thick at the eastern fringe (i.e. in the vicinity of GA-EP-02 and GA-EP-03).

The alluvium within the MPE site is typically a high plasticity clay with some granular and lower plasticity zones, particularly at the southern extent of the site. The alluvial clay contains ironstone nodules and is typically very stiff or hard consistency. Where granular bands or seams were encountered in boreholes, they were assessed as dense or very dense. These seams and bands ranged in thickness from 300mm to 1.2m.

In some cases it is difficult to distinguish between Older Alluvium and residual soils, particularly close to the rock contact. Compared to the Unit 4a residual soils, the Unit 3 materials are generally slightly lower consistency (e.g. Very Stiff cf. Hard), have higher moisture contents (e.g. closer to the plastic limit rather than dry of the plastic limit) and are of higher plasticity (e.g. Liquid Limits of up to 82%).

However, at many of the investigation locations, there is a sharp transition from alluvial sediments into the underlying bedrock. This implies that residual soils have been scoured from the bedrock surface and transported alluvial soils (which may have been produced from the same parent rocks upstream) have been deposited onto weathered rock.

The depositional environment of the alluvium means that the lateral extent and thickness of individual layers is likely to be highly variable. Therefore, for the magnitude of Unit 3A bands encountered and the relative spacing of boreholes undertaken, the geological cross-sections in APPENDIX B do not differentiate between sand (Unit 3A) and clay (Unit 3B).

Although, the lateral extent and thickness of individual layers of the alluvial soils is likely to be highly variable, the dominant nature and characteristic of the Unit 3 alluvial soils is cohesive and highly plastic.

Unit 3B material was characterised by very high plasticity (e.g. Liquid Limit of 82%), high swells and variable but typically low California Bearing Ratio (CBR) values (on 4 day soak testing). The CBR values recorded for Unit 3B material for 10 tests undertaken ranged from 1.5 % to 7 % (with a mean of 3 %), with swells of 0.8% to 5.5% (and a mean of 2.4%).

Optimum Moisture Contents (OMC) are typically in the range of 17 % to 32% with comparable field moisture contents of about 17 % to 34%. However some field moisture contents of discrete samples varied significantly from dry to wet of their respective OMCs.

Dispersion tests indicated the Emerson Class is 4 or 5 (based on 3 tests).

5.4 Shale – Unit 4

Unit 4 includes sub-units 4A (Residual Soil), 4B (very low to low strength siltstone) and 4C (medium strength or higher siltstone). In general the residual soils below the MPE site appear to be relatively thin, with a relatively abrupt transition from the older alluvium to Unit 4B.

5.4.1 Unit 4A – Residual Soil

Residual soil (Unit 4A) was identified at three locations;

- Borehole GA-BH-02, from 5.4m to 8.7m depth below ground level (although it is noted that this material was logged as borderline Unit 4B) and
- Excavator Pits GA-EP-02 and GA-EP-03 from 0.7m to 1.5m and 0.9m to 1.4m depths below ground surface respectively.

Residual soil was inferred to be absent over the depth of investigation at other locations. It was not possible to distinguish residual soil and alluvium from the CPT data, and soils penetrated by CPTs have been inferred to be Unit 3.

The Unit 4A material was characterised by high plasticity, high swells and variable but typically low CBR values on 4 day soak testing. The CBR values recorded for Unit 4A material from 5 tests undertaken ranged from 0.5 % to 11 % (with a mean of 4 %), with swells of 0.1% to 6.4% (and a mean of 2.9%).

Optimum Moisture Contents (OMC) are typically in the range of 15.5 % to 27.3% with comparable field moisture contents of about 12 % to 24%. However some field moisture contents of discrete samples varied significantly from dry to wet of their respective OMCs.

Dispersion tests indicated the Emerson Class is 5 (based on 4 tests).

5.4.2 Unit 4B – Extremely Low to Low Strength Shale

The Unit 4B material contains decomposed zones, comprising clays of variable plasticity and reactivity. The result of this is the potential for variable CBR test results that do not necessarily improve with depth, as might be expected. This is common for the near surface weathered Shale which would likely classify as "Class V Shale" in accordance with the Pells classification system (Pells, 1993).

Unit 4B material was encountered within Excavator Pits GA-EP-02 and GA-EP-03 from 1.5m and 1.4m (respectively) to termination depth of 5m depth below surface level.

The CBR values recorded for Unit 4B material from 6 tests undertaken ranged from 3 % to 12 % (with a mean of 7 %), with swells of 0.0% to 2.5% (and a mean of 1.0%).

Optimum Moisture Contents (OMC) are typically in the range of 11.6 % to 14% with comparable field moisture contents of about 9.6 % to 13.6%. Field moisture varied from dry to approximately equal to OMC.

Dispersion tests indicated the Emerson Class is 4 or 5 (based on 2 tests).

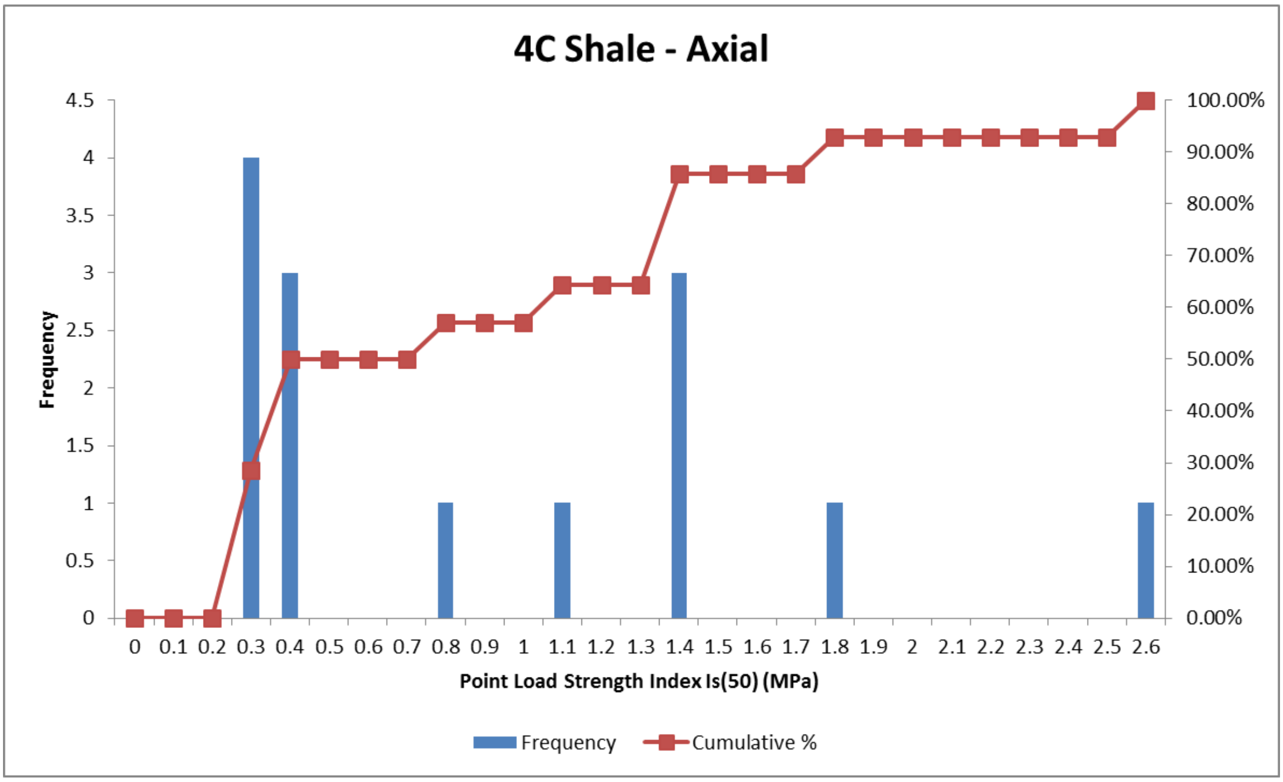
5.4.3 Unit 4C – Medium Strength or Higher Shale

The Unit 4C material improved from moderately weathered on first contact becoming slightly weathered to fresh with depth. In both boreholes (GA-BH-01 and 02) rock strength improved from typically medium strength to high strength with depth.

Defects identified within the rock core comprised numerous sub-horizontal bedding partings, with clay coatings and veneers and predominantly low angle joints typically dipping 20º to 45º.from the horizontal. Infrequent steeper joints dipping 60º to 80º were also identified.

- Within borehole GA-BH-01 the bedrock comprises Unit 4C from 20.5m below ground surface to completion at 26.46m depth. Defect frequency reduced from typically 30mm to 100mm spacing up to 23.9m depth to 1000mm to 3000mm spacing from 23.9m to completion
- Within borehole GA-BH-02 the bedrock comprises Unit 4C from 10.1m below ground surface to 16.1m depth. Defect spacing was typically 30mm to 100mm up to 11.9m below which defect frequency decreased to a typically spacing of 100mm to 300mm.

Figure 5 shows a summary of the Is_{50} rock strength results obtained from point load testing carried out on Unit 4C samples during the Stage 2 investigation. As expected the data shows a strong anisotropy related to the horizontally fissile nature of the shale.



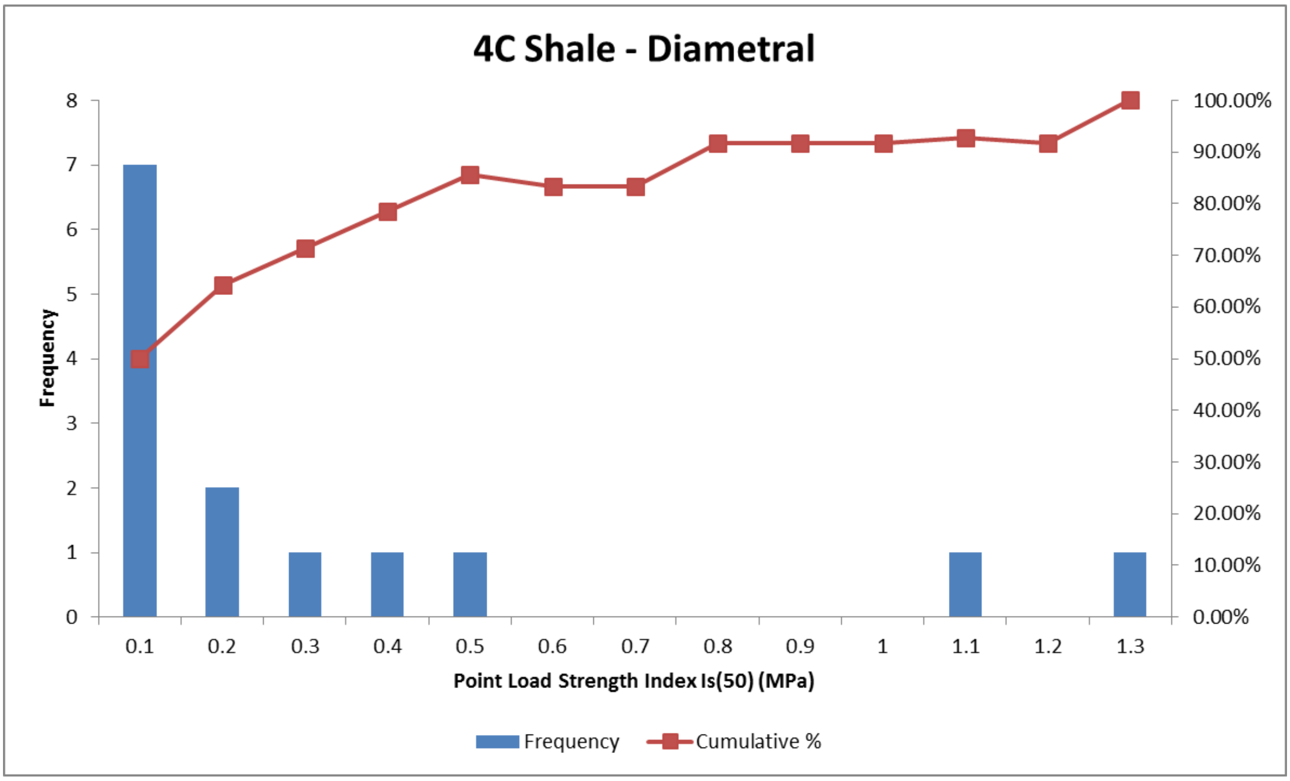


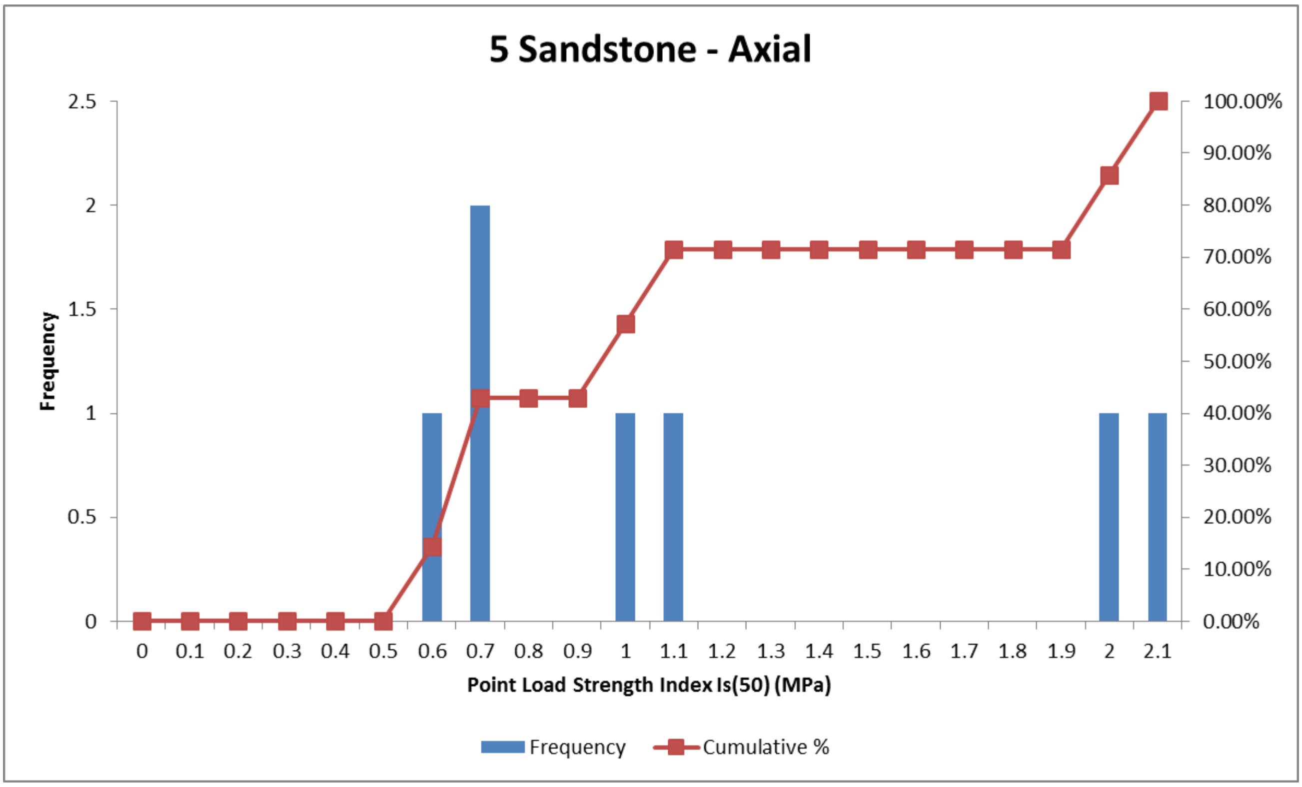
Figure 5: Point Load Test Results for Unit 4C

5.5 Unit 5 –Sandstone

Sandstone was encountered in borehole GA-BH-03 only, at 22.5 m depth below ground level, underlying the Older Alluvium (Unit 3). The sandstone is highly weathered (with extremely weathered bands) and low strength on first contact (i.e. Unit 5B), improving to be slightly weathered or fresh and high strength (i.e. Unit 5C) from 24.1m depth to completion of the borehole at 30m depth. The sandstone contains siltstone laminations (at times carbonaceous) and bedding typically dips approximately 5º – 10º from horizontal, with some zones of cross-bedding observed to dip up to 20º.

Defects identified within the rock core predominantly comprise sub-horizontal bedding partings, with defect spacing typically 30mm to 100mm up to 24.1m depth below surface level and typically 300mm to 1000mm to borehole completion at 30m.

Figure 6 shows a summary of the Is_{50} rock strength results obtained from point load testing carried out on Unit 5 samples during the Stage 2 investigation.



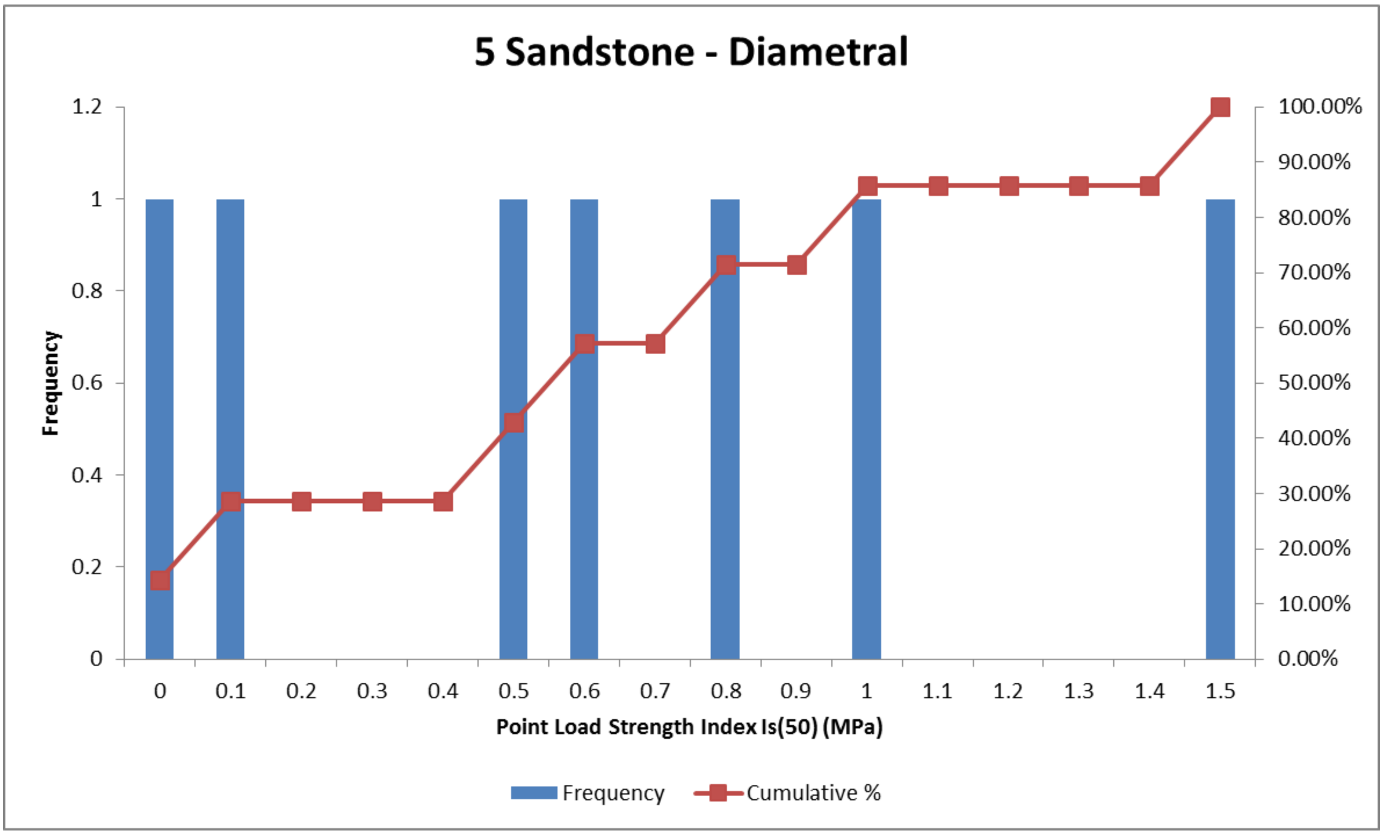


Figure 6: Point Load Test Results for Unit 5

As mentioned above, there is a published mapped boundary between Ashfield Shale and Hawkesbury sandstone, inferred to run beneath the southern portion of the site, and as such there is the potential for rock head to comprise Mittagong Formation rock over a significant portion of the MPE site.

6.0 DESIGN PARAMETERS

Geotechnical design parameters are nominated in Table 3, below. The following notes should be considered when using these parameters:

1) All values of K assume level ground above the wall. Higher coefficients would apply where the ground surface slopes above the wall, or alternatively this should be modelled as a surcharge load.
2) Appropriate vehicle/structural surcharge pressures should be added to the above earth pressures.
3) Appropriate water pressures should be added unless effective drainage at the rear of the wall is provided.
4) K_0 values are appropriate for rigid wall design; lower values may apply on consideration of wall movements and development of partial or full active pressures. Design tools should be used that allow for modelling of staged excavation processes and stress relaxation. Where design methods do not account for this, alternative pressure envelopes are suggested in Figure 8, below for propped/anchored retaining systems (refer to Figure E5 of AS4678 for further information). Water pressures and appropriate vehicle / surcharge pressures would need to be added to the earth pressure design profiles in Figure 8.
5) Active and passive earth pressure coefficients based on Caquot and Kerisel, 1948, assuming zero soil / wall friction, as the wall is to be designed for no or negligible wall movement. Golder note that generally 0.1%H to 0.4%H movement (4 to 16 mm for a 4m wall) is required to develop active pressures, but that 5 to 10%H movement (200 mm to 400 mm for a 4 m wall) is required to develop full passive pressures. The stability and serviceability performance of walls should both be assessed.
6) Preliminary geotechnical design parameters for piles are summarised in Table 3 below, including serviceability limit state end bearing and ultimate limit state shaft resistance.
7) Ultimate load capacities can be estimated by multiplying the above serviceability end bearing values by a factor of 4. The geotechnical reduction factor ϕ_g to be applied to the ultimate capacities will depend on the foundation type, structural redundancy and level of testing proposed, in accordance with Australian Standard AS2159 (2009). Higher load capacities may be able to be adopted if a limit state approach is adopted and settlements calculated using higher end pressures and shaft adhesion values are found to be acceptable. Trial piles and/or pile testing may be necessary to justify adoption of high ϕ_g factors and/or higher design parameters than suggested in Table 3.
8) Piles should penetrate into the design founding strata a minimum of one pile diameter (or minimum 500mm). Piles designed using the serviceability end bearing pressure below should result in settlements less than 1% of the pile diameter. Ultimate bearing resistance will occur at larger settlements, typically up to 5% of the pile diameter.
9) We note that the installation method for CFA piles requires elevated concrete pressures during concreting. This effectively results in a "prestressing" of the base and shaft resistance as these elevated pressures are locked into the concrete as the concrete sets. This may explain the higher shaft and base resistances obtained when testing properly installed CFA piles. It has been our experience that the shaft and base resistance values obtained from testing properly installed CFA piles are similar to those expected for driven piles and significantly higher than design values which would normally be adopted for bored piles; this may be considered in pile design for the site, and should be verified by pile testing (PDA and CAPWAP).
10) Soil properties are derived from typical values, based on laboratory classification of the soils encountered during borehole excavations, the in-situ tests (SPT N values and CPT results) and engineering judgement.

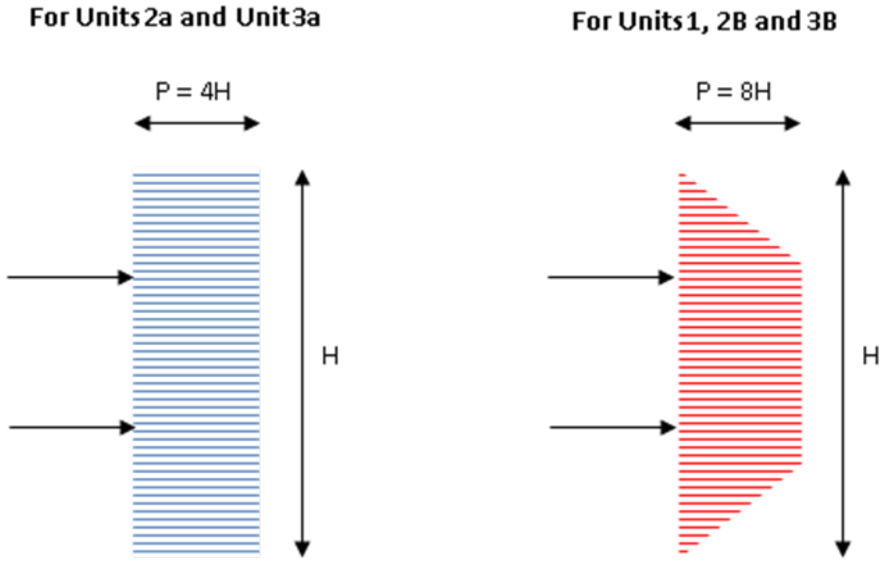


Figure 7: Alternative Earth Pressure Envelopes

6.1 Geotechnical Design Parameters

Table 6: Design Parameters

Unit	Description	Moist Unit Weight γ (kN/m^3)	*Undrained Strength*		*Drained Strength*		Undrained Modulus E_u (MPa)	Drained Modulus E'_s (MPa)	Poisson's Ratio v'	At-rest coefficient K_0 [1,2,3,4]	Active Earth Pressure Coefficient K_a [1,2,3,5]	Passive Earth Pressure Coefficient K_p [1,3,5]	Serviceability End Bearing Pressure (kPa) [6,7]	Ultimate End Bearing Pressure (kPa) (rock only)	Ultimate Shaft Adhesion (kPa) [6, 7, 8]
			Su (kPa)	Φ_u (°)	c' (kPa)	Φ' (°)									
1A	Topsoil	16	N/A	N/A	0	25	NA	10	0.3	0.53	0.45	2.2	N/A	N/A	N/A
1B	Anthropogenic Fill	17	NA	NA	0	28	N/A	5	0.3	0.55	0.4	2	N/A	N/A	N/A
1C	Granular Fill	18	N/A	N/A	0	32	N/A	15	0.3	0.47	0.3	3.2	N/A	N/A	N/A
1D	Cohesive Fill	18	75	0	0	25	15	10	0.3	0.53	0.45	2.2	N/A	N/A	N/A
3A	Dense Sand	20	N/A	N/A	0	38	N/A	100	0.3	0.38	0.24	4	300Z (Max 2,500)	N/A	4Z (Max 60, bored) 8Z (Max 120, driven)
3B	Very Stiff Clay	20	150	0	5	28	60	40	0.3	1.73	0.33	2.5	400	N/A	75
4A	Residual Shale Soil	20	150	0	5	28	N/A	40	0.3	1.73	0.33	2.5	400	N/A	75
4B	Extremely Low to Low Strength Shale	22	N/A	N/A	25	35	N/A	150	0.25	0.3	0.2	3	1,000	3,000	300
4C	Shale of medium strength or higher	24	N/A	N/A	50	40	N/A	1,000	0.2	-	-	6	4,000	10,000	1,000
5A	Residual Sandstone Soil	20	150	0	5	28	N/A	40	0.3	1.73	0.33	2.5	400	N/A	75
5B	Very Low to Low Strength Sandstone	22	N/A	N/A	50	35	N/A	200	0.25	0.3	0.2	3	1,500	3,000	500
5C	Sandstone of medium strength or higher	24	N/A	N/A	100	42	N/A	2,000	0.2	-	-	9	8,000	20,000	2,500

6.2 Soil, Rock and Water Aggressivity to Concrete and Steel

The laboratory test results were compared with the guidelines for durability presented in Tables 6.4.2 (C) and 6.5.2 (C) of AS 2159-2009 *Piling – Design and Installation*. A summary of the aggressivity exposure classification for the soil is presented in Table 7 below:

Table 7: Aggressivity Exposure Classification

Sample ID*	Exposure Classification	
	For Concrete Piles	For Steel Piles
TP1202, 0.1-0.5 m, FILL (Unit 1C)	Non-aggressive	Non-aggressive
TP1206, 1.0-1.3 m, (Unit 3B)	Moderate	Non-aggressive
GA-EP-01 0.6-0.7 m, Clay (Unit 3B)	Mild	Mild
GA-EP-01 3.6-3.7 m Clay(Unit 3B)	Mild	Moderate
GA-EP-02 1.3-1.4 m, Clay (Unit 4A)	Mild	Non-aggressive
GA-EP-02 3.5-3.6 m Clay/Shale (Unit 4B)	Mild	Non-aggressive
GA-EP-03 1.1-1.2 m, Clay (Unit 3B)	Mild	Non-aggressive

* Depth below ground surface level

6.3 Earthquake Parameters

6.3.1 Design Earthquake (PGA)

The subsurface profile of the MPE site generally comprises very stiff / dense alluvial soils over bedrock. Based on AS1170.4 (Standards Australia, 2010) the following parameters are recommended for earthquake design:

- Probability Factor, K_p = 1.0 (assuming a 1 in 500 Annual Probability of Exceedance);
- Hazard Factor, Z = 0.08 for Sydney;
- Site Sub-soil Class = Ce (Shallow Soil Site).

6.3.2 Preliminary Liquefaction Assessment

Based on the generally dense nature of the granular soils on the site, we consider that there is a low risk of liquefaction being triggered under the 1 in 500 year AEP event.

7.0 EXCAVATIONS

7.1 Excavation Conditions

Temporary excavations will likely be required for the removal of existing structures, services and unsuitable soils. Minor excavation will also be required in the elevated area to the east of the site. Basement parking within the Freight Village at the north western corner of the site would also require excavation.

It is noted that depth to rock typically reduces from west to east across the site and towards the centre of the site, with the most shallow rock being encountered in the elevated central eastern portion of the MPE site. Some variability in depth to rock is anticipated across the footprint of the proposed basement car parking within the Freight Village, with shallower rock depths (and potentially higher ground water levels) likely towards the east of the footprint. However, given the relatively shallow depth of excavation currently proposed (less than 2m) such features are likely to have greater influence on foundation considerations (e.g. piling depths and casing requirements, should piled foundation be adopted) than excavation conditions. The impact of varied depth to rock will require consideration in detailed design settlement calculations.

A conventional bulldozer or hydraulic excavator can be used to excavate the soils. Removal of obstructions in the fill such as building foundations may require the assistance of a rock breaker.

Excavation within the Unit 3 soils, should typically be achievable with conventional hydraulic excavators of sufficient capacity and fitted with appropriate buckets (e.g. ‘tiger toothed’ should zones of iron cemented seams or nodules be encountered). Should extensive bands of iron cemented material be encountered, a rock breaker or a dozer with a ripper may be required. As an example, excavator pits were relatively rapidly excavated during the Stage 2 investigation within Unit 3 and up to Unit 4B material with a 21T excavator utilising a flat toothed bucket. Should extensive excavation into Unit 4 material be required, a rock breaker or a dozer with ripper would be needed. In the interests of economic production rates, such machinery may be required for Unit 3 material where extensive iron cemented bands exist within the excavation depth range.

Demolition and removal of existing facilities including structures, their foundations, associated services and pavements (asphaltic and concrete) will require heavy duty earthworks equipment (e.g. large excavators, with appropriate bucket teeth and ripping hooks, rock breakers and dozers). The advice of a specialist earthworks contractor should be sought to determine appropriate equipment and methods for demolition and removal of existing surface and near surface features. Consideration should be given to the impact of proposed equipment and methods on opportunities for re-use or sorting of demolition material, as well as planning and approval requirements (e.g. operation hours, dust and noise).

Limited dispersion testing gave Emerson Numbers of 4 and 5, indicating some dispersion potential. Visual observations are that soils in the local area have been eroded due to surface run-off. Where possible, topsoil and grassed areas should be left in place until construction works start.

7.2 Vibration

Care should be taken during excavation (and backfilling compaction) to limit the vibration impacts on structures that need to be retained (or new structures). In addition, the potential vibrations from construction may need to be considered with respect to buried services, nearby commercial and residential properties. We recommend that the following measures are taken to assess and manage vibration risks:

- Carry out an assessment of the proximity of vibration sensitive structures to the site;
- Carry out dilapidation surveys on vibration sensitive structures before work commences and after work has been completed; and
- Prepare a vibration management plan setting limits on Peak Particle Velocity (PPV) and install, where required, monitoring systems to assess vibrations.

7.3 Groundwater

At the time of the geotechnical investigation, observations made in probe holes created by CPT investigation equipment and boreholes prior to the introduction of drilling fluids indicated groundwater typically at about 4 m to 7 m depth below the existing ground levels (about RL 9 to 12 m AHD) over the majority of the MPE Stage 2 site. However groundwater was encountered within 1.5m of the ground surface at GA-CPT-14 (i.e. about RL 14.5m AHD) at the south eastern corner of the MPE Stage 2 site in proximity to Anzac Creek.

Inferred groundwater levels based on field records at discrete locations are shown on the geological sections of APPENDIX B.

Groundwater is anticipated to be deeper than the expected depth of bulk excavations over the majority of the MPE Stage 2 site. However, groundwater is likely to be encountered within the depth of bored piles, if used (see Chapter 9.4.2). Groundwater may also be encountered within excavations undertaken towards the southern corner of the MPE Stage 2 site (i.e. in proximity to GA-CPT-14 and Anzac Creek) for depths greater than approximately 1.5m. Should bulk excavation to such depths (or greater) be required in this area, consideration will need to be given to the potential for, and management during construction of groundwater inflows.

The implication of variation in groundwater level and rock head across the site (refer to the geological sections of APPENDIX B) should be borne in mind as it relates to the design. It is possible that groundwater will be encountered within service trenches in some parts of the site (dependent on their location and depth). This is particularly pertinent to the eastern area of the proposed footprint for basement car parking in the Freight Village. Although not encountered during Stage 2 investigation of the elevated central eastern portion of the site (Moorebank Hill), groundwater may be encountered at relatively shallow depth if extensive excavation is undertaken within the central and eastern area of the site. The potential for seasonal variations in groundwater level should be borne in mind.

7.4 Surface Water Management

Management of surface water will be required during earthworks. Management methods to limit impacts of water on proposed excavations may include:

- Diverting surface water flows away from excavations; and
- Using sediment controls and pumping from excavation sumps to manage inflows from rainfall, local surface water runoff and seepages from the face of cut slopes.

7.5 Excavation Support Requirements

Recommendations on suitable batter slopes are provided in Chapter 8.6. In areas of the Proposal site where excavation induced movements must be kept as low as practical (i.e. to protect existing or new structures and services), or insufficient space exists to accommodate batter slopes the following temporary retention options may be considered:

- Proprietary shoring systems (i.e. hydraulic trench boxes or shoring systems); or
- Anchored/braced sheet pile walls (achieving toe embedment with these walls may require pre-boring if iron cemented layers are encountered).

In areas where permanent structures are required (for example deep pumping stations), the following options could be considered:

- Anchored/braced reinforced concrete contiguous pile walls; or
- Anchored/braced reinforced concrete soldier pile walls with shotcrete infill panels.

Cantilevered sheet pile wall options may be problematic due to uncertainty of achieving toe embedment due to iron cemented layers within Unit 3 soils. For this reason, contiguous bored concrete walls or shallower braced or anchored sheet pile solutions may be preferred. For rigid/propped walls, we recommend adopting at-rest (K_0) pressure coefficients provided in Table 6, above.

Other retention options such as gravity wall, soil nailing or cantilevered concrete pile wall options could be considered. The appropriateness of such systems will depend on the details of the area to be retained and performance, aesthetic and maintenance requirements.

The earth pressure envelopes shown in Figure 7 assume that effective drainage is provided at the base of, and behind the retaining walls. If this cannot be provided, allowance for hydrostatic pressure should also be included. Applicable temporary surcharges should be added to the soil pressures, using the values nominated, as appropriate to the permitted deformation condition.

The excavation contractor should undertake a risk assessment for buried services and take appropriate steps to mitigate adverse impacts as appropriate to the excavation geometry and support method adopted.

8.0 EARTHWORKS

Earthworks should be carried out in accordance with AS3798-2007, "*Guidelines on Earthworks for Commercial and Residential Developments*", the recommendations in this report and a site specific Earthworks Specification.

Based on our current understanding of performance requirements for warehouses and pavements, there will be a need to provide an Earthworks Platform to the underside of pavement/warehouse slabs and foundations. The need for a layer of engineered Structural Fill (ripped or crushed sandstone) below warehouse slabs and footings is discussed in Chapter 9.1 and 9.4.1.

There is an opportunity for re-use of site won material and this is discussed in Chapter 8.2.

Dependent on the final performance requirements adopted for detail design, it may also be possible to leave some or all of the relatively low organic content topsoil layer in place as discussed in Chapter 8.1.

8.1 Management of Existing Fill

The existing fill on the MPE Stage 2 site was found to be generally 0.5 m deep, but locally up to 1.25 m thick at the test pit locations, comprising silty sand, clayey sand and clay. It is possible that deeper fill, with poorer compaction, is present locally.

The history of placement of the existing fill is not known, and we do not know if it was placed as engineered fill in accordance with an engineering specification, even though the investigation data tends to indicate that it is typically at least moderately compacted. There is some uncertainty as to the how the fill might behave under the additional load of new fill plus floor or pavement loads, and whether adverse total and differential settlements could occur that would damage the floor slabs and pavements.

As the Proposal includes the placement of fill across the MPE Stage 2 site, a decision will need to be made about whether to remove and replace the existing fill prior to placing the new fill. There are several methods of verification and/or improvement of the existing fill that could be adopted alone or in combination to limit the risk of adverse settlements arising from leaving the existing fill in-situ:

1) Further geotechnical investigation to verify fill thicknesses, composition and compaction – test pits with Dynamic Cone Penetrometer tests and a combination of in situ density testing and laboratory compaction tests to establish the relative compaction of the fill.

2) Leave the existing fill in place and proof rolling with a 10 tonne conventional static roller prior to placement of new fill. If during proof rolling, excessive deflection was observed over large areas, more intensive soil improvement may be required, as discussed below. If localised areas of excessive deflection are observed, the existing fill in that area should be excavated and replaced in accordance with an engineering specification. A geotechnical engineer should be present during the proof-rolling. This is a relatively high risk option if used alone, and is not recommended unless done in association with additional testing (Item 1).

3) Excavation and replacement of some or all of the existing fill in accordance with an engineering specification. We would anticipate that the existing fill could be reused, with some sorting to remove unsuitable materials, and moisture conditioning so that the material is placed at or near to the optimum moisture content. This would be the lowest risk option from a sub-grade performance perspective but we appreciate that given the former usage of the site and the potential for Unexploded Ordnance (UXO) and Exploded Ordnance Waste (EOW), this option may not be favoured.

4) As an alternative to excavate and replace, and only if further geotechnical investigation indicates it is warranted, and compatible, soil improvement by preloading. This method would involve "pre-stressing" the soil prior to constructing floor slabs and pavements, by building a temporary fill layer above the final design surface that has a load equivalent to the design pavement and floor loadings. The limitation of

this method is that it would take weeks or months to complete. It would possibly require importation of additional fill, and staged improvement of different parts of the site.

5) As an alternative to excavate and replace, and if further geotechnical investigation indicates it is warranted, and compatible, soil improvement using a high energy impact roller (HEIC). HEIC has the benefit of being able to compact soils to greater thickness than conventional rollers. In our experience compaction to at least 1 m depth should be feasible. A variety of methods could be used to verify the effectiveness of the compaction, including Dynamic Cone Penetrometer, CPT, and geophysical methods.

Typically topsoil will need to be removed. However, it may be possible to leave some of the lower organic content sandy topsoil in place (once stripped of surficial vegetative matter). This would only be possible where a sufficiently thick Structural Fill Earthworks Platform can be provided above to the underside of warehouse slabs/footings or pavements. The required thickness of the Structural Fill Earthworks Platform would be dependent on detail design performance requirements for the overlying development but might potentially be 1.2m thick. Such an approach would require careful consideration and development of an appropriate methodology, likely incorporating HEIC in accordance with item 5 above, in order to sufficiently compact underlying strata and identify zones of poor material which may require special treatment or removal and replacement.

Development of the above options will require consideration of contamination issues and geotechnical issues, as the preferred geotechnical solution may not be possible due to contamination constraints.

8.2 Potential Fill Sources

8.2.1 Existing Fill

The existing fill encountered on the MPE Stage 2 site is a mixture of Unit 1 C and Unit 1D material. From a geotechnical perspective, the fill would be suitable for reuse as General Fill provided it is moisture conditioned and sorted to remove unsuitable, oversize, and/or and deleterious inclusions. Unsuitable materials that should not be used as General Fill include:

- Humic topsoil and primarily silt;
- Fill which contains wood, metal, plastic, boulders, ash, decaying vegetation and other deleterious substances; or
- Rock fragments or boulders greater than about 200 mm across (or more than ⅔ of the intended compacted layer thickness).

It is noted that topsoils across the MPE Stage 2 site have recorded test results for organic content in the range 0.1% to 1.2% with an average of 0.5%. Higher organic contents are at times accepted within general fill specifications, dependent on the particular application of the fill and the sensitivities of design to performance of the fill (e.g. decay and/or swelling of the organic matter and potential for development of roots within the fill mass). As an example for a material with 0.5% organic content which was formed in an embankment of 2m height, complete decay of the material might result in movement of 10mm. There is also the potential for swelling of the material and this may be a more significant issue for relatively thin fill layers.

Given the relatively low organic content (8 of the 22 samples tested recorded values of 0.2% or less), blending of some ‘topsoil’ material with other material to produce general fill may be possible. Where present, the surficial zone of material comprising the bulk of vegetative matter and any ‘root mat’ (as might be present within turfed areas), would need to be excluded and the blended product would need to be subject to lot based testing to confirm suitability prior to use.

Blending of the topsoil obtained from below the upper surficial vegetative layer with the overall fill strata may be acceptable. Care would need to be exercised in excising zones of high organic content material (such as may occur around established vegetated areas and tree zones) from such operations. This would require careful management by an experienced earthworks foreman, in conjunction with lot based acceptance testing in accordance with an Earthworks Specification. The Earthworks Specification should include acceptance criteria (e.g. allowable organic content) for the blended product.

8.2.2 Elevated Central Eastern Area

Investigations within the elevated eastern part of the MPE site known as “Moorebank Hill” indicate shallow rock from TP1205 (refusal at 1.25m below surface level) south to GA-EP-03 (Unit 4B encountered at 1.4m below ground surface). However north of TP1205 there is a relatively sharp drop off in rock head level as evidenced by GA-EP-01, which recorded alluvial material over its full depth to 5m below ground surface. Rock head also drops off relatively sharply to the south from GA-EP-03 towards GA-CPT-14 where a depth of penetration (indicative of soil) of approximately 14m was recorded.

Fifteen CBR tests (4 day soak), with 4.5kg surcharge were undertaken on the materials obtained from excavator pits carried out across Moorebank Hill. The results of the CBR tests, Standard Maximum Dry Densities (SMDD), Optimum Moisture Contents (OMC) and percentage swells are summarised in Table 8.

Table 8: Earthworks Test Results from Elevated Central Portion of the Site

Unit	Material	No. of Tests	CBR			SMDD (t/m^3)			OMC (%)			Swell (%)		
			Min	Max	Average	Min	Max	Average	Min	Max	Average	Min	Max	Average
Unit 1D[1]	Fill - Cohesive	2	5	16	10.5	1.5	1.6	1.57	21	28	24	0.4	1.8	1.1
Unit 3B	Older Alluvium - Clay	6	0.5	3	1.7	1.6	1.8	1.7	16	25	19	2.1	6.4	3.5
Unit 4A[1]	Residual Shale soil	1	0.5	0.5	N/A	1.5	1.5	N/A	27	27	N/A	2.4	2.4	N/A
Unit 4B	Extremely Low to Low Strength Shale	6	3.0	12	7.4	1.8	2	1.9	12	16	14	0	2.5	1.1

1. Note that these values should be viewed with particular caution due to the small size of the data set.

The existing fill, natural clays and weathered rock within the depth of the excavator pits can be used as General Fill provided they are compacted in accordance with the requirements of AS3798-2007 and project specific earthworks specifications are developed. However, these materials should not be used within 600 mm below the base of structures, such as footings or floor slabs because they are highly reactive and susceptible to shrink and swell with changes in moisture content. In addition, the suitability of the Unit 1D, 3 and 4A material as a pavement sub-grade would need careful consideration also due to the potential low CBR values and high swells. Consideration of further improvement, for example by stabilisation may be necessary. If these materials are to be utilised, careful consideration of appropriate drainage measures to manage exposure of these materials to groundwater and infiltration will be required due to their cohesive and potentially expansive nature.

Should the Moorebank Hill area be pursued as a source of fill materials to raise the level of the site, additional investigation to provide a sufficient data set of results (e.g. CBR tests, including 10 day soak and potential stabilisation trials) and delineation of the material boundaries (e.g. variation in rock head and extent of alluvium) should be undertaken. If greater use of the Unit 3 and Unit 4A is sought, a laboratory stabilisation trial could be carried out on samples with varying percentages of stabilising agent (e.g. lime and cement) subjected to 4 and 10 day soak CBRs and UCS testing, to assess the feasibility of stabilisation.

Emerson Crumb testing indicates the materials from the central elevated eastern portion of the site have some dispersion potential (Emerson Class Numbers of 4 and 5). As reported previously (Golder, 2015b) there are soils in the local area that have been eroded by surface run-off and observations of this type of material in excavations in the Glenfield Waste Facility indicate that similar clay fills are susceptible to erosion. To limit the potential for erosion, we recommend that where the fill is not concealed beneath structures or pavements, it should be protected from erosion by planting vegetation or alternative erosion protection measures. It is noted that stabilisation of the material would potentially improve both its performance as a sub-grade (i.e. CBR values) and susceptibility to scour.

8.2.3 Imported Fill

Imported fill may comprise a range of materials, including sand, gravel, crushed or ripped sandstone, crushed or ripped shale. Depending on the timing of construction on the site, large quantities of sandstone may be available from tunnelling projects in the Sydney metropolitan area.

Crushed sandstone and shale are typically used as fill materials in Sydney, as they are widely available. Usually the type of fill that is used depends on availability at the time of construction, and the constraints placed on fill types in the design. The geotechnical analyses presented in this report have been developed on the basis that a layer of imported crushed sandstone (Structural Fill) would be used to raise site levels.

Depending on the materials available at the time of construction, it may be worth considering using a specification that allows the potential of reusing recycled aggregates. These could either be sourced from demolition works in the Sydney area, or potentially from demolishing and processing the current construction materials on the former DNSDC site, from buildings, slabs and pavements. An earthworks specification for such materials is available to download from the following link:

http://www.environment.nsw.gov.au/resources/warr/104SupplyofRecycledMaterial.pdf.

Some older road pavement materials may need to be tested for the presence of coal tar prior to acceptance for reuse.

8.3 Proof Rolling and Compaction of Fill

New fill beneath structures (including pavements) should be compacted to be equivalent to a minimum Standard Maximum Dry Density (SMDD) of 98% (AS1289.5.1.1-2003) at a moisture ratio of 60 to 90 % of Standard Optimum Moisture Content (SOMC). The upper 600 mm below floor slabs of warehouses should be compacted to 100% SMDD and should be crushed sandstone or similar. This is to provide a suitable subgrade and drainage layer beneath for floor slabs and to support heavy equipment loads during construction and in operation.

Two methods of compaction that could be considered are:

- Conventional compaction in layers using a static or dynamic roller;
- Dynamic impact roller (high energy impact compaction) could be feasible given the size of the site. The use of this method would become more efficient the larger the area to be compacted.

Conventional compaction would follow the process described in AS3798-2007. After removal of vegetation, topsoil layer (where considered necessary) and treatment of the subgrade (as described in Chapter 8.1 above), new fill should be placed and compacted, with a maximum loose lift thickness of 300 mm, except the upper 600 mm below warehouse floor slabs, which should be 150 mm loose lift thickness. Over-excavation and replacement of existing fill and/or natural material may be required dependent upon final design levels (i.e. new fill thicknesses), new fill characteristics, performance requirements of the completed fill platform and nature of the existing fill/natural material within the area in question. Chapter 9.1 provides discussion relating to particular levels of performance relating to slabs and footings and associated material replacement requirements.

In proposed fill areas where the existing slopes are steeper than 1V:8H the fill should be keyed-in by excavating horizontal benches on which the fill should be placed.

Conventional compaction should be carried out in the full time presence of a Geotechnical Inspection and Testing Authority (GITA) in accordance with the requirements for Level 1 supervision described in AS3798-2007. AS3798-2007 also sets out the minimum requirements for field density and compaction control testing. The GITA should be appointed by the earthworks contractor and be responsible for carrying out the required testing. The GITA should be audited on a regular basis by the geotechnical design consultant.

Dynamic impact roller compaction, also known as High Energy Impact Compaction (HEIC), has the potential to achieve compaction of thicker layers than under conventional compaction. From our experience of HEIC, a compaction trial, completed prior to main site compaction works can help to select the most appropriate plant and compaction methodology for the site, as this will depend on factors that predominantly vary between sites. Generally, dynamic impact rolling is most effective in soils with low fines content (sandy soils), as the effective depth of the compaction is reduced where the fines content increases. The objective on this site would be to develop a methodology to compact fill thicknesses of say up to about 1 m, subject to verification of trial pads.

For efficiency, it may be possible to reduce the frequency of standard earthworks testing regimes, if augmented, by a combination of other testing methods, such as geophysical methods, CPT testing, plate loading tests or Falling Weight Deflectometer (FWD) testing. A compaction trial could be used to assess or correlate these methods and the most efficient layer thicknesses for placement of fill. Dynamic Impact Compaction should be carried out in the full time presence of the geotechnical design consultant responsible for the earthworks specification for the site.

Conventional compaction equipment (large vibratory smooth drum rollers) may be required to complete the final surface compaction below floor slabs to achieve level control and a uniform surface prior to pouring floor slabs.

8.4 Pavement Subgrade

Internal access roads on the site are to be designed by others. Pavement thickness will be heavily influenced by the number of truck movements experienced during the life of the pavement and its subgrade condition. To assist in assessment of various pavement configurations, we recommend that the following assumptions are used in assessment of preliminary pavement thickness designs. Further testing may be required as the design is developed.

- The CBR test results from alluvial clays indicate variable (0.5% to 3%) but low CBR values for this material, with an average of 2 % and a median value of 1.5 %, based on the results from four samples tested during the Stage 1 investigation, 2 sample tested as part of the Stage 2 pavement investigation and 5 samples tested from the elevated central eastern area. Consideration would need to be given to stabilisation of these materials where they form the subgrade for new pavements. This would improve settlement performance for sensitive structures, moisture stability of the subgrade and would improve pavement performance. It is noted that the alluvial soil CBR samples displayed significant swell on soaking (with a maximum of 5.5% and an average of 3%).
    - If pavements are to be constructed on alluvial clays, which are expansive, an effective subsurface drainage system will be required. The subgrade should also be graded in such a way to minimise ponding of water and to allow the water to migrate to the outer edge of the pavement where it can be removed by the subsurface drainage system. This subsurface drainage system should be constructed parallel and along the edge/s of the pavement
- The current site has been filled using a combination of granular fill (Unit 1C) and cohesive fill (Unit 1D). Based on laboratory testing of similar fill materials on adjacent sites and our experience of other sites, we consider that a design CBR value of 10% would be appropriate for Unit 1C. Based on available laboratory test results, the Unit 1D material records an average CBR value of 7%. Caution should be taken in adopting subgrade values for Unit 1D material based upon the limited investigation of this unit

undertaken to date. Unit 1D (and 1C material) is likely to be variable in lateral extent and thickness and may not be present in sufficient volumes of consistent characteristics to warrant adoption of average CBR values. The minimum CBR value recorded to date for the Unit 1D material is 3 %.

- Recommendations provided in Chapter 8.2 should be considered by the pavement designer and it would be necessary to validate the assumed CBR value via laboratory testing of the fill strata in the area of proposed pavement design and with appropriate consideration of likely pavement thicknesses and moisture ingress potential (i.e. to adopt appropriate CBR test surcharge weights and soak periods)
- Existing site derived materials identified during Stage 1 and Stage 2 investigation are all expected to be reusable as General Fill with processing in accordance with an Earthworks Specification. However, there may not be sufficient volumes to achieve a cut / fill balance. Selective use of better quality materials towards the top of the new formation may provide some benefit to the pavements. Further testing of these areas would be required to assess their extent and geotechnical and contamination suitability.
- The pavement should be finished with suitable cross-fall and adequate surface drainage to minimise ponding on the surface of the pavement.
- Designers should use laboratory testing results in the GDR and future investigations to assess appropriate design CBR values. The selection of design values should also take into account additional fill materials imported to raise ground levels to underside of pavement materials.

8.5 Bulking Factors

We suggest selecting values from Table 9 which are based on a combination of published values and experience with local materials.

Table 9: Suggested Bulking / Compaction Factors

Unit	Geological Origin	Predominant Material Type/ Rock Weathering Condition	Consistency / Density / Inferred Strength	Volumetric Bulking Factor[2] (*in situ* to truck)	Volumetric Compaction Factor[2] (*in situ* to re-compacted)
3	Tertiary Alluvium, Fill	Cohesive / granular	Mainly Very Stiff / dense	1.2-1.4	1.0-1.2
4A	Residual Soil	Mainly Cohesive/ fine grained	Stiff to Hard	1.2-1.4	1.0-1.2
4B and 4C	Mainly Siltstone/ laminite	Ext. to Highly Weathered	Extremely Low to Low Strength	1.3	1.1-1.2
		Mod. Weathered to Fresh	Mostly Medium to High Strength	1.3-1.4	1.1-1.2
5B and 5C	Sandstone	Ext. to Highly Weathered	Extremely Low to Low Strength	1.3	1.1-1.2
		Mod. Weathered to Fresh	Mostly Medium to High Strength	1.5	1.2-1.3

Notes:

1. Excluding landfill materials, for which bulking factor is uncertain due to intrinsic variability.

2. Based on estimated values published in McNally (1998)[1].

No bulking factor tests were carried out in materials sampled from site. The bulking factor is the ratio of in situ density of soil or rock against its dry density following excavation or compaction. A bulking factor of less than 1 implies that the insitu dry density of the material is less than the re-compacted material. This generally applies to soil materials as modern compaction plant often compacts soil to a density in excess of that at which it occurs in the natural state. Under this circumstance, we have referred to this as a "compaction" factor in Table 9.

A bulking factor of greater than 1 implies that the insitu dry density of the material is greater than the re-compacted material; which generally occurs for many rocks.

Given the lack of site specific data, we recommend that base case values in the mid-range of the above bulking / compaction factors are adopted along with sensitivity analyses within the range of suggested values above. When considering earthworks volumes, appropriate allowance should also be made for wastage due to unsuitable material, fill rejection, embankment overfilling and haul road construction.

8.6 Cut and Fill Batter Slopes

Excavations close to existing and new structures will need to be designed to control ground movements, and may require installation of a rigid shoring/retaining system, prior to excavation commencing.

Alternatively, where space allows, the excavation may be formed using battered side slopes, see Table 10, below, but these are only recommended in areas that do not have nearby movement sensitive structures or services.

Table 10: Recommended Unsupported Batter Slopes[1]

Unit	Material	Permanent Batter Slope	Temporary Batter Slope
Units 1, 3A	Fill and sandy Alluvial Soil	1(v):2(h)	1(v):1.5(h)
Unit 3B	Older Clayey Alluvium	1(v):2(h)	1(v):1(h)
Units 4 and 5	Shale and Sandstone	N/A	

Notes:

1. For excavations / slopes up to 3 m vertical depth/height.

Surcharge loads (including site traffic loads and spoil) should be kept away from the excavation crest (i.e. a distance equal to the depth excavation).

If slopes other than those recommended in Table 10 are to be used, or slopes deeper than 3 m are planned, then additional slope stability assessments should be completed. Limit-equilibrium analysis (using software program Slope/W or similar) could be used to assess the stability of the slope and the impact of any vehicles, plant or structures at the crest of the slope.

Steeper temporary and permanent batter slopes than 1V:1H may be achievable for excavation in rock and might be desirable to minimise hard rock excavation volumes. For assessment of steeper temporary batter slopes and assessment of appropriate permanent batter slopes in rock, specific geotechnical consideration of the following aspects will be required:

- The extent of the excavation
- The depth of the excavation

[1] McNally, G. "Soil and Rock Construction materials, CRC Press, PP187

- The location of the excavation with respect to:
  - Surface features (e.g. people and property)
  - Sub-surface features (e.g. buried services)
- The rock mass properties of the slopes (e.g. rock strength and defects)
- The potential erodibility of the excavated material
- Maintenance requirements
- Permanent batter slopes will need to be designed to adequately limit the risk posed by the excavation to people and property.

8.7 Structures for Stormwater Detention Ponds

Embankments or bunds for stormwater detention ponds, if required, could be constructed to form the detention areas using site–won or imported materials. We expect that ponds would need to be lined because the on-site soil materials that could be used as fill sources generally include granular seams/layers and have some dispersive potential. However, with appropriate design and detailing based on consideration of the characteristics of the particular material to be used and construction methodologies adopted (potentially including zoned construction) it may be possible to form detention ponds utilising site won material.

Geotechnical design of embankments for detention ponds would be required. The design would need to include recommendations on the maintenance and inspection requirements during operation. An assessment should be made upon the suitability of the design parameters of Table 6 above for use in design calculations (such as stability analysis), once the location, extent and details of the detail design for the detention ponds is available (including the particular materials to be used for the embankment construction). Such an assessment will need to be undertaken by a suitably qualified geotechnical engineer in consideration of the likely variability of foundation and construction materials and the potential need for additional investigation and testing.

8.8 Upgrade to Moorebank Avenue

The Proposal includes proposed upgrade works to Moorebank Avenue, comprising the raising of the vertical alignment by approximately 1.5 m and some local widening works for part of the road. The horizontal alignment of the road will remain the same, meaning that the existing road will form the foundation of the new road.

The construction of the new road would need to be completed using the requirements of RMS specification R44. The existing road should be an acceptable foundation for construction of the new road, the existing pavement materials may need to be removed to avoid water ponding on top of the existing road and to promote drainage of the new road construction. Once removed, these materials could potentially be recycled into the new road select or upper zone of formation (UZF) layers once processed.

Existing services and drains running along the road may need to be decommissioned and/or relocated as part of the works. Where widening of the new road extends its footprint over areas without existing carriageway, existing site investigation information is available to inform selection of design CBR values for pavement design.

Settlement of the proposed filling of the road should be similar in magnitude to those discussed in Section 10.0 of this report as ground conditions and fill height assumptions are similar.

Constructability and staging considerations will need to be taken into account during upgrading of the road, temporary retention and traffic management is likely to be required during the raising of the road.

9.0 STRUCTURAL FOOTINGS

9.1 Site Classification

Due to the variability in soils below the Proposal site, the site classification in accordance with AS2870 should be considered for each separate development area and additional testing at each lot may be required, depending on the final locations of structures. The geotechnical model for soils within the Proposal site is discussed in Chapter 5.0. The distribution of the soils beneath the Proposal site is shown on cross sections in APPENDIX B.

9.1.1 Granular Soils

In areas of the site where granular materials are present within the full 2 m below the final surface level, a site classification of Class S in accordance with AS2870 is considered appropriate. This assumes that a minimum of 600 mm of new granular Structural Fill comprising ripped or crushed sandstone is placed below structures to the surface of the granular material.

9.1.2 Cohesive Soils

AS2870 Table D2 indicates that Sydney sites underlain by clay soils greater than1.8 m thick should be classified as Class H1 or H2. However, re-classification is possible with additional analysis to quantify the shrink/swell movements based on site specific material properties obtained through laboratory testing.

Therefore, based on previous shrinkage index testing of samples conducted at the site (Golder, 2015b), a Class M classification is appropriate for the site provided that at least 1 m of fill below the warehouse floor slabs and foundations is granular Structural Fill comprising a ripped or crushed sandstone. This would need to be validated by additional Shrinkage index testing and/or correlation with more basic index tests such as Linear Shrinkage test and Atterberg Limits conducted at a sufficient frequency over the area of warehouse slabs and foundations.

Where lighter weight structures do not include a granular layer below floor slabs or foundations and they rest directly on natural cohesive soils, either additional testing should be completed at the site of the structure, or alternatively, they should be designed for a site classification of H1.

9.1.3 Uncontrolled Fill

Areas containing Unit 1 fill would be classified as Class P, requiring engineering measures such as ground improvement or foundations supported on underlying Unit 3 materials (to satisfy foundation performance requirements). Where foundations are supported on the underlying Unit 3 materials, consideration would need to be given to the character of the Unit 3 material in accordance with Chapters 9.1.1 and 9.1.2 to determine requirements. Depending on the selected engineering option chosen in these areas, they could potentially be reclassified if movements from engineered fill and underlying soils in response to long term equilibrium moisture conditions are assessed.

9.2 Design Loading

The following design loading assumptions have been adopted:

- Floor loads of warehouses, 40 kPa;
- Pad or strip footing loads, 150 kPa; and
- Ground levels to be raised approximately 1.5 m above existing levels over the western portion of the site.

9.3 Lightweight Structures

It should be possible to found lightly loaded structures (i.e. single storey office buildings, small storage buildings, gatehouses etc.) that are not settlement sensitive on either piers or strip footings embedded in new General or Structural Fill layers or directly on natural Unit 3A or 3B soils, if they can be designed to achieve bearing pressures of less than 100 kPa and accommodate anticipated surface movements in response to changes in soil moisture content in accordance with Chapter 9.1 above. We do not recommend supporting footings in the existing fill because of the risk of unacceptable total and differential settlements, unless the fill has been treated as described in Chapter 8.0, over the depth of influence below footings, so that it can be considered “engineered fill”. Again, anticipated surface movements in response to changes in soil moisture content must be able to be accommodated by the structure. Footings should have a minimum embedment depth of 500 mm below finished ground level.

The base of footing excavations should be dry and free of debris and loosened soil. Concrete for shallow footings should be poured within 24 hours of excavating the footing.

9.4 Warehouse Foundations

Although, the specifics of the proposed designs are not yet available, however for reference, column loads from warehouses are typically relatively high, depending on the chosen arrangement of columns within the structures. Based on our experience of design of similar sized structures, columns loads can range between 1,250 kN to 7,500 kN or higher. The viability of shallow footings to support columns will need to be assessed during interaction between geotechnical and structural designers.

9.4.1 Option 1 - Shallow Footings

For column loads at the lower end of the above range, it should be possible to found warehouse column footings at shallow depth in Structural Fill, provided that there is an adequate thickness of Structural Fill beneath the base of footings, and provided the footing can be economically dimensioned to achieve bearing pressures no greater than 150 kPa. Where a footing has a width of “B”, there must be a thickness of Structural fill (or in-situ Unit 3A or 3B materials) of at least 1.3 times B below the base of the footing. Depending on proposed levels, this may require some excavation of existing fill materials.

Alternatively, warehouse footings could be founded directly on Unit 3 soils, following moisture conditioning and re-compaction of the uppermost 300mm below footings, and designed for allowable bearing pressures of less than 150 kPa, although this would likely result in footing excavations at least 2 m deep to penetrate through new fill and the existing fill.

All footings should have a minimum embedment depth of 500 mm below finished ground level.

If serviceability or stability considerations cannot be met with shallow foundations, then some columns may need to be founded on piles.

9.4.2 Option 2 - Piled Foundations

Where it is not feasible to support column loads on shallow footings, piles will be required. In selecting piles for the proposed site, the view of piling contractors should be sought, as this will be useful in identifying the most appropriate system. In general, we consider that bored piles may be most appropriate for use on the site, given the potential for driven piles to refuse on iron cemented bands within the Unit 3 soils and within Unit 1 fill. It is noted that a number of CPT tests refused at relatively shallow depths which might be indicative of iron cemented layers within the Unit 3 soils.

If bored piles extend below the groundwater table they will likely need to be cased to prevent groundwater inflow and maintain wall stability prior to concreting. This will typically affect piles of lengths greater than 5m (groundwater was typically at about 5 m to 7 m depth based on CPTs). However shallower piles may potentially be affected over the ‘upslope’ eastern portion of the Stage 2 and the southern corner towards Anzac Creek (e.g.in the vicinity of GA-CPT-14 which recorded groundwater at 1.5m depth below surface level).

Table 6 above provides parameters for deep foundation (i.e. piles) within both Unit 3A and Unit 3B. However, based upon the current results of investigation it is likely that given the relatively thin nature of Unit 3A seams and bands within the older Alluvium, foundations would be taken to bear on Unit 3B material with their shaft adhesion dominated by the characteristic of Unit 3B strata. Notwithstanding this there will be granular layers present within the Unit 3 material and this is a particular consideration for piling methodologies in that where deep foundations are proposed below the water table, inflows may be locally high and slumping/collapse of pile walls likely if bored piles are attempted without adequate lining. Should piled foundations within the Unit 3 material be adopted in detail design development, it will be appropriate to carry out additional investigation in the immediate vicinity of the proposed piled foundations.

Continuous flight auger (CFA) piles may be an appropriate choice, with either single piles or pile groups used, depending on the magnitude of column loads. CFA would avoid some of the issues that bored piles could face if high groundwater flows are experienced. Depending on column loads, floating piles founded in Unit 3 may be possible. Alternatively piles would be able to be advanced to extremely weathered rock, but would likely refuse on rock of medium strength or higher. To accommodate higher column loads a pile group in soil could be used, but this would need to take into account potential reduced capacity, due to group effects.

Another option, depending on the magnitude of column loads required, would be to have a single larger diameter bored pile socketed into rock of medium strength or greater. As an initial example of potential capacity, a 1,000 mm diameter bored pile could have a working (serviceability) load of up to about 6 MN.

Consideration could be given to procuring piling works through a design and construct delivery model, as this often gives piling contractors the ability to innovate as well as take on a higher degree of risk for the installation and performance of the piles.

9.4.2.1 Pile Type Selection

In selecting suitable pile types for the site there are a range of advantages and disadvantages that need to be considered. We have summarised some of the site specific considerations in Table 11.

Table 11: Advantages and Disadvantages of Different Pile Types

Driven Piles (precast concrete)

Advantage	Disadvantage
Ability to make visual observation of pile quality prior to installation	Risk of refusal on iron cemented bands
Cheap and readily available	Noise and vibration
	May adversely impact adjacent piles when driven in groups

Driven Piles (steel I-section)

Advantage	Disadvantage
Greater penetration of cemented layers possible	Smaller section area for cost, compared with precast or bored piles
Driving equipment readily available	Noise and vibration during installation

Bored Piles (CFA)

Advantage	Disadvantage
Greater penetration of cemented layers possible, depending on type of equipment	Cannot penetrate far into high strength rock, so lower capacities than bored pile.
No casing, dewatering or cleaning required	Expensive relative to driven piles

Bored Piles (Open bored)

Advantage	Disadvantage
Penetration of cemented layers possible	Temporary liners, dewatering and cleaning required
Potential to construct sockets into bedrock for higher pile capacity	Expensive relative to driven piles
Higher capacity piles available by socketing into rock	

9.4.2.2 Pile Load Tests

Pile load testing should be completed in accordance with the recommendations in AS2159. Detailed pile design should take into account the type and quantity of pile testing in assessing the available pile capacity. In general terms a higher cost pile testing methodology can result in reduced pile lengths, the cost / benefit balance would need to be assessed during the design process.

9.4.2.3 Supervision of Bored Pile Construction

Prior to concreting, all piles on rock should be inspected by a geotechnical engineer or engineering geologist to assess the exposed rock at toe level.

10.0 SETTLEMENT ASSESSMENT

Our preliminary estimates of settlement under slabs of the large scale industrial warehouses are in the range 20 mm to 40 mm. Differential settlements across similarly loaded areas are expected to be about half of the total settlements. Our calculations were based on one dimensional loading conditions, with an anticipated loading of 70 kPa over existing soils, comprising a slab loading of 40 kPa and 30 kPa to allow for an average filling height of approximately 1.5 m. The calculations were based on the results of the investigations undertaken across the MPE site. The soil stiffness assumptions used in these calculations are generally as shown in Table 6.

The magnitude of the calculated settlements is within the typical tolerance limits for industrial structures.

During detailed design of structures, additional considerations will need to be made, including:

- Checks by structural engineer to assess the compatibility of predicted movements with the sensitivity and tolerance of each proposed structure, super-imposing any expected long term settlements and shrink/swell movements, as appropriate.
- Undertaking investigations for specific structures to confirm foundation compressibility. This would need to consider the consistency (and variability) of foundation materials over the depth of influence associated with any given strip / slab loading. The study would also need to delineate uncontrolled fill areas within the vicinity of structures.

11.0 GEOTECHNICAL MANAGEMENT MEASURES

- Undertake further targeted detailed investigations (in addition to those already completed on the MPE site during the Proposal and the MPE Stage 1 Proposal) based on the final developed detailed design in order to:
  - determine the extent and composition of topsoil and project specific criteria for its use in earthworks materials
  - characterise and delineate key strata intended to be utilised as General Fill or founding strata to structures and embankments. This testing should focus primarily on the Unit 3 and Unit 4 underlying elevated area of "Moorebank Hill".
  - verify thicknesses, composition and compaction of existing fill to be able to inform methods on treatment of the existing fill. This may include test pits with Dynamic Cone Penetrometer (DCP) tests and a combination of in-situ density testing, laboratory compaction and CBR tests to establish the relative compaction and likely performance characteristics of the existing fill.
  - assess the rate of liming or cement stabilisation that may be required for subgrade treatment and the impact of stabilisation on CBR values.
- Where practical and feasible, consideration to settlement reducing piles, rather than conventional piles, during detailed design in order to reduce overall pile length across the site.
- Develop an earthworks specification, during detailed design, which defines appropriate project specific criteria for the use of existing fill material, imported fill, existing topsoil and other geotechnical materials to be used during the construction of the Proposal.

12.0 REFERENCES

- Golder Associates (2016a). SIMTA Intermodal Terminal, Stage 2 (DNSDC) Geotechnical Data Report. Report Reference 1416224_022_R_Rev 0, dated 08 April 2016.
- Golder Associates (2016b). SIMTA Stage 2 – Pavement Investigation Results. Memorandum Reference 1416224_024_Rev 0
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- IPWEA (2010). Specification for Supply of Recycled Material for Pavements, Earthworks and Drainage http://www.environment.nsw.gov.au/resources/warr/104SupplyofRecycledMaterial.pdf. accessed 11/12/2014.
- NSW Department of Minerals (1991). 1:100 000 Geological Series Sheet for Penrith (1st edition) – Sheet 9030.
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- Standards Australia (2007b). AS3798 Guidelines on earthworks for commercial and residential developments.
- Standards Australia (2009). AS2159 Piling – Design and installation.
- Standards Australia (2011). AS2870 Residential slabs and footings.

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Report Signature Page

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NS/JDM/nrs

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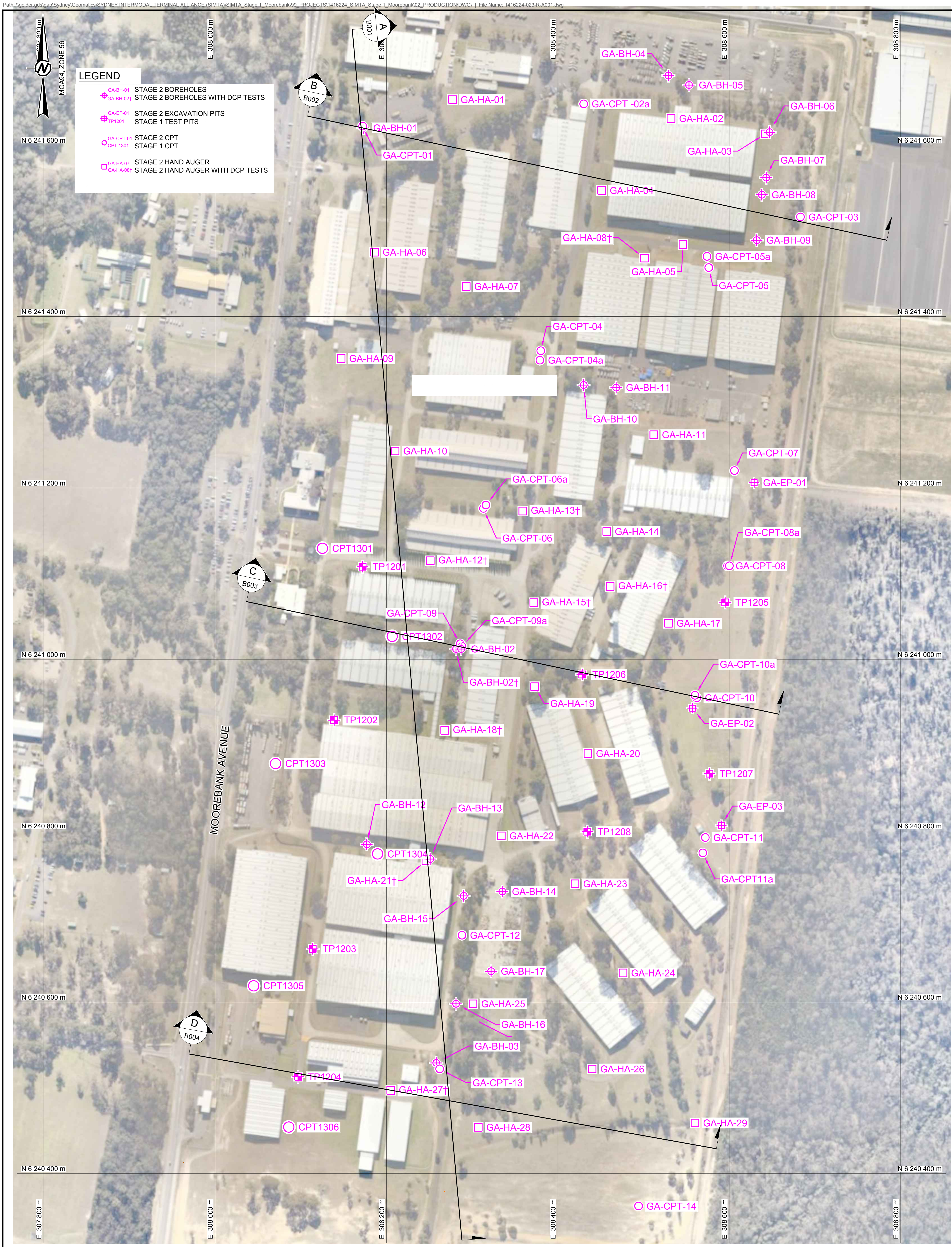
APPENDIX A

Geotechnical Site Investigation Plans

A001	GEOTECHNICAL SITE INVESTIGATION PLAN: SHEET 1 OF 1
A002	INFERRED TOPSOIL CONTOURS: SHEET 1 OF 1

Path: \\golder.gds\gap\Sydney\Geomatics\SYDNEY INTERMODAL TERMINAL ALLIANCE (SIMTA)\SIMTA_Stage_1_Moorebank\99_PROJECTS\1416224_SIMTA_Stage 1_Moorebank\02_PRODUCTION\DWG\ | File Name: 1416224-023-R-A001.dwg



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1:4,000 METRES

IMAGE DATED 2014-06-30
SOURCED WITH PERMISSION FROM NEARMAP ON 2014-09-30
IMAGE GEOREFERENCED BY GOLDER AND INTENDED FOR INDICATIVE PURPOSES ONLY
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nearmap

CLIENT	TACTICAL GROUP	PROJECT	MPE STAGE 2 MOOREBANK, NSW
CONSULTANT	Golder Associates	TITLE	GEOTECHNICAL SITE INVESTIGATION LOCATION PLAN
YYYY-MM-DD	2016-07-28		
PREPARED	EJJ		
DESIGN	MJM		
REVIEW	NRS		
APPROVED	BJF		

PROJECT No.	REPORT	Rev.	FIGURE
1416224	023-R	1	A001

308000.000000 308500.000000 309000.000000

6241500.000000 6241000.000000 6240500.000000

308000.000000 308500.000000 309000.000000

LEGEND

Topsoil thickness (m)

- 0.1
- 0.2
- 0.3
- 0.4
- 0.5
- + Topsoil thickness observed

NOTES

1. The Approximate Site Boundary represents the spatial extent of the Golder investigation area on this project.

REFERENCE

1. Aerial Photography Copyright NearMap Pty Ltd.

REFERENCE SCALE: 1:8,000 (at A4)
PROJECTION: GDA 1994 MGA Zone 56

CLIENT
TACTICAL GROUP (FOR SIMTA)

PROJECT
MPE STAGE 2

TITLE
MPE STAGE 2 INVESTIGATION INFERRED TOPSOIL CONTOURS

CONSULTANT

Golder Associates

YYYY-MM-DD	2016-07-29
PREPARED	KJS
DESIGN	-
REVIEW	MM
APPROVED	NS

PROJECT	DOCUMENT	Rev.	FIGURE
1416224	023	1	A002

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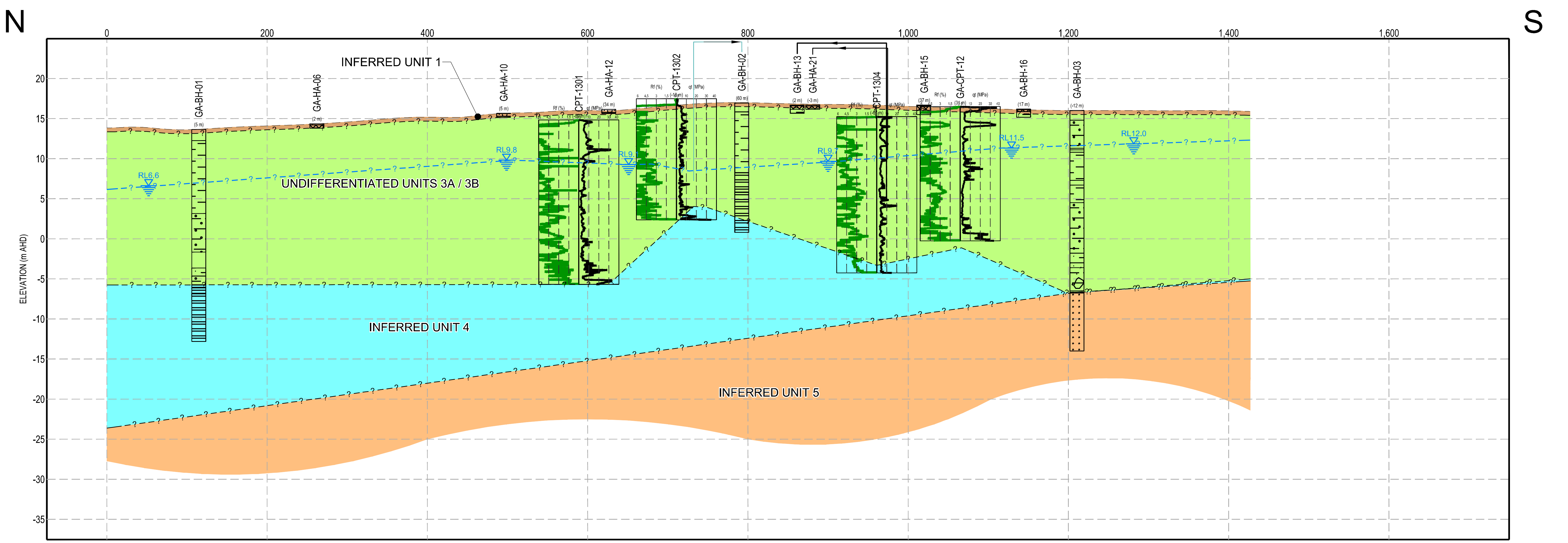
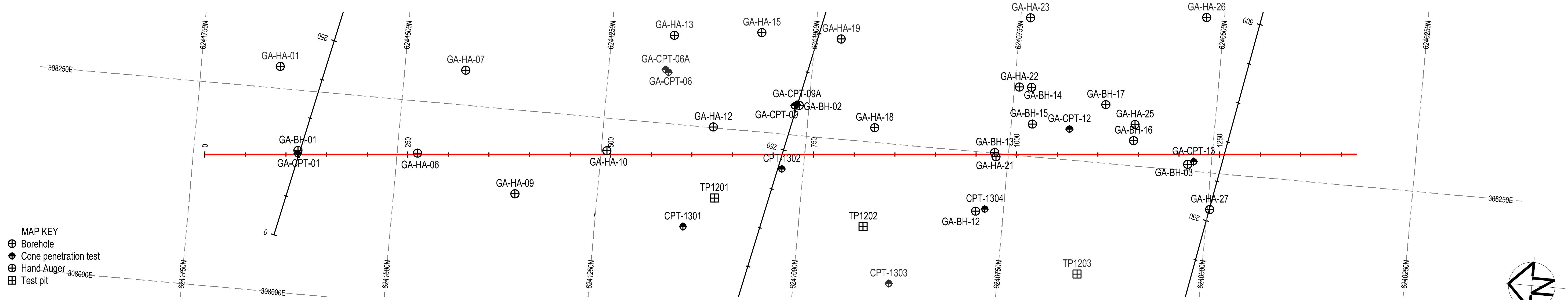
APPENDIX B

Geotechnical Cross Sections

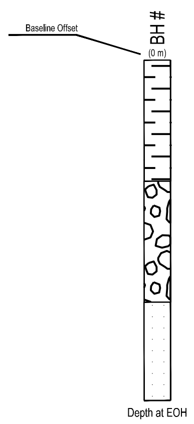
B001	SECTION A – STAGE 2 SITE – NORTH SOUTH SECTION 1
B002	SECTION B – STAGE 2 SITE – EAST WEST SECTION 1
B003	SECTION C – STAGE 2 SITE – EAST WEST SECTION 2
B004	SECTION D – STAGE 2 SITE – EAST WEST SECTION 3





MAP KEY
- Borehole
- Cone penetration test
- Hand Auger
- Test pit

POST LEGEND



SHALE
CLAY
Gravelly CLAY
Sandy CLAY
Sandy Gravelly CLAY
TOPSOIL
CORE LOSS
FILL
Clayey SAND
Clayey Gravelly SAND
Clayey Silty SAND
SANDSTONE

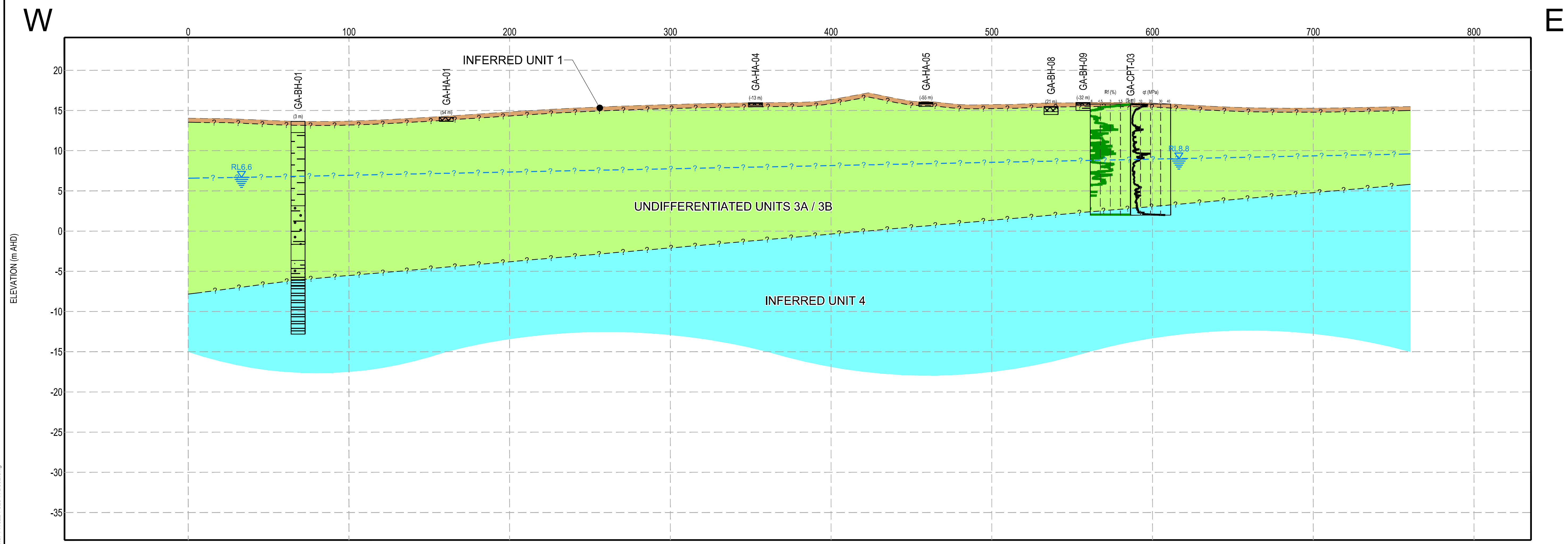
WATER LEVEL
WATER INFLOW

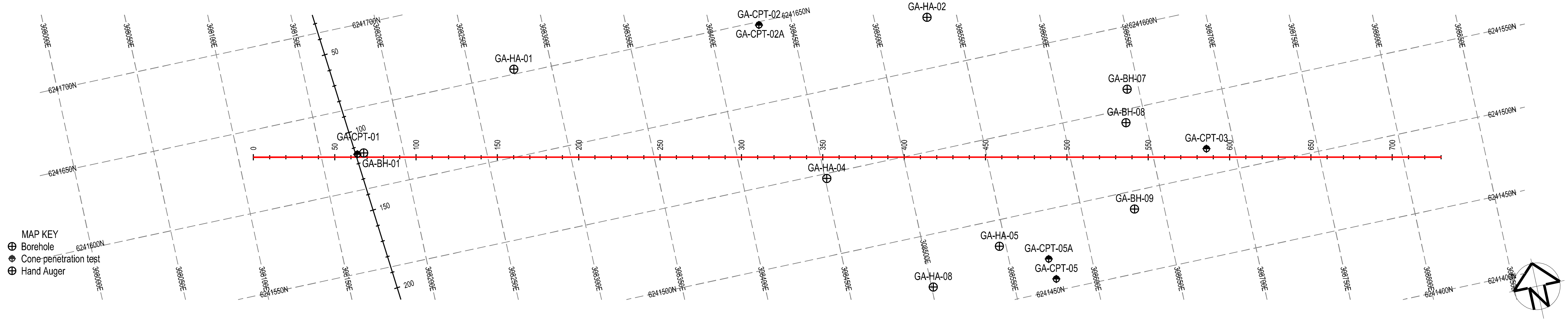
GEOTECHNICAL UNITS
- UNIT 1 - FILL
- UNIT 2 - ALLUVIAL SAND AND CLAY
- UNIT 3 - ALLUVIAL SAND AND CLAY
- UNIT 4 - SILTSTONE
- UNIT 5 - SANDSTONE

H 1:5000
0 60 120 180 240 300m
V 1:500
0 5 10 15 20 25m
Coord.System: 31000 Zone 56 Height Datum: 31000

CLIENT	TACTICAL GROUP	PROJECT	MPE STAGE 2 MOOREBANK, NSW
CONSULTANT	Golder Associates		
YYYY-MM-DD	2016-07-28	TITLE	SECTION A - STAGE 2 SITE - NORTH SOUTH SECTION
PREPARED	EJJ		
DESIGN	MJM		
REVIEW	NRS		
APPROVED	BJF		

PROJECT No.	REPORT	Rev.	FIGURE
1416224	023-R	1	B001





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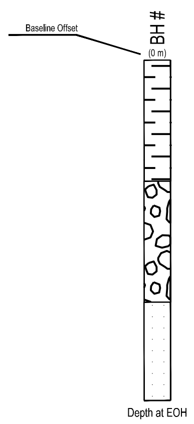

MATERIAL GRAPHIC

- CLAY
- Sandy CLAY
- Clayey SAND
- SHALE
- CORE LOSS
- TOPSOIL
- FILL
- WATER LEVEL
- WATER INFLOW

GEOTECHNICAL UNITS

- UNIT 1 - FILL
- UNIT 2 - ALLUVIAL SAND AND CLAY
- UNIT 3 - ALLUVIAL SAND AND CLAY
- UNIT 4 - SILTSTONE
- UNIT 5 - SANDSTONE

H 1:2500

0 20 40 60 80 100m

V 1:500

0 5 10 15 20 25m

Coord.System: 31000 Zone 56 Height Datum: 31000

CLIENT
TACTICAL GROUP

PROJECT
MPE STAGE 2
MOOREBANK, NSW

CONSULTANT

Golder Associates

YYYY-MM-DD	2016-07-28
PREPARED	EJJ
DESIGN	MJM
REVIEW	NRS
APPROVED	BJF

TITLE
SECTION B - STAGE 2 SITE - EAST WEST SECTION 1

PROJECT No.	REPORT	Rev.	FIGURE
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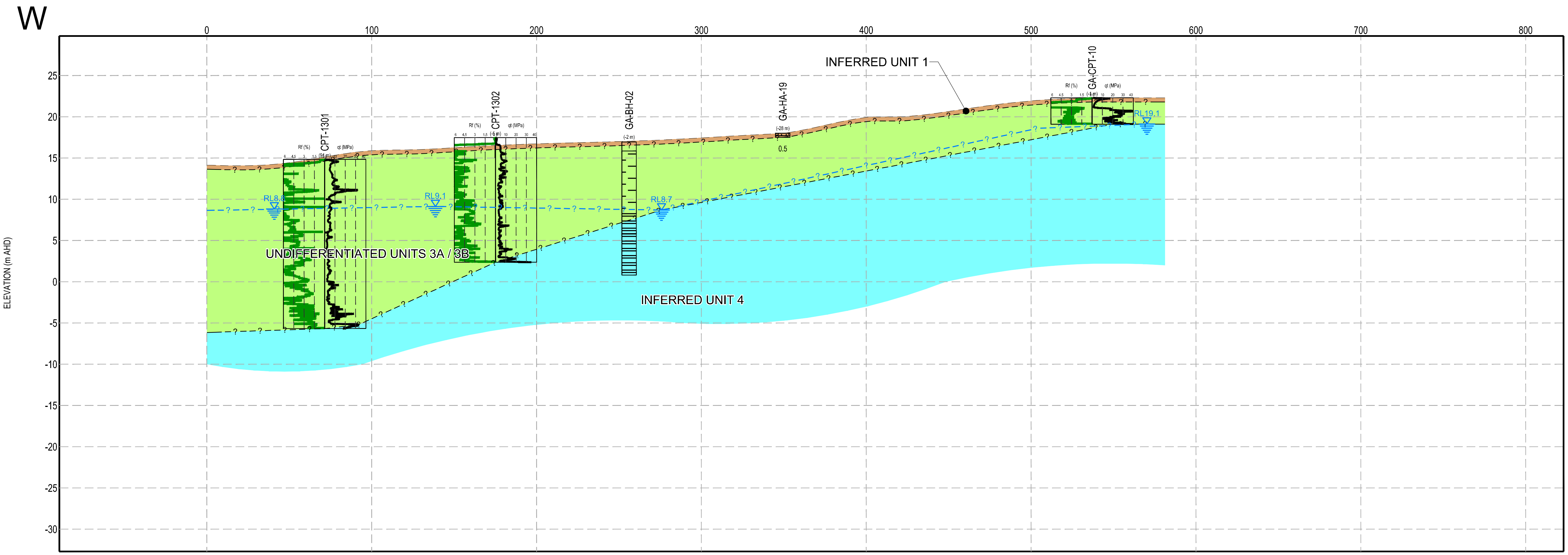

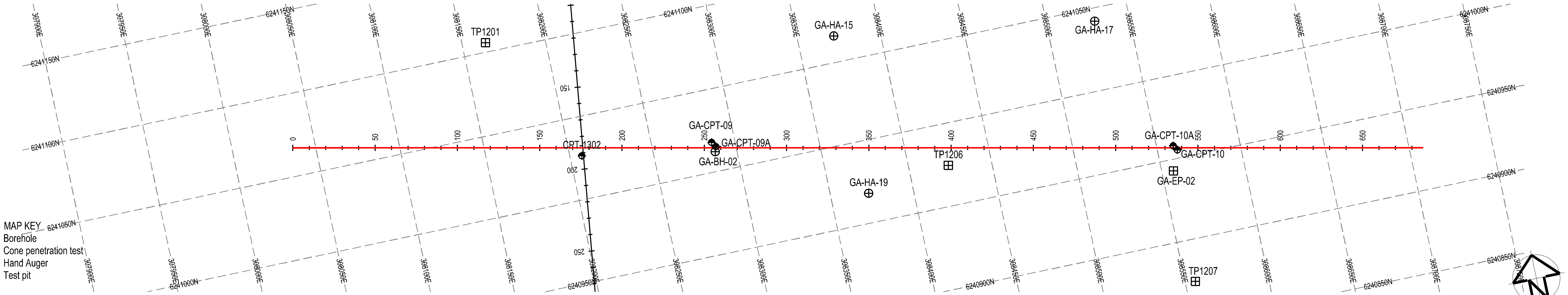

MAP KEY
- Borehole
- Cone penetration test
- Hand Auger
- Test pit

POST LEGEND

Baseline Offset
BH #
Depth at EOH

- CLAY
- SHALE
- CORE LOSS
- TOPSOIL
- FILL
- Silty CLAY
- WATER LEVEL
- WATER INFLOW

GEOTECHNICAL UNITS

- UNIT 1 - FILL
- UNIT 2 - ALLUVIAL SAND AND CLAY
- UNIT 3 - ALLUVIAL SAND AND CLAY
- UNIT 4 - SILTSTONE
- UNIT 5 - SANDSTONE

H 1:2500
0 20 40 60 80 100m
V 1:500
0 5 10 15 20 25m
Coord.System: 31000 Zone 56 Height Datum: 31000

CLIENT
TACTICAL GROUP

CONSULTANT
Golder Associates

YYYY-MM-DD	2016-07-28
PREPARED	EJJ
DESIGN	MJM
REVIEW	NRS
APPROVED	BJF

PROJECT
MPE STAGE 2
MOOREBANK, NSW

TITLE
SECTION C - STAGE 2 SITE - EAST WEST SECTION 2

PROJECT No.	REPORT	Rev.	FIGURE
1416224	023-R	1	B003

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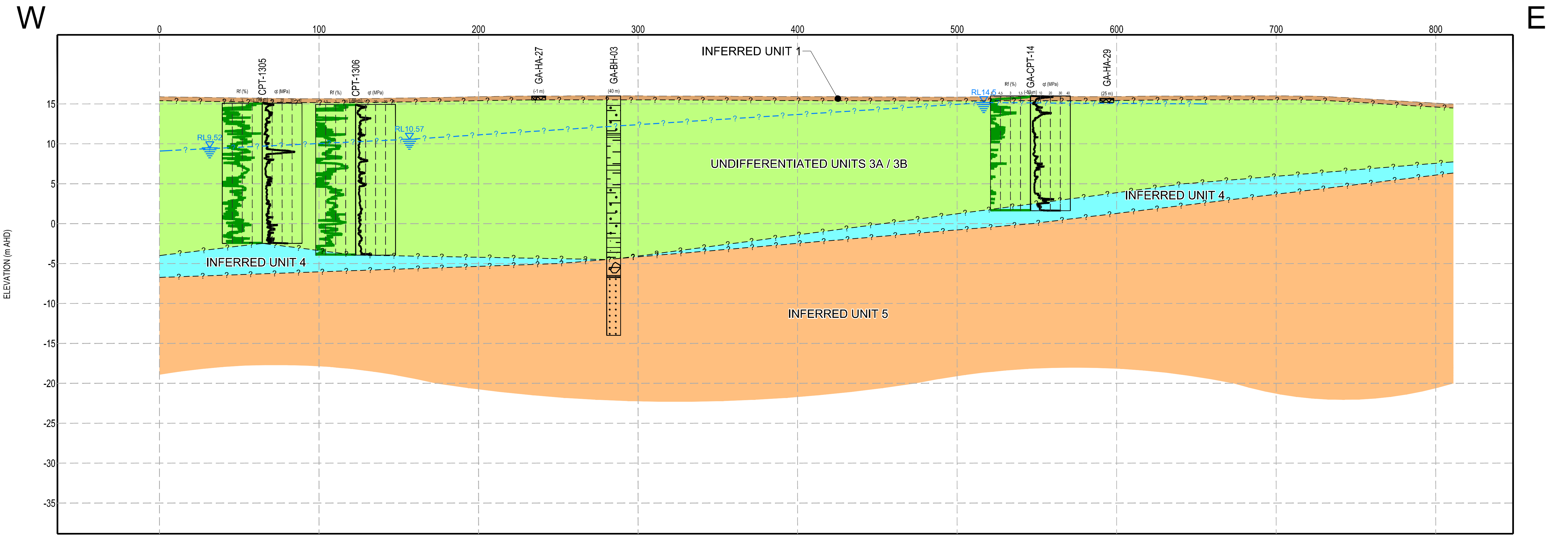
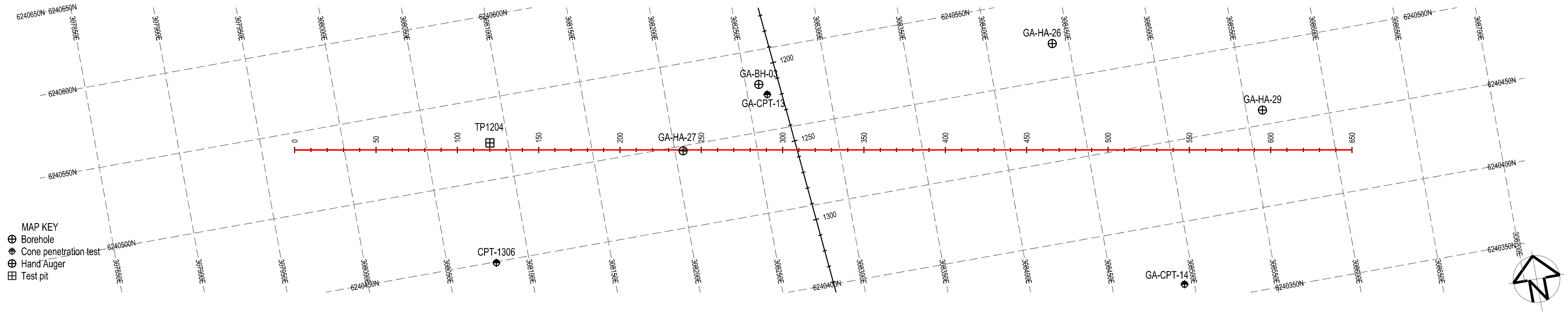
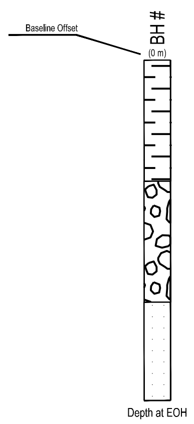
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A3





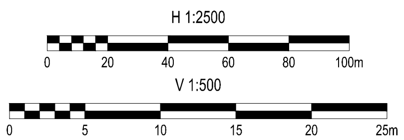
POST LEGEND



Sandy CLAY
CLAY
Gravelly CLAY
Clayey SAND
Clayey Gravelly SAND
Sandy Gravelly CLAY
SANDSTONE
CORE LOSS
TOPSOIL
FILL
Silty CLAY
WATER LEVEL
WATER INFLOW

GEOTECHNICAL UNITS

UNIT 1 - FILL
UNIT 2 - ALLUVIAL SAND AND CLAY
UNIT 3 - ALLUVIAL SAND AND CLAY
UNIT 4 - SILTSTONE
UNIT 5 - SANDSTONE



Coord.System: 31000 Zone 56 Height Datum: 31000

CLIENT
TACTICAL GROUP

CONSULTANT
Golder Associates

YYYY-MM-DD	2016-07-28
PREPARED	EJJ
DESIGN	MJM
REVIEW	NRS
APPROVED	BJF

PROJECT
MPE STAGE 2
MOOREBANK, NSW

TITLE
SECTION D - STAGE 2 SITE - EAST WEST SECTION 3

PROJECT No.	REPORT	Rev.	FIGURE
1416224	023-R	1	B004

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