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REPORT
ON
GEOTECHNICAL AND HYDROGEOLOGICAL ASSESSMENT

EASTERN CREEK QUARANTINE STATION
60 WALLGROVE ROAD
EASTERN CREEK

Prepared for
AFTERON LTD

Project 71089.00
3 June 2009

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REPORT ON GEOTECHNICAL AND HYDROGEOLOGICAL ASSESSMENT
EASTERN CREEK QUARANTINE STATION
60 WALLGROVE ROAD, EASTERN CREEK

1. INTRODUCTION

This report presents the results of a geotechnical and hydrogeological assessment for the proposed development site currently occupied by the Eastern Creek Quarantine Station at 60 Wallgrove Road, Eastern Creek. The work has been conducted by Douglas Partners Pty Ltd (DP) on behalf of Afteron Ltd and their project managers, ICPS Pty Ltd.

The purpose of the study is to assess the potential impacts of the development on geology, soils and groundwater and to provide recommendations for further assessment where required.

2. SITE DESCRIPTION

The Eastern Creek Quarantine Station site has a total area of approximately 15.3 hectares (approximately 38 acres) and is approximately rectangular in shape with maximum plan dimensions of 430 m (east-west) at the northern boundary and 380 m (north-south) about one-third the way into the site from the eastern boundary. The topography of the site comprises gently undulating terrain which is governed by:

- A localised ridge line that extends from the south-western corner to the centre and then turns north to north-east and continues beyond the northern boundary.

- An east-facing slope on the eastern side of the ridge line that falls to the boundary adjoining Wallgrove Road.
- A west to north-west facing slope on the western side of the ridge line that falls to beyond the western boundary, originally to meet a meandering gully/creek within the adjoining cemetery but now interrupted by filling embankments on the cemetery land.
- Tributaries of Eastern Creek that bypass the site to its north and west and delineate the toe of the natural west-facing slopes, together with modified ground surfaces (cut and filled ground) within the adjoining cemetery.
- The alignment of Eastern Creek, which lies approximately 600 m east of the site and delineates the toe of the east-facing natural slopes, together with modified ground surfaces (filled embankments) along the M7 Motorway alignment.

Site grades are gentle with most areas exhibiting slopes of less than 10% (≈ 6 degrees). Localised slopes about the dam within the south-western corner of the site are slightly steeper and reach approximately 10 degrees.

Existing developments at site have caused only minor disturbance of the land surface by way of localised cutting and filling for the construction of building pads, pavements and associated site services. Whilst there are many buildings (office buildings, stables, kennels, catteries, greenhouses, maintenance buildings, residences, etc) on-site, most have required excavations and/or filling depths of about 1 m or less. More significant excavations and filling has occurred within the central western, south-western and southern portions of the site where developments have included the construction of dams/ponds, dressage areas, service trenching, various pits and buried tanks. Excavations and filling within these areas is estimated to affect up to 3 m or so of the current/previous land surface.

3. PROPOSED DEVELOPMENT

Afteron Ltd is proposing to develop the Eastern Creek Quarantine Station site for employment purposes. The predominant land use would be light industrial including warehousing and distribution, with ancillary office space.

Afteron Ltd, in conjunction with ICPS Pty Ltd has prepared a preliminary site layout plan for the development of the Quarantine Station site. The proposed preliminary plan of the subdivision layout is shown on Drawing 1. The plan indicates 20 industrial lots serviced by one new estate road with access from Wallgrove Road at the north-eastern part of the site. The north-western portion of the site will be retained as an ecological conservation area.

At some point in time, during subdivision construction or during individual development of each industrial lot, it is likely that earthworks will be undertaken to create a levelled building pad across each lot. This will involve cutting and filling of the land to suit the requirements of each development.

4. SCOPE AND METHODOLOGY

The objective of the assessment is to assess the potential impacts of the proposed industrial estate project on geological and hydrogeological resources, including impacts and development constraints in relation to:

- slope stability;
- site preparation and earthworks;
- salinity;
- acid sulphate soils;
- erosion and sedimentation; and
- groundwater.

The assessment is based on the results of:

- review of available published geological, soils, salinity and groundwater literature and data;
- review of geotechnical and salinity investigations of nearby lands, including DP projects for the adjacent M7 Hub, Old Wallgrove Road and the Western Sydney Parklands sites; and
- site inspection.

The current study has not involved any additional sub-surface testing or investigation. Where warranted, the study will recommend additional sub-surface investigation to be carried out during subsequent development of the concept plan.

4.1 Published Geological and Hydrological Data

The principal geological reference for the site is the Penrith 1:100 000 Geological Series Sheet (Ref. 1) and accompanying notes. Additionally, rock exposures and the results of geotechnical investigations undertaken for nearby sites provide further assistance with determining the likely geological conditions affecting this site.

The principal sources of groundwater information are the online data base of the (former) Department of Land and Water Conservation (DLWC).

Relevant results from review of the data sources described above are included within the following report sections.

4.2 Previous Douglas Partner Investigations

Previous DP investigations within the nearby M7 Hub, Old Wallgrove Road and Western Sydney Parklands sites have included cone penetration tests using a truck-mounted rig, test pits excavated by tractor-mounted backhoes, cored and non-cored bores, dynamic cone penetrometer (DCP) testing and electromagnetic surveys using a Geonics EM31 system. The locations of the nearby sites are shown on Drawing 2.

4.3 Field Work Methods

The field work for the current assessment comprised an inspection of the Eastern Creek Quarantine Station site by a senior associate (geotechnical engineer) on 27 March 2009.

Site features were considered with reference to existing site developments and a map of the site provided by the manager of the quarantine station.

5. GEOLOGICAL AND HYDROGEOLOGICAL CONTEXT

5.1 Published Geology and Soils Data

The Penrith 1:100 000 Geological Series Sheet (Ref. 1) indicates that the site is entirely underlain by Bringelly Shale of the Wianamatta Group of Triassic age. The Bringelly Shale comprises an interbedded sequence of shale, laminite, siltstone, fine sandstone and some minor coaly bands.

Geotechnical subsurface investigations at nearby sites intersected up to 4 m of clay overlying predominantly shale, with occasional siltstones and fine grained sandstones. The siltstones and sandstones were described as generally massive with limited apparent porosity and permeability and with limited fracturing or jointing.

The Bringelly Shale is mantled by alluvial deposits, comprising sand, silt and clay with some gravel bands, along the course of the nearby Eastern Creek. Geological maps indicate the alluvium to lie wholly outside the site boundary, although minor alluvial deposits may be present on site. The distribution of the closest alluvium to the site surrounds Eastern Creek and is shown on Drawing 3.

The Penrith 1:100 000 Soil Landscape Sheet (Ref. 2) indicates the site is wholly underlain by the Blacktown soil landscape unit, which is summarised as follows:

Blacktown Soil Landscape – a residual soil landscape developed on a landscape typically comprising gently undulating rises with local relief to 30 m and slopes usually less than 5% on Wianamatta Group shales. The Blacktown soils are shallow to moderately deep (<1 m), red and brown podsolic soils on crests, upper slopes and well drained areas. Deep (1.5 m – 3 m) yellow podsolic soils are located on lower areas and in areas of poor drainage. These soils are derived from weathering of the underlying (typically shaly) bedrock and are highly plastic, moderately reactive, of low soil fertility, poor soil drainage and include localised areas of salinity or sodicity and moderate erodibility.

5.2 Previous Site Investigations

The principal, relevant items of note from previous DP site investigations of nearby areas are summarised in the following sections.

5.2.1 M7 Hub

The central and western sections of the M7 Hub site are characterised by residual clay, silty clay, sandy clay or shaly clay to depths ranging between 0.5 m and 7.5 m. The clays are generally stiff to hard and contained traces of ironstone gravel and overlie highly weathered to slightly weathered shale, siltstone, and/or sandstone. The strength of the rock is generally extremely low to very low, increasing with depth to low to medium and medium strength, with some high strength materials being intersected at depth. No groundwater was encountered in the test pits and bores.

5.2.2 Western Sydney Parklands

The Western Sydney Parklands site is characterised by alluvial and residual silty clay, clay and gravely clay to depths ranging between 0.5 m and 3 m. These clays re generally stiff to hard and contained traces of ironstone gravel and overlie weathered to slightly weathered shale, siltstone and/or sandstone. The strength of the rock is generally extremely low to very low, increasing with depth to low strength. No groundwater was encountered in the test pits.

5.2.3 Old Wallgrove Road

The section of Old Wallgrove Road extending some 1300 m west from the M7 Motorway is characterised by pavement material comprising a 0.025 m thick bituminous wearing surface at the surface underlain by roadbase gravel to depths generally between 0.1 m and 0.25 m. Crushed sandstone filling was encountered below the roadbase to depths between 0.2 m and 0.75 m and was inturn underlain by roadbase gravel.

The pavement and road shoulders were underlain by up to 0.9 m of general filling, comprising gravelly sand or silty clay with traces of shale or sandstone gravel and sand. It was in turn underlain by residual, mostly stiff to very stiff silty clays. Very low to medium strength, highly and moderately weathered sandstone was intersected at depths of 0.7 m at two locations below the pavement and depths between 0.15 m and 1.75 m in the road shoulders.

5.3 Previous Laboratory Testing

No laboratory testing has been carried out during the current assessment. However, extensive testing has formed part of the previous DP investigations of the adjacent properties. Summaries of the testing are given in the following sections.

5.3.1 M7 Hub and Old Wallgrove Road

The laboratory testing of materials from the M7 Hub and adjacent section of Old Wallgrove Road indicated:

- residual clays of medium or high plasticity which are likely to have moderate to high susceptibility to shrinkage and swell movements resulting from changes in soil moisture content.
- residual clays (ten samples) of Emerson Class Number 2, indicating that the clay filling is potential dispersive and two samples with Emerson Class Number 5 or 6 (slightly to non-dispersive).
- California bearing ratio (CBR) values ranging from 1.0% to 8% for residual clay samples prepared to a dry density ratio of approximately 100% relative to standard compaction and to approximately optimum moisture content, and soaked for four days under a surcharge load of 9 kg.
- pH, sulphate and chloride contents, which when compared to requirements of AS2159 – 1995; Table 6.1 (Ref. 3), indicates that the soils on the site are non-aggressive with regards to concrete structures.
- exchangeable sodium and Cation Exchange Capacity (CEC) values indicating, when compared with the sodicity classes given in DLWC (Ref. 4), that the soils are highly sodic.

5.3.2 Western Sydney Parklands

The laboratory testing of materials from the Western Sydney Parklands site indicated:

- silty clays and clays of moderate to high plasticity, likely to have high susceptibility to shrinkage and swell movement resulting from changes in soil moisture content.
- silty clays and clays that are non to slightly dispersive, although some dispersive behaviour should be expected.

5.4 Salinity Potential

5.4.1 Background

McNally (Ref. 5) describes the general hydrogeological framework relevant to Western Sydney, including this site, where the shale terrain is known for saline groundwater (due to connate salt in shales of marine origin or to windblown sea salt) and the salt accumulates by evapo-transpiration (mostly in the B-horizon of residual soils). In areas of urban development, this can lead to damage to building foundations, lower course brickwork, road surfaces and underground services, where these impact on the saline zone or where the salts are mobilised by changing groundwater levels. Seasonal water level changes of 1 m - 2 m can occur in a shallow regolith aquifer or a deeper shale aquifer due to natural causes, however urban development should be carried out with a view to maintaining the natural water balance (between surface infiltration, runoff, lateral through-flow in the regolith, and evapo-transpiration) so that long term rises do not occur in the saline groundwater level.

The former Department of Infrastructure Planning and Natural Resources (DIPNR) infers a “moderate salinity potential” for the entire site area on mapping entitled “Salinity Potential in Western Sydney 2002” (Ref. 6). Salinity potentials are based on soil types, surface levels and general groundwater considerations but are not in general ground-truthed. Hence, it is not generally known if actual soil salinities are consistent with the potential salinities, although given site topography and results of salinity testing undertaken at nearby sites, it is likely that the moderate salinity potential would describe the typical upper salinity potential of this site.

For purposes of description, a saline soil may be defined as containing sufficient soluble salts to adversely affect plant growth and/or land use. Generally, a level of electrical conductance of a saturated extract (ECe) in excess of 4 dS/m at 25°C is regarded as the defining characteristic of a saline soil. The boundaries of salinity classes defined by Richards (Ref. 7) are given in Table 1.

Table 1 – Salinity Classes

Class	ECe (dS/m)	Implication
Non Saline	<2	Salinity effects mostly negligible
Slightly Saline	2 – 4	Yields of sensitive crops effected
Moderately Saline	4 – 8	Yields of many crops effected
Very Saline	8 – 16	Only tolerate crops yield satisfactorily
Highly Saline	>16	Only a few very tolerant crops yield satisfactorily

Note: to convert from dS/m to μS/cm multiple ECe values by 1000.

Generally, soil salinity (ECe) in excess of 4 dS/m (moderately, very or highly saline) is regarded as posing some risk to an urban development or to a down-gradient area, requiring the formulation of a salinity management plan.

5.4.2 M7 Hub Site

Studies by SMEC Australia Pty Ltd for Blacktown City Council (Ref. 8) and the SEPP59 Landowner Group (Ref. 9) covered a broad area of the Eastern Creek catchment but included sampling and testing of soil and groundwater along Reedy Creek and lower slopes of the north-western portion of the M7 Hub development. The testing indicated that:

- salinity levels in Reedy Creek were slightly elevated and in excess of the (then DLWC) water quality threshold of 2500 μS/cm;
- groundwater was in general highly saline and unsuitable for most purposes;
- maximum soil salinities of 3.65 - 5.11 dS/m (slightly – moderately saline) and 5.28 dS/m (moderately saline) were determined from bore samples respectively at depths of 0.5 - 4.0 m along Reedy Creek at a depth of 1.5 m in the north-western portion of the M7 Hub.

Salinity investigations by DP (Ref. 10 and 11) in the M7 Hub and adjacent section of Old Wallgrove Road generally confirmed the previous SMEC studies (Ref. 8 and 9) with the principal findings being that:

- very saline soil (8.3 dS/m) was identified in natural soils at a single location, at a depth of 2.5 m adjacent to Reedy Creek.

- elsewhere moderately saline conditions (at worst) were assessed adjacent to the riparian (alluvial) zone of Reedy Creek and adjacent lower slopes adjacent to Reedy Creek, on the lower slopes of the western topographic ridge and in a narrow zone in the northeast of the area between Reedy Creek and Old Wallgrove Road, where inferred underlying shales may have an elevated salt content. Highest salinities are developed in the zone from 1.0 m – 2.5 m below ground surface, generally in firm to very stiff silty clays of the B and C soil horizons.
- the western section of the area is underlain for the most part by non-saline to slightly saline soils.

5.4.3 Western Sydney Parklands

Salinity investigations by DP (Ref. 12) in the Western Sydney Parklands site indicated:

- variously non-saline to very saline soil conditions, with moderately and above (very and highly saline) soils identified within the 0.5 m to 2.0 m depth zone, correlating to the B horizon soils. The very and highly saline soils were typically present along the local creek alignments.
- electromagnetic survey data showed that of the 42 000 corrected and filtered apparent conductivity measurements over the study area, 57% fell in the non-saline to slightly saline classes, with 39% in the moderately saline class and 4% in the very to extremely saline classes. Following linear regression of the apparent conductivity measurements and factoring to estimate EC_e values from the electromagnetic survey data, 97% fell in the non-saline to slightly saline classes and 3% fall in the moderately saline class.

5.5 Acid Sulphate Soils

The site is located approximately 50 m above the level of estuarine soil development in which acid sulphate soils are developed. Reference to the *Prospect/Parramatta River Acid Sulphate Soil Risk Map* (Ref. 13) indicates that the closest known or potential acid sulphate soil deposits are located some 12 km to the north-west of the site.

5.6 Groundwater

The site inspection has indicated a minor waterlogging potential in areas of flat terrain where run-off is limited by the local topography. In such cases, waterlogging has resulted due to unfavourable ground surface contours and apparent overland flow from rainfall or localised open stormwater drainage. There were no obvious signs of waterlogging resulting from subsurface groundwater flow/seepage.

Assessment by Old (Ref. 14) of the groundwater in the Wianamatta Group, and Bringelly Shale in particular, indicated that:

- groundwater is typically brackish to saline with TDS values in the range 4000 – 5000 mg/L (but with cases of TDS up to 31 750 mg/l being reported), the dominant ions being sodium and chloride, the water being generally unsuitable for livestock or irrigation).
- the shales have a very low intrinsic permeability and groundwater flow is likely to be dominated by fracture flow with resulting typically low yields (<1 L/s) in bores.

6. IMPACT ASSESSMENT AND MITIGATION STRATEGIES

6.1 Site Inspection Results

Notes describing classification methods and descriptive terms used in the current investigation and referred to in the following sections are included in Appendix A.

6.1.1 Portion of Quarantine Station Proposed for Development

Within the portion of the quarantine station that is proposed for development, the principal items of note are:

- a dam and filled dam wall at the south-western corner of the site (refer Drawing 1 within proposed Lot 8). The dam appears to have been constructed by excavation and placement of soil filling of up to 3 m thick.

- a filled and levelled dressage area east of the above dam and adjacent to the site's southern boundary. The maximum depth of filling is estimated to be approximately 2 m, which includes imported sand filling at the dressage area ground surface.
- a filled embankment midway along and adjacent to the site's southern boundary. Filling is estimated to be up to 5 m thick. This filling is of unknown origin and purpose. Site representatives reported a possible horse burial area within this portion of the site. Another possible reason for the filling is that it contains stockpiled filling from on-site excavations (dams, or similar).
- the presence of many localised cut to filled building pads for various site structures and pavements. Cutting and filling is generally limited to about 1 m depth.
- the presence of several overland flow drainage channels transferring excess stormwater, greenhouse irrigation water and kennel/cattery washout water to on-site drainage pits, sumps and scattered shallow storage dams. Most channels are lined with grass and all are about 0.6 m deep or less.
- the presence of above ground storage tanks and maintenance buildings that may have led to contaminating activities on site during the life of the quarantine station.

6.1.2 Proposed Ecological Conservation Area

Within the western portion of the site proposed for retention as an ecological conservation area, the principal items of note are:

- a dam/storage pond approximately midway along the site's western boundary and adjacent to the proposed rear boundary of proposed Lot 6. This dam is estimated to be up to 1 m deep and appear to collect overland and possible piped flow from higher site levels in the southwest of the site.
- a shallow channel that discharges the two dam overflows north and offsite through a culvert under the northern site boundary and into the adjoining cemetery site. This channel also collects overland flow from within the north-western corner of the quarantine station's horse compound.
- isolated areas of 'boggy' ground surrounding the dam and channel areas, primarily due to insufficient grade to discharge all surface stormwater efficiently to prevent localised surface ponding.

- an area of disturbed ground within the central northern section of the ecological conservation area. This area is approximately 15 m in diameter and is covered with weed and stockpiles of refuse vegetation.
- the filled embankment and continuing raised ground surface within the cemetery lands to the west of the quarantine station site. This filling has changed the original ground surface topography, preventing onward natural runoff in a westerly to north-westerly direction. It is apparent that some runoff from the cemetery site possibly enters the proposed ecological conservation area. Filling within the adjoining cemetery site includes refuse and building debris and therefore possibly contains contaminants of unknown origin.
- dead or dying trees within the central part of the proposed ecological conservation area. Many trees appear healthy, although some have died, possibly as a result of saline soil conditions, although grass growth surrounding the dead trees appeared thick and healthy.
- Salt tolerant vegetation (she-oaks and paperbarks) along the western boundary and also surrounding the surface water discharge point at the northern boundary.

6.2 Geological and Hydrogeological Model of the Eastern Creek Site

The site is characterised by low lying, gently undulating topography that is underlain by the shales and sandstones of the Bringelly Shale. Based on the site observations, data from field investigation and associated laboratory testing of the nearby and similar sites within similar terrain in the Sydney area, it is considered that the principal features of geological and hydrogeological significance for the site are:

- highly variable weathering profiles with clay profiles ranging from less than 1 m to about 9 m deep on the Bringelly Shale, which is characterised by rapid lateral and vertical variation in the lithology. Dispersive conditions, high susceptibility to shrink-swell movements and low bearing strength (particularly for road pavements) are expected within the residual soils.
- shale and siltstone bedrock initially ranging in strength from very low to low strength and potentially medium to high strength below about 5 m in many areas.
- weathering and erosion resistant, medium to very high strength sandstone bands irregularly distributed in the stratigraphic sequence, but probably at depths greater than 10 m at this site.

- very saline soils along poorly drained sections of Eastern Creek and tributary gullies outside of the site. Within the site it is anticipated that most of the site will be classified as non or slightly saline, but with a scattering of moderately saline areas, particularly in footslope locations or where shale bands are preferentially salt rich.
- groundwater within the shale sequence being fracture controlled and very saline.

6.3 Slope Stability Constraints

The following slope stability assessment is based on the results of the geological inspection and DP's involvement in similar projects. It includes consideration of bedrock geology, observed or anticipated soil depth, steepness of slopes relative to historical or ancient slope failures in similar materials, the disturbance of soil and vegetation cover during development, the influence of groundwater or surface saturation, and the effects of earthquake forces.

No deep seated slope stability hazards were identified within the residual profiles or the underlying Bringelly Shale. Similarly, localised soil creep is unlikely in natural ground, although minor creep within filling of unknown condition, such as the filled dam walls and embankment in the south-western corner of the site, is possible.

It is anticipated that site preparation will effectively remove existing areas of creep affected soils. Localised cut slope instability may develop during benching, however it is anticipated that the management of the slope hazards will be suitably achieved by flattening of batters or the construction of retaining walls. Following completion of the development, it is anticipated that there will be a very low risk of slope instability when assessed in accordance with the methods of the Australian Geomechanics Society (AGS) Sub-committee on Landslide Risk Management (Ref. 15).

6.4 Salinity Constraints

6.4.1 Causes of Salinity

Although saline soils and groundwater are a natural part of the Australian landscape, land management practices are now increasingly recognised as significant contributors to the expansion of salt affected areas. In particular, urban salinity is increasingly occurring around populated areas due to clearing and site development.

Salinity occurs when salts found naturally in the soil or groundwater are mobilised. Capillary rise and evaporation concentrate the salt on, and close to, the ground surface. Urban salinity becomes a problem when the natural hydrogeological balance is disturbed by human interaction. This may occur in urban areas due to changes to the water balance, increases in the volume of water into a natural system altering subsurface groundwater flows and levels, exposure of saline soils, and removal of deep rooted vegetation reducing rates of evapo-transpiration. Even small changes in sensitive areas can result in the balance being irrecoverably altered and salinisation occurring.

6.4.2 Effects of Salinity in an Urban Environment

Some building methods may also contribute to the process of urban salinity. For example, compaction of ground surfaces and fills in a manner which restricts groundwater flow could result in a concentration of salt in one area; cutting into slopes for buildings can result in saline soils or groundwater being exposed and intercepted; and the use of imported filling may be an additional source of salt or the filling may be less permeable, preventing good drainage. These issues may also result in problems with the design and construction of roads. In particular, the building of embankments and the compaction of layers can interfere with groundwater flow. Also, the inappropriate positioning, grading and construction of drains can result in surface and groundwater mixing and stagnant pools forming, that evaporate, leaving salt encrusted ground.

Excess salinity in an urban environment can result in significant problems. It can manifest itself in a number of ways resulting in damage to buildings, vegetation, soils and roads.

The effects of salinity can be observed on building materials, infrastructure including pipe work and roads, as well as in vegetation. The effect of urban salinity is the result of both physical and chemical actions of the salt on concrete, bricks and metals. Salt moves into the pores of concrete and bricks and becomes concentrated when the water evaporates and can result in breakdown of materials and corrosion. Evidence of this may include crumbling, eroding or powdering of mortar or bricks, flaking of brick facing and cracking or corrosion of bricks.

High levels of salinity may also affect soil structure, chemistry and productivity. This can reduce plant growth, which in turn alters soil structure, chemistry and nutrient levels. As soils become more saline, plant and micro-organisms decline and soil structure deteriorates. Waterlogging may also occur following a decline in nutrient levels. Over time, the alteration of soil structure can lead to the formation of gullies and other forms of soil erosion.

Salinity may also result in the corrosion of steel pipes, structural steel and reinforcement and can damage underground service pipes resulting in significant financial costs. Salinity can also have a significant effect on roads and pavements, including deterioration of the bitumen seal, blistering, which can lead to the formation of cracks and potholes, staining, cracking, deformation, potholes and cracking and spalling of reinforced concrete pavements.

6.4.3 Salinity Management Strategies for the Eastern Creek Site

A moderate salinity potential is inferred on published mapping (Ref. 6) across the entire site. In general, investigations of the nearby M7 Hub site to the south indicated that a high salinity potential was inferred by the published mapping for the lower slopes and drainage areas of Reedy Creek. This was not realised, except within some alluvial sediments immediately adjacent to the creek. Such soil conditions do not represent the quarantine station site and hence moderately saline or less saline soils are likely.

Additionally, it is likely that the predominantly silty clay soils will be sodic and non-aggressive to steel and mostly non-aggressive to concrete, with some potential for localised increased aggressivity if soil salinity does increase above the expected moderate class.

Although moderate or less saline soil conditions are inferred for this site, efforts should be made to prevent or restrict changes to the site's water balance that could result in rises in groundwater levels. As a precaution, development must be planned to mitigate the effects of any potential salinisation that could occur. These efforts need to be directed at all levels of the development process including:

- site design, vegetation and landscaping;
- commercial building and infrastructure construction.

In general, the following strategies are directed at:

- maintaining the natural water balance;
- maintaining good drainage;
- avoiding disturbance or exposure of sensitive soils;
- retaining or increasing appropriate native vegetation in strategic areas;
- implementing building controls and engineering responses where appropriate.

Planning of the proposed development at this site requires careful management with view to controlling drainage and infiltration of both surface waters and groundwater to prevent rises in groundwater levels and to minimise the potential for soil erosion. Precautionary measures, applicable to the whole development area to reduce the potential for salinity problems, include:

- avoiding water collecting in low lying areas, along drainage channels, dam floodways, in ponds, depressions, or behind fill embankments or near trenches on the uphill sides of roads. This can lead to water logging of the soils, evaporative concentration of salts, and eventual breakdown in soil structure resulting in accelerated erosion.
- roads and the shoulder areas should also be designed to be well drained, particularly with regard to drainage of surface water. There should not be excessive concentrations of runoff or ponding that would lead to waterlogging of the pavement or additional recharge to the groundwater. Road shoulders should be included in the sealing program.
- surface drains should generally be provided along the top of batter slopes of greater than 2.5 m height to reduce the potential for concentrated flows of water down slopes possibly causing scour. Well graded subsoil drainage should be provided at the base of all slopes where there are road pavements below the slope to reduce the risk of waterlogging.
- with regard to surface slopes, a minimum grade of 1V:100H (1%) is suggested.
- where possible, materials and waters used in the construction of roads and fill embankments should be selected to contain minimal or no salt. This may be difficult for cuts and fills in lower areas where saline soils are exposed in cuts, or excavated then placed as filling. Under these circumstances, where salinisation could be a problem, a capping layer of either topsoil or sandy materials should be placed to reduce capillary rise, to act as a drainage layer and also reduce the potential for dispersive behaviour in any sodic soils.

- gypsum should be mixed into filling containing sodic soils and cuts where sodic soils are exposed on slopes to improve soil structure.
- salt tolerant grasses and trees should be considered if re-planting close to higher than slightly saline soil areas, where present, to reduce soil erosion and maintain the existing evapo-transpiration and groundwater levels. Reference should be made to an experienced landscape planner or agronomist.
- consideration of salinity during detailed geotechnical studies for proposed buildings and infrastructure, and incorporation of appropriate design mitigation measures, as required.

The entire site area is shown on published mapping (Ref. 6) as being of moderate salinity potential, but previous DP investigations of nearby sites indicates that salinity of moderate classification or greater is likely to have only a scattered distribution. To ensure that potential salinity impacts are appropriately considered during development of the Eastern Creek Quarantine Station site, it is recommended that any application for development within the site be accompanied by a salinity report prepared by a suitably qualified geoconsultant. The report should:

- assess actual salinity levels across the site;
- evaluate the impacts of the development on saline land; and
- outline measures that would be adopted to mitigate and manage these impacts.

In addition, it is recommended that Afteron Ltd develop a Soil and Water Management Plan (SWMP) for the Eastern Creek site, including:

- provision for Erosion and Sediment Control Plans to be developed for each development involving ground disturbance;
- a Surface Water Monitoring Program including provision for salinity/conductivity monitoring, as required; and
- a Groundwater Monitoring Program, including provision for salinity/conductivity monitoring.

6.5 Site Preparation

Bulk earthworks are expected to involve moderate scale cut-to-fill operations to profile the site to create level building pads at each of the proposed lots shown on the site layout presented in Drawing 1. Additional filling is also likely to be required for the backfilling of existing dams and other areas of existing site cutting.

Outlined in the following sections are comments related to appropriate engineering works for minimising or mitigating environmental effects during the excavation and placement of filling materials.

6.5.1 Excavation Conditions

It is expected that the excavations will encounter mostly clays and interbedded shale, siltstone and sandstone rock ranging from extremely low to possibly high strength.

It is anticipated that most of the site will not be subject to significant excavation constraints. However, excavation in low to high strength bedrock will probably require the use of heavy ripping with D9 class or heavier dozers. It is likely that ripping of high strength sandstone, if encountered, may result in very low productivity and as such, excavations within such material should be minimised.

NSW EPA guidelines require that all material to be disposed off site, such as may be required during excavation of existing filling volumes if unsuitable materials are identified, should be the subject of a Waste Classification Assessment. To minimise the risk of unexpected delays and disposal costs it may be preferable to conduct sampling and testing on any excess excavated soils prior to commencement of excavation, if such soils can be identified.

Significant groundwater inflows into site excavations are not anticipated and any seepage, if present, is likely to be minor. It is recommended that drainage be provided to limit ponding of both seepage and stormwater runoff, with the collected water being passed through sedimentation basins and monitored to ensure compliance with water quality requirements prior to discharge or reuse for moisture conditioning of filling within the site.

The presence of high plasticity clay and weathered shale at bulk excavation levels is likely to result in poor trafficability conditions during and after rainfall. Hence, the provision of adequate site drainage and a surface capping layer (e.g. ripped sandstone) during construction will be required to reduce the inconvenience caused by poor trafficability.

6.5.2 Filling Conditions

The preparation of surfaces to receive new filling and the placement of the filling should be carried out under Level 1 engineering control in general accordance with AS 3798-2007 Guidelines on Earthworks for Commercial and Residential Developments (Ref. 16).

The natural silty clays and weathered rock appear suitable (subject to assessment of salinity) for reuse as fill material provided the rock is crushed during compaction to a maximum size of 100 mm. Due to the expected high plasticity of the clays, care should be taken not to over-compact the clay, due to the risk of subsequent swelling. The moisture content should also be maintained after compaction by spray coating or placement of a granular surfacing. This will minimise the erosion potential of the prepared surface until the areas are covered with buildings and/or pavements.

Any imported filling should preferably include granular materials, such as ripped or crushed rock, or low plasticity clays of non-saline or low salinity classification that is free of deleterious substances.

6.6 Erosion and Sedimentation

It is anticipated that a variation in material types and the probable varying bench levels between adjacent commercial lots will result in a combination of boundary batters and retaining walls to minimise unproductive space in the development. Cut faces should be battered back to minimise the risk of slope failures during the construction of retaining walls, or alternatively, where retaining walls are not required, they should be graded at appropriate temporary or permanent batter slopes. A short-term batter slope of 1:1 (H:V) is suggested for batter heights of up to 4 m in clay and extremely weathered rock. For long-term conditions, a minimum batter slope of 2:1 (H:V) is suggested, with 3:1 (H:V) being preferred to permit surface protection (e.g.

by grassing) to reduce the potential for erosion of soil batters. For very low strength or better rock a batter slope of 0.75:1 (H:V) is suggested for short term and 1:1 (H:V) for long term.

Erosion of permanent batter slopes in clays is likely unless the faces of the slopes are protected. This is also applicable to the faces of shale which tend to fret readily when subjected to alternate wetting and drying. In cut faces developed in sandstone of at least medium strength, it may be possible to leave the face exposed but this will require assessment of individual faces for determination of any required protection (e.g. shotcrete) or support (e.g. rock bolts) during site development. For all cuts, provision should be made for drainage at the top and at the base of the slopes to control any run-off and potential face erosion.

6.7 Acid Sulphate Soils

The site generally lies at an elevation some 50 m or more above the formation environment of estuarine acid sulphate soils. As such, it is considered that there is no risk of estuarine acid sulphate soil conditions.

6.8 Groundwater

The intersection of free groundwater is anticipated only for the minor seepage of perched groundwater that may be expected along the soil to rock interface and within the fractured zones of rock that are intersected by cut batters.

It is considered that bulk excavation works will generally not intersect and will have no direct impact upon the groundwater table, provided standard procedures are followed to manage storm water runoff and subgrade drainage.

It should be noted that saline and high salinity groundwater is natural for the area and there are no management techniques which can eliminate this problem. Certain site management options are recommended to reduce the offsite environmental effects. The options include:

- the monitoring of groundwater at both upstream and downstream limits of the site, as required.

- the maintenance and improvement of native vegetation along drainage courses and careful water management of landscaping areas associated with individual lot developments.
- minimisation of exposure of saline and sodic soils in temporary faces or stockpiles during site preparation works.
- the collection and controlled discharge of seepage from cut faces and storm water from hard surfaces of the proposed development such that the potential for localised ponding or waterlogging is minimised.
- the provision of lining of temporary or permanent ponds to minimise groundwater recharge through gravelly bands within the soil profile, if identified.
- the application of gypsum to areas of exposed soils or unpaved landscaping areas.
- protection of in-ground structures from potential salt attack.

6.9 Construction of New Roads

It is assumed that the proposed estate road will be carried out at or about the current ground surface to minimise cutting and filing of the site, thus cutting and filling would likely be limited to within about 2 m of existing ground surface levels.

No testing has yet been carried out along the length of the road, however investigation of nearby sites indicate that constraints will include:

- low subgrade strength
- localised poor drainage with potential for localised salinity.

Appropriate methods to address these constraints include:

- provision of drains along the perimeter of the pavements with a longitudinal fall to discharge points to minimise the risk of ponding.
- measures to limit ingress of salt and moisture into the substrate. This can be addressed by consideration of the type and amount of materials and water used during construction. Methods of achieving this may include limiting the salt content of material and water to less than 0.25% of soluble salts by dry mass of aggregate for at least 0.5 m depth and/or

minimising permeability through compaction, stabilisation and seal characteristics and ensuring the seal is applied as soon as possible after the pavement has been compacted.

- the location of sedimentation and detention basins away from salinity and waterlogging susceptible areas (if applicable).

7. CONCLUSIONS AND RECOMMENDATIONS

The site inspection, review of published literature and the results of investigations of nearby sites has indicated that the principal geotechnical constraints affecting the proposed Eastern Creek Quarantine Station development site are:

- localised areas subject to waterlogging, which may cause increased salinity.
- shallow to deep clay profiles of moderately to high shrink-swell susceptibility and probably localised dispersion potential.
- possible variations in the weathered bedrock profile, which may require changes in the requirements for battering, retaining works, foundations and pavements.
- the potential requirement for heavy ripping and rock breaker use in medium and high strength rocks within deeper sections of potential site excavations, including pipeline excavations in cuttings.
- saline conditions are present within the groundwater.
- saline conditions will be present in some residual soils, and possibly some rock strata.
- low subgrade CBR values within residual soils and potentially within sections of the underlying bedrock.

To address the geotechnical constraints, it will be necessary to:

- develop a comprehensive Soil and Water Management Plan including:
    - provision for detailed Erosion and Sedimentation Control Plans to be prepared for each development prior to commencement of construction;

  - a surface Water Monitoring Program, including provision for suspended solids and salinity monitoring, as required; and
  - a Groundwater Monitoring Program, including provision for groundwater level and quality monitoring.
- prepare a salinity assessment report to accompany each project/development application.
- conduct detailed geotechnical investigations to inform the detailed design of buildings and infrastructure.

In summary, the constraints described above may be addressed by appropriate engineering works and consequently, it is considered that the proposed development can be successfully constructed, as have been the adjacent developments, such as the M7 Hub, which are located within effectively equivalent topographic terrain and stratigraphic sequence and are subject to similar salinity and hydrogeological conditions.

DOUGLAS PARTNERS PTY LTD

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NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00 mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300 mm)	CPT Cone Value (q_c — MPa)
Very loose	less than 5	less than 2
Loose	5—10	2—5
Medium dense	10—30	5—15
Dense	30—50	15—25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling are given in the report.

Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Test Pits — these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling — the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" — Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm

  as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil.

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) \text{ N (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18)\ c_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is carried out in accordance with Australian Standard 1289 “Methods of Testing Soil for Engineering Purposes”. Details of the test procedure used are given on the individual report forms.

Bore Logs

The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than ‘straight line’ variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions — the potential for this will depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

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

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

Reproduction of Information for Contractual Purposes

Attention is drawn to the document “Guidelines for the Provision of Geotechnical Information in Tender Documents”, published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

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

Douglas Partners
Geotechnics • Environment • Groundwater

DESCRIPTION AND CLASSIFICATION OF ROCKS FOR ENGINEERING PURPOSES

DEGREE OF WEATHERING

Term	Symbol	Definition
Extremely Weathered	EW	Rock substance affected by weathering to the extent that the rock exhibits soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.
Highly Weathered	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decreased compared to the fresh rock usually as a result of iron leaching or deposition. The colour and strength of the original fresh rock substance is no longer recognisable.
Moderately Weathered	MW	Rock substance affected by weathering to the extent that staining or discolouration of the rock substance usually by limonite has taken place. The colour of the fresh rock is no longer recognisable.
Slightly Weathered	SW	Rock substance affected by weathering to the extent that partial staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable.
Fresh Stained	Fs	Rock substance unaffected by weathering, but showing limonite staining along joints.
Fresh	Fr	Rock substance unaffected by weathering.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index ($I_{s(50)}$) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by Australian Standard 4133.4.1 - 1993.

Term	Symbol	Field Guide*	Point Load Index $I_{s(50)}$ MPa	Approx Unconfined Compressive Strength q_u ** MPa
Extremely low	EL	Easily remoulded by hand to a material with soil properties	<0.03	< 0.6
Very low	VL	Material crumbles under firm blows with sharp end of pick; can be peeled with a knife; too hard to cut a triaxial sample by hand. SPT will refuse. Pieces up to 3 cm thick can be broken by finger pressure.	0.03-0.1	0.6-2
Low	L	Easily scored with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150 mm long 40 mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.	0.1-0.3	2-6
Medium	M	Readily scored with a knife; a piece of core 150 mm long by 50 mm diameter can be broken by hand with difficulty.	0.3-1.0	6-20
High	H	Can be slightly scratched with a knife. A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken with pick with a single firm blow, rock rings under hammer.	1 - 3	20-60
Very high	VH	Cannot be scratched with a knife. Hand specimen breaks with pick after more than one blow, rock rings under hammer.	3 - 10	60-200
Extremely high	EH	Specimen requires many blows with geological pick to break through intact material, rock rings under hammer.	>10	> 200

Note that these terms refer to strength of rock material and not to the strength of the rock mass, which may be considerably weaker due to rock defects.

* The field guide assessment of rock strength may be used for preliminary assessment or when point load testing is not able to be done.

** The approximate unconfined compressive strength (q_u) shown in the table is based on an assumed ratio to the point load index of 20:1. This ratio may vary widely.

STRATIFICATION SPACING

Term	Separation of Stratification Planes
Thinly laminated	<6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	>2 m

DEGREE OF FRACTURING

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude known artificial fractures such as drilling breaks. The orientation of rock defects is measured as an angle relative to a plane perpendicular to the core axis. Note that where possible, recordings of the actual defect spacing or range of spacings is preferred to the general terms given below.

Term	Description
Fragmented	The core consists mainly of fragments with dimensions less than 20 mm.
Highly Fractured	Core lengths are generally less than 20 mm - 40 mm with occasional fragments.
Fractured	Core lengths are mainly 40 mm - 200 mm with occasional shorter and longer sections.
Slightly Fractured	Core lengths are generally 200 mm - 1000 mm with occasional shorter and longer sections.
Unbroken	The core does not contain any fracture.

ROCK QUALITY DESIGNATION (RQD)

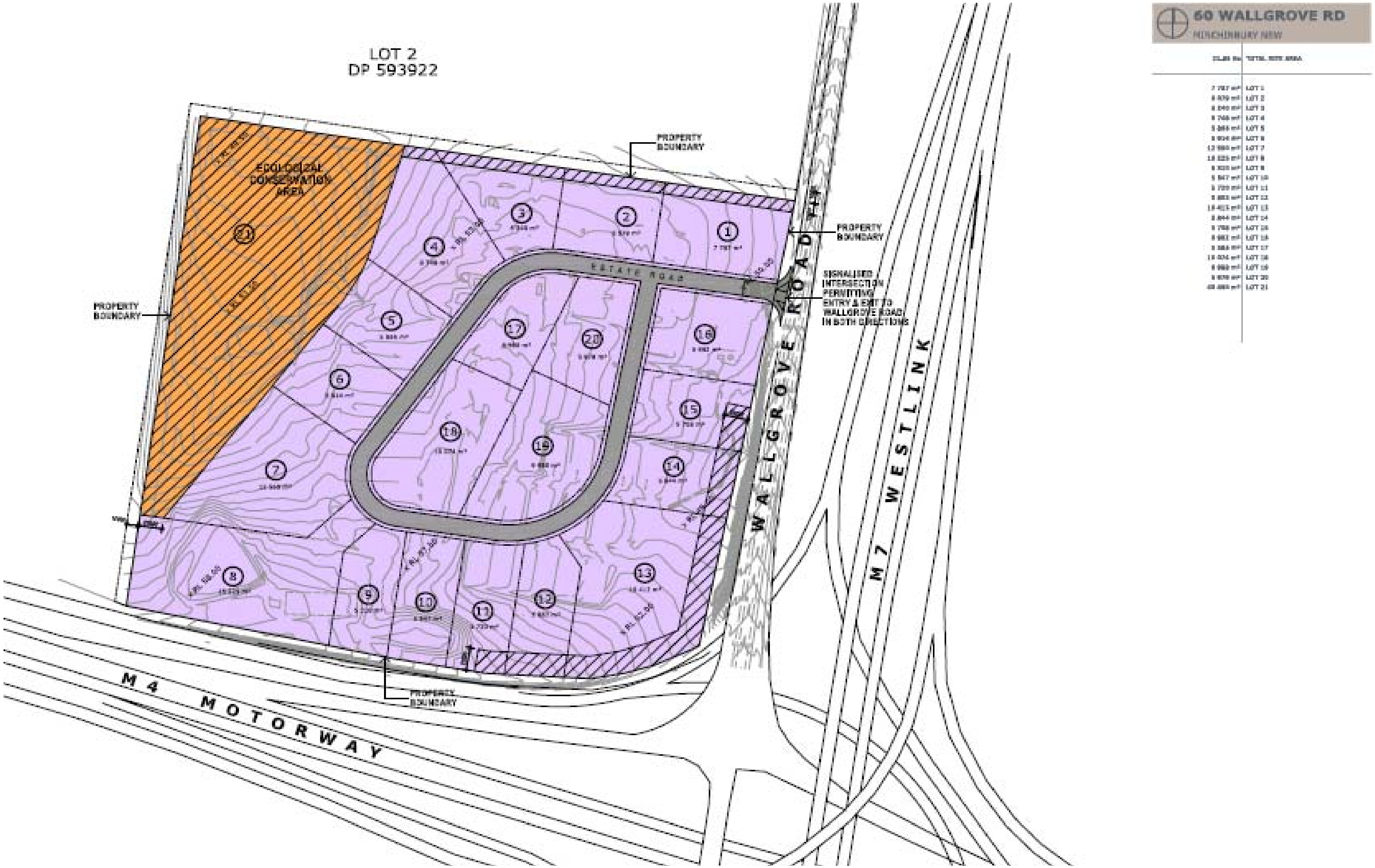
This is defined as the ratio of sound (i.e. low strength or better) core in lengths of greater than 100 mm to the total length of the core, expressed in percent. If the core is broken by handling or by the drilling process (i.e. the fracture surfaces are fresh, irregular breaks rather than joint surfaces) the fresh broken pieces are fitted together and counted as one piece.

SEDIMENTARY ROCK TYPES

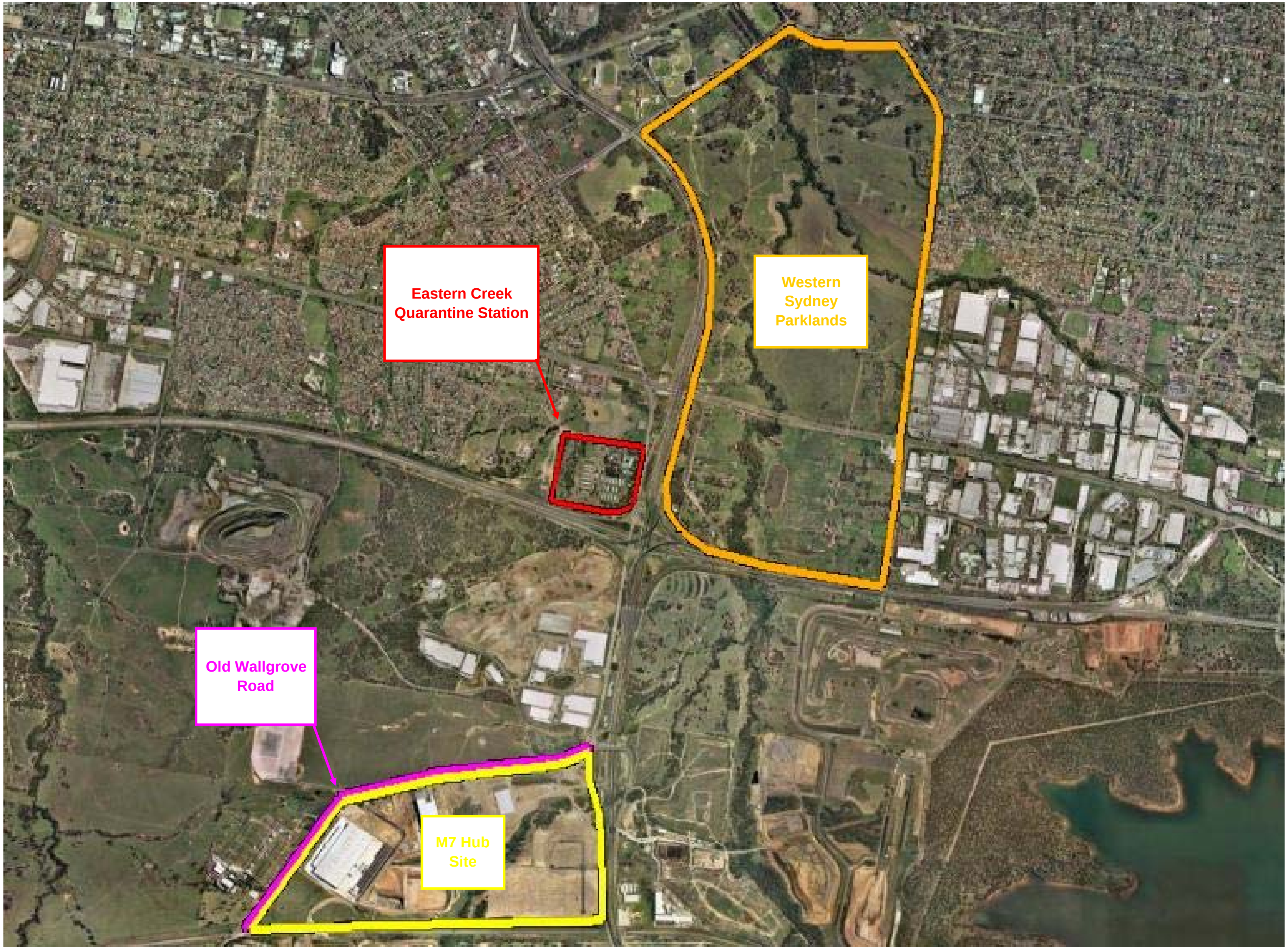
This classification system provides a standardised terminology for the engineering description of sandstone and shales, particularly in the Sydney area, but the terms and definitions may be used elsewhere when applicable.

Rock Type	Definition
Conglomerate	More than 50% of the rock consists of gravel-sized (greater than 2 mm) fragments
Sandstone:	More than 50% of the rock consists of sand-sized (0.06 to 2 mm) grains
Siltstone:	More than 50% of the rock consists of silt-sized (less than 0.06 mm) granular particles and the rock is not laminated.
Claystone:	More than 50% of the rock consists of clay or sericitic material and the rock is not laminated.
Shale:	More than 50% of the rock consists of silt or clay-sized particles and the rock is laminated.

Rocks possessing characteristics of two groups are described by their predominant particle size with reference also to the minor constituents, eg. clayey sandstone, sandy shale.



Douglas Partners Geotechnics · Environment · Groundwater	Client: Afteron Ltd			Title: **Proposed Site Layout**	Project No: 71089.00
	Drawn By: RCB	Scale: As shown	Office: Sydney	**Eastern Creek Quarantine Station**	Drawing No: 1
	Approved By:		Date: 2.6.09	**60 Wallgrove Road, Eastern Creek**	Revision: A



Douglas Partners Geotechnics • Environment • Groundwater	Client: Afteron Ltd		Title: **Referenced Nearby Site Locations**	Project No: 71089.00
	Drawn By: RCB	Scale: As shown / Office: Sydney	**Eastern Creek Quarantine Station**	Drawing No: 2
	Approved By:	Date: 2.6.09	**60 Wallgrove Road, Eastern Creek**	Revision: A

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	Drawn By: RCB	Scale: As shown	Office: Sydney	**Eastern Creek Quarantine Station**	Drawing No: 3
	Approved By:		Date: 2.6.09	**60 Wallgrove Road, Eastern Creek**	Revision: A