Moorebank Precinct West (MPW) – Stage 2 Proposal

Geotechnical Interpretive Report

SIMTA

SYDNEY INTERMODAL TERMINAL ALLIANCE

Part 4, Division 4.1, State Significant Development

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MOOREBANK PRECINCT WEST (MPW) - STAGE 2

Geotechnical Interpretive Report

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Executive Summary

This report provides geotechnical advice for the Moorebank Precinct West (MPW) Stage 2 Proposal of the Moorebank Precinct (MP), which involves redevelopment of approximately 220 hectares of land for an intermodal terminal, associated infrastructure facilities, and warehousing. The results of a geotechnical investigation, completed by Golder in 2014 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. This report will provide the basis of the submission for the MPW Stage 2 State Significant Development (SSD) application.

The geology of the site generally comprises a thin layer of fill material at the existing ground surface, generally 0.5 m thick (but up to 2 m or more locally) over alluvium comprising stiff to very stiff clays or dense to very dense sands. The site is generally underlain by shale bedrock at depths between 5 m to 24 m below existing ground level. Sandstone rock was located in some boreholes at the southern end of the site. Groundwater is expected between about 9 m to 12 m depth below the existing ground surface, however perched water is likely to be encountered at higher levels in the vicinity of established ponds and Anzac Creek.

Most aspects of the development will involve relatively routine geotechnical design and construction procedures. An aspect that will require particular attention is the treatment of the existing fill (including the topsoil layer, where present), which is of variable compaction and composition. In the report we present alternative techniques for managing the risks associated with the potential for adverse settlement of the fill under the weight of new fill, pavements, floor slabs and structural footings.

Excavations are expected to be relatively shallow and typically above the water table. 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 excavations, with a provision for breaking-out occasional ironstone layers.

Fill materials will need to be imported onto the site to raise site levels, particularly over the lower lying western portion of the site, in close proximity to Georges River. Importation of good quality fill (e.g. VENM ripped rock, such as sandstone tunnel excavation spoil) will present less risk than the reuse of existing site fill. This is due to portions of the existing fill being contaminated and/or having been placed as uncontrolled fill of potentially variable and deleterious composition. Notwithstanding this, existing fill on site may be suitable for reuse as general fill provided it is treated to remove unsuitable materials and re-compacted to meet the requirements of AS3798. There are areas of existing fill within the site that will need contamination remediation in accordance with the Remediation Action Plan (RAP).

In the report we have provided indicative pavement designs based on different assumed subgrade conditions. A fully flexible pavement with thin asphalt surfacing (non-structural wearing course) and granular base and sub-base is expected to have lower capital cost but higher maintenance costs for the wearing surface. A thick asphaltic concrete pavement with cement stabilised base and granular sub-base typically has higher capital cost but lower maintenance costs. Indicative pavement thicknesses based on a 20 years design life are 450 millimetres (mm) (flexible) and 590 mm (rigid) with an assumed subgrade CBR of 10% and 750 mm (flexible) to 1,050 mm (rigid) for a subgrade CBR of 3%.

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 5 mm to 35 mm, which is within the typical tolerance limits for industrial structures.

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1.0 INTRODUCTION

1.1 The Project

On the 3 June 2016 Concept Plan Approval (SSD 5066) was granted, under Part 4, Division 4.1 of the *Environmental Planning and Assessment Act 1979* (EP&A Act), to develop the Moorebank Precinct West Project (MPW Project) on the western side of Moorebank Avenue, Moorebank, in south-western Sydney (the MPW site).

The MPW Project involves the development of intermodal freight terminal facilities (IMT), linked to Port Botany, the interstate and intrastate freight rail network. The MPW Project includes associated commercial infrastructure (i.e. warehousing), a rail link connecting the MPW site to the Southern Sydney Freight Line (SSFL), and a road entry and exit point from Moorebank Avenue.

Under the Concept Plan Approval, the MPW Project is to be developed in four phases, being:

1) Early Works development phase, comprising:

   - The demolition of existing buildings and structures
   - Service utility terminations and diversion/relocation
   - Removal of existing hardstand/roads/pavements and infrastructure associated with existing buildings
   - Rehabilitation of the excavation/earthmoving training area (i.e. 'dust bowl')
   - Remediation of contaminated land and hotspots, including areas known to contain asbestos, and the removal of:
     - Underground storage tanks (USTs)
     - Unexploded ordnance (UXO) and explosive ordnance waste (EOW) if found
     - Asbestos contaminated buildings
   - Archaeological salvage of Aboriginal and European sites
   - Establishment of a conservation area along the Georges River
   - Establishment of construction facilities (which may include a construction laydown area, site offices, hygiene units, kitchen facilities, wheel wash and staff parking) and access, including site security
   - Vegetation removal, including the relocation of hollow-bearing trees, as required for remediation and demolition purposes

2) Development of the intermodal terminal (IMT) facility and initial warehousing facilities

3) 'Ramp up' of the IMT capacity and warehousing

4) Development of further warehousing.

Approval for the Early Works phase (MPW Concept Plan Approval) was granted as the first stage of the MPW Project within the Concept Plan Approval. Works, approved as part of this stage are anticipated to commence in the third quarter of 2016.

Commonwealth Approval (No. 2011/6086), under the *Environmental Protection Biodiversity Conservation Act 1999* (EPBC Act), was also granted in mid-2016 (soon after the Concept Plan Approval) for the MPW Project. In addition to this, the Planning Proposal (PP_2012_LPOOL_004_00) which provided a rezoning of part of the MPW site, and surrounds, was gazetted on 24 June 2016 into the *Liverpool Local Environmental Plan 2008* (Amendment No. 62).

On 5 December 2014, Moorebank Intermodal Terminal Company (MIC) and SIMTA announced their in-principle agreement to develop the Moorebank IMT Precinct on a whole of precinct basis. This agreement is subject to satisfying several conditions which both parties are currently working towards. SIMTA is therefore seeking approval to build and operate the IMT facility and warehousing under the MPW Project Concept Approval, known as the MPW Stage 2 Proposal (the Proposal).

Key terms used within this report are defined in Table 1 below. Table 1 includes Key Terms as applied in the Environmental Impact Statement (EIS) for the MPW Stage 2 Proposal.

Table 1: Key Terms Table

Moorebank Precinct West (MPW) Concept Plan Approval (Concept approval and Early Works)	MPW Concept Plan and Stage 1 Approval (SSD 5066) granted on 3 June 2016 for the development of the MPW Intermodal terminal facility at Moorebank and the undertaking of the Early Works. Granted under Part 4, Division 4.1 of the *Environmental Planning and Assessment Act 1979*. This reference also includes associated Conditions of Approval and Revised Environmental Management Measures, which form part of the documentation for the approval. N.B. Previously the MIC Concept Plan Approval
Moorebank Precinct West (MPW) Planning Proposal	Planning Proposal (PP_2012_LPOOL_004_00) to rezone the MPW site from 'SP2- Defence to 'IN1- Light Industrial' and 'E3- Management', as part of an amendment to the *Liverpool Local Environmental Plan 2008* (as amended) gazetted on 24 June 2016.
Moorebank Precinct West (MPW) Project	The MPW Intermodal Terminal Facility as approved under the MPW Concept Plan Approval (5066) and the MPW EPBC Approval (2011/6086). N.B. Previously the MIC Project
Moorebank Precinct West (MPW) site	The site which is the subject of the MPW Concept Plan Approval, MPW EPBC Proposal and MPW Planning Proposal (comprising Lot 1 DP1197707 and Lots 100, 101 DP1049508 and Lot 2 DP 1197707). The MPW site does not include the rail link as referenced in the MPW Concept Plan Approval or MPE Concept Plan Approval. N.B. Previously the MIC site.
Early Works	Works approved under Stage 1 of the MPW Concept Plan Approval (SSD 5066), within the MPW site, including: establishment of construction compounds, building demolition, remediation, heritage impact mitigation works and establishment of the conservation area.
Early Works Approval	Approval for the Early Works (Stage 1) component of the MPW Project under the MPW Concept Plan Approval (SSD 5066) and the MPW EPBC Approval. Largely contained in Schedule 3 of the MPW Concept Plan Approval.
Early Works area	Includes the area of the MPW site subject to the Early works approved under the MPW Concept Plan Approval (SSD 5066).
Proposal	MPW Stage 2 Proposal (the subject of the EIS), namely Stage 2 of the MPW Concept Plan Approval (SSD 5066) including construction and operation of an IMT facility, warehouses, a Rail link connection and Moorebank Avenue/Anzac Road intersection works.
Proposal site	The subject of the EIS, the part of the MPW site which includes all areas to be disturbed by the MPW Stage 2 Proposal (including the operational area and construction area).
IMT facility	The Intermodal terminal facility on the Proposal site, including truck processing, holding and loading areas, rail loading and container storage areas, nine rail sidings, loco shifter and an administration facility and workshop.
internal road	Main internal road through the Proposal site which generally travels along the western perimeter of the site. Provides access between Moorebank Avenue and the IMT and warehouses.
Rail link connection	Rail connection located within the Proposal site which connects to the Rail link included in the MPE Stage 1 Proposal (SSD 14-6766).
Proposal operational rail line	The section of the Rail link connection and Rail link between the SSFL and the Rail link connection (included in the MPE Stage 1 Proposal) to be utilised for the operation of the Proposal.

construction area	Extent of construction works, namely areas to be disturbed during the construction of the Proposal.
operational area	Extent of operational activities for the operation of the Proposal.
Moorebank conservation area/conservation area	Vegetated area to remain to the west of the Georges River, to be subject to biodiversity offset, as part of the MPW Project.
Moorebank Precinct (MP)	Refers to the whole Moorebank intermodal precinct, i.e. the MPE site and the MPW site.
Moorebank Precinct East (MPE) Project	The Intermodal terminal facility on the MPE site as approved by the MPE Concept Plan Approval (MP 10_0913) and including the MPE Stage 1 Proposal (14-6766). N.B. Previously the SIMTA Concept Plan Approval
Moorebank Precinct East (MPE) site	The site which is the subject of the MPE Concept Plan Approval, and includes the site which is the subject of the MPE Stage 1 Approval. N.B. Previously the SIMTA site
Moorebank Precinct East (MPE) Stage 1 Proposal	MPE Stage 1 Proposal (14-6766) for the development of the Intermodal terminal facility at Moorebank. This reference also includes associated conditions of approval and environmental management measures which form part of the documentation for the approval. N.B. Previously the SIMTA Stage 1 Proposal
Rail link	Part of the MPE Stage 1 Proposal (14-6766), connecting the MPE site to the SSFL. The Rail link (as discussed above) is to be utilised for the operation of the Proposal.

1.2 Objective of Geotechnical Investigations

Golder has undertaken geotechnical investigation across the Moorebank Precinct (MP) under a number of campaigns, at various stages during the planning and concept design development.

Two campaigns of geotechnical investigation have been undertaken within the MPW site, as follows:

- Stage 1 – 2014/2015 Campaign (factual results contained within Golder 2015a)
- Stage 2 – 2016 Campaign (factual results contained within Golder 2016a)

The overall objective of the geotechnical investigations was to collect subsurface information to inform detailed design of key features of the project, including earthworks, structural foundations, major drainage structures, internal road pavements and rail subgrade preparation and container handling and storage areas.

Due to the current conceptual nature of design layout and design features, additional geotechnical investigation and geotechnical advice is likely to be required to finalise detail design of specific individual design elements.

1.3 This Report

The report has been prepared to support the Environmental Impact Statement (EIS) for approval of the Proposal and provide geotechnical advice to inform:

- The Proposal;
- Planning and design, so that the requirements for engineering, management and geotechnical requirements can be better defined during further stages of the development;
- Concept and detail designs for earthworks, foundations, engineered fill, pavement and rail subgrade, and other structures;
- Development of an Earthworks Specification for the Proposal Site;

- Geotechnical solutions that will allow approval of the site to be used for commercial and industrial uses; and
- Contractors and developers during a tendering and construction process.

This report has been updated to its current (Rev 1) form in consideration of the results of the Stage 2 – 2016 geotechnical investigation campaign referred to in Chapter 1.2 above.

1.4 Scope of Work

The scopes of work for the two stages of geotechnical investigation are discussed in Golder Associates' Geotechnical Data Reports (GDR, Golder, 2015a and 2016a) and summarised below. The locations of geotechnical site investigations completed are shown in Figure A003.

- Hand-auger boreholes to 1.2 m depth at proposed exploratory locations;
- Hand dug test pits to 0.5 m target depth;
- Borehole drilling and sampling including recovery of rock core;
- Machine excavated test pits;
- Cone penetration tests (CPT) and dilatometer tests (DMT);
- Seismic refraction profiling;
- Survey of all borehole locations using GPS equipment;
- Laboratory testing of soil and rock samples for geotechnical, contamination and acid sulphate soil purposes.

This geotechnical interpretive report includes:

- Geological and geotechnical interpretation and assessment of site investigation results;
- A geological site model, including a description of the ground conditions and cross-sections illustrating the ground conditions beneath the site;
- Recommended geotechnical engineering design parameters;
- Recommendations for earthworks, foundations, engineered fill, pavement and rail subgrade, other structures; and
- Assessment of geotechnical conditions that will be encountered that affect the design, construction and ongoing performance of the MPW Project.

2.0 REVIEW OF EXISTING INFORMATION

2.1 Topography and Terrain

The Proposal site is generally bounded by the Georges River to the west, Moorebank Avenue to the east, the East Hills Railway Line to the south and the M5 Motorway to the north. It is located on Moorebank Avenue, Moorebank (which runs approximately north-south along the Proposal site's eastern boundary) and forms Lot 1 in Deposited Plan (DP) 1197707[1]. The Proposal site also contains Lots 100 and 101 DP1049508, which are located north of Bapaume Road and west of Moorebank Avenue.

The Proposal site is located wholly within Commonwealth Land. The site is approximately rectangular in plan, occupying an area of about 220 hectares, and about 2,950 m from north to south and 960 m from east to west at its widest point.

Key existing features of the site include:

- Relatively flat topography, with a slight decline towards the Georges River along the western boundary to the MPW site. The majority of the site is situated on a terrace above the Georges River at an elevation of approximately RL+15m AHD;
- A number of linked ponds in the south-west corner of the Proposal site, within the existing golf course, that link to Anzac Creek, which is an ephemeral tributary of the Georges River;
- An existing stormwater system comprising pits, pipes and open channels ;
- Direct frontage to Moorebank Avenue, which is a publicly used private road, south of Anzac Road and a publicly owned and used road north of Anzac Road;
- The majority of the site has been developed and comprises low-rise buildings (including warehouses, administrative offices, operative buildings and residential buildings), access roads, open areas and landscaped fields for the former School of Military Engineering (SME) and the Royal Australian Engineers (RAE) Golf Course and Club. Defence has since vacated and all buildings on the site are currently unoccupied and will be removed during the Early Works;
- Native and exotic vegetation is scattered across the Proposal site;
- The riparian area of the Georges River lies to the west of the Proposal site and contains a substantial corridor of native and introduced vegetation. The riparian vegetation corridor provides a wildlife corridor and a buffer for the protection of soil stability, water quality and aquatic habitats. This area has been defined as a conservation area as part of the MPW Concept Plan Approval;
- As stated above, the majority of the Proposal site has been developed, however heritage and biodiversity values still remain on the site; and
- A strip of land (up to approximately 250 metres wide) along the western edge of the MPW site lies below the 1% annual exceedance probability (AEP) flood level.

The ground surface levels of the Proposal site are shown on Figure A004. The area adjacent to the Georges River is terraced. Some modification of the natural ground surface may have occurred in this area as part of the development of training facilities. These training facilities included an area where earthmoving plant driver training was completed, known as "The Dust Bowl", where soils were reworked and fill may have been imported.

Asbestos was found within localised contamination 'hot spots' and areas of anthropogenic fill, for further discussion of this refer to the ESAR (Golder, 2015b) and the AMP (Golder 2016b).

[1] Previously legally described as "Lot 3001, DP 1125930" in the MPW Concept Plan Approval (SSD 5066), however has since been subdivided.

The sides of the Georges River valley are relatively steep and heavily vegetated. Rock outcrops were locally observed on the western bank of the river, which generally lies at higher elevations than the eastern bank.

Within the site there is a small creek (Anzac Creek) which runs through the golf course to the northeast. There are also some small dams in the northern part of the site, some of which have had their sides steepened and retained with sheet piles. With the exception of Anzac Creek, the drainage systems drain west towards Georges River.

2.2 Land Use

This section of the report presents a general overview of the historic development of the site and changes in land use. For more detailed discussion refer to the ESAR (Golder, 2015b), the UXO Risk and Management Plan (G-Tek, 2016a) and the AMP (Golder 2016b). Changes in land use of the site can be assessed using historical aerial photographs (Refer to Figures A005 to A015).

The earliest available aerial photographs are from 1930 and these show the land to be cleared bushland and fields. There are small tracks and paths across the site area and meandering streams cross the site. The area appears to have been cleared of trees and bush up to the edge of the Georges River. Sand banks and bars are visible within the Georges River. Moorebank Avenue is present on this photograph.

By 1956, the military facility had been developed on the site, comprising Steele Barracks. The Defence National Storage Distribution Centre (DNSDC) is present on the eastern side of Moorebank Avenue. A road bridge is present, which crosses the Georges River immediately to the south of the Casula Power Station.

The 1965 aerial photograph shows additional development of the site area, the bridge adjacent to the power station is not present and there is a sand quarry on the site of the current day Glenfield Waste Facility. From the aerial photographs it is apparent that from the 1970's dredging methods were in use on the Glenfield site, as it is flooded. The golf course is present on the MPW site in the 1978 aerial photograph.

By 1986 the road bridge on the current M5 alignment had been constructed to the north of the site. The overall site arrangement appears to be similar to that of the present day. The Glenfield Waste site does not have flooded areas on it, and possible shale and sandstone outcrops are visible in the base of some excavations.

Current aerial photographs show the site in its present arrangement. As of December 2014, military units on the MPW site were relocating to new facilities at Holsworthy and the DNSDC was being gradually relocated in preparation for the proposed change in land use from a military facility to an intermodal terminal.

However, due to training activities involving blank small arms ammunition (SAA) and bomb disposal training activities, Inert Ammunition (IA), Unexploded Ordnance (UXO) and Exploded Ordnance Waste (EOW) risks require management on the Proposal Site (G-Tek 2016a).

2.3 Drainage and Ponds

The dominant water feature of the area is the Georges River which is not locally tidal. Other water systems in the area include ponds and creeks, some of which run towards the Georges River. Anzac Creek is notable in that it runs away from the Georges River in a north easterly direction, re-joining the river at Lake Moore. Drainage systems are shown on Figure A016, attached.

The natural state of the Georges River has been modified since the early 1800s, and the weir constructed at Liverpool marks the present upstream tidal limit (Navin, 2014).

2.4 Climate and Meteorology

2.4.1 Overview

The Holsworthy area experiences relatively mild temperatures and moderate rainfall, with a yearly average rainfall of about 880 mm, based on records from the nearest observation site at Bankstown Airport since 1968 (Refer to Figure 1). Typically, the wettest month (mean rainfall) is February, and driest usually August. The annual mean minimum temperature is 12.0°C and the mean maximum temperature is 23.2°C. The hottest month is usually January (mean maximum of 28.2°C) and the coldest month is usually July (mean minimum of 5.1°C).

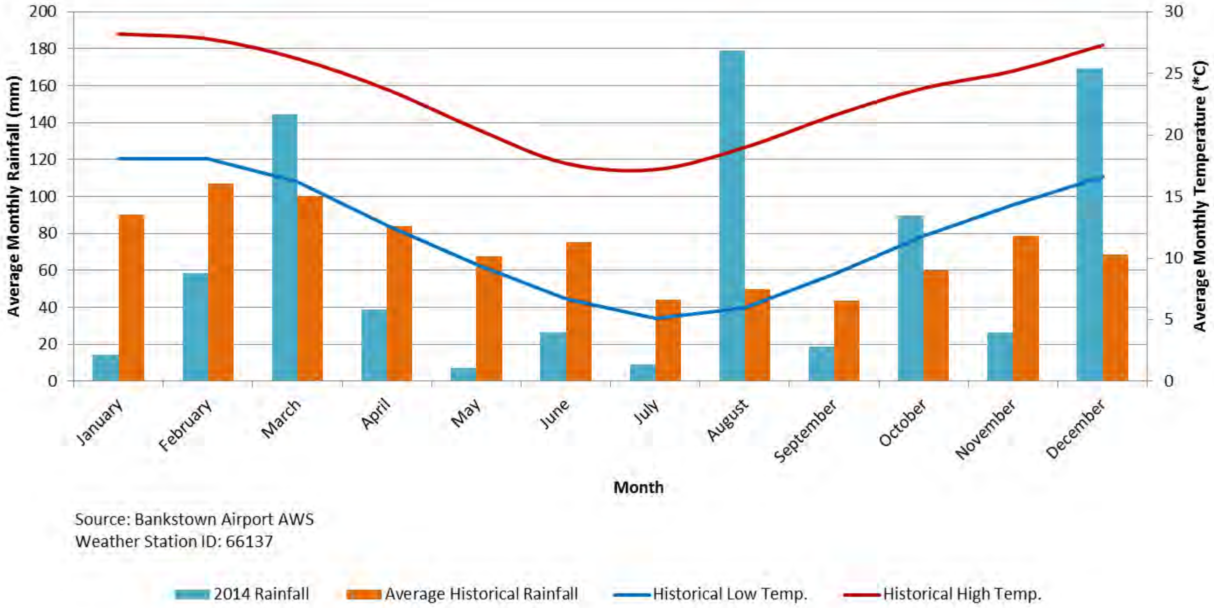


Figure 1: Monthly Rainfall and Temperature Records from Bankstown Airport (7.1 km from MPW Site)

2.4.2 Rainfall Records

Plots showing daily rainfall data over the duration of the Stage 1 and Stage 2 geotechnical fieldwork campaigns are presented as Figure 2 and Figure 3, below. The rainfall conditions around the time of the site investigation may impact groundwater levels measured during that time. Based on the rainfall records, the rainfall during the MPW Stage 1 investigation was less than average during November but more than average during December. For the MPW Stage 2 investigation, the combined rainfall over the preceding 3 months was approximately equal to historical averages for that period. Over the period of the MPW Stage investigation rainfall experienced was slightly lower than historical averages for that period.

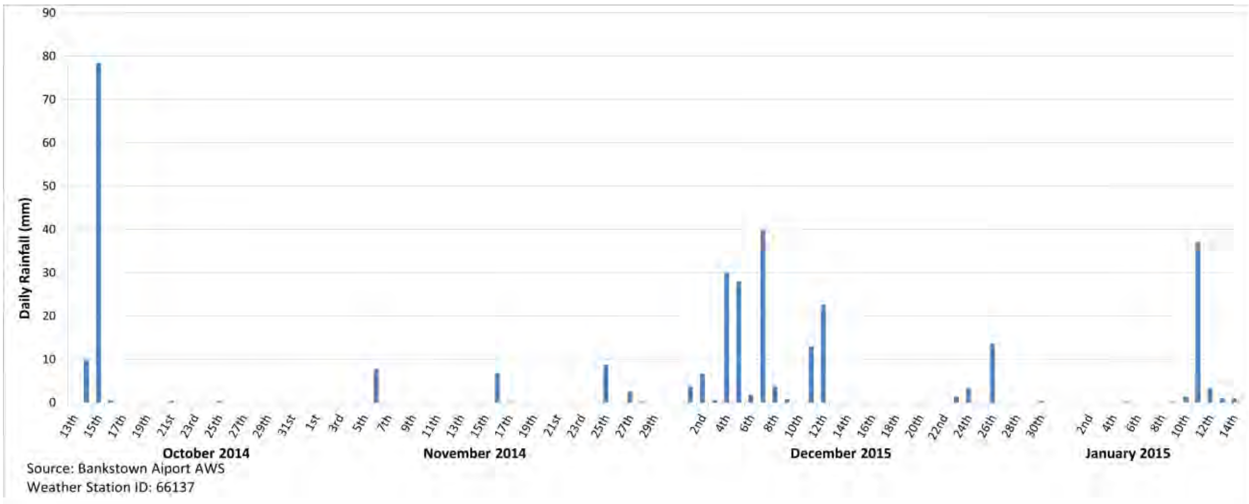


Figure 2: 2014/2015 (Stage 1 Geotechnical Investigation Campaign) Daily Rainfall Records from Bankstown Airport (7.1 km from MPW Site)

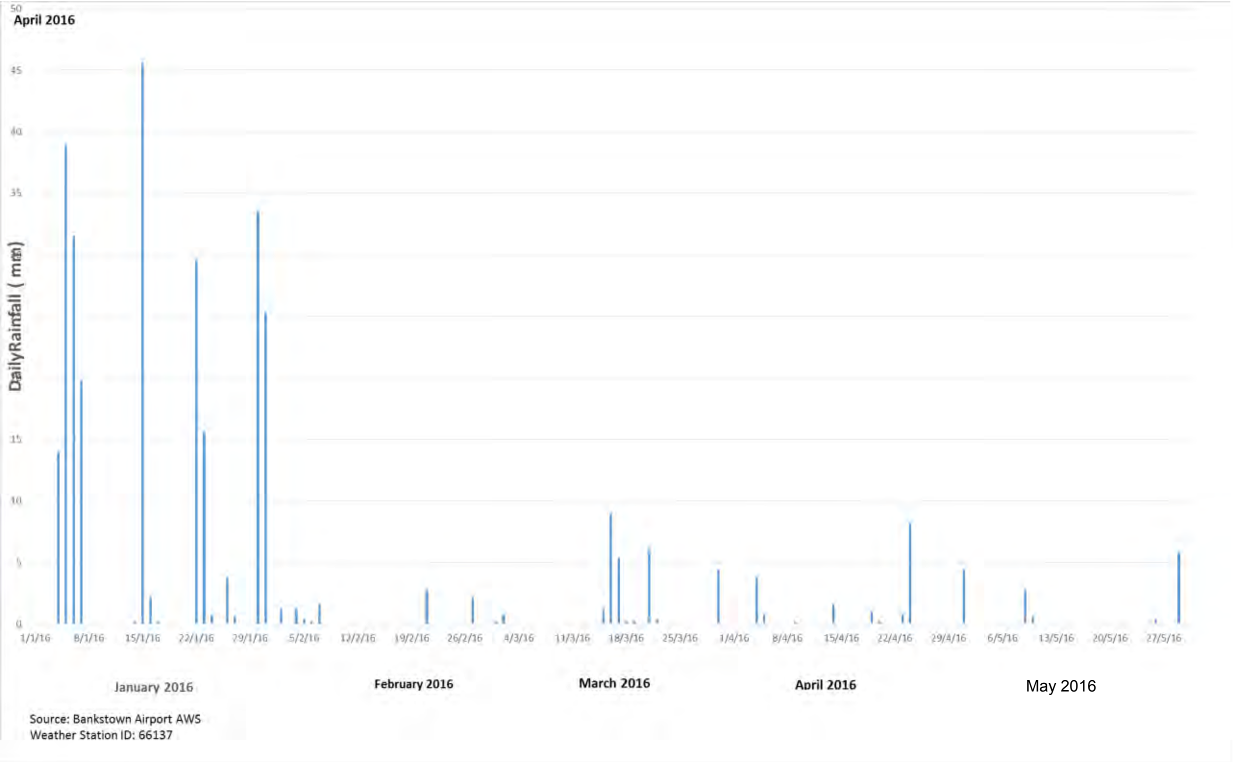


Figure 3: 2016 (Stage 2 Geotechnical Investigation Campaign) Daily Rainfall Records from Bankstown Airport (7.1 km from MPW Site)

2.5 Previous Investigations

2.5.1 Information Sources

Several geotechnical and geochemical investigations have been previously carried out at the MPW site (refer to Table 2). Information from previous geotechnical and geochemical investigations was supplied to Golder by MIC. Previous geotechnical investigation exploratory locations are shown, together with the Golder 2014 and 2016 locations on Figure A017.

Earth Tech (2006) reviewed investigations completed prior to its 2006 investigation, and PB (2015a) included a detailed review of the Earth Tech investigation and partial reviews of other selected investigations completed prior to the Earth Tech (2006) investigation.

Some of the previous data is not in current Association of Geotechnical & Geoenvironmental Specialists Data Interchange Format (AGS) format. Soils were not always described in accordance with the Australian Standard for Geotechnical Site Investigations (Standards Australia (1993) AS 1726). However the information, does provides some valuable information on near surface conditions, and therefore, this existing information has been considered, together with the information obtained from our current field investigations, in developing our geotechnical model for the MPW Project.

Table 2: Previous Investigations

Author	Report Title
Groundwater Technology (1994)	Environmental Site Assessment
Dames and Moore (1996)	Environmental Management Plan and Environmental Audit
CMPS&F, July (1998)	School of Military Engineering (SME) and adjoining areas, Preliminary Environmental Investigation

Author	Report Title
Egis Consulting Australia (2000)	Stage 1 Preliminary Site Investigation, Moorebank Defence Site
HLA Envirosciences (2002)	Soil & Groundwater Investigation Precinct H (DNSDC) Moorebank Defence Land
HLA (2003)	Preliminary Groundwater Study, Moorebank Defence Land (2003)
URS (2003)	Investigation of Potential Sources of TCE, North West Precinct of Moorebank Defence Lands
GHD (2003)	Asbestos Report and Register for the Liverpool Military Area, Updated Registers
GHD (2004a)	Estimated Asbestos Removal and Reinstatement Costs, Liverpool Military Area
GHD (2004b)	Groundwater Investigation of the North Western Portion of the Moorebank Defence Land
GHD (2005)	Proposed Intermodal Freight Hub, Moorebank, Summary of Environmental Planning Reports
HLA Envirosciences (2005)	AST and UST Management Plan, Volume 10, Sydney West Defence Region
Earth Tech (2006)	Stage 2 Environmental Investigation
ERM (2006)	Technical Advice Document, related to Earth Tech (2006) Stage 2 Environmental Investigation
HLA Envirosciences (2006)	Defence Integrated Distribution System (DIDS) Baseline Investigation
GHD (2006)	Proposed Inter-modal Freight Hub Moorebank – Summary of Environmental Planning Reports
G-tek (2011)	Explosive Ordnance Assessment and Safeguarding, Moorebank Intermodal Terminal – Post Activity Report
Parsons Brinckerhoff (2011)	Moorebank Intermodal Terminal - Geotechnical Investigation Report (document no. 2103829A_PR_036)
Parsons Brinckerhoff (2013)	Steele Barracks Moorebank – Dust Bowl Asbestos Management Plan
Parsons Brinckerhoff (2015a)	Phase 2 Environmental Site Assessment, Moorebank Intermodal Terminal (document no. 2103829A-CLM-REP-1 Rev B)
Parsons Brinckerhoff (2015b)	Preliminary Remedial Action Plan (RAP), Moorebank Intermodal Terminal (document no. 2189293C-CLM-REP-2 Rev C)
Parsons Brinckerhoff (2014c)	Phase 1 Environmental Site Assessment, Moorebank Intermodal Terminal (document no. 2103829C-CLM-REP-3321 Rev C) – *included within PB 2015a*
AECOM (2014)	Site Audit Report and Site Audit Statement, Moorebank Intermodal Terminal, Moorebank, NSW (document no. 60327260_SAR_10JUL2014)
Navin Officer Heritage Consultants (2014)	Moorebank Intermodal Terminal – Aboriginal Heritage Assessment Report, June 2014.

2.5.2 Navin Officer Heritage Consultants (2014)

Navin Officer Heritage Consultants completed an aboriginal heritage assessment report (Navin, 2014). The report is useful for geotechnical purposes because it contains logs of test pit excavations from which we have been able to assess shallow soil conditions (approximate topsoil and fill thicknesses) in selected parts of the MPW site.

The main objective of the Navin investigation was to identify geomorphological conditions and to allow recovery and analysis of artefacts. The nature of this investigation means that the sampling points are relatively densely spaced around discrete areas, rather than providing a general coverage of topsoil and fill thickness of the entire Proposal site.

2.5.3 PB Geotechnical Investigation Report (2011)

PB completed a geotechnical investigation of the site in June 2011, comprising twenty CPTs and seventeen boreholes. Of the twenty CPTs, three refused at depths less than 3 m, with a note that refusal was in fill.

The PB investigation characterised the site as comprising very stiff to hard clays with minor bands of medium dense to dense sands, generally at the southern and eastern site boundaries. Closer to the Georges River, along the site's western boundary, the ground conditions were medium dense to very dense sands, with minor bands of very stiff to hard clay. Rock depths varied from 7.6 m below ground level at PB_BH24, in the south east of the site, to a maximum rock depth of 25.8 m below ground level, which was found in PB_BH13, below the centre of the site.

The topsoil thickness was reported to vary between 50 mm and 500 mm, with an average thickness of 200 mm in the seventeen boreholes.

The fill thicknesses varied between 0 m and 1.8 m (PB_BH21), with an average thickness of 0.5 m, but six out of seventeen boreholes were logged as not having fill.

The PB report also included discussion of areas of loose / soft ground. These can be split into two different categories:

- Shallow soils, inferred to be topsoil, fill or alluvium. The soils identified in the PB table ranged in thickness from 0.2 m to 2 m thick; and
- Deeper soils, inferred to be 0.1 m to 1.1 m thick were inferred in CPTs at depths generally greater than 10 m below existing ground level. The layers identified by PB are relatively thin and generally correspond to low cone resistance values between stiffer / denser zones. Due to the deposition environment in which they were laid down it is possible that these layers represent over consolidated organic materials or layers of ash, which can occur within alluvial soils.

3.0 GEOLOGY AND SOILS

3.1 Regional Geology

According to the published 1:100,000 Penrith Geological Map (NSW Department of Minerals, 1991), the Project site is underlain alluvial sediments over rock. Adjacent to the Georges River the alluvial sediments are Quaternary (Holocene) age (<10,000 years) (Qha). These lay above a stratum of Tertiary (Pliocene) age fluvial deposits, consisting of clayey quartzose sand and clay (Ta). The geological map indicates that the underlying rock conditions in the area are either Triassic Hawkesbury Sandstone (Rh) or Ashfield Shale (Rwa).

Within the Sydney region, a relatively thin layer known as the Mittagong Formation (Rm) is sometimes present between the Hawkesbury Sandstone and Ashfield Shale units. It is not shown on the geological map in the site vicinity as being a significant near surface bedrock unit. The Mittagong Formation is typically transitional between the Hawkesbury Sandstone and the overlying Ashfield Shale with a maximum thickness of 10m. We have found what we interpret to be Mittagong Formation in borehole BHBI at the south eastern corner of the site. We found similar rock conditions in a borehole for another client at the Moorebank Avenue railway overbridge. In general, the Ashfield Shale occurs in areas of higher elevation, where it forms a cap over the Hawkesbury Sandstone or Mittagong Formations. An example close to the MP site is the western bank of the Georges River, which geological maps show to comprise shale. The general geological sequence in the area was observed during a site walkover of the Glenfield Waste Facility in December 2014. A photograph taken during this inspection shows the Ashfield Shale overlying Sandstone (with heavily eroded alluvial soils with ironstone bands in the background).

Figure 4: Photograph in Glenfield Waste Facility showing general stratigraphy looking northwest

The bedrock conditions that are anticipated as being present below the site are shown in Table 3, with the formations expected to occur in the vicinity of the site shown by green shading below. The bedding of the sedimentary sequence generally dips between 0° and 15° to the west although localised variations can occur with steeper bedding planes often associated with cross-beds.

Table 3: 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 geology beneath the site is illustrated on geological sections (Refer to Figures A023 to A035) that we have developed using the existing information. The geological sections form the basis of our geotechnical model, discussed later in the report.

3.1.1 Quarternary / Holocene Fluvial and Estuarine deposits

Geological maps indicate that soil deposits comprise sands, clays and silts and that they are present on terraces adjacent to the Georges River and associated with other creeks and ponds in the area.

Over recent geological time, the Georges River has laid down sediments in the form of channel deposits, sand banks and silt flats in the general area of the Proposal site. On the nearby Glenfield Waste site, the Georges River sediments were dredged through the mid to late 1900s as a source of construction materials. The channel of the Georges River is likely to have moved over geological time and hence buried former channels and associated deposits may be present below the MPW site. The fluvial deposits derived from the Georges River are likely to be laterally and vertically variable with gravels and sands being found close to the river and in old buried channels and silts and clays being found further from the river across the floodplain.

These shallow soil deposits have likely been impacted and reworked by natural and man-made activities. They may have been originally deposited by flooding, but may have been impacted by dredging for building resources (Glenfield Waste Facility), vegetation removal and regrowth, agricultural development of the site and then due to the development and use of the site as a military base. A photograph showing these soils on the adjacent Glenfield Waste Facility is included as Figure 4, above.

3.1.2 Rock Formations

The 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 the unit 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 metres to 1.5 metres thick) over a lower unit of fine grained sandstone and siltstone (typically one metre to three metres thick, but can be up to ten 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 thin 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.

The published geological map indicates that the structural contour of the Hawkesbury Sandstone and Ashfield Shale interface lies at approximately RL 0m AHD below the southern end of the Proposal site and that this surface dips to the north to approximately RL-20m AHD at the northern boundary.

3.2 Faults and Dykes

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 shown on the geological plan in the vicinity of the MPW site, however, two lineaments are shown on the geological map to the west and east of the site. These may imply that faults are present in the bedrock close to these features and they may have had some impact on the present route of the Georges River, as the course of rivers can be affected by the presence of weaker zones in bedrock, such as faults.

There are no major (regional) faults or dykes shown within or close to the Proposal site on published geology maps. It is possible, though, 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.

Published geological information (Pells, 1993) indicates that small scale faulting occurs within shales of the Wianamatta Group, but that usually they are of limited continuity (i.e. less than 10 m).

No fault zones or dykes were encountered during the recent field investigations. If localised faults or dykes are encountered during construction, advice should be sought from an experienced Geotechnical Engineer.

3.3 Soil Landscapes

The Penrith Soils Landscape Map (Soil Conservation Service of NSW, 1989) indicates that the soils on the MPW site are of the Berkshire Park Group. These are soils 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. Surface water erosion comprising gully, sheet and rill erosion can also occur in exposed areas.

On lower river terraces, soils of the Richmond Group are present. These are described as poorly structured orange to red clay loams and mapping also indicates that ironstone nodules may be present. These soils are potentially erodible.

On the site of the Glenfield Waste Site, Freemans Reach Group soils are mapped. These are associated with active floodplains that are level with minor (<10 m) relief. They are typically deep brown sands and loams, which have a high potential for stream bank erosion, are prone to flooding and/or high water tables. The mapping of these units also indicates that they are associated with extractive industries, such as sand and gravel mining.

An extract of the Soils Landscape Map is included as Figure A019.

3.4 Hydrogeology

The overall geotechnical and geochemical investigation scope included monitoring of water levels within existing monitoring wells on the Proposal site. Full details of the monitoring procedure and the results of the monitoring are included in the ESAR (Golder, 2015b). The majority of wells sampled were installed with screens in the soils overlying rock.

There are two main aquifer systems on the site; a perched system within alluvial soils and; a deeper aquifer within the bedrock. Based on contouring of the results from groundwater monitoring wells on the site, groundwater in the shallow alluvial aquifer flows towards the Georges River. Two contoured groundwater plans for the site are attached as Figures A020 and A021, showing monitored groundwater levels in 2011 and 2014.

Ashfield Shale has a very low rock mass permeability and may act as an aquitard (barrier to groundwater flow). On the MPW site this unit may well reduce the infiltration of groundwater into underlying sandstone, although some groundwater may flow within this unit through joints or faults. Groundwater in the unit is saline and hard, salinity levels up to 3100 mg/l have 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 paleochannels.

Groundwater in sandstone is generally of reasonable quality (typical salinity: 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.

3.5 Slope Stability and Erosional Processes

There are no known areas of natural slope instability (landslide) within the site area. With the site being located immediately adjacent to the Georges River, some accumulation of soil by the river banks will have occurred over geological time and the river's course may have changed. As a result of these fluvial processes, some weathering and erosional processes will have impacted areas of the site close to the river. Although the river banks are heavily vegetated, older colluvial deposits may have formed when sea levels were higher than they presently are now. From field observation there appears to be low likelihood that colluvial slopes of significant depth have formed in the area and colluvium is most likely to have been stabilised or modified by human activity.

An area of soil erosion was observed on the western bank of the Georges River, this suggests that soils formed in the local area can be prone to erosion when exposed to concentrated water flow or where not otherwise protected (Refer Figure 5).

Figure 5: Area of erosion within Glenfield Waste Facility

No rock outcrops were observed on the MPW site area, although some areas of the western bank of the Georges River have outcrops of sandstone close to the road bridge at Cambridge Avenue (Refer Figure 6).

Figure 6: Sandstone outcrop to the south of Cambridge Avenue road bridge

3.6 Contaminated Soils and Acid Sulphate Soils

The extent and nature of possible contaminated soils and acid sulphate soils in the project area are discussed in the ESAR (Golder, 2015b).

3.7 Geological Units

For the purpose of geotechnical characterisation of the subsurface conditions, we have generalised the soil and rock types at the site into the following units, as illustrated in Table 4.

The geotechnical characteristics of each of these units are discussed in the following sections of the report, followed by a description of the ground conditions encountered beneath different parts of the site.

Table 4: Geotechnical Model

Unit		Sub-unit	
1	Surficial Soils	1A	Topsoil
		1B	Anthropogenic Fill
		1C	Granular Fill
		1D	Cohesive Fill
2	Recent Alluvium	2A	Sand
		2B	Clay
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
		5B	Very Low to Low Strength Sandstone
		5C	Sandstone of medium strength or higher

3.8 Surficial Soils

3.8.1 Unit 1A – Topsoil

A thin surface layer of topsoil was generally encountered on the site, varying in thickness up to approximately 0.5 m. It is generally associated with well-established landscaped areas with grass cover. It generally comprised brown silty sand or clayey sand with rootlets. The colour of the topsoil varied from pale to dark grey and brown, with darker material typically, but not always, containing greater organic content. Interpreted topsoil thickness contours, calculated from discrete investigation locations are shown in Figure A036.

It is noted that apparent ‘buried’ layers of older topsoil were encountered beneath a thin layer (typically 0.3 m thick or less) of fill at a number of locations (e.g. GA-HP-6009, GA-HP-6026 and GA-HP-6040). ‘Older’ topsoil also appeared to have been worked into filling/re-grading of the surface at some locations (e.g. GA-HP-6015)

The topsoil layer can be further sub-divided on the basis of soil horizon definitions as applied by the Soil Conservation Services of NSW, based on the humic content. This further sub-division may be applied to facilitate rationalisation of topsoil stripping in accordance with a site specific Earthworks Specification.

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. Characterisation of material as topsoil, where it implies a lack of suitability as an engineered material, should be based upon an assessment of appropriate characteristics and parameters pertinent to the desired usage (or rejection) of that material.

For the purposes of field logging, topsoil has been taken as the near surface layer of material with an observed higher proportion of organic material (be that humic material, roots or rootlets) in contrast to the underlying fill or natural material. Organic Content testing was undertaken to inform re-use testing potential for the topsoil (particularly the lower portions of topsoil or older 'buried' topsoil layers below fill). Organic Content testing was also undertaken on Unit 1B, 1C and 1D material and those results are discussed in the following Chapters.

A total of 30 Organic Content tests were undertaken on Unit 1A samples with results ranging from 0.4% to 2.5% (GA-HP-6024), with an average value of 1.3%. 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.

Standard compaction and California Bearing Ratio (CBR) testing was also undertaken on two samples of Unit 1A material, returning CBR values (after 4 day soak) of 10 % and 20 %. This reflects the dominantly sandy nature of the topsoil, as does the recorded swells of 0% and 0.1%. Standard Maximum Dry Density (SMDD) testing returned values of 1.64 and 1.77 tonnes/m^3 and Optimum Moisture Contents (OMC) of 11.2% and 14.9% were recorded. It should be noted that the testing was undertaken on samples of soil which were obtained from beneath the surficial vegetative layer (such as the root mat of turf overlying topsoil layers). Significantly higher organic contents would be recorded should such vegetation be included in topsoil samples.

3.8.2 Unit 1B – Anthropogenic Fill

In some parts of the site there are above or below ground areas where waste materials (anthropogenic fill) have been placed. These include areas where former valleys have been in-filled and above ground waste stockpiles, as shown in Figure A037. Our ESAR contamination report (Golder 2015b) identifies and discusses areas of anthropogenic fill in more detail. Where encountered, Unit 1B material typically extended to depths of more than 2m and up to 4m (GA-BH-3102).

The compaction of anthropogenic fill zones is expected to be poor and variable as evidenced at GA-BH-3102 which reported a mix of SPT hammer refusal (on obstructions within the fill such as sheet metal) and low blow counts (initial seating under rod and hammer weight alone followed by an N value of 1) over the 4 m thickness of Unit 1B encountered.

A number of test pits were undertaken during the MPW Stage 2 campaign within areas of anthropogenic fill 'hotspots', with laboratory testing undertaken to assist in assessing the opportunity for re-use of the Unit 1B material, subject to processing and screening for unsuitable material.

Mixing of material Units has occurred in zones of Unit 1B filling. As such topsoil will likely be mixed in with some of the fill and evidence of this was observed within the MPW Stage 2 test pits. Three organic content tests were undertaken on Unit 1B samples as with results ranging from 0.4% to 1.0% (GA-TP-3113), and an average value of 0.7%.

The material encountered was variable but typically sandy with silt and clay. Inclusions ranged in size and included steel, concrete and timber to boulder size as well as apparent domestic waste such as a dilapidated pram and butter knife (refer to Golder 2016a for conditions encountered at specific locations).

Standard compaction and California Bearing Ratio (CBR) testing was also undertaken on two samples of Unit 1B material, returning CBR values (after 4 day soak) of 10 % and 19 %. This reflects the dominantly sandy nature of the fill (excluding the included waste material and debris), as does the fact that no swell was recorded. Standard Maximum Dry Density (SMDD) testing returned values of 1.89 and 2.0 tonnes/m^3 and Optimum Moisture Contents (OMC) of 12.1% and 12.2% were recorded.

3.8.3 Units 1C and 1D – Fill

In developing the site into its current form it is likely that cut / filling operations have been completed to produce level working areas and in the construction of structures over an extended period of time. Fill areas include existing road pavements and hard stand areas. Most of the fill encountered on the MPW Stage 2 site is granular (primarily sand, Unit 1C) although the PB (PB, 2011) geotechnical investigation did record sandy clay / clayey sand fill material in Boreholes 3, 7, 10, 13, 16, 18 and 24 (Unit 1D), as did the 2016, MPW Stage 2 geotechnical investigation.

As the site has been in use since the 1940s compaction of these fill materials will likely have been completed using different equipment and to different specifications than those used currently.

Inferred fill thicknesses over the site area are shown in Figure A038 and are up to approximately 0.5 m thick. The contouring process used in the generation of Figure A038 may lead to overestimation of fill thickness, where a locally thicker fill material was found, for instance, thicker granular fills associated with existing hardstand or internal access roads of the site. However, the figure is useful in that it shows a correlation between greater fill thicknesses and areas of the site that have been extensively developed, with buildings, services and roads constructed.

The thickness of granular fill materials are summarised below:

- Gravels were generally associated with paved or hardstand areas, with thickness ranging between 0.36 m (BH102) and 0.45 m (BH110).
- Sands, silty sands or clayey sands, generally inferred to have been reworked from natural soils had thicknesses ranging between 0.28 m (BH108) to 2 m (BH111).

Greater fill thicknesses may also be expected along former valleys and creeks across the site, behind retaining structures that are present on the lower terraced area and in areas of the site where there are slopes, which fill materials may have been end tipped historically to provide new working areas.

A summary of laboratory testing on sands from Unit 1C is presented in Table 5. These soils were generally dry of optimum moisture content (2 to 10% dry, based on lab testing), so depending on climatic conditions at the time, some moisture conditioning will be required if effective compaction of these soils is to be achieved.

Table 5: Summary of Lab Testing Results for Unit 1C

Test	Minimum	Maximum	Average	Median	Number of Tests
Moisture Content, (%)	1	20.9	8.0	5.9	9
California Bearing Ratio (CBR) (5.0mm) (%)	5	30	15.5	N/A	10
Optimum Moisture Content (OMC) (%)	9.5	17	12.1	11.9	13
Maximum Dry Density (MDD), (t/m3)	1.7	2	1.9	1.9	13
Emerson Class Number	5	6	5.5	N/A	6
Organic Content (%)	0.2	0.8	0.6	0.7	7

3.9 Unit 2 – Recent Alluvium

Unit 2 comprises Recent Alluvium (inferred Holocene age) characterised by very loose to loose sands or silts (Unit 2A) or very soft to soft clays (Unit 2B). This unit was not encountered during the Stage 1 and Stage 2 EPW geotechnical investigation. However, the ESAR (Golder 2015b) included some sediment probing work along existing drains and watercourses on the site, which contained Unit 2 materials. In addition, we consider that existing creeks such as Anzac Creek and the ponds at the northern end of the MPW site are likely to contain recent alluvial materials.

PB (PB, 2011) may have encountered recent alluvial materials in some boreholes during their investigation, based on Table 3 of their report. Generally these layers appear to be relatively thin, or they are associated with fill deposits, which may make differentiating between fill and recent alluvium difficult.

3.10 Unit 3 – Older Alluvium

Older Alluvium (inferred Tertiary age) is found beneath the Unit 1 surficial soils and Unit 2 alluvium (where present) and overly residual soils and bedrock. In some cases it is difficult to distinguish between recent (Unit 2) and older (Unit 3) alluvium. The Unit 3 materials are generally denser or stiffer than the Unit 2 materials.

Unit 3 comprises sub-units of medium dense to very dense sands and silty sands (Unit 3A) and very stiff to hard silty clays (Unit 3B). In general, the unit is formed from interbedded sands and clays. At many of the investigation locations, there is a sharp transition from alluvial sediments into the underlying bedrock, which implies that residual soils have been scoured from the bedrock surface and that transported alluvial soils (which may have been produced from the same parent rocks upstream) have been deposited onto weathered rock.

Both units are inferred to contain iron cemented bands or dense materials, through which CPTs could not penetrate. Numerous CPTs are inferred to have refused at depths ranging between 0.1 m and 15.5 m below existing ground level. Due to the variability in rock head across the site (refer to the Cross Sections of APPENDIX A) and the broad spacing of discrete test locations, it is difficult to infer in all instances whether CPT refusal reflects hard, potentially iron cemented bands with the Unit 3B material or Unit 4 or Unit 5 residual material or bedrock. For more information on CPT refusal refer to the CPT logs in the GDR (Golder, 2016a). Test pits that were excavated using a backhoe at the locations of selected refused CPTs, also refused on inferred iron cemented layers. The presence of these hard layers will need to be considered when contractors consider excavation and piling options for the site.

The CPTs by PB (PB, 2011) identified loose or soft materials at depth within the very stiff or very dense materials. These may be thin clay layers or organic layers.

While we endeavoured to distinguish between sand (Unit 3A) and clay (Unit 3B) layers on the geological sections, due to the complexity and potential variability of the former alluvial depositional environment and the relatively widely spaced position of the boreholes, it was not possible to delineate the Unit 3A and 3B on the interpretive geological sections. The presence and differing engineering behaviour of the units will need to be considered locally during design of each specific facility at the site.

3.10.1 Unit 3A – Silty Sands and Clayey Sands

The unit was a maximum of approximately 18 m thick (BH114).

Table 6 presents a summary of the results of laboratory testing on Unit 3A materials. Atterberg limit tests were carried out on three samples of material described as clayey sand and grouped within Unit 3A. The tests indicated that the samples tested classify as CL (low plasticity clay). Typically a soil can demonstrate clay-like behaviour even if it contains a relatively low percentage of clay size particles and appears to be primarily sand in composition. The composition of the Unit 3 materials is highly variable, with gradational properties between sand and clay. Results of aggression testing undertaken on Unit 3A materials are presented in Chapter 4.6.

CBR test results are likely to vary substantially within this unit dependent upon the relative amount of clay fines within the sample. Where the material is dominantly sand, the CBR test value will be high. For example, a single test was undertaken on a sand sample of Unit 3A material, with a reported 4 day soaked CBR value of 40. MDD and OMC were 1.92 t/m^3 and 10.8 % respectively.

Table 6: Summary of Lab Testing Results for Unit 3A

Test	Minimum	Maximum	Average	Median	Number of Tests
Moisture Content, (%)	3.5	20	10.5	10.3	37
Liquid Limit (LL), (%)	15	42	28.6	N/A	12
Plastic Limit (PL), (%)	8	17	13.6	N/A	12

Test	Minimum	Maximum	Average	Median	Number of Tests
Plasticity Index (PI), (%)	3	29	15	N/A	12
Linear Shrinkage (LS), (%)	7	10.5	8.8	9	3
Emerson Class Number	5	6	5.6	N/A	8

A plot of Standard Penetration Test (SPT) "N" values versus depth is given in Figure 7 for the Unit 3A sands. The N value is a representation of the density of the soils, and indicates that the sands are typically medium dense to dense, although there is no obvious trend of increasing density with depth. Each point on this graph represents an individual SPT test value.

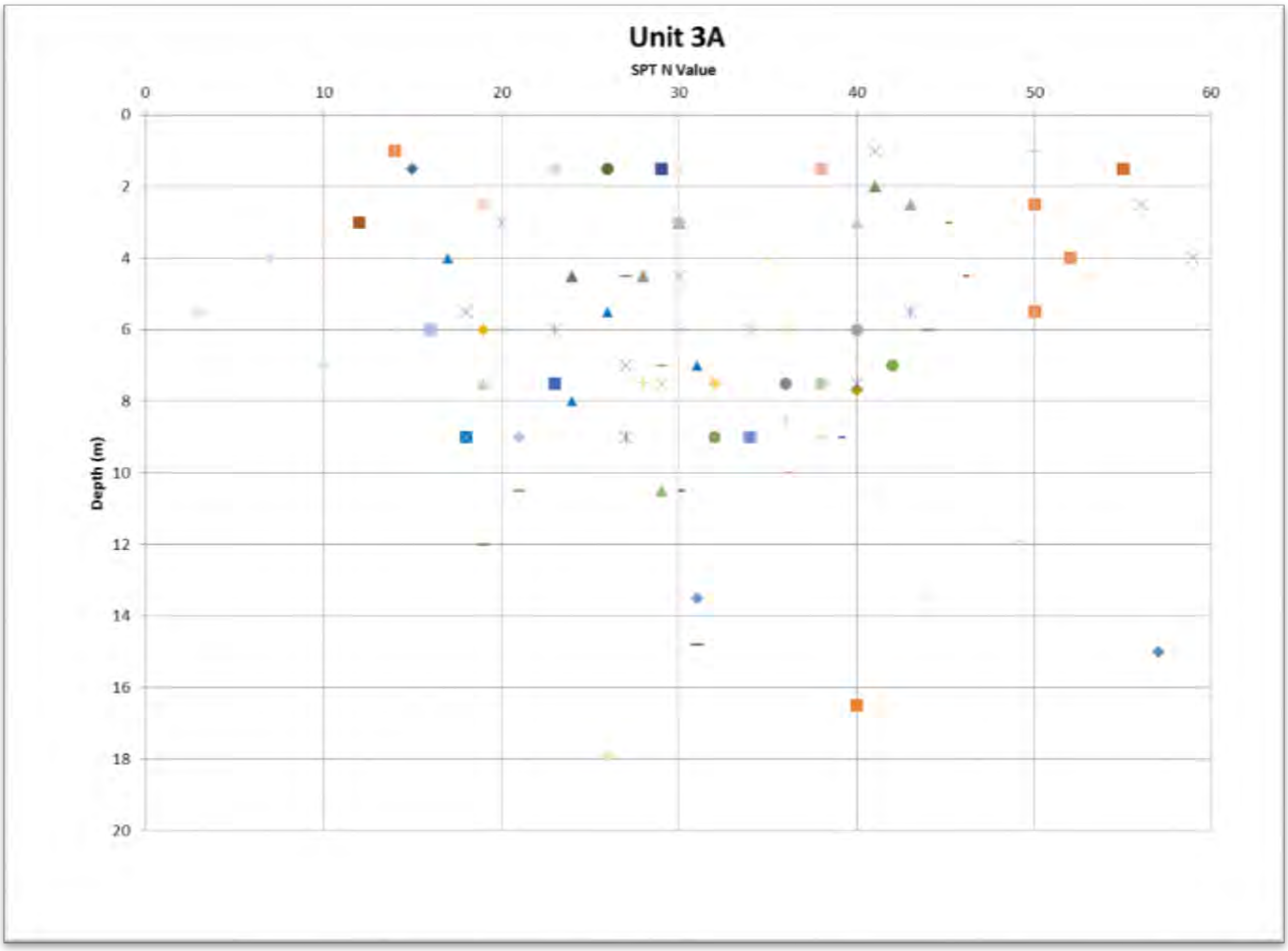


Figure 7: Summary of SPT results in Unit 3A

3.10.2 Unit 3B – Silty Clays and Sandy Clays

Unit 3B clays were a maximum of approximately 20 m thick (BH101) and included interbedded sand or clayey sand layers, based on CPT results. CPT134 is a good example of an interbedded Unit 3A and Unit 3B profile. A summary of laboratory test results for tests on clays from Unit 3B is presented in Table 7.

The samples taken from relatively shallow depth (2 m deep or less) had a relatively wide range in results for both moisture content and optimum moisture content. In general the soils were slightly dry of optimum content (approximately 2%). However in some areas these materials may also require drying-back, particularly where earthworks are conducted during periods of wet weather. The average OMC of Table 7 is shown as a line on Figure 8. The majority of samples tested were dry of the inferred average OMC for Unit 3B.

Table 7: Summary of Lab Testing Results for Unit 3B

Test	Minimum	Maximum	Average	Median	Number of Tests
Moisture Content, (%)	11.2	30.2	18	17.5	20
Liquid Limit (LL), (%)	18	97	50.8	47	35
Plastic Limit (PL), (%)	8	26	15.5	15	35
Plasticity Index (PI), (%)	3	77	35.2	32	35
Linear Shrinkage (LS), (%)	1.5	19.5	12.8	14	22
% < 0.075 mm	47	92	63.7	N/A	10
California Bearing Ratio (CBR) (5.0mm) (%)	1	7	3.2	2.5	5
Optimum Moisture Content (OMC) (%)	13.6	30	20.8	20.5	5
Maximum Dry Density (MDD), (t/m3)	1.44	1.86	1.68	1.72	5
Emerson Class Number	5	6	5.5	N/A	16
Shrinkage index	0.7	2.8	1.3	1.3	6

The field moisture content of the clay is plotted versus depth in Figure 8 below.

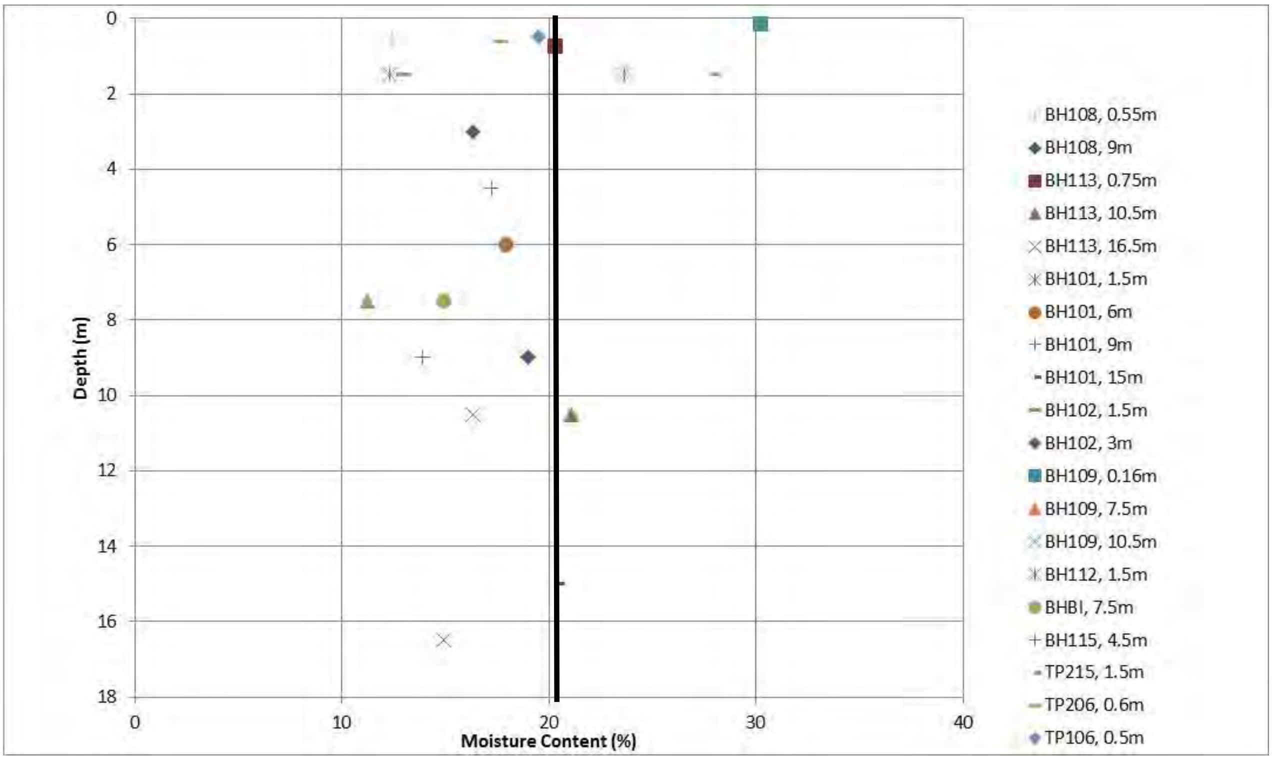


Figure 8: Field Moisture Content vs Depth – Unit 3B

As illustrated on the plasticity chart in Figure 9, the Unit 3B clay has a Unified Soil Classification System (USCS) symbol typically of CI to CH (medium to high plasticity). This is consistent with the average and median values calculated for Table 7. However as can be seen in Figure 9 a number of very high plasticity outliers (e.g. Liquid Limit greater than 80%) have been recorded. Care should be taken in applying average values. Given the size of the site and the relatively widely spaced nature of investigation and testing undertaken to date it is possible that locally relatively large areas of material may be encountered which did not conform well to average values.

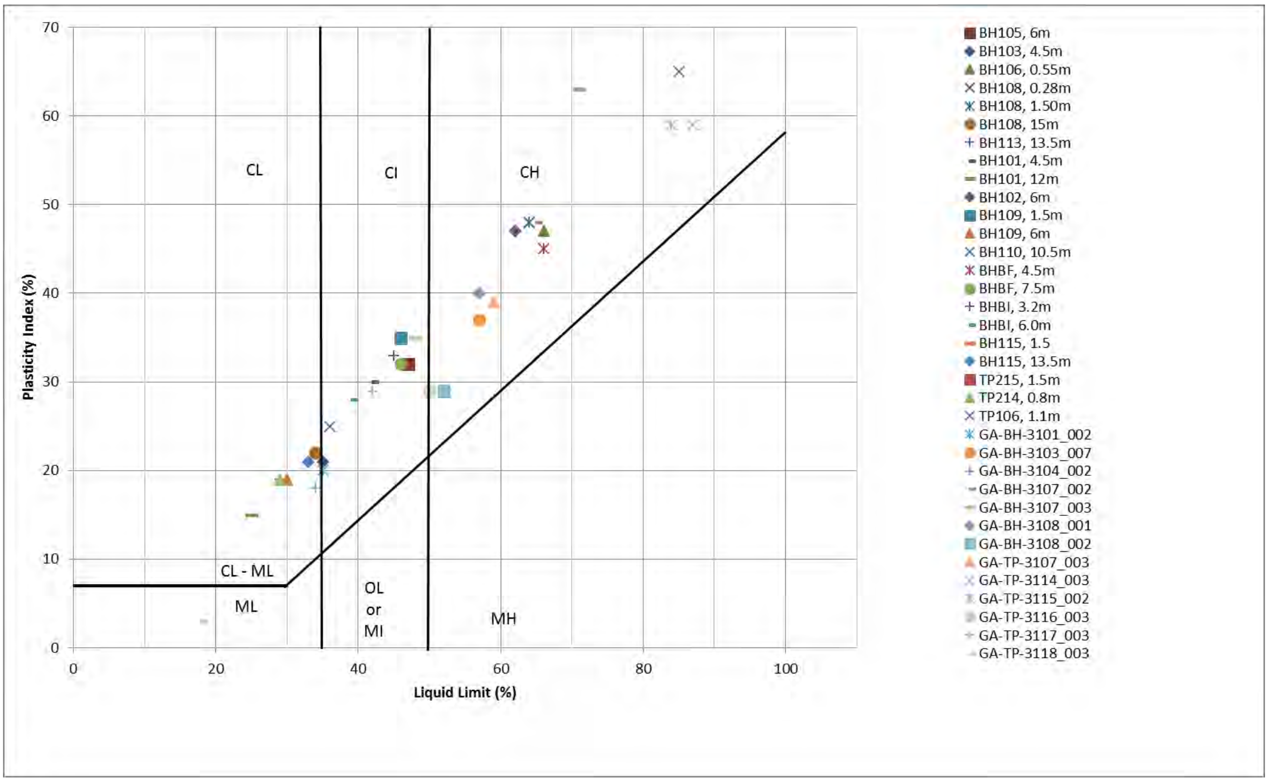


Figure 9: Liquid Limit vs Plastic Limit – Unit 3B

A plot of Standard Penetration Test (SPT) "N" values versus depth is given in Figure 10. The N value is a representation of the strength of the soils, and indicates that the clays are stiff to hard. Each point on this graph represents an individual SPT test value.

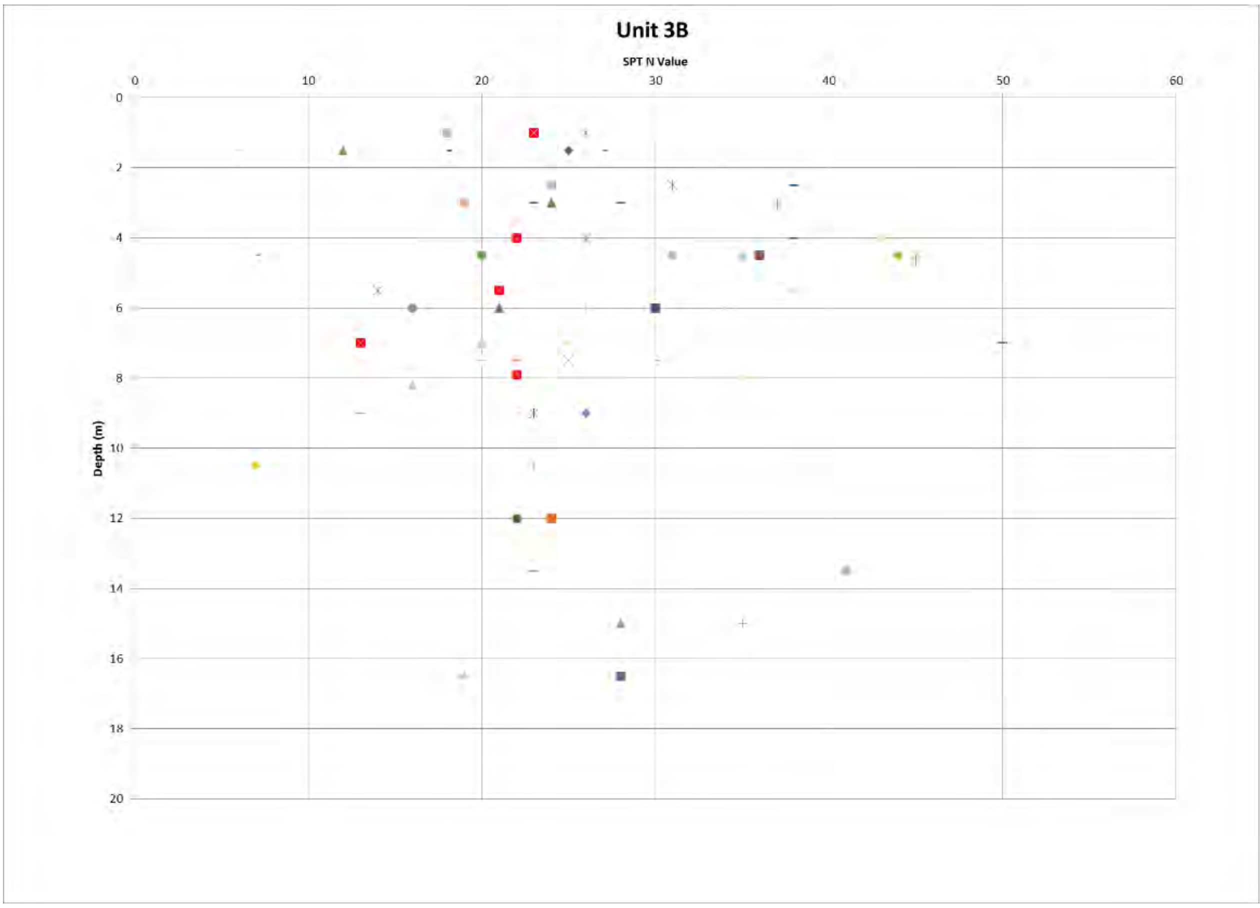


Figure 10: Summary of SPT results in Unit 3B

Dilatometer (DMT) Testing was completed at locations adjacent to boreholes BH109, BH111 and BH114. In two of the three proposed test locations the DMT refused at relatively shallow depth (3 m and 5 m). Both of these locations were underlain by denser Unit 3A sand. At the third location, underlain by very stiff Unit 3B clay a DMT test was completed successfully to a depth of 12 m. The results of DMT testing are included in the GDR (Golder, 2015a).The results of DMT testing corroborate the SPT and CPT data which indicate that the Unit 3A and 3B materials typically comprise stiff to hard sandy clay and dense to very dense sand.

3.11 Unit 4 – Ashfield Shale and associated Residual Soils

3.11.1 Unit 4A – Residual Soils

Unit 4 includes sub-units 4A (Residual Soil), 4B (very low to low strength siltstone) to 4C (medium strength or higher siltstone).

In general the residual soils below the site appear to be relatively thin, with a relatively abrupt transition from the older alluvium to extremely weathered siltstone, which also generally quickly improves in strength to medium to high strength. Figure 11 shows the transition between the alluvial soils and extremely weathered shale rock in BH111. A thin layer of possible residual soil, approximately 150 mm in thickness was observed in this borehole, which we consider to be typical for the area, considering its geological history and the results of boreholes.

The geological profiles interpreted from the seismic reflection surveys and the borehole information correlate reasonably well. The seismic results also appear to confirm that residual soils are thin or absent.

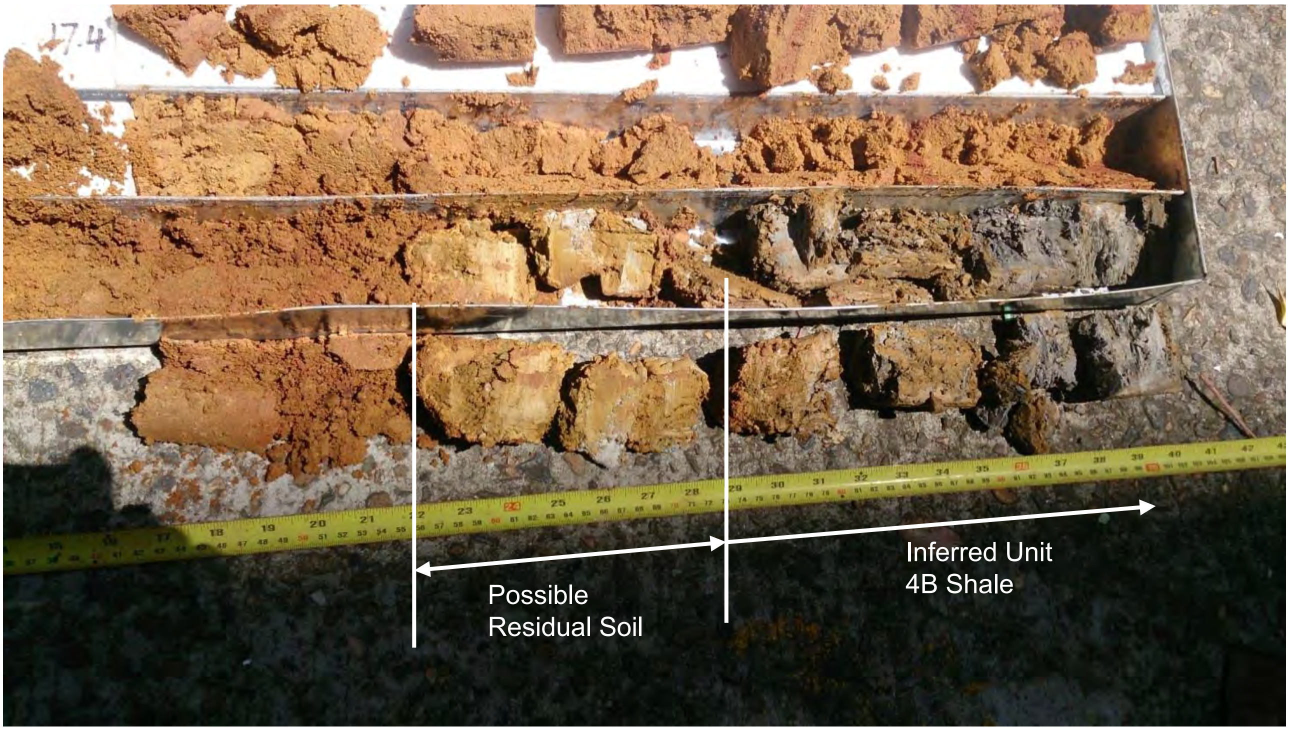


Figure 11: Transition between alluvium and shale bedrock

3.11.2 Unit 4B – Extremely Low to Low Strength Shale

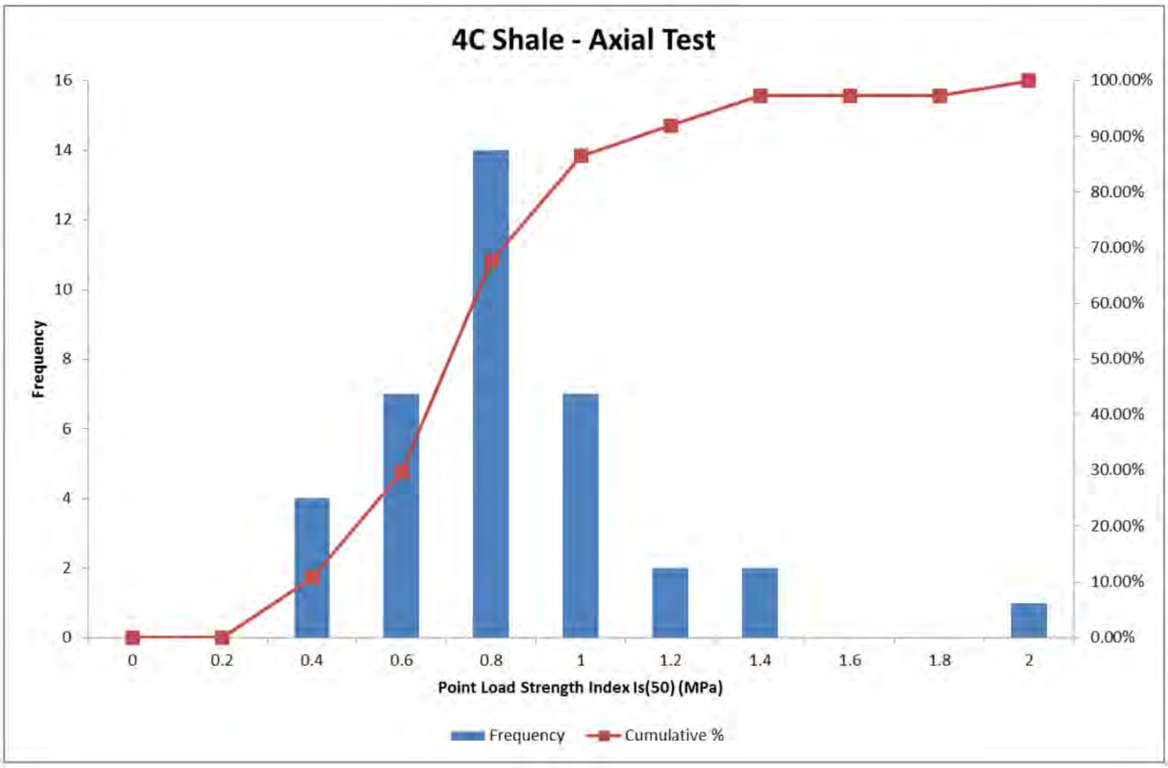
Shale was found in the majority of boreholes over the site at depths ranging from 8.5 m to 21.8 m. Generally, the shale encountered across the main investigation site does not exhibit deep weathering, with slightly weathered to fresh and medium to high strength shale encountered within approximately 2 m of the top of the unit in the majority of boreholes. The shale encountered in the southern end of the site exhibited a deeper weathering profile with Unit 4B shale inferred to be up to 5 m thick.

Contours of the top of rock are included in Figure A039. Figure A039 does not distinguish between the top of sandstone or shale. We have inferred the potential shale / sandstone boundary on cross sections A023 to A027 based on the results of boreholes and published geological maps.

3.11.3 Unit 4C – Shale of Medium Strength or Higher

Unit 4C shale observed during the investigations was generally slightly weathered to fresh and medium to high strength. Figure 12 below gives a summary of the Is_{50} rock strength results obtained from point load testing carried out on shale samples during the investigation. As expected the data shows a strong anisotropy related to the horizontally fissile nature of the shale.

Three UCS tests were carried out on shale samples which indicated compressive strengths of 7 MPa, 17.4 MPa and 25.7 MPa.



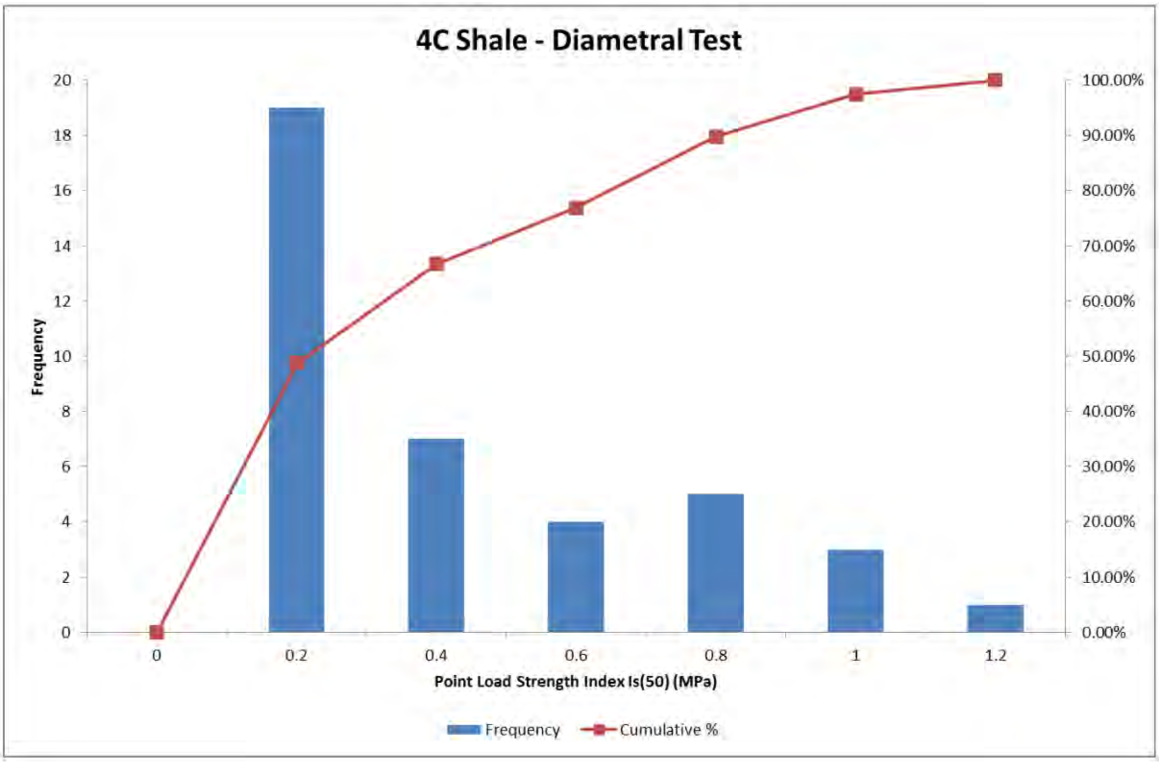


Figure 12: Point Load Test Results for Unit 4C

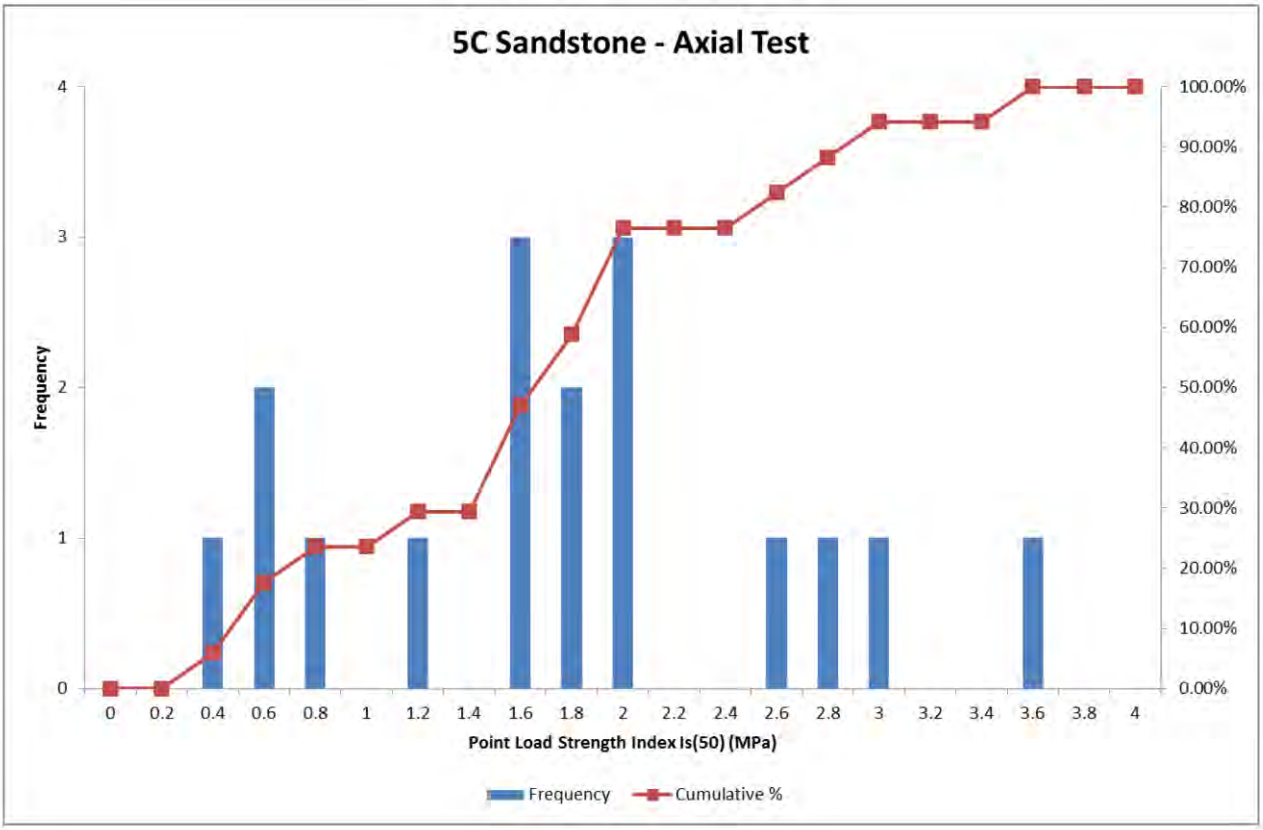
3.12 Unit 5 – Hawkesbury Sandstone and associated Residual Soils

The findings of the geotechnical investigation seem to be consistent with published geological information. Sandstone (in the absence of a shale cap) was only encountered below the southern end of the site (BH101). A thin layer of residual soil 1 m thick was observed in this borehole comprising silty clay of hard consistency. Elsewhere, the residual soil was likely eroded prior to deposition of the overlying alluvial sediments.

The Hawkesbury Sandstone was also observed in other locations, below a shale cap (BH103, BH108 and BHBI).

The majority of the Hawkesbury Sandstone encountered during the investigations was slightly weathered to fresh and medium to high strength. Generally the sandstone encountered does not exhibit deep weathering, with Unit 5C sandstone encountered within approximately 2 m of the top of the unit in the four boreholes in which it was encountered (BH101, BH103, BH108 and BHBI).

Figure 13 below gives a summary of the Is_{50} rock strength results obtained from point load testing carried out on sandstone samples during the investigation. Two UCS tests were carried out on sandstone samples which indicated compressive strengths of 20.2 MPa and 22.6 MPa.



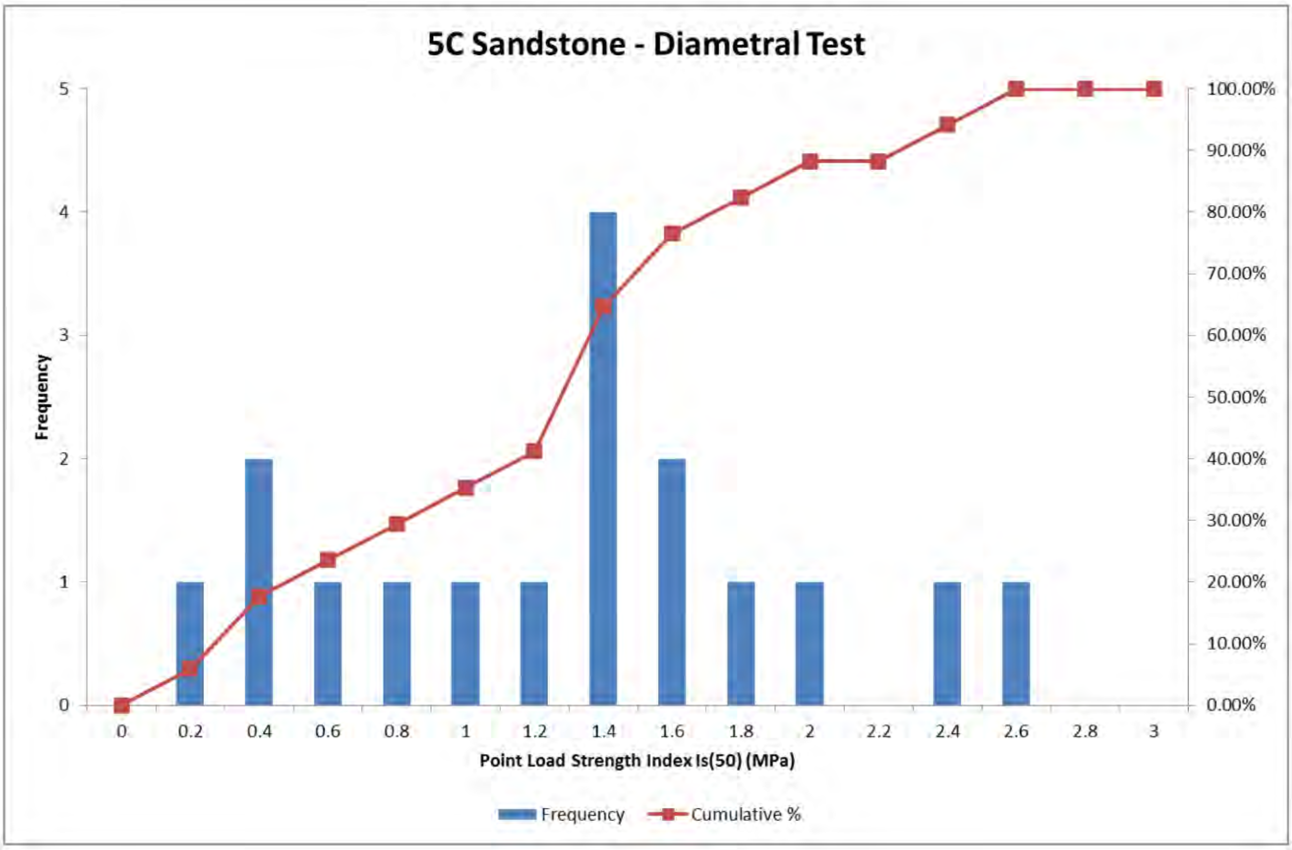


Figure 13: Point Load Test Results for Unit 5C

3.13 Rock Defects

Generally the rock defects encountered during the field investigation were associated with the bedding features in the sedimentary rocks. The majority of defects dip between 0° and 15°. Based on the borehole information it is difficult to assess the presence of any major defect sets that may be present, other than the sub-horizontal bedding defects.

A typical characteristic of weathered sandstone and shale is planar weathered seams running parallel with bedding. These defects are a major factor in the engineering classification of rocks in the Sydney Basin using Pells (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. In general, on the MPW site the rock conditions immediately below rock-head level include weathered seams, but rock quality generally increased rapidly within 1 to 3 m. This appears to be consistent with the generally thin residual profile over the site inferred to be due to erosion.

3.14 Acid Sulphate Soils

An extract from acid sulphate soil mapping is attached as Figure A040. In general this shows recent alluvial soils within or close to the Georges River as having the greatest risk of containing acid sulphate soils. Further discussion of acid sulphate soils is included in the Golder ESAR report (Golder, 2015b).

3.15 Summary of Ground Conditions

The MPW site has a relatively thin surficial fill layer (i.e. Unit 1 materials per Table 4 above), generally being approximately 0.5 m thick, but up to 4 m or more in some areas of the site, generally related to filling pre-existing depressions in the site or disposal of waste materials (typically Units 1B and 1C). There is a relatively rapid transition to stiff / dense alluvial deposits, comprising sands or clays (Units 3A and 3B). In general greater depths of alluvial material were encountered towards the northern end of the site (up to approximately 20m) compared to the south (typically 10m or less). Although both sands and clays were interbedded, which is consistent with the variable alluvial conditions under which they were deposited, the proportion of sand was found to be greater towards the northern end of the site (with some locations comprising nearly all sand) than at the southern end (where selected locations comprised only clay). These soils exhibited a low potential of erodibility when subjected to water.

Ashfield shale rock (Units 4B and 4C) was generally found below the overlying alluvium for the majority of the site area (to depths of up to 25 m). The exception to this is the southern end of the site, where Hawkesbury sandstone was observed (Unit 5C) below the overlying alluvial material. The shale rock forms a cap above the sandstone. The depth to rock varies between approximately 8 m to 21 m below existing ground level. The results from the current investigation appear consistent with earlier seismic refraction surveys completed by PB in 2011, which indicated rock levels varying by a similar range as the existing survey, with a maximum rock elevation difference over the survey runs completed of about 10 m.

4.0 DESIGN PARAMETERS

4.1 Design Loading

The following design loading assumptions have been adopted in this report:

- Floor loads of warehouses, 40 kPa;
- Pad or strip footing loads, >150 kPa; and
- Ground levels to be raised to achieve a typical design level of RL16m (Arcadis, 2016a).

4.2 Performance Criteria

The following performance criteria have been considered in the preparation of this report:

- Long term post-construction differential settlements of top of the surface (in areas of fill or virgin material) equal to or less than 1 in 400 over 30 years;
- Future industrial lots may be subjected to characteristic ground movements similar to those anticipated for a Class M site as defined in AS 2870 – Residential Slabs and Footings.

4.3 Ground Stiffness

Ground stiffness parameters (modulus) are required for the estimate of foundation performance (settlement) and the design of piles.

Our assessment of ground stiffness parameters has focused on Unit 3A and Unit 3B materials, as these materials, along with the nature of the new fill used to form the Earthworks Platform (including the Structural fill layer and any underlying General Fill) are most likely to influence foundation performance. Our assessment of ground stiffness has been made directly, or indirectly using published correlations, from the results of in situ testing: SPT “N” values, CPT cone tip resistance, dilatometer (DMT) tests and downhole seismic testing.

Ground stiffness is a soil property which is strain dependant. Where strains are small the soil stiffness tends to be high and conversely where strains are large the soil stiffness reduces. Different site investigation techniques assess soil properties at different strain levels; we have included an indicative summary of the testing methods used during the current investigation below:

Table 8: Indicative Strain Levels for Investigation Techniques

Testing Method	Approximate Strain Level	Comment
Geophysics	0.0001 to 0.001 %	Maximum Modulus (E_0)
DMT	0.01 to 0.1 %	Used for deformation analyses
CPT, SPT	1 to 10 %	Used for bearing capacity and stability analyses

Due to this strain dependency, there can be a large variation in modulus for the same material. In assessing appropriate parameters to use, the nature of the material and the type of assessment required need to be taken into account. Hence, while for assessment of stability or bearing capacity mechanisms a lower modulus may be used, for some deformation analyses a higher value may be appropriate.

Plots showing our interpretation of soil stiffness versus depth for the granular Unit 3A soils and clay of Unit 3B are presented in Figure 14 to Figure 16, below:

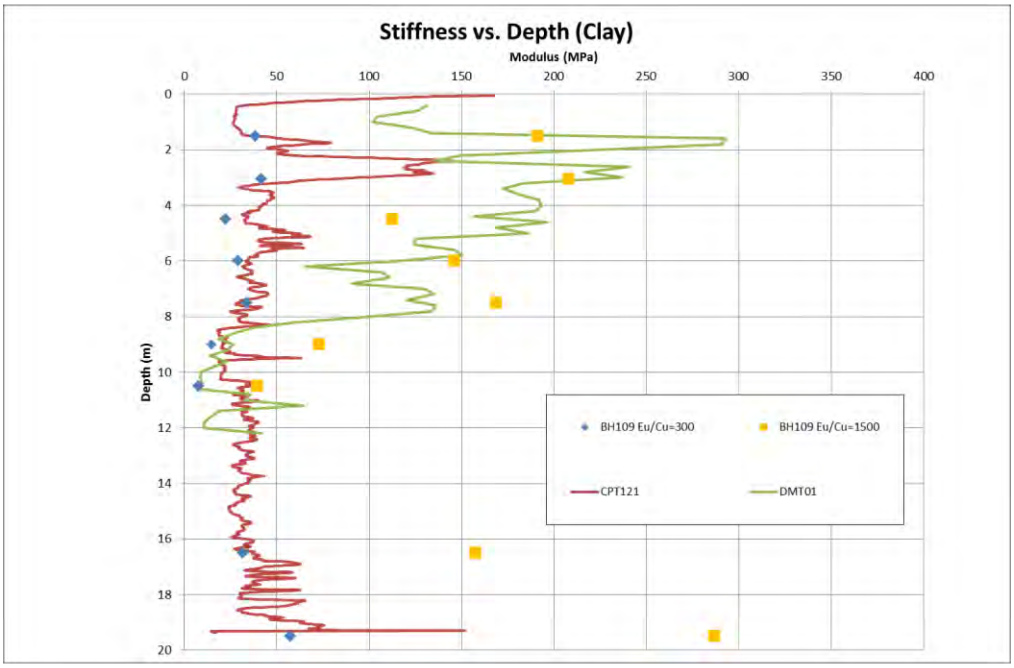


Figure 14: Stiffness vs Depth Plot for BH109 / CPT121 / DMT01 (Unit 3B Clay Profile)

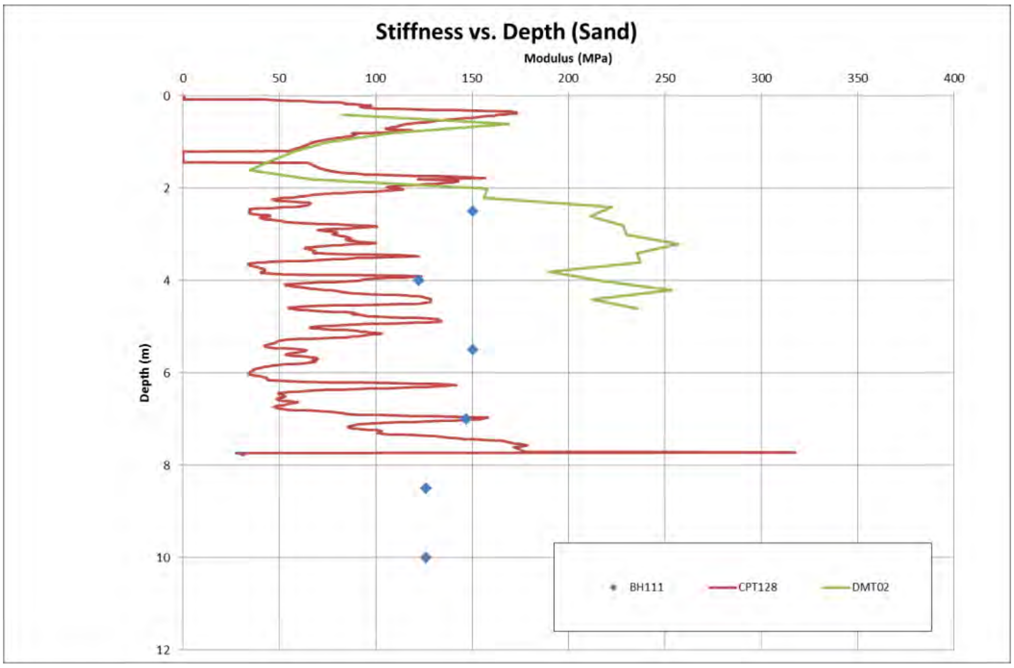


Figure 15: Stiffness vs Depth Plot for BH111 / CPT128 / DMT02 (Unit 3A Sand Profile)

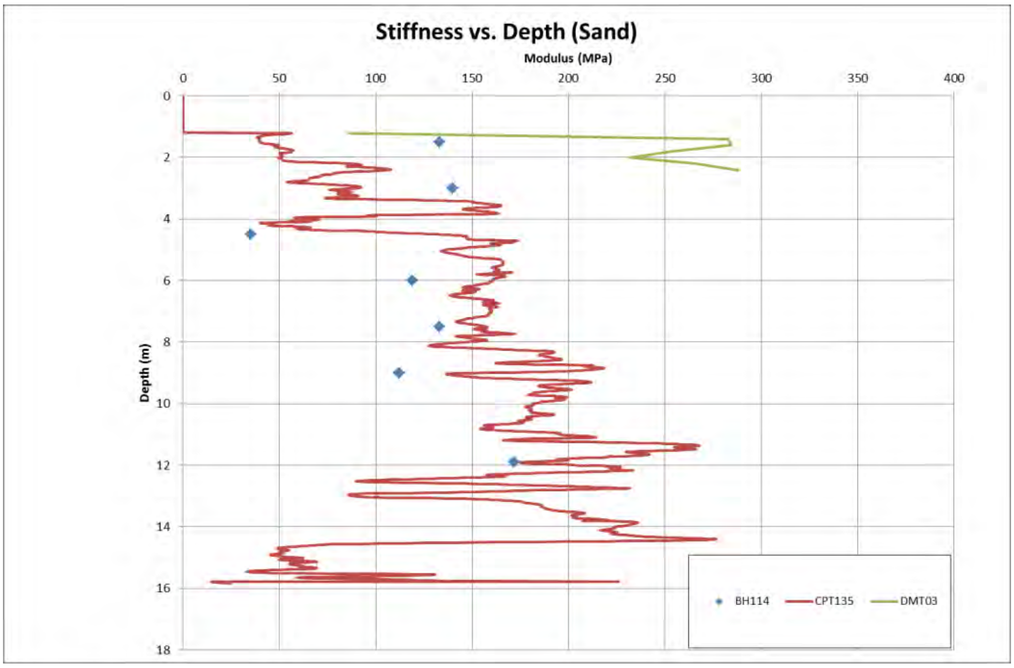


Figure 16: Stiffness vs Depth Plot for BH114 / CPT135 / DMT03 (Unit 3A Sand Profile)

4.4 Geotechnical Engineering Parameters

Design parameters are nominated in Table 9, below.

4.5 Geotechnical design parameters

Table 9: Design Parameters

Unit	Description	Moist Unit Weight γ (kN/m³)	*Undrained Strength* Su (kPa)	*Undrained Strength* Φ_u (°)	*Drained Strength* c' (kPa)	*Drained Strength* Φ' (°)	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]	Overconsolidation Ratio OCR	Serviceability End Bearing Pressure (kPa) [6,7]	Ultimate End Bearing Pressure (kPa) (rock only)	Ultimate Shaft Adhesion (kPa) [6, 7]
1A	Topsoil	16	N/A	N/A	0	25	NA	5	0.3	0.5	0.5	2.2	N/A	N/A	N/A	N/A
1B	Anthropogenic Fill	17	NA	NA	0	28	N/A	5	0.3	0.6	0.4	2.5	N/A	N/A	N/A	N/A
1C	Granular Fill	18	N/A	N/A	0	32	N/A	15	0.3	0.5	0.3	3.2	N/A	N/A	N/A	N/A
1D	Cohesive Fill	18	75	0	0	25	15	10	0.3	0.5	0.5	2.2	N/A	N/A	N/A	N/A
2A	Loose Sand	18	N/A	N/A	0	30	N/A	10	0.3	0.5	0.3	3	N/A	N/A	N/A	N/A
2B	Firm Clay	18	30	0	0	23	7	5	0.3	0.6	0.5	2	N/A	N/A	N/A	N/A
3A	Dense Sand	20	N/A	N/A	0	38	N/A	100	0.3	0.7	0.25	4	N/A	300Z (Max 2,500)	N/A	4Z (Max 60, bored) 8Z (Max 120, driven)
3B	Very Stiff Clay	20	150	0	5	28	55	40	0.3	1.2	0.3	2.8	3	200 (shallow footing) 400 (pile footings)	N/A	75
4A	Residual Shale Soil	20	150	0	5	28	40	40	0.3	1.2	0.3	2.8	3	400 (pile footings)	N/A	75
4B	Extremely Low to Low Strength Shale	22	N/A	N/A	25	35	N/A	150 to 500	0.25	1.2	0.2	4	N/A	700 to 1,500	1,500 to 3,000	100 to 300
4C	Shale of medium strength or higher	24	N/A	N/A	50	40	N/A	500 to 1,000	0.2	-	-	-	N/A	4,000 to 8,000	10,000 to 20,000	300 to 1,000
5A	Residual Sandstone Soil	20	150	0	5	28	40	40	0.3	1.2	0.3	2.8	3	400 (pile footings)	N/A	75
5B	Very Low to Low Strength Sandstone	22	N/A	N/A	50	35	N/A	200 to 600	0.25	1.2	0.2	-5	N/A	1,000 to 1,500	3,000 to 5,000	300 to 500
5C	Sandstone of medium strength or higher	24	N/A	N/A	100	42	N/A	600 to 2,000	0.2	-	-	-	N/A	8,000 to 12,000	20,000 to 50,000	300 to 2,000

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 16, 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 16, below.
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 9 below, including serviceability and ultimate limit state end bearing and ultimate limit state shaft resistance.
7) 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 9.
8) 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.

For Units 2A and 3A **For Units 1, 2B and 3B**

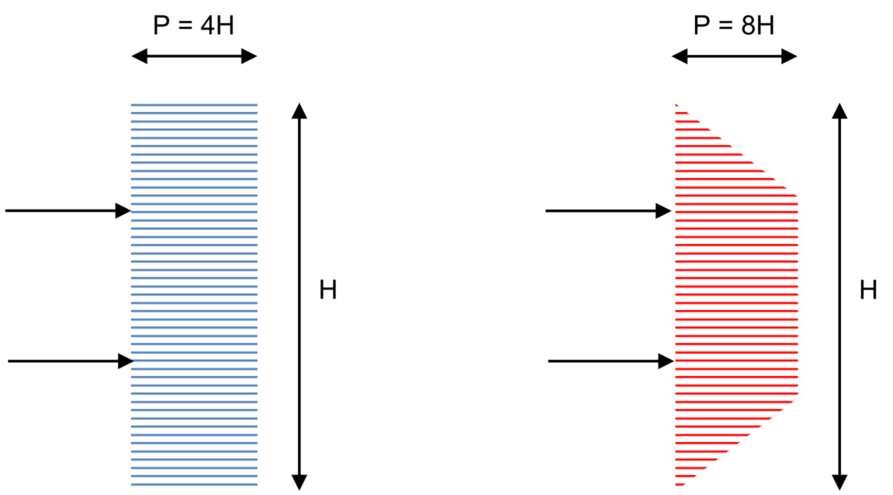


Figure 17: Alternative Earth Pressure Envelopes

4.6 Soil, Rock and Water Aggressivity to Concrete and Steel

The laboratory test results for aggressivity testing 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 10.

Table 10: Aggressivity Exposure Classification

Sample ID	Exposure Classification			
	For Concrete Piles		For Steel Piles	
	Above Groundwater	Below Groundwater	Above Groundwater	Below Groundwater
BH111 0.3-2m (Unit 1C)	Non-aggressive	Mild	Non-aggressive	
BH111 8.5-8.95m (Unit 3A)	Mild	Moderate	Non-aggressive	Mild
BH101 1.5-1.95m (Unit 3B)	Mild	Moderate	Non-aggressive	Mild
BH107 0.20-0.35m (Unit 1C)	Non-aggressive	Mild	Non-aggressive	
BH109 7.50-7.95m (Unit 3B)	Non-aggressive	Mild	Mild	Moderate
BH109 10.50-10.95m (Unit 3B)	Mild	Moderate	Moderate	Severe
BH110 0.65-0.95m (Unit 1C)	Mild	Moderate	Non-aggressive	Mild
BH112 0.07-0.60m (Unit 1C)	Mild	Moderate	Non-aggressive	
BH112 1.50-1.95m (Unit 3B)	Non-aggressive	Mild	Non-aggressive	Mild
BHBF 0.00-0.30m (Unit 1C)	Mild	Moderate	Non-aggressive	
BHBI 0.30-1.0m (Unit 1C)	Mild	Moderate	Non-aggressive	
BH114 6.00-6.45m (Unit 3A)	Mild	Moderate	Non-aggressive	Mild
BH114 14.90-15.21m (Unit 3A)	Mild	Moderate	Non-aggressive	Mild
GA-TP-3102_002 (Unit 3A)	Non-aggressive	Mild	Non-aggressive	
GA-TP-3104_03 (Unit 1B)	Non-aggressive	Mild	Non-aggressive	
GA-TP-3106_001 (Unit 1C)	Non-aggressive	Mild	Non-aggressive	
GA-TP-3107_02 (Unit 3A)	Non-aggressive	Mild	Non-aggressive	
GA-TP-3111_03 (Unit 1C)	Non-aggressive	Mild	Non-aggressive	
GA-TP-3112_01 (Unit 1B)	Non-aggressive	Mild	Non-aggressive	
GA-TP-3118_003 (Unit 3B)	Non-aggressive	Mild	Non-aggressive	
GA-TP-3120_002 (Unit 1B)	Non-aggressive	Mild	Non-aggressive	

In general, exposure classifications for the site above groundwater level are non-aggressive to mild for concrete and steel piles. Below the groundwater table, exposure conditions are more severe, with moderate exposure conditions for concrete piles and mild to severe exposure conditions for steel piles.

The soil exposure classification for both concrete and steel piles is governed by acidic pH values. These findings appear consistent with groundwater monitoring results and the potential for acid generation in the soils. Refer to the ESAR (Golder, 2015b) for further discussion of this.

4.7 Earthquake Parameters

4.7.1 Design Earthquake (PGA)

The subsurface profile 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).

4.7.2 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 a 1 in 500 year AEP event.

4.8 Erodibility of Soil and Weathered Rock

Unit 1C and Unit 3B exhibited low to no dispersive potential during laboratory testing. Unit 4 and Unit 5 rocks are also typically non-dispersive. Re-moulding of Unit 4B and Unit 4C at a moisture content near optimum (i.e. excavation and re-compaction) does not increase potential for dispersive behavior, however further breakdown of the soil may occur, by water turbulence or concentrated rapid water flow. We therefore recommend that these materials not be exposed to concentrated water flow over or through the soil profile (e.g. by lining drainage channels).

5.0 EXCAVATIONS

5.1 Excavation Conditions

Temporary excavations will be required for the removal of existing redundant structures, services and unsuitable soils. Excavations up to 4 m deep may be required for installation of new drainage and sewer systems for the site.

The fill deposits on the site are generally up to 2 m deep. There may be localised site areas, possibly in in-filled former valleys or in areas with Unit 1B waste fill, where a greater excavation depth is required to remove unsuitable soils.

A conventional bulldozer or hydraulic excavator can be used to excavate the Unit 1 surficial soils. Removal of obstructions in the fill such as building foundations may require the assistance of a rock breaker.

If Unit 2 soils have to be removed, then a conventional bulldozer or hydraulic excavator should be able to excavate the material. Some pre-treatment or drying of the material may be required at the time of excavation to make the material easier to handle for re-use or disposal.

If excavations need to extend into the Unit 3 soils, iron cemented bands may be encountered at shallow depth. A rock breaker or a dozer with ripper may be needed to excavate through the iron cemented bands.

Emerson Crumb testing indicates relatively low erosion potential, but there are soils in the local area that have been eroded due to surface run-off. Where possible, topsoil and grassed areas should be left in place until construction works start.

5.2 Vibration

Care should be taken during excavation (and backfilling compaction) to limit the vibration impacts on new structures that may be built as a part of progressive staging of the development works. In addition, the potential vibrations from construction, such as driving piles, impact roller compaction or use of a hydraulic rock breaker may need to be considered with respect to buried services, nearby commercial, industrial, 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.

5.3 Groundwater

Groundwater beneath the MPW site area was about 8 to 12 m below the existing ground levels at the time of the geotechnical investigation, which is deeper than the expected depth of excavations.

However, higher water levels were encountered in the vicinity of established ponds on the site (e.g. 0.8m below surface at GA-BH-3102 and 2.8m below surface at GA-TP-3112). Relatively higher groundwater was also encountered in the vicinity of Anzac Creek at GA-CPT-3116, where groundwater was recorded at approximately 2m below ground level.

Groundwater is likely to be encountered within the depth of bored piles, if used (see Chapter 8.3.2).

Groundwater monitoring was carried out by PB (PB, 2011) and a monitoring round was completed by Golder (Golder 2015b). The results of the PB groundwater monitoring indicated the groundwater within the alluvial soils generally flows westwards towards the Georges River with groundwater levels recorded in 2011 of between RL6mAHD and RL2mAHD. These results are consistent with the results of the Golder monitoring.

As the alluvial soils on the site contain granular horizons, there may be seasonally elevated perched water tables in fill materials and sand layers. These perched water systems could impact retaining walls, excavations for slopes and foundations. Elevated or perched groundwater levels are also expected in the vicinity of established ponds on the MPW site. Perched groundwater inflows could potentially lead to softening of natural alluvial clays in footing excavations, so concrete for footings should be placed as soon as practicable. Potential for perched groundwater should be considered in the design of slopes and retaining walls and control measures such as sump pumping may be required during construction.

5.4 Surface Water Management

Management of surface water will be required during earthworks. Management methods to limit impacts of water on the proposed excavation 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.

5.5 Excavation Support Requirements

Recommendations on suitable batter slopes are provided in Chapter 6.7. In areas of the 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 9, above.

However, 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 17 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. Any 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.

6.0 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 8.0.

There is an opportunity for re-use of site won material as General Fill (i.e. engineered fill below the Structural Fill layer) and this is discussed in Chapter 6.2 and 6.3.

Dependent on the final performance requirements adopted for detail design, it may also be possible to leave some of the relatively low organic content topsoil layer in place as discussed in Chapter 6.3.

6.1 Stripping of Unsuitable Material

Prior to placing new fill materials, the existing Unit 1A topsoil should be stripped from the surface of the site, in accordance with a site specific Earthworks Specification (which may provide for assessment and further sub-division of Unit 1A for foundation preparation purposes). Subject to assessment of suitability from a contamination viewpoint, stripped topsoil should be stockpiled for reuse in landscaped areas of the site.

Based on the recorded properties of Unit 1A, opportunity exists for incorporation of the lower topsoil in General Fill, subject to UXO/EOW and contamination considerations and a sufficiently low final organic content being achievable. With appropriate blending a high proportion of the topsoil encountered (excluding the surficial layer comprising a high proportion of vegetative matter, such as the root mat for areas of turf) should be able to be re-used as General Fill, subject to the performance requirements of overlying development and the heights of filling required.

Unit 1B anthropogenic fill should be managed in accordance with the Remediation Action Plan (RAP).

The extent to which the topsoil and anthropogenic fill is removed should be undertaken in consideration of the performance requirements of the area and the nature of the topsoil and anthropogenic fill in that area.

Alternative options for areas containing anthropogenic fill include:

- Excavating, sorting and then re-using the Unit 1B as fill material.
- Excavating and replacing the Unit 1B material, with excavated 1B material either:
  - re-used on site below landscaped areas of the site.
  - disposed off site.
- Improving the Unit 1B material *in-situ*, using methods such as high energy impaction compaction.

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

It is anticipated that the main areas where unsuitable material requiring treatment or removal and replacement will be at or in the immediate vicinity of anthropogenic ‘hotspots’ and established ponds.

6.2 Existing Fill Materials

The fill encountered during investigation of the MPW Stage 2 site was found to typically be about 0.5 m to 1.2 m thick, comprising mainly sand or clayey sand. It is possible that deeper fill, with poorer compaction, is present locally. Locally at anthropogenic ‘hotspots’ fill up to approximately 5m deep was encountered.

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. There is some uncertainty as to 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.

Most of the existing fill encountered on site is mainly granular (sandy). 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 deleterious inclusions. Unsuitable materials that should not be used as General Fill include:

- Topsoil and 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).

Where Unit 1C fill needs to be excavated to level sections of the site, it could potentially be reused on site as General fill to refill areas that have been excavated (for example old pond areas or areas where Unit 1B fill has been removed). Additional testing and screening of this material may be required on site during construction to comply with the Earthworks Specification.

6.3 Management of Existing Fill Materials

It is noted that in areas of proposed filling, the impact of the underlying existing fill could be mitigated by the thickness of the new fill above. For example, where the thickness of overlying Structural Fill is such that loading is carried substantially within the new fill. However, such benefits would need to be considered in light of the specific design details (e.g. pavement, warehouse slab and foundation requirements, including footing width and depth).

With respect to Unit 1C (and 1D material where encountered), due to its shallow and moderate, but variably compacted nature, the opportunity exists to leave this material in place, where adequate thickness (and quality) of overlying fill can be provided. This would be subject to adequate compaction being achieved and zones of unsuitable material being identified and treated. Management in accordance with a site specific Earthworks Specification would be necessary. The Earthworks Specification must include a means for assessing and treating the foundation to overlying fill for adequacy. Different 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 include:

1) Excavation and replacement of some or all of the existing fill in accordance with an engineering specification. This would be the lowest risk option.

2) As an alternative to excavate and replace, and if further geotechnical investigation indicates it is viable, and compatible, soil improvement using conventional or High Energy Impact Compaction (HEIC). HEIC has the benefit of being able to compact soils to greater thickness than conventional rollers. In our experience compaction to at least 2 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.

Unit 1C (and 1D material where encountered) presents an opportunity for re-use as General Fill. This would be subject to meeting the requirements of the Earthworks Specification and UXO/EOW and contamination considerations.

The need for a high quality Structural Fill layer below areas of new development to satisfy that performance criteria of Chapter 4.2 is discussed in Chapter 8.0 below. Within the areas proposed for excavation and the depth ranges envisaged material appropriate for use as Structural Fill to meet the performance criteria of Chapter 4.2 is not anticipated to be available from site won material.

Based on our current understanding of design earthworks levels (Arcadis, 2016a) sufficient volume exists within the areas of filling to accommodate the full volume of excavated material from cut areas whilst still maintaining allowance for an overlying Structural Fill layer.

Prior to placing new fill materials, the existing Unit 1A topsoil, or portion(s) thereof, should be stripped from the surface of the site in accordance with a site specific Earthworks Specification and Unit 1B fill should be treated or removed as required by the RAP and Earthworks Specification. Topsoil should be stockpiled for reuse in landscaped areas of the site where contamination considerations allow. As discussed in Chapter 6.1, an opportunity exists for re-use of lower topsoil layers as General Fill, subject to adequate blending to achieve acceptable organic content and conformance with the Earthworks Specification.

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, however this layer could 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 2 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.

6.4 Imported Fill Materials

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 currently active tunnelling projects in the Sydney metropolitan area. If tunnel spoil is to be used, then it may not require crushing, possibly only screening to remove large rocks. We understand that on previous projects, fresh sandstone spoil from the Cross City Tunnel was placed directly into reinforced soil walls from trucks without screening or moisture conditioning.

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. As discussed later in the report, the geotechnical analyses presented in this report have been developed on the basis that a layer of sandstone fill would be used.

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 MPW 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.

Unsuitable materials that should not be used as engineered fill include:

- Topsoil and 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).

6.5 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. We note that sandstone spoil can have a tight compaction curve and moisture contents above optimum can lead to heaving in the sandstone layers, this should be considered when developing an earthworks specification for the site, tighter moisture conditioning requirements may be required for some materials.

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 topsoil and treatment of the subgrade (as described in Chapter 6.1 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. 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 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.

6.6 Bulking Factors

We suggest selecting values from Table 11, which are based on a combination of published values and experience with local materials.

Table 11: Suggested Bulking / Compaction Factors

Unit [1]	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)
2 and 3	Quaternary Alluvium, Fill	Cohesive / granular	Mainly Firm to Stiff / loose to dense	1.1-1.3	0.9-1.1
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. Excludes fill materials, for which bulking factor is uncertain due to intrinsic variability.
2. Based on estimated values published in McNally (1998).

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 11.

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.

6.7 Cut and Fill Batter Slopes

In accordance with Chapter 1.1, we understand that existing structures on the site will be demolished and removed as part of the Early Works. Dependent on the staging of the works, 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 Chapter 6.7, below, but these are only recommended in areas that do not have nearby movement sensitive structures or services.

Table 12: Recommended Batter Slopes (excavations / slopes up to 3 m)

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

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

If slopes other than those in Table 12 are to be used, or higher slopes 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 any vehicles, plant or structures at the crest of the slope.

6.8 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 9 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.

7.0 PAVEMENTS

Internal access roads on the site are proposed to carry several thousand fully loaded B-double vehicles per day. 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 pricing of various pavement configurations, we have carried out preliminary pavement thickness designs using the following parameters and assumptions:

- Subgrade conditions based on a soaked CBR value of 3%, which is the average subgrade CBR value obtained from laboratory tests. Adopting the average subgrade CBR strength implies that there is a 50% probability that subgrade is weaker or stronger than assumed. A lower design CBR value may need to be considered during detail design to reduce the likelihood of early pavement failure and improve design reliability. Based on the limited laboratory test results available to date on the Unit 3B material, a design subgrade CBR of 2% would reduce the risk of early pavement failure from 50% to 10%.
- We have considered the effect on pavement thickness for a subgrade CBR value of 10% reflecting improved subgrade strength in areas of imported granular fill. For this increased subgrade CBR value to apply, the granular fill should be at least 600 mm thick.
- The suggested number of daily truck passes will result in a high number of design axle repetitions. A review of whole of site traffic movements will allow refinement of vehicle passes and design axle repetitions and optimisation of pavement thickness design.
- We have considered a pavement design life of 10, 15 or 20 years. For this site, once more information is known on vehicle movements, a design life of 30 years may need to be considered.
- We have considered two different pavement profile types, as follows:
  - Fully flexible pavement with thin asphalt surfacing (non-structural wearing course) and granular base and sub-base. This option has lower capital cost but higher maintenance costs for the wearing surface.
  - A thick asphaltic concrete pavement with cement stabilised base and granular sub-base. This option has higher capital cost but lower maintenance costs.

7.1 Design Traffic Calculation

Preliminary design traffic calculations have been carried out based on the following:

- 6000 B-double truck passes per day and no growth rate per year on the number of vehicle passes. All trucks are assumed to be travelling fully loaded, in one direction within one lane.
- B-double axle configuration comprising one 6 tonne single axle with single wheels (SAST), a 16.5 tonne tandem axle with dual wheels (TADT) and two 20 tonne triple axles with dual wheels (TRDT).

It is noted that the above assumptions are understood to be reflective of a high level ultimate precinct external traffic trip estimate. As such those numbers may be greater than the final detail design stage traffic volume calculated for and appropriate for use in detail design of pavements within the MPW precinct itself.

Using the Austroads Guide to Pavement Technology (2012), the following table summarises design axle repetitions for design life of 10, 15 and 20 years considering the two pavement options outlined above.

Table 13: Summary of Design Axle Repetitions

Design Life	10 years	15 years	20 years
Design ESA for Empirical Design	1.3×10^8	1.9×10^8	2.5×10^8
Design SAR_7 for Subgrade Failure,	1.4×10^8	2.1×10^8	2.8×10^8
Design SAR_5 for Asphalt Fatigue,	1.8×10^8	2.8×10^8	3.7×10^8

Design Life	10 years	15 years	20 years
Design SAR_{12} for Cracking of Cemented materials	$3.9x10^8$	$5.9x10^8$	$7.8x10^8$

7.2 Preliminary Pavement Thickness Design

Based on the design traffic summarised above we have carried out a number of mechanistic pavement design analyses using the commercially available pavement design software CIRCLY. Table 14 and Table 15 below summarise the results of these analyses as preliminary options for pavement thickness design. Pavement materials considered in preliminary analysis included:

- Unbound gravel layers (base and sub-base layers) with young's moduli values ranging from 150 MPa to 500 MPa
- Asphalt with a young's modulus of 2,000 MPa
- Heavily bound cemented sub-base with a young's modulus of 5,000 MPa.
- Subgrade of CBR 3% or CBR 10%.

Table 14: Summary of Preliminary Pavement Thickness Design – Subgrade CBR 3%

	Layer Thickness (mm) for Subgrade CBR 3%				Total Pavement Thickness (mm)
Design Life	Full Depth Granular	Structural Asphalt	Cemented Base	Granular Subbase	
10 years	730	-	-	-	730
	-	250	200	300	750
15 years	750	-	-	-	750
	-	250	200	450	900
20 years	770	-	-	-	770
	-	250	200	600	1,050

Table 15: Summary of Preliminary Pavement Thickness Design – Subgrade CBR 10%

	Layer Thickness (mm) for Subgrade CBR 10%				Total Pavement Thickness (mm)
Design Life	Full Depth Granular	Structural Asphalt	Cemented Base	Granular Subbase	
10 years	420	-	-	-	420
	-	175	200	150	525
15 years	435	-	-		435
	-	185	200	150	535
20 years	450	-	-	-	450
	-	190	200	200	590

7.3 Other Considerations

Consideration would need to be given to stabilisation (e.g. by lime or cement) of the upper 300 mm of subgrade in areas where a CBR of 3% or less is anticipated. This will improve moisture stability of the subgrade and improve pavement performance.

Where pavement is constructed on Unit 3B material, which is 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 pavement should be finished with suitable cross-fall and adequate surface drainage to minimise ponding on the surface of the pavement.

All pavement materials should satisfy RMS requirements, in particular, Specification QA3051. For the full depth granular pavement, the upper base layer should be a minimum of 200 mm. The materials should be compacted in loose layers not more than 150 mm or less than 100 mm at 100% Modified Maximum Dry Density (MMDD) in accordance with RMS Specification R71.

7.4 Container Terminal Areas

Designers of future container terminal should use laboratory testing results in this report and the GDR (Golder 2016a) 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.

8.0 STRUCTURAL FOOTINGS

8.1 Site Classification

The advice below is based on the current proposed site arrangement. Due to the variability in soils below the site, the site classification should be considered for each separate development area of the site and additional testing at each lot may be required, depending on the final locations of structures.

Most of the proposed development will comprise commercial buildings. Advice on site classification has been provided in this report with reference to AS2870, the scope of which covers industrial and commercial buildings of similar scale to residential properties. Specific assessment of appropriate investigation and testing densities and methods will be required once the area, extent and articulation characteristics of slabs and footings are further developed. The minimum number of exploration positions nominated within Clause 2.4.4 of AS2870 will be inadequate for warehouse slabs of the scale contemplated for the MPW Project. Accordingly, in lieu of detailed investigation and assessment, the most conservative (i.e. greatest shrink-swell potential) classification should be adopted from the available data.

8.1.1 Granular Soils

In areas of the site, where granular materials are present over the top 2 m below the final surface level of the site, a site classification of Class S is considered appropriate. This assumes that new granular Structural Fill comprising ripped or crushed sandstone is placed below structures to the surface of the granular material.

8.1.2 Cohesive Soils

AS2870 Table D2 indicates that Sydney sites underlain by clay soils greater than 1.8 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. Our initial calculations of shrink/swell movements using the method prescribed in AS2870 are discussed below.

The shrinkage index of samples tested ranged from 0.7 to 2.8. The testing was completed on tube samples and as it was difficult to retrieve samples within some of the very stiff / dense materials on the site, these results may be biased towards softer or more plastic soils.

Assuming that at least 1 m of Structural Fill comprising a ripped or crushed sandstone is provided below the level of warehouse floor slabs and foundations, and the fill is not susceptible to movement caused by moisture changes, we consider that a Class M classification is appropriate for the site.

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.

8.1.3 Uncontrolled Fill

Areas containing Unit 1B or 1C 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 8.1.1 and 8.1.2 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.

8.2 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 engineered 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 8.1. We do not recommend supporting footings in the existing fill or Unit 2 clays or sand because of the risk of unacceptable total and differential settlements, unless the fill has been treated as described in Chapter 6.0, over the depth of influence below footings, so that it can be considered “engineered fill”. 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.

8.3 Warehouse Foundations

We anticipate that column loads from the proposed warehouses will be 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 between geotechnical and structural designers during the detailed design phase.

8.3.1 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 engineered 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 1.3B 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 3A or Unit 3B 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 may result in footing excavations at least 2m deep to penetrate through new fill and the existing fill.

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 (See Chapter 8.3.2).

8.3.2 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 would be most appropriate for use on the site, given the presence of iron cemented bands within the Unit 3A and 3B soils. CPTs consistently refused on these layers, and this can be a good indicator of where driven precast concrete piles will also refuse. Bored piles should be able to penetrate these layers, however, piling contractors should be asked for advice about the most appropriate drilling methods to penetrate these layers.

If bored piles extend below the groundwater table (at about 9 to 12 m depth), they will likely need to be cased to prevent groundwater inflow and maintain wall stability prior to concreting. As discussed in Chapter 5.3 groundwater may be encountered at higher levels, particularly in the vicinity of Anzac Creek and established ponds. If piling is proposed in these areas precautions against groundwater inflows may be encountered at relatively shallow depths.

Depending on the magnitude of loading required, driven piles may be able to be used for some structures as the capacity achieved can be assessed immediately after the driving process. In general we consider that this would be a riskier option if uniform load-displacement behaviour is required from the piles (i.e. driven piles should be avoided for a piled raft) but they could be considered as an option for some structural columns. These pile types would likely refuse on iron cemented layers as discussed above, which could lead to different piles or piles groups resting at different levels, the design of the structure supported by driven piles would need to have the potential to accommodate this.

Continuous flight auger (CFA) piles could be a good option for the site, with either single piles or pile groups used, depending on the magnitude of column loads. CFA piles 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 3A/3B may be possible. Alternatively, piles could be advanced into bedrock, with the achievable depths depending on the equipment being used and the experience of the piling contractor. CFA piles may be able to be advanced into Unit 4C shale and 5B sandstone, based on experience on recent projects. 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.

The installation method for CFA piles requires elevated concrete pressures during concreting. This can lead to higher shaft and base resistances obtained being higher than those normally adopted for bored piles. This may be considered in pile design for the site, and should be verified by pile testing (PDA and CAPWAP).

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.

8.3.2.1 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 16, below.

Table 16: 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 piles
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	

8.3.2.2 Pile Load Tests

Pile load testing should be completed in accordance with the recommendations in AS1259. 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.

8.3.2.3 Supervision of Bored Piles 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.

9.0 SETTLEMENT ASSESSMENT

We have reviewed our initial settlement assessment, (which was based on the MPW Stage 1 investigations and presented in Figure A041) for the additional investigation undertaken for the MPW Stage 2 Proposal, using the proposed indicative cut to fill diagram provided by Arcadis (Arcadis, 2016a) which assumes a 1 m Structural Fill Earthworks Platform is provided between the stripped earthworks surface and the underside of pavement level.

One dimensional settlement calculations have been used to assess potential settlement under loading comprising changes in ground level, plus slab loading of 40 kPa. The soil stiffnesses used in calculations were based on results of 57 CPTs that are inferred to have refused close to or on rock. The soil stiffness assumptions used in these calculations are as shown in Table 9.

Preliminary estimates of settlement under the slabs of proposed large scale industrial warehouses are in the range of 5 mm to 35 mm. South of GA-CPT-3111 estimated settlements are typically less than 10 mm. North of GA-CPT-3111, estimated settlements are typically in the range of 10 mm to 30 mm. Estimated settlements at GA-CPT-3102 were well in excess of 50 mm, reflective of the poor ground conditions encountered over the upper 4m. However, it is noted that this is an area of a known drainage feature/pond and outside the currently proposed earthworks zone. It is illustrative of the need to carefully identify and manage zones of potentially poor ground/fill (such as may be associated with established ponds) in accordance with a site specific Earthworks Specification,

The magnitude of the calculated settlements within the zone of proposed development is likely to be 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 of foundation materials over the depth of influence associated with any give strip / slab loading. The study would also need to delineate uncontrolled fill areas within the vicinity of structures (if not already completed) and take account of imported fill being used to replace existing uncontrolled fill.

10.0 GEOTECHNICAL RISKS AND OPPORTUNITIES

- There are likely areas of Unit 1B fill that have not been found during the geotechnical and ESAR investigations. This would require proactive management and good geotechnical supervision on site to identify and address each occurrence. The opportunity would be to plan for this eventuality prior to construction commencing so well understood procedures are in place during construction.
- Removing anthropogenic fill and retain on site in a contained area (for example the “dust bowl”, or potentially below a stormwater detention basin (assuming contamination risks are acceptable).
- Subject to removal of vegetation (e.g. turf cover and trees) and the root affected zones (potentially extensive for mature trees) and compaction/ground improvement, largely leaving the ‘topsoil/fill’ layer in place could be feasible. This would reduce the scale of risk across the overall site relating to interaction of potential contamination and UXO/EOW, as some areas may not need to be excavated other than to prepare the surface as required by a site specific Earthworks Specification. Note, for this approach to be adopted, final design levels would need to allow for the installation of an Earthworks Platform of sufficient quality and thickness to satisfy the detail design performance requirements.
- Designing piles as settlement reducing piles, rather than conventional piles. This could result in an overall reduction in pile length for the site.
- Raising site levels, such that the need for removal of potentially contaminated material is limited.

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