Douglas Partners

Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

Proposed New Maitland Hospital
Metford Road, Metford

Prepared for
Health Infrastructure

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Integrated Practical Solutions

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

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1. Introduction

This report presents the results of a geotechnical investigation undertaken by Douglas Partners Pty Ltd (DP) for the proposed New Maitland Hospital at Metford. This report expands upon DPs initial report, Project 81719.01 dated December 2015 (Ref 15), and includes the results of supplementary subsurface investigation at client nominated locations. The supplementary investigation was commissioned in an email dated 20 June 2017 by Mark Davis of AECOM acting on behalf of NSW Health Infrastructure and was undertaken in accordance with Douglas Partners' proposal NCL170350 dated 4 July 2017.

The key features of the Concept design include the following:

- A building envelope within the north-western part of Lot 7314 mostly on previously disturbed areas;
- Building mass of up to eight storeys (including lower ground but excluding plant level);
- Approximate gross floor area of 60,000 square metres;
- Primary access from Metford Road at a new roundabout junction opposite Fieldsend Street;
- Secondary access for ambulances from Metford Road, south of the Primary access. This allows for accessing the hospital from both points;
- An on-grade helipad located to the northeast of the hospital;
- Two car parking facilities;
- Stormwater drainage to the east of the hospital, to manage stormwater flows through the site from offsite lands as well as from the hospital precinct;
- Partial retention of vegetation at the southern, western and eastern end of the site, with some clearing and understorey removal to meet bushfire mitigation requirements (APZ).

A number of geotechnical risks to the proposed development were identified during formation of the scope of work for this geotechnical investigation, as follows:

- Presence of deep filling in Lot 7314;
- Suitability of stockpiled material for reuse as engineered filling;
- Depth of sediment in base of former quarry;
- Instability in the existing quarry high wall; and
- Presence of high strength bedrock.

These geotechnical risks have been assessed during the present investigation as well as the following geotechnical issues:

- Review of previous report in light of the repositioning of the hospital;
- Excavation conditions for areas of earthworks to bring the site to approximately RL 18 m AHD;
- Identification of areas of filling;
- Estimates of settlements in areas of deep filling subject to proposed building loads;
- Geotechnical parameters for high level and piled footings;
- Subgrade conditions for proposed pavement areas;
- Indicative pavement thickness designs;
- Risk of instability in the existing quarry excavation faces;
- Appropriate stabilisation/mitigation measures against quarry face instability;
- Construction methodology for filling of existing quarry, if required;
- Material quality and compaction requirements for engineered filling; and
- Subgrade preparation measures.

The initial investigation comprised the drilling of six bores, installation of two groundwater wells, excavation of 21 test pits, laboratory testing of selected samples, engineering analysis and reporting.

The supplementary investigation comprised the drilling of three bores and installation of one groundwater well.

2. Background Data

The previous geotechnical assessment carried out by DP (Ref 1) provided information on geotechnical conditions based on previous investigations within and near the site together with preliminary design parameters (end bearing, shaft adhesion parameters) for various foundation types as well as comments on a range of other geotechnical items.

Advice provided in the previous report which is relevant to the current development concept has been incorporated in the present investigation report.

DP has provided a previous revision of this report (Ref 15). The current revision includes the results of supplementary subsurface investigation comprising boreholes drilled during July 2017.

3. Site Description

The site is located within the former PGH Bricks site at Metford and is identified as Lot 7314 DP 1162607. The site is irregular in shape with an approximate area of 16.4ha. Figure 1 shows the PGH Bricks site (outlined black) and the current site extent (red).

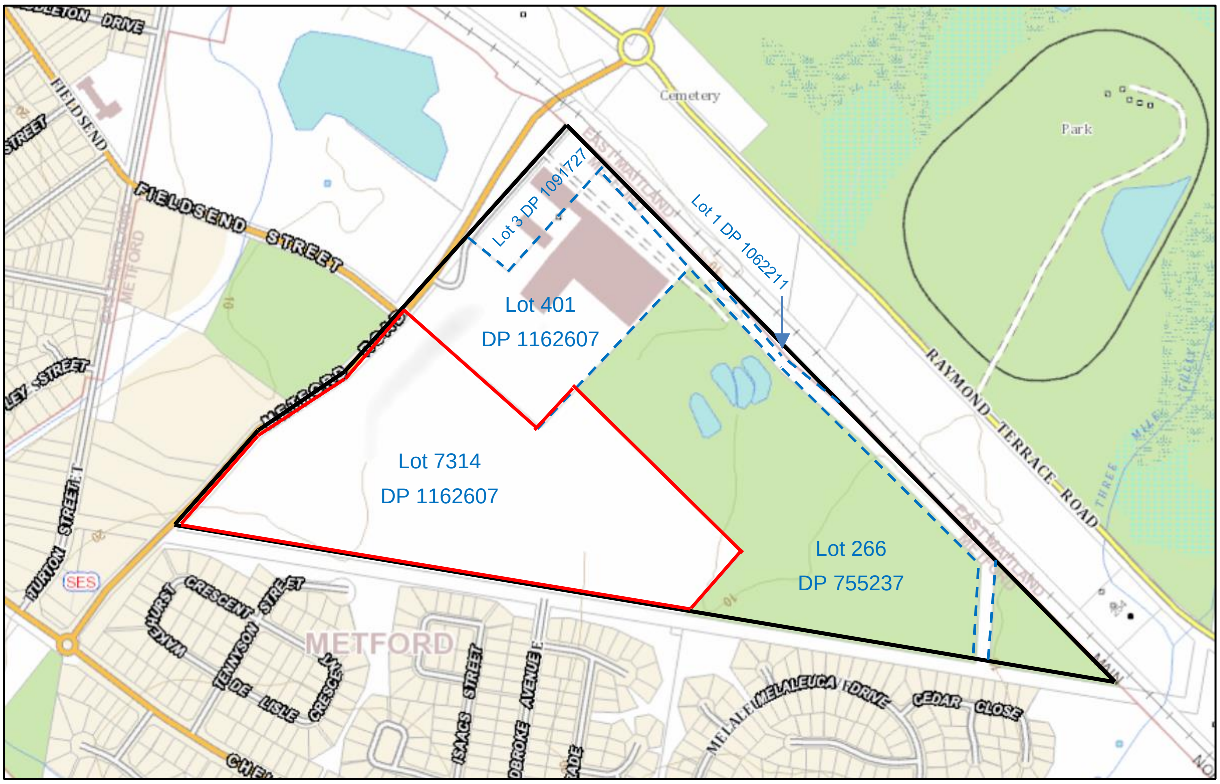

Figure 1: Location and Extent of the Site

It is understood that the hospital is proposed to be situated within the western half of Lot 7314, which is located within the south-western area of the former PGH Bricks site. The proposed hospital layout is shown on the attached test location plan in Appendix D. The site is bounded to the north and east by the former PGH Bricks site, which still has remnants of the former CSR Bricks and Roofing selection centre and sales office and the Northern Railway. The site is bounded to the south by existing residential developments and Metford Road to the west.

The site ranges in surface level from 8 m AHD to 26 m AHD, generally comprised areas of fill above ground level (stockpiles), rock outcrops, water bodies, unsealed access roads, areas of dense vegetation (south-western area) as well as stripped / bare areas.

The site has been divided into several terrain units of similar conditions, as follows:

- South-western Area;
- North-western Area;
- Eastern Area; and
- Southern Strip.

These are discussed in Sections 3.1 to 3.4 below. Site features are also presented in Drawing 2 in Appendix D.

3.1 South-Western Area

The western area of the site covers approximately one-third of Lot 7314.

This area was covered by a dense cover of mature trees. Surface levels within this area of the site range from about RL 20 m AHD along its eastern side to RL 12 m near Metford Road.

A natural drainage path was observed within the densely vegetated area in the south-western part of the site. As shown in Figure 2 below, the rock outcrop serves to facilitate the variation in surface level. This low cliff line defines the western boundary of a broad gully, within which overland flow passes roughly south-east to north-west through this area of the site. This drainage path is discussed further in Section 9.8.

Figure 2: Natural drainage line in south-western part of the site (2015)

Several small ponds were present within the drainage path (refer Figure 3).

Figure 3: A typical pond in the south-western area of the site (2015)

Fingers of fill material were observed to extend into this vegetated area from a constructed berm wall which segregates the disturbed central area of the site from the south-western area (refer Figure 4).

Figure 4: Typical fill mound observed behind the berm wall (2015)

The extent of these fingers of filling have not been fully characterised during the present investigation. An estimate of the affected area is shown on Drawing 2 in Appendix D.

Several areas of outcropping rock were noted in the south-western area of the site. Figure 5 below shows a typical exposure of bedrock

Figure 5: Low to medium strength sandstone exposure in the southern part of the site (2015)

3.2 North-Western Area

The north-western area of the site has been highly disturbed as is characterised by bare soil and rock surfaces with numerous stockpiles. It is understood that the majority of the structures for the new hospital will be constructed in this area of the site. The stockpiles are typically about 5 m to 10 m in height above the surrounding ground level. The stockpiled material is variable but predominantly comprised either cohesive soils and ripped bedrock (clay, sandy clay, silty clay and weathered rock); or gravel, chitter (coarse coal reject) and coal.

Figure 6, below shows a view of this area of the site in 2015 from the northern boundary.

Figure 6: Looking south from the site boundary, excavator at Pit 301 (2015)

Surface levels in this area of the site are generally RL 20 m AHD to RL 24 m AHD.

Figure 7 shows a typical stockpile within this area of the site.

Figure 7: Stockpile adjacent Pit 301, comprising cohesive soils and ripped bedrock (2015)

Several areas beyond the stockpiles exposed sandstone bedrock, particularly towards the eastern and south-eastern sides of this area (refer Figure 8). The sandstone was generally very low strength or stronger.

Figure 8: Exposed sandstone bedrock in north-western area (2015)

3.3 Eastern Area

The eastern area of the site contains the former quarry floor.

Figures 9 and 10, below show views across this area of the site in 2015.

Figure 9: Looking north across the site from Borehole 606 (2015)

Figure 10: Looking south from top of a stockpile, carbonaceous stockpiles (left), water body (left), existing quarry cutting (back), stockpile (right) (2015)

Based on review of available topographic mapping, the surface levels within this area of the site generally range from about 8 m AHD to 12 m AHD, rising to the west, south and east out of the floor of the former quarry.

During the investigation in 2015, a large area of ponded water (Main Pond) was located in this area of the site (middle of Figure 9). The depth of the water was not known but based on probing around the edges it was anticipated to be greater than 1 m to 2 m. During the further investigation in July 2017, it was noted that the sediment in the base of the pond was in the process of being removed.

The remainder of this area is covered by accumulated sediments and low vegetation. Surface water is present in many parts of this area of the site (refer Figure 11).

Figure 11: Typical view of eastern area (2015)

A gravel access track, raised about 1 m above the surrounding ground surface, runs approximately north-west to south-east across this area.

Within the southern part of this area of the site, a large stockpile of carbonaceous material was present. The height of the stockpile varied from less than 1 m to up to about 5 m. Erosion of the carbonaceous stockpiles, as shown in Figure 12, revealed coal below in some areas.

Figure 12: In-situ coal exposed beneath edge of carbonaceous material stockpile in eastern area of the site (2015)

Surface water was present in various water bodies / ponds across this area of the site and was most likely perched. There may be some connection between the ponds and groundwater however the rock permeability would be relatively low and flow rates would also be low. Preferential flow potentially occurs along strata interfaces within the rock, particularly coal and shale layers which tend to have higher permeability than neighbouring sandstone, siltstone and claystone layers.

3.4 Southern Strip

The southern strip of the site, which varies in width of between 40 m to 90 m in width, is located between the former quarry floor and the southern boundary of the site.

This area is generally covered by dense bush (refer Figure 13).

Figure 13: Existing vegetation at Pit 312 (2015)

The drainage gully which traverses through the south-western area of the site, enters the site within the southern strip (refer Drawing 2). This gully is incised up to 3 m into the surrounding landscape, with sandstone bedrock of at least low strength exposed at its base (refer Figure 14).

Figure 14: Sandstone bedrock exposed in crossing of gully (2015)

The eastern area and southern strip are separated by a rock batter of approximately 10 m in height (refer Figures 15 and 16).

Figure 15: Rock batter between eastern area and southern strip (2015)

Figure 16: Sandstone and siltstone exposed in batter (2015)

More comments in relation to this batter are presented in Section 9.16.

4. Regional Geology

The Newcastle Coalfields 1:100,000 Geology map indicates that the site is underlain by the Tomago Coal Measures of Permian age (geology code Pt). This formation is described as comprising laminated sandstone, claystone, siltstone, coal and tuff. Figure 17 below shows the site overlain with the geological map and site topography (contours at 2 m intervals). Based on information provided by the project surveyor, these 2 m contours are not considered accurate for this site and should be treated as indicative only.

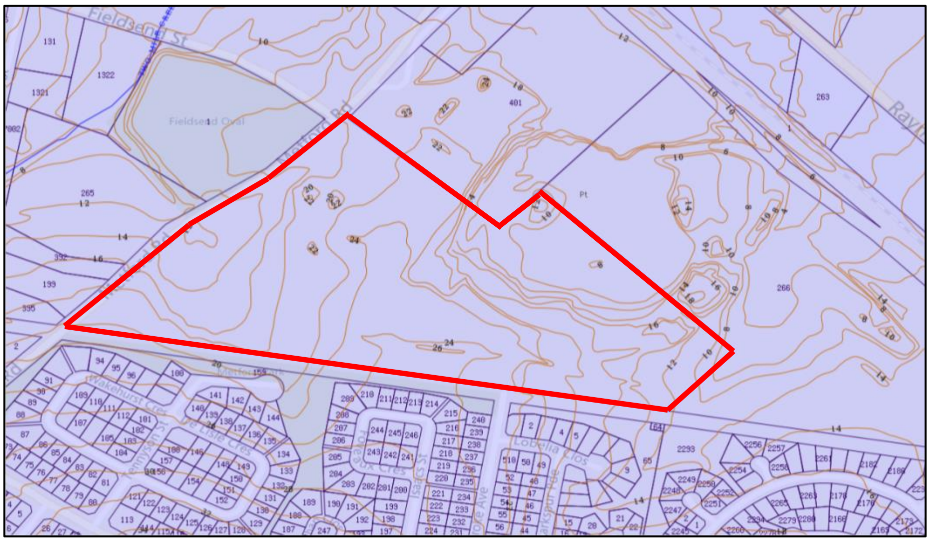

Figure 17: Geological Setting of the Site

The site topography also indicates where the natural drainage path exits the site along the western boundary, along Metford Road.

The Newcastle 1:100,000 Soil Landscapes Sheet indicated that the site includes two soil landscape types, as shown blow in Figure 18.

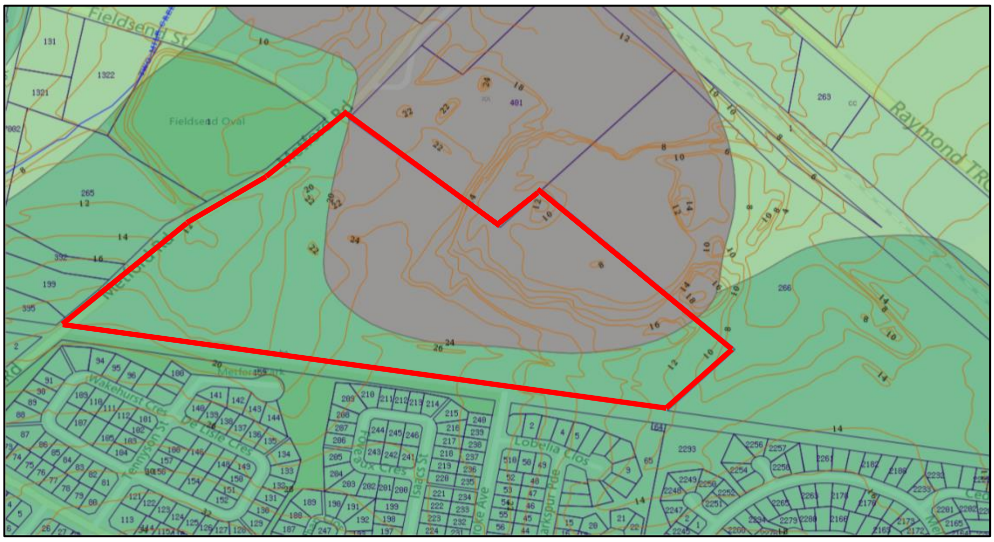

Figure 18: Soil Landscape Setting of the Site

The features of these soil landscapes are summarised in Table 1.

Table 1: Soil Landscape Features

Feature	Description		
Colour (in Figure 15)	Dark green	Light green	Grey
Soil Landscape Code	Be	cc	xx
Soil Process Group	RESIDUAL	ALLUVIAL	DISTURBED TERRAIN
Soil Landscape Name	Beresfield	Cockle Creek	Disturbed Terrain
Landscape Description	Undulating low hills and rises on Permian sediments in the East Maitland Hills region. Slope gradients 3-15%, local relief to 50m, elevation is 20-50m. Partially cleared tall open-forest.	Narrow floodplains, alluvial fan deposits and broad delta deposits. Slope gradients are 0-2%, elevation is <1-50m, local relief is <1m; Cleared open forest.	Level plain to hummocky terrain, extensively disturbed by human activity, including complete disturbance, removal or burial of soil. Local relief variable. Land fill includes soil, rock, building and waste materials. Original vegetation completely cleared.
Soils Description (summary only)	Crests: moderately deep (<120cm), Yellow Podzolic Soils, Brown Podzolic Soils and brown Soloths. Upper Slopes: moderately deep (<120cm) Red Podzolic Soils and red Soloths. Sideslopes: brown Soloths and yellow Soloths. Lower Slopes: deep (>200cm), imperfectly to poorly drained Yellow Podzolic Soils, yellow Soloths and Gleyed Podzolic Soils.	Floodplains: deep (>200cm), imperfectly to poorly drained yellow Soloths (Dy3.41) and Yellow Podzolic soils. Delta / Fan Deposits: deep (>200cm), moderately well to poorly drained Yellow Earths and Grey Earths; with deep (>200cm) imperfectly drained, to well-drained Yellow Podzolic Soils.	Highly variable.
Limitations	High foundation hazard, water erosion hazard, seasonal waterlogging and high run-on localised lower slopes, highly acid soils of low fertility.	Flood hazard, water erosion hazard, permanently high watertables (localised), periodic to permanent waterlogging (localised), high run-on, acid, and infertile sodic / dispersible soils of low wet strength.	Dependent on nature of site. Limitations may include mass movement hazard, steep slopes, foundation hazard, unconsolidated low wet bearing strength materials, potential acid sulfate soils, impermeable soils, poor drainage or erosion hazard.

The NSW Acid Sulfate Soils Risk map indicates no risk of actual or potential acid sulfate soils.

5. Field Work Methods

The initial field work was carried out in the period of 12 October 2015 to 5 November 2015. A supplementary investigation was then carried out from 11 August 2017 to 12 August 2017.

The initial investigation comprised the following:

- Drilling of six boreholes (Boreholes 601 to 606) to depths of between 8.0 m and 21.9 m;
- Excavation of 20 test pits (Pits 301 to 318 and 320 to 321) to depths of between 0.6 m and 2.8 m;
- Drilling of one borehole (Borehole 319) to 0.65 m using hand tools;
- Installation of groundwater wells in bores 605 and 606; and
- Walk-over inspection by a Principal geotechnical engineer to assess site conditions and possible instability in the existing quarry cutting.

The supplementary investigation comprised the following:

- Drilling of three boreholes (Boreholes 701 to 703) to depths of ranging from 8.55 m to 12.0 m; and
- Installation of a groundwater well in Bore 703.

It is noted that for the initial investigation, limited access was available to the majority of the site owing to the presence of trees, overly wet ground and stockpiled materials, subsurface investigation was limited to accessible locations.

The supplementary investigation bores were drilled at client nominated locations.

The test pits were excavated with a 4.5 tonne excavator fitted with a 450 mm bucket with teeth.

Boreholes 601 to 603 and 701 to 703 were drilled with a using a truck-mounted rotary drilling rig equipped with solid flight augers and wash boring equipment for drilling in soil and weathered rock, as well as NMLC diamond coring equipment for coring bedrock (refer Figure 19). Standard penetration tests (SPTs) were performed at selected bore locations and depths. A bobcat-mounted drilling rig was used for Boreholes 604 to 606.

Figure 19: Drilling Rig set up at Borehole 601 (looking west)

Groundwater monitoring wells constructed of 50 mm PVC casing and machine slotted 50 mm PVC screen were installed in Boreholes 605, 606 and 703 to depths of 8 m to 12.0 m. The wells were completed with a gravel pack extending several metres above the well screen and a bentonite plug of at least 0.5 m thickness in each well. Details of the monitoring well construction are provided on the detailed borehole logs in Appendix B.

The test locations were set out by a geotechnical engineer from DP. The engineer also logged the subsurface conditions encountered at each test location and collected samples for subsequent laboratory testing and identification purposes. The engineer boxed and photographed the rock core and carried out point load strength index tests on the core. Pocket penetrometer tests and dynamic penetrometer tests were performed at selected depths and locations.

No survey information for the site has been provided by the client. Reduction of surface levels from the 2 m digital contour mapping was not possible as the limited survey provided by ADW Johnson for Bores 601 to 605 reveals that the 2 m digital mapping is not accurate for this site. The MGA coordinates at each pit location were recorded using a hand held GPS unit which is normally accurate to within about ±5 to 10 m depending on satellite coverage. The coordinates and elevation of Bores 601 to 605 were provided by the project surveyor. It is recommended that all test locations are accurately surveyed prior to further design.

The approximate locations of the boreholes and test pits are indicated on the Test Location Plan, Drawing 1, in Appendix D.

6. Field Work Results

The previous geotechnical assessment (Ref 1) indicated the principal geotechnical conditions at the site would comprise sequences of four main elements: fill above general ground level (stockpiles), filled depressions / voids, residual clays and bedrock.

The subsurface conditions encountered in the bores and test pits undertaken for the present investigation are presented in detail in the borehole logs and test pit logs in Appendix B. These should be read in conjunction with the accompanying general notes in Appendix A which explain the descriptive terms and classification methods used in the logs. The results of the dynamic penetrometer tests are also included in Appendix B. The subsurface conditions encountered in the bores and pits are summarised in Tables 2 to 4 below. For the descriptions of the bedrock, the following general classifications were utilised:

- Class V Extremely low strength sandstone or siltstone and all coal;
- Class IV Very low to low strength sandstone or siltstone; and
- Class III Medium strength or stronger sandstone or siltstone.

Table 2: Summary of Subsurface Conditions – Undeveloped Area (South-western Corner and Southern Strip)

Test Pits	Depth From (m)	Depth to (m)	Soil Description
307	0	0.3	TOPSOIL - grey silty sand
310, 314, 315	0	0.7 / 2.4	FILLING - generally comprising brown / dark brown / grey clayey gravel / sandy gravel / gravel (varying fractions of coal / siltstone/ sandstone / brick) / silty sand filling with varying fractions of sand, silt, and clay, cobbles (sandstone / bricks / siltstone) and various anthropogenic inclusions and sulphidic staining at various depths
307 to 313, 321	0 / 0.3	0.6 / 0.95	SANDY CLAY / SILTY CLAY / CLAYEY SILT / CLAY - brown / orange / light grey, typically very stiff to hard
308 to 311, 313, 321	0 / 1.2	0.15 / 1.4	SILTY SAND / SANDY SILT / CLAYEY SAND - orange / grey / brown, typically medium dense
312	0.95	1.8	SILT - light grey silt (extremely weathered siltstone with soil like properties)
307 to 311, 314, 321	0.6 / 1.4	0.75 / 1.65*	SANDSTONE - orange / light grey sandstone, typically very low to low strength, test pits refusing in low to medium strength sandstone
312, 313	0.9 / 1.8	1.2 / 2.0*	SILTSTONE - light grey siltstone

Notes to Table 2:

* - Termination Depth

Table 3: Summary of Subsurface Conditions – Stripped Area (North-Western Area)

Test Pits/Bores	Depth From (m)	Depth to (m)	Soil Description
301 to 305, 601 to 606, 701 to 703	0	0.1 / 3.8	FILLING - generally comprising light grey / dark grey / grey / brown / black clay / sand / clayey sand / sandy clay / gravel / gravelly sand / silty sand / silt / coal filling with varying fractions of sandstone / siltstone gravel / cobbles, gravel, clay, brick and coal
306	0	0.15	TOPSOIL - light grey silty sand
306 / 703	0.15 / 0.3	0.6 / 0.7	CARBONACEOUS SANDY CLAY - black carbonaceous clay and carbonaceous sandy clay with interbedded bands of grey clayey silt
306 / 701	0.15 / 0.6	0.6 / 1.1	SANDY CLAY - brown sandy clay interbedded with grey silty clay
301, 302 to 305, 601, 602, 603, 606,701, 702, 703	0.5 / 3.8	21.85*	SANDSTONE - typically very low to low strength, becoming low to medium below 4m, with varying fractions of siltstone / carbonaceous lenses (Extremely low strength carbonaceous sandstone from 3.8 m to 4.4 m in 702)
304, 306, 602, 603, 606, 701, 703	0.4 / 8.07	20.65*	SILTSTONE - typically extremely low to very low strength, becoming very low to low strength below 3m, with varying fractions of coal and carbonaceous siltstone (Extremely low strength carbonaceous siltstone from 0.7 m to 1.3 m in 703)

Notes to Table 3:

* - Termination Depth

Table 4: Summary of Subsurface Conditions – Low Lying Area (Eastern Area)

Test Pits	Depth From (m)	Depth to (m)	Soil Description
317 to 320	0	0.3 / 0.8	FILLING - generally comprising brown / dark brown / grey clayey silt / red silty clay / sandy clay filling with varying fractions of black coal fines / coal / carbonaceous siltstone, gravel, sandstone cobbles, silt and bricks
316	0.3	0.6	COAL - very low strength, black coal
316 to 319	0 / 1.05	0.3 / 1.2	SILTSTONE / CARBONACEOUS SILTSTONE - typically very low to low strength, grey siltstone / carbonaceous siltstone with some bands of sandstone (Pit 317)
318	0.8	1.05	SILTY CLAY - hard, brown / grey siltstone with some carbonaceous siltstone fragments
319, 320	0.5	1.8	SANDY CLAY - hard, red / brown sandy clay with varying fractions of silt, gravel and cobbles

Inspections by DP during construction of the weighbridge adjacent to Metford Road encountered an abrupt change from shallow rock to deep fill (i.e. a backfilled quarry void). Anecdotal reports indicated the depth to be substantial (much greater than 4 m) but the depth was not confirmed. However, the results of test pits and bores in this area (Pits 305 and 306) indicated bedrock at 0.5 m and 1.1 m respectively. Hence, if a previous void has been filled in this area, it is likely to be of limited area. Given the history of extraction throughout the site, however, there may be similar areas of deep filling elsewhere on site, between test locations.

No free groundwater was observed in the test pits, with the exception of Pit 318 which encountered groundwater at 0.3 m and Pit 319 which encountered groundwater at the surface. Groundwater standpipes were installed in Boreholes 605, 606 and 703. Monitoring of standpipes 605 and 606 was undertaken on 5 November 2015, at which time Bore 606 was dry and Bore 605 had a groundwater at a depth of 7.2 m (RL 17.1 m AHD). Based on information received from GHD, groundwater was recorded in the installed wells at levels ranging from about 6.5 m AHD to 8 m AHD (wells located within the north-western and eastern areas of the site) and RL 16 to 17 m AHD in a well installed in the western corner of the site. No information on well 703 groundwater level has been provided.

It should be noted that groundwater levels are affected by factors such as climatic conditions and soil permeability and will therefore vary with time.

Mapping of the rock batter located between the eastern area and the southern strip was undertaken by a Principal Geotechnical Engineer on 5 November 2015. The inspection revealed the following:

- The batter was generally formed at a slope ranging from about 20° to 25°;
- Interbedded siltstone and sandstone was exposed within the batter;
- Based on tactile assessment, the strength of the bedrock was generally extremely low to low within the upper half of the batter. This is consistent with the rock core retrieved in the boreholes from this area of the site;
- Within the upper portion of the batter, bedding was observed with a dip of 5° to 15° and a dip direction of about 310° to 340° M;
- The dominant jointing in this lower portion of the batter was sub-vertical to 80° with a dip direction of 080° to 090° M and a spacing ranging from about 0.05 m to 0.2 m;
- Slightly stronger siltstone and sandstone, assessed to be low strength, was present in the lower half of the exposed batter;
- Within this lower half of the batter, bedding was observed with a dip of 0 to 5° and a dip direction ranging from about 000° to 050° M;
- The dominant joints were dipping at about 65° with a dip direction of around 060° M and a spacing ranging from 200 mm to 1000 mm;
- Coal was exposed within the lower approximately 1 m towards the eastern visible extent of the batter (refer Figures 20 and 21).

Figure 20: Coal exposed in lower area of batter

Figure 21: Coal in exposed batter

7. Laboratory Testing

7.1 Analytical Program

Laboratory testing for the proposed development comprised the following:

- Three shrink-swell tests;
- One Atterberg limits test;
- One linear shrinkage test;
- Six standard compaction tests;
- Four soaked California bearing ratio (CBR) tests;
- Five soil aggressiveness tests;
- One water aggressiveness test; and
- Seven combustibility tests.

Detailed laboratory test reports are included in Appendix C and the results are summarised in Tables 5 to 7.

7.2 Analytical Results- Geotechnical

The detailed results are included in Appendix C, and are summarised in Tables 5 to 7 below.

Table 5: Summary of CBR and Standard Compaction Test Results

Pit	Depth (m)	Description	FMC (%)	SOMC (%)	SMDD (t/m^3)	CBR (%)	Swell under 4.5 kg Surcharge (%)
301A	0.5-0.7	Filling (ripped sandstone, sand and clay)	16.5	18.5	1.71	6	1.2
301	1.0-1.3	Filling (ripped sandstone, sand and clay)	-	14.0	1.87	-	-
303	0.9-1.1	Sandstone	-	14.5	1.83	-	-
306	0.7-1.1	Brown Sandy Clay	21.5	21.0	1.58	9	0.6
311	0.3-0.7	Red, brown and grey Silty Clay	18.9	20.0	1.71	5	0.8
313	0.9-1.2	Siltstone	16.6	17.0	1.78	3.5	1.6

Notes to Table 5:

FMC - Field moisture content
SOMC - Standard optimum moisture content
SMDD - Standard maximum dry density
CBR - California bearing ratio (4 day soaked)

Table 6: Summary of Shrink-Swell, Atterberg and Linear Shrinkage Test Results

Pit	Depth (m)	Description	FMC (%)	W_L (%)	W_P (%)	PI (%)	LS (%)	Iss (% per ΔpF)
306	0.3-0.7	Carbonaceous Sandy Clay	36.5	-	-	-	-	3.0
306	0.6-1.0	Sandy Clay	26.1	-	-	-	-	1.5
307	0.3-0.45	Sandy Clay	12.2	36	14	22	8.5	-
311	0.4-0.66	Clay	16.6	-	-	-	-	2.2

Notes to Table 6:

FMC – Field Moisture Content
W_L – Liquid Limit
W_P – Plastic Limit
PI – Plasticity Index
LS – Linear Shrinkage
Iss - Shrink/Swell Index

Table 7: Results of Soil and Water Aggressiveness Tests

Pit / Bore	Depth (m)	Soil Type (A or B)	pH	EC (µS/cm)	Cl (mg/kg)	SO_4 (mg/kg)	Classification for Concrete	Classification for Steel
302	1.0	B	9	430	390	260	Non-aggressive	Non-aggressive
302	2.0	B	8.9	430	350	300	Non-aggressive	Non-aggressive
303	0.5	B	7.8	340	55	570	Non-aggressive	Non-aggressive
307	0.6	B	5.5	370	330	280	Mild	Non-aggressive
312	0.4	B	4.8	200	130	190	Mild	Non-aggressive
602	-	Water	6.9	9200	3000	250	Non-aggressive	Severe

Notes to Table 7:
EC: Electrical Conductivity SO_4: Sulphates   Cl: Chlorides
Soil Type A: High permeability soils (e.g. sands and gravels) which are in groundwater
Soil Type B: Low permeability soils (e.g. silts and clays) or all soils above groundwater

The classifications for concrete and steel are based on the Piling Code AS 2159 (Ref 2).

7.3 Analytical Results - Combustibility

Laboratory testing was undertaken by SGS Australia, a NATA registered laboratory. Analytical methods used are shown in the laboratory sheets in Appendix C.

A total of seven soil samples were selected to provide an assessment of combustibility conditions at potential areas of risk. The samples were analysed for a range of analytes as follows:

- Ash;
- Volatile Matter;
- Fixed carbon;
- Total sulphur;
- Gross calorific value.

The results of analysis undertaken on soil samples are summarised in Table 8 below.

Table 8: Results of Soil Combustibility Testing

Location	Co-ordinates		Soil Description	Ash %	Combustibles %	Volatile Matter %	Fixed Carbon %	Total Sulphur %	Gross Calorific Value (MJ/kg)
	Easting	Northing							
TP303/0.2	369305	6374607	Black silty sand (coal fines)	34.09	**65.9**	29.0	71.0	0.64	19.69
TP305/0.1	369302	6374626	Black fine to medium grained sand (coal fines)	76.8	23.2	9.9	90.1	0.17	5.56
TP310/0.4	369203	6374465	Brown, mixture of clay, sand and gravel (coal reject, carbonaceous siltstone and sandstone)	69.76	30.2	13.5	86.5	0.23	8.16
TP314/0.3	369428	6374335	Dark brown / black gravel (coal reject) with some sand, clay and cobbles (carbonaceous siltstone), some sulphidic staining	65.7	34.3	15.4	84.6	0.54	9.29
TP316/0.6	369375	6374427	Black coal, highly weathered	11.85	**88.2**	34.0	66.0	1.72	29.34
Stockpile 1	369372	6374366	Dark grey to black, silty sand (coal reject)	79.69	20.3	9.9	90.1	0.24	3.81
Stockpile 2	369433	6374379	Dark grey to black, silty sand (coal reject)	48.57	**51.4**	22.1	77.9	0.48	14.06

Notes to Table 8:

Results on a dry weight basis

Bold results exceed Wollongong City Council DCP Guidelines Chapter E19 Earthworks (Land Reshaping Works) for Absolute Maximum Combustibles - 40% (Ref 3)

8. Proposed Development

The key features of the Concept design include the following:

- A building envelope within the north-western part of Lot 7314 mostly on previously disturbed areas;
- Building mass of up to eight storeys (including lower ground but excluding plant level);
- Approximate gross floor area of 60,000 square metres;
- Primary access from Metford Road at a new roundabout junction opposite Fieldsend Street;
- Secondary access for ambulances from Metford Road, south of the Primary access. This allows for accessing the hospital from both points;
- An on-grade helipad located to the northeast of the hospital;
- Two car parking facilities;
- Stormwater drainage to the east of the hospital, to manage stormwater flows through the site from offsite lands as well as from the hospital precinct;
- Partial retention of vegetation at the southern, western and eastern end of the site, with some clearing and understorey removal to meet bushfire mitigation requirements (APZ).

There were no detailed plans on the buildings or car parks at the time of the investigation, but it is believed that the final bulk earthworks level for the development will be approximately RL 18. Column loads are not known at this stage.

9. Comments

The geotechnical risks identified for the proposed development together with the issues outlined in Section 1 are discussed in the following sections.

9.1 Earthworks and Site Preparation

The earthworks associated with development of the site are expected to include excavation and re-compaction of existing fill materials and the placement and compaction of relocated or imported fill materials.

It is understood that bulk excavation is likely to be required to level the site to about RL 18 m AHD. Areas of the site, particularly within the north-western and southern strip, are at levels of up to RL 24 m AHD. Hence excavation of up to 6 m depth may be required. Based on conditions encountered within the bores, the strength of the bedrock within the upper 6 m is generally extremely low to low strength and would be commensurate to Class V and Class IV sandstone, as defined in Ref 13.

Depending on the final layout of the development, placement of up to 8 m of filling may be required within the former quarry floor.

Excavation through fill materials, in-situ clays and the zones of weaker rock (extremely low to low strength) is expected to be relatively straightforward using conventional excavation equipment such as excavators fitted with rock teeth, possibly a pneumatic or hydraulic hammer for harder zones and detailed excavation. It is noted the rock strength encountered in the boreholes ranged from very low to medium strength, generally the sandstone layers exhibit higher strength than siltstone and claystone which tend to weather more rapidly.

The production rate is expected to decrease as rock strength increases and fracture spacing is wider, such as the medium strength sandstone observed at the site. The excavation of Class III sandstone or Class II siltstone is likely to require a rock hammer or medium to heavy ripping by D9 dozer (or larger).

The following procedure is generally suggested for placement and compaction of engineered filling over existing natural soils/rock but the final specification should be confirmed once the development layout has been confirmed:

- Remove vegetation, topsoil, uncontrolled filling and deleterious materials;
- Test roll the surface in order to determine any soft zones and assess moisture condition. Moisture contents should be in the range OMC -2% (dry) to OMC+2% where OMC is the optimum content at standard compaction;
- Compact the tyned natural surface (where rock is not exposed) to a dry density ratio of at least 100% Standard. The re-compacted clay should be left exposed for a minimum of time prior to placement of additional fill layers, to minimise the occurrence of desiccation cracking or softening;
- Suitable filling should be placed in horizontal layers not exceeding 300 mm loose thickness and compacted to a dry density ratio of at least 95% to 100% Standard up to within 0.5 m of design bulk earthworks level and to at least 100% Standard in the upper 0.5 m. Moisture content should be in the range as stated above;
- Particle size should generally by less than 150 mm, however, an occasional absolute maximum particle size of up to 200 mm could be used;
- Unsuitable material at cut / fill transitions should be over excavated and replaced with select fill;
- Where carbonaceous materials are to be used as controlled fill, the filling should be blended with non-carbonaceous materials and placed in horizontal layers not exceeding 300 mm loose thickness and compacted to a dry density ratio of at least 100% Standard, as determined by test methods AS1289.5.1.1 (Ref 4) and AS1289.5.4.1 (Ref 11). Moisture content determinations are to be in accordance with AS1289.2.1.1 (Ref 12) with the exception that a 50°C oven is to be used (Ref 3).

It is noted that the existing clay and silty clay soils are of medium to high plasticity and likely be difficult to work, particularly when wet. Site trafficability will be reduced when these soils become wet. If the soils become wet, they should be tyned and allowed to dry. Careful control of moisture will be required during compaction of these soils.

In the event that unfavourable weather conditions occur prior to and during construction, trafficability for non-tracked plant is expected to be very poor in the lower parts of the site and therefore the use of a layer of granular crushed rock, crushed recycled concrete, or similar may be required over the natural clays to provide a working platform for temporary access roads.

Alternatively, structures could be supported on piles installed through the fill to rock, as discussed in Section 9.11.

9.2 Additional Earthworks Procedures for Filling of Former Quarry

It is understood that filling of the former quarry may be required as part of the development of the site. In order to raise site levels to RL 18 m AHD, this could involve the placement of up to 8 m of filling.

If the filling is to be utilised to support structures or pavements, then it is recommended that the filling is placed in accordance with the procedures outlined in Section 9.1 above. In the event that all structures will be supported on piles, founded in the sandstone or siltstone below the filling and near surface coal layers, the thickness of each layer of filling may be able to be increased, depending on the compaction equipment used during earthworks.

The most appropriate methods will depend on the depth of filling to be placed, expected settlements and structural tolerances to settlement, however shallow dynamic compaction using a purpose-built impact roller may be suitable. The effective depth of compaction varies with soil conditions but is typically in the order of 1 m.

Any proposed ground improvement measures should be preceded by specific investigation and design, and possibly trials to determine the most suitable method.

Regardless of the depth of filling to be placed within the base of the former quarry, the following additional preparation measures will be required:

- Dewatering of main pond and other areas of ponded water;
- Given the low lying nature of this area of the site, a dewatering plan is anticipated to be required to allow earthworks;
- Removal of all accumulated sediments and filling, following appropriate assessment for disposal or reuse. In this regard, up to 0.8 m of filling, comprising soft silty clay with significant anthropogenic inclusions such as brick fragments were encountered in the pits undertaken in this area of the site (refer Figure 22). Penetrometer testing at Location 322 indicated that the sediments were less than 0.2 m thick at this location.

Figure 22: Filling encountered in low lying area of the site

9.3 Retaining Structures

The design parameters for retaining structures for both short-term and long-term situations are shown in Table 9 for each of the principal geological units. These would apply to the design of cantilevered walls or shoring, contiguous pile walls and soldier pile walls. Any walls that will form part of a permanent structure should be based on long-term parameters.

The design values given are based on level ground behind the wall and do not include any surcharge loads that may be imposed near the top of the wall.

Passive pressures are given as either an earth pressure coefficient (Kp) for granular soils and / or ultimate passive pressures for residual clays and rock. The values given below are unfactored and because passive pressures are used as a resisting force, an appropriate factor of safety should be applied to determine working values and to limit deflections.

Table 9 provides suggested preliminary design parameters.

Table 9: Preliminary Design Parameters for Retaining Structures

Description	Design Parameters (unfactored)								
	γ_b kN/m^3	Short Term				Long Term			
		K_a	K_0	K_p	P_p kPa	K_a	K_0	K_p	P_p kPa
Filling –cohesive	18	0.25	0.40	3.5	-	0.30	0.60	3.5	-
Filling - granular	20	0.30	0.50	3.3	-	0.30	0.50	3.3	-
Clay – stiff to very stiff	19	0.25	0.40	-	200	0.30	0.60	2.5	-
Clay – hard / extremely weathered rock	19	0.25	0.40	-	200	0.35	0.55	3.0	-
Class V Sandstone / Class IV Shale (extremely low strength sandstone or siltstone)	22	0.15	0.30	-	400	0.20	0.35	-	400
Class IV Sandstone / Class III Shale (very low to low strength sandstone or siltstone)	22	0.00	0.10	-	2000	0.10	0.25	-	2000
Class III Sandstone / Class II Shale (medium strength sandstone or siltstone)	22	0.00	0.10	-	3000	0.10	0.25	-	3000

Notes to Table 9:

K_a – Active earth pressure coefficient

K_0 – 'At-rest' earth pressure coefficient

K_p – Passive earth pressure coefficient

P_p – Passive earth pressure

The use of active pressure coefficients (K_a) requires that there will be sufficient deflection of the retaining system during construction to reach active conditions. If lateral deflections are prevented or restricted, at-rest coefficients (K_0) should be used.

Any surcharge loads such as pavements, construction equipment or sloping backfill should be added to the design pressures determined for the soil / rock profile alone. Below the water table the additional load due to hydrostatic pressure should be added.

The parameters provided above are based on the provision of full drainage behind the retaining walls.

9.4 Batter Slope Stability

During site works temporary batters may be needed to facilitate construction, such as building retaining walls or filling in quarry voids with controlled fill that will cover the slope.

Permanent batter slopes may be required where they form part of the completed development. Preliminary assessment may be based on the slopes provided in Table 10.

Table 10: Temporary and Permanent Batter Slopes

Stratum	Short Term (Temporary)(1)	Long Term (Permanent)(1)
Fill - Compacted	2H:1V	2.5H:1V
Residual Clays	1.5H:1V	2H:1V
Class V Sandstone / Class IV Shale (extremely low strength sandstone or siltstone)2	1H:1V	1.5H:1V
Class IV Sandstone / Class III Shale (very low to low strength sandstone or siltstone)2	0.5H:1V	1H:1V
Class III Sandstone / Class II Shale (medium strength sandstone or siltstone)2	Vertical	0.25H:1V to Vertical (3)

Notes to Table 10:

1. Above values are for a maximum vertical height of 3 m. Greater depths to be specifically assessed, and may require measures for stability and drainage.
2. Batters in rock are dependent on jointing and will require confirmation at time of excavation.
3. Vertical cuts may be feasible subject to geological mapping and rock bolting or similar, if needed.

All batter slopes will require appropriate erosion protection measures, depending on the location, depth, drainage conditions and longevity of the slope in question.

In this regard, the former quarry excavation face has been formed at around 22°, which is about 2.5H:1V. No signs of gross instability were observed within this batter, although it is noted that some erosion has taken place. It is noted that some soils on the site returned high pH values. Careful selection of vegetation which is tolerant of these pH levels will be required on these slopes.

9.5 Existing Filling

Based on the conditions encountered in the pits and bores, filling to depths (below surrounding ground surface levels) of greater than about 1 m is generally restricted to the north-western area and south-western area. Stockpiles of filling, which are about 5 m to 10 m in height are also present.

Filling of up to 2.5 m depth was encountered in pits excavated in the south-western area of the site, close to the boundary with the north-western area of the site. This filling is associated with the fingers of filling which appear to be present in this area.

Within the eastern, low lying area, filling was encountered to depths of up to 0.8 m and generally comprised sandy clay with some coal fines, occasional bricks and gravel.

9.6 Suitability of Site Materials for Reuse as Filling

9.6.1 Soils and Extremely Weathered Rock

The results of the investigation suggest a clayey profile up to a depth of greater than 1 m in the proposed location of the New Hospital (southern portion) and up to 1.8 m elsewhere in the site. The clay material was typically described as having very stiff or better consistency.

Figures 23 and 24, below, present a graphical summary of soil plasticity and shrink-swell data obtained for soils in the vicinity of the site during the previous investigation by DP (Ref 1).

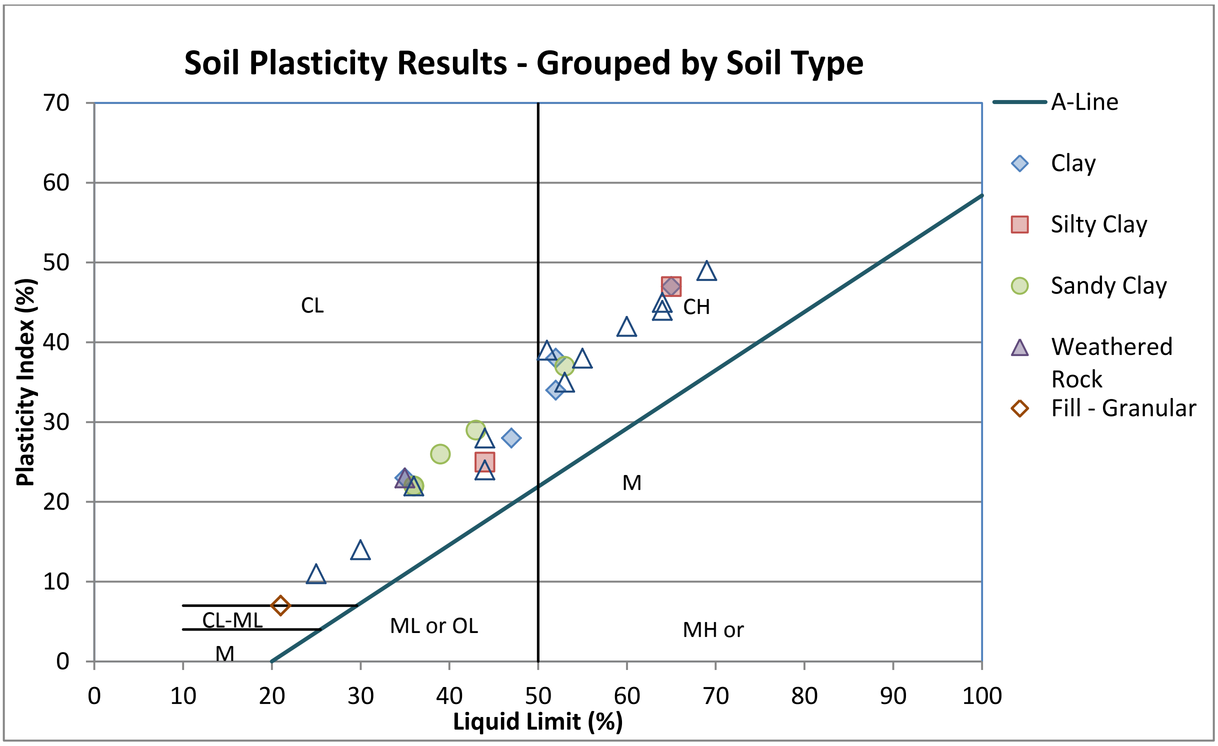

Figure 23: Chart of Soil Plasticity Tests

Available data on clay reactivity in terms of the shrink-swell index is shown plotted in Figure 24. California bearing ratio (CBR) results are graphed in Figure 29.

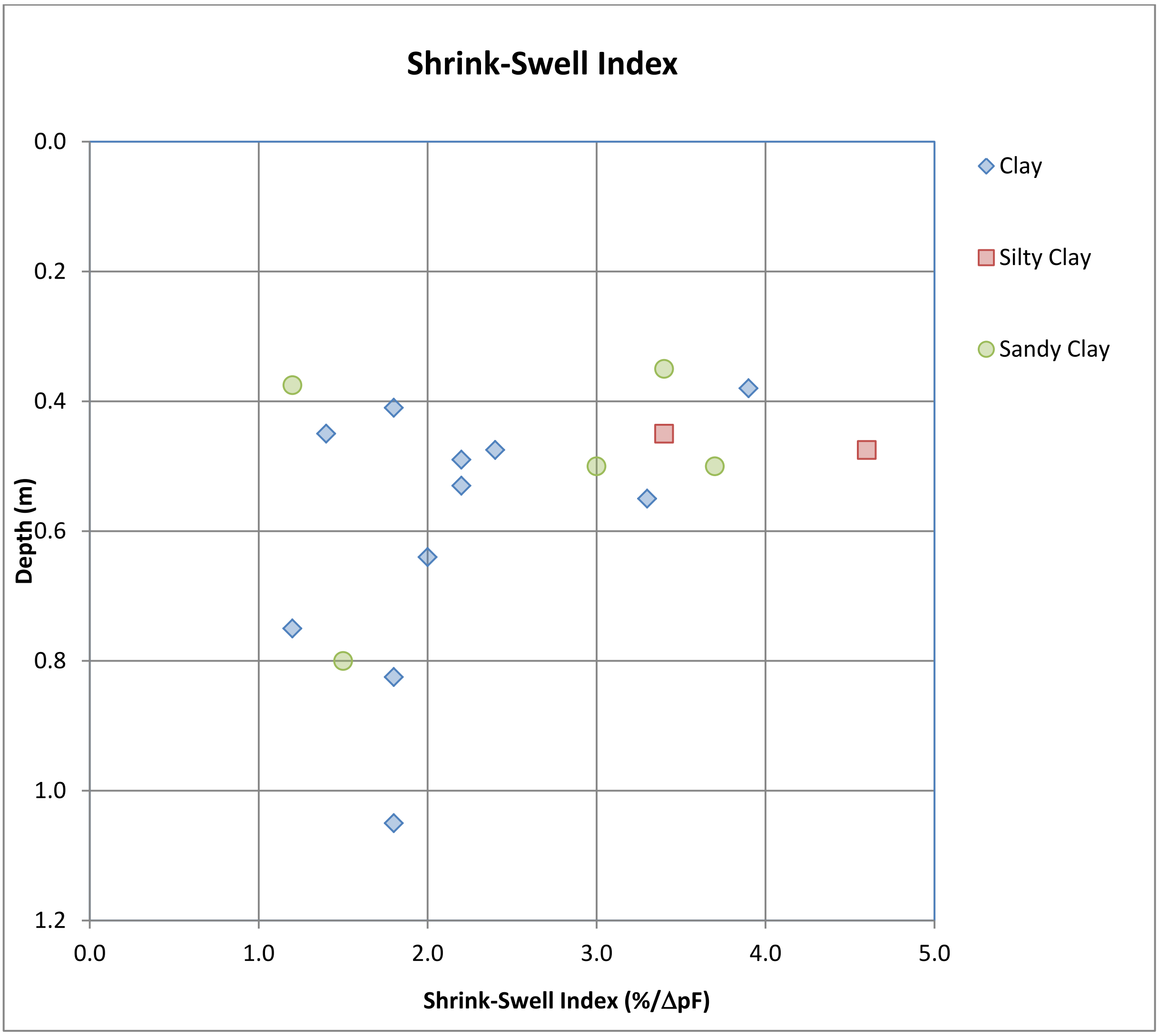

Figure 24: Chart of Shrink-Swell Index Tests

Laboratory testing on samples retrieved from the current investigation and the information provided above indicate that the clay has a moderate propensity to change in volume with changes with moisture content, as indicated by high shrink-swell values on remoulded samples up to 3.0% and swells of up to 1.2% during soaked CBR testing. The results also indicate that the underlying clays have a moderate potential to soften on exposure to moisture, as indicated by the low soaked CBR values (as low as 5%).

As indicated above and presented in Section 9.9, large characteristic surface movements associated with the use of the high plasticity clays needs to be considered if the clay is to be used beneath building areas. The use of at least 1.0 m of low-reactivity filling over the high plasticity clay will reduce the characteristic surface movement of the final profile.

The soils present within the eastern, low lying, area of the site are anticipated to have elevated moisture content and are not likely to be suitable for re-use without significant moisture reconditioning.

9.6.2 Weathered Rock

The ability to re-use weathered rock material won from bulk earthworks will be a function of the particle size achieved during excavation. In general, particle size should not exceed 150 mm for re-use as engineered filling. The purpose of this is to reduce the likelihood of difficulties if excavations are required at a later date, for example, to install footings and to reduce void spaces which could lead to piping/erosion as well as to assist with compaction. The material should be well-graded to control the compacted density and void ratio. All material to be re-used as engineered filling should be free of organics and other deleterious materials and debris.

It is anticipated that very low to medium strength weathered siltstone and sandstone encountered in Pits 301 to 305, and Bores 601 to 606, could be readily excavated / ripped to a particle size of less than about 0.3 m but more likely less than 0.15 m. It is suggested that cross-ripping methods and heavy pad foot rollers could be used to further break down excavated bedrock to material with a maximum particle size of less than 150 mm.

It may be possible to use the coal within the bulk filling provided it is blended with other non-combustible material (refer Section 9.7).

9.6.3 Stockpiled Material

It is considered that the existing stockpiled material (predominantly ripped sandstone and siltstone with some clay) would be suitable for reuse as engineered fill in embankments or in building pads, provided that the material is moisture conditioned to achieve a moisture content at the time of placement as recommended in Section 9.1. Coarse material within the stockpile would need to be broken down such that the maximum particle size is no greater than 200 mm.

9.7 Assessment of Soil Combustibility

Based on the results of the investigation, potentially combustible material was encountered at the following locations:

- Pits 303 and 305 Black coal filling to depths of up to 0.25 m in the north-western area of the site;
- Pit 310 Clayey gravelly filling with some coal and carbonaceous siltstone to 1.2 m depth, within the “fingers” of filling reaching into the south-western area of the site;
- Pit 314 Coal and carbonaceous gravel filling to 0.7 m depth over bedrock in the southern strip, adjacent to the former quarry face; and
- Pit 316 Exposed coal, as encountered in Pit 316 and also within the floor of the former quarry to the east of the pond, together with the lower sections of the former quarry face.

The results of laboratory testing on the collected samples indicated the presence of combustible materials in all of the samples tested. Approximately of the samples tested (3 out of 7 tests) also exhibited combustible percentages exceeding the adopted criteria of 40% combustible material (Ref 3).

Based on the assessment the stockpiled carbonaceous materials within the southern and central part of the site will require some form of management and remediation to minimise and mitigate future combustibility risks.

DP has not considered chemical concentrations of the carbonaceous materials in this assessment. Recommendations from GHD in relation to contamination issues at the site may render the material unsuitable for re-use. However, provided that the material is deemed suitable for re-use from a contamination viewpoint, it may be suitable as bulk filling provided the recommendations outlined below are followed.

Based on the findings of the assessment, remediation of the identified combustion risk areas is likely to comprise one or more of the following mitigation measures:

- Capping – placement and compaction of a non-combustible layer of at least 0.5 m thickness over the identified potentially combustible materials;
- Removal of coal materials – excavation of accessible areas of concentrated combustible materials for disposal and/or reuse as part of blending operations;
- Reshaping works – earthworks in areas of potential combustible materials to reduce slope angles and compact loose materials to minimise the potential for oxygen ingress and subsequent combustion;
- Blending – mixing of highly combustible material with non-combustible material to reduce the overall combustible percentage of the material and minimise the risk of combustion. Blending may be with underlying soil/rock (e.g. ripping of near-surface coal with underlying soil/rock) or mixing with imported materials.

In addition to the above remediation recommendations, areas of capping, coal removal, reshaping and blending should be mulched and vegetated as part of remediation works.

Development of detailed procedures and specifications for remediation will be required prior to remediation works. A combustion risk mitigation program, including a Remediation Action Plan will be required which will outline the procedures, specifications and responsibilities for remediation works at the site.

It is also recommended that a management plan is prepared for future management of combustion risk at the sites.

9.8 Drainage

A site inspection was undertaken by a Principal Geotechnical Engineer from DP on 5 November 2015 after a rain event. Water ponding was observed within the densely vegetated area in the south-western portion of the site, adjacent the natural drainage path. Figures 25 to 27 show the area at the time of the inspection.

Figure 25: Looking SSW at water ponding (co-ordinates: 369127, 6374429)

Figure 26: Looking SSE at overland flow (co-ordinates: 369112, 6374419)

Figure 27: Looking north at water ponding (co-ordinates: 369091, 6374344)

The area of overland flow / ponding observed is highlighted in Figure 28 below.

Figure 28: Aerial with 2 m contours, site boundary (red), area of ponding / overland flow (blue)

It is understood that placement of up to 5 m of filling may be required in this area to raise the site to RL 18 m AHD. Consideration should be given to the design of earthworks to maintain flow of groundwater and surface water within this natural drainage path. Installation of an appropriately sized geotextile encapsulated gravel ‘drainage blanket’ or pipes would be appropriate prior to placing controlled fill in this area.

The proposed development should be designed such that all proposed excavations, such as those for footings and installation of services etc, will not intersect the drainage blanket, thus preserving the integrity of the drain. If any accidental damage is caused to the drainage blanket, it should be fully repaired to ensure that the functionality of the drain is not compromised. Satisfactory repair should be verified by a geotechnical engineer.

Alternatively, construction of an appropriately designed subsurface drainage system could be adopted.

9.9 Site Classification

Site classification to AS 2870-2011 (Ref 5) is not strictly applicable to this site due to it being a hospital development rather than a residential development. However, the principles of footing design and site maintenance presented therein are considered relevant for the type of buildings anticipated to be proposed for the site.
Site classification of foundation soil reactivity provides an indication of the propensity of the ground surface to move with seasonal variation in moisture. The site classification is based on procedures presented in AS 2870-2011 (Ref 5), the typical soil profiles revealed in the boreholes and test pits and the results of laboratory testing.

Owing to the presence of filling to a depth of greater than 0.4 m, the site is classified as **Class P**.

It is understood that excavations up to 6 m depth and fills of up to 8 m may be required for the development.

Consideration should be given to the possibility of reactive movements under constructed buildings. Where filling is to be used, it is recommended a low reactive material (i.e. sandy gravelly material) (I_{SS} <1.0%) be used within the upper 1 m of the profile (at least) to reduce potential shrink swell movements. It is anticipated that the excavated sandstone material may be suitable for this purposes, provided it has a maximum particles size of 200 mm but preferably less than 100 mm. Provided at least 1 m of low reactivity material is placed at the top of the fill profile, the range of characteristic surface movements, y_s values, is estimated to be about 15 mm to 20 mm but creep settlement of compacted filling could be of a similar magnitude.

Masonry walls should be articulated in accordance with TN61 (Ref 6) to minimise the effects of differential movement.

9.10 High Level Footings

It is preferable that all footings found on material of similar stiffness, (i.e. all footings to bear on the bedrock). Careful consideration of detailing and articulation of the structures would be needed during the design to allow the structures to tolerate the differential movements which could result from founding on materials of differing stiffness, such as bedrock and clay or filling.

Spread footings founded within the relevant strata should be designed for the following ultimate limit state bearing pressures:

- Extremely low to very low strength siltstone or sandstone 3000 kPa;

- Low strength siltstone or sandstone 5000 kPa;
- Medium strength or stronger sandstone 10000 kPa.

A factor of safety of 3 should be applied to these values to derive an allowable bearing pressure but serviceability (settlement) analysis needs to be undertaken during detailed design.

Carbonaceous sandy clay (weathered coal) was encountered in Pit 306 from 0.15 m to 0.6 m, the shear strength of which is highly sensitive to changes in moisture, and while stiff to very stiff in their current state, they will soften appreciably if subjected to increased moisture.

Therefore, if carbonaceous clays are encountered, the design and construction of the footings must account for the moisture-sensitive ground conditions. Associated risks include a loss of bearing capacity if the soils become saturated for any reason, and differential movement of footings with variations in soil moisture.

9.11 Deep Footings / Piles

Deep foundation systems would be appropriate for the support of major loads and where the presence of uncontrolled fill precludes the use of shallow footings. Bored piles would be suitable founded within the very low to low strength sandstone bedrock. Given the presence of shallow bedrock driven piles are not considered suitable for this site.

The recommended preliminary design parameters for piles are shown in Table 11.

Table 11: Design Parameters for Piles

Stratum	Ultimate		Serviceability (Working Loads)	
	End Bearing (kPa)	Shaft Adhesion (kPa)	End Bearing (kPa)	Shaft Adhesion (kPa)
Clay – hard / extremely weathered rock	1800	80	600	50
Class V Sandstone / Class IV Shale (extremely low strength sandstone or siltstone)[2]	4000	200	1200	100
Class IV Sandstone / Class III Shale (very low strength sandstone or siltstone)[2]	10000	500	2500	250
Class III Sandstone / Class II Shale medium strength sandstone or siltstone)[2]	20000	1100	4000	450

Notes to Table 11:

1. The design bearing pressures should be adjusted to account for weaker layers below the bearing layer if present.
2. Piles founded on coal or claystone should be avoided due to potential for softening and excessive settlement.
3. Ultimate Values occur at large settlements (> 5% of minimum pile diameter / width)
4. Design geotechnical strength ($R_{d,g}$) should initially be based on a strength reduction factor of $\phi_g = 0.55$
5. Shaft adhesion values based on a shaft roughness of R2 or better
6. Serviceability / Max Allowable end bearing to cause settlement of < 1% of minimum pile diameter / width
7. AS 2159 – 2009 requires that the contribution of the shaft from ground surface to 1.5 times pile diameter or 1 m (whichever is greater) shall be ignored

It should be noted that the above design parameters given in Table 11 are primarily for bored piles with clean sockets and bases. Specific cleaning buckets and grooving tools should be used in pile construction. Pile installation could be affected by the possible presence of obstructions within existing fill such as concrete, steel and other coarse inclusions. The available information suggests that this will not be a widespread problem however the possibility cannot be precluded. Piles founded within the coal or within 2 m above the coal should be designed for Class V parameters. More detailed investigations should be undertaken once the final design of the building has been established to confirm suitable foundation depths.

For piles in tension, the shaft adhesion parameters should be reduced by 25%.

During construction the design bearing pressures should be confirmed by geotechnical inspection and / or quality assurance testing relevant to the type of pile and method of installation.

9.12 Assessment of Soil Aggressiveness

The results of laboratory testing of soil samples collected during field work have been compared to the exposure classifications for steel and concrete as outlined in AS 2159–2009, Piling–Design and Installation (Ref 2), and results are provided in Table 7 of Section 7.2.

The soil results indicate a non-aggressive to mild classification for concrete and a non-aggressive classification for steel. The water results indicate a non-aggressive classification for concrete and a severe classification for steel.

All items in contact with the ground should be suitably protected against decay or corrosion, possibly by allowing for a sacrificial thickness, reducing working loads accordingly and / or designing for a more severe classification as the aggressiveness may change over time.

9.13 Pavements

The possible pavement layout, traffic spectrum or other design parameters for pavement is not known at this stage. The following advice in relation to pavement design is present for preliminary planning purposes only and should be reviewed once the layout of the development is known.

The following pavement thickness design is in accordance with Austroads – Guide to Pavement Technology (Ref 7).

9.13.1 Design Traffic

Details on proposed traffic loadings were not available at the time of this assessment.

Flexible pavement thickness designs have been provided for design traffic loadings as follows:

- 4×10^3 Equivalent Standard Axles (ESA); and
- 9×10^5 Equivalent Standard Axles (ESA).

If the traffic loading is to be significantly different from the above, the pavement thickness should be reviewed.

9.13.2 Subgrade CBR

The results of the investigation indicate a range of possible subgrade types across the site, from residual clay, exposed bedrock to future controlled filling.

Laboratory testing on samples retrieved from the pits indicate a CBR range of about 3% to 9%.

The previous Geotechnical Assessment (Ref 1) suggested a CBR range of 3% to 5%, based on a data assessment of CBR laboratory testing taken at and adjacent to the site, as shown in Figure 29.

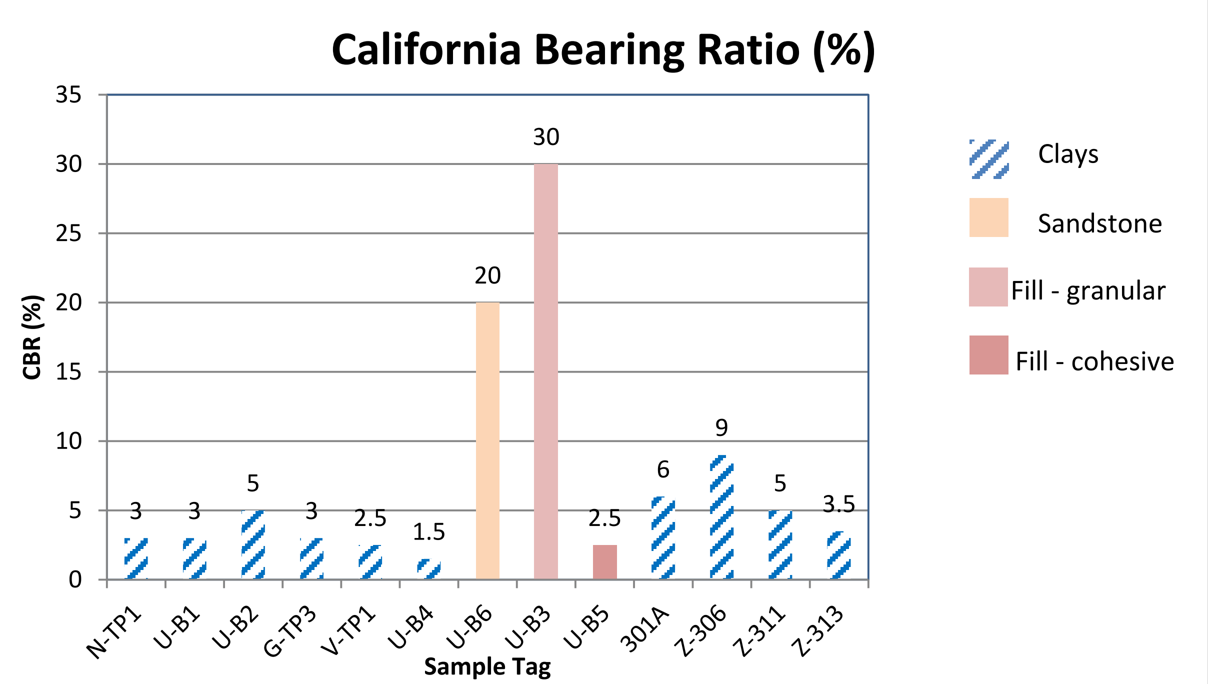

Figure 29: Graph of CBR Test Results from previous investigation

As can been seen from Figure 28, for clay soils, CBR values ranging from 1.5% to 5% were returned, which is generally consistent with the results of testing from the present investigation.

Therefore, a design subgrade CBR of 3% has been adopted.

9.14 Flexible Pavement Thickness Design

The indicative flexible pavement thickness design is presented in Table 12, below.

Table 12: Flexible Pavement Thickness

Pavement Layer	Thickness (mm)	
	4×10^3 ESA	9×10^5 ESA
Wearing Course	30[(1)]	40[(1)]
Basecourse	100	120
Subbase	170	350
Total	300	510

Notes to Table 12:
(1) Thickness of AC10. A 7 mm prime seal should be placed over the basecourse prior to placement of the AC.

The pavement thickness presented above is dependent on the provision and maintenance of adequate surface and subsurface drainage. Surface grades should be sufficient to prevent ponding of stormwater.

It is expected that there may be an increased maintenance requirement in areas of tightly turning vehicles due to the high shear / torsional stresses applied to the pavement surface. The use of a stiffer binder (i.e. Class 320 bitumen) in the asphalt would be expected to reduce the damage to the asphalt surface in areas of tightly turning vehicles.

The recommended material quality and compaction requirements for sealed flexible pavement are presented in Table 13, below.

Table 13: Material Quality and Compaction Requirements – Sealed Flexible Pavement

Pavement Layer	Material Quality	Compaction Requirements
Basecourse	CBR ≥ 80%, PI ≤ 6%. Grading in accordance with SR41 (Ref 8)	Compact to at least 98% dry density ratio Modified (AS 1289.5.2.1, Ref 9)
Subbase	CBR ≥ 30%, PI ≤ 12%. Grading in accordance with SR41 (Ref 8) or Maitland City Council specifications	Compact to at least 95% dry density ratio Modified (AS 1289.5.2.1, Ref 9)
Select Subgrade	Soaked CBR ≥ 15%	Compact to 100% dry density ratio Standard (AS 1289.5.1.1, Ref 4)
Subgrade	CBR ≥ 3.0%	Compact to at least 100% dry density ratio Standard (AS 1289.5.1.1, Ref 4)

Notes to Table 13:
CBR – California bearing ratio (4 day soaked)
PI – Plasticity Index

9.15 Subgrade Preparation

The following procedure is recommended for preparation of the pavement subgrades:

- Excavate to design subgrade level. To reduce the risk of rutting and subgrade damage it is recommended that wheeled / rubber tyred vehicles and equipment should be kept off the subgrade, where possible;
- Remove any additional topsoil or deleterious materials;
- Where bedrock is encountered at subgrade level, it should be tyned to 150 mm depth to destroy any possible preferential moisture flow paths;
- Proof roll the excavated surface to assess moisture content and soft zones. Remove soft zones and replace with compacted approved filling. Moisture contents should be in the range -3% (dry) to OMC, for pavements where OMC is the optimum moisture content at standard compaction. If wet subgrade conditions are encountered, the material should either be tyned and allowed to dry or removed and replaced with a select subgrade (CBR>15%). The depth of any excavation should be confirmed by geotechnical inspection;

- Compact the subgrade to a minimum dry density ratio of 100% Standard (AS 1289.5.1.1, Ref 4). The compacted subgrade should be left exposed for a minimum amount of time prior to placement of pavement layers to minimise the occurrence of subgrade deterioration due to the effects of weather;
- If raising of the subgrade level is required, all deleterious materials should be removed. Approved filling should then be placed in layers not exceeding 250 mm loose thickness and compacted to a minimum dry density ratio of 100% Standard at the moisture content described above.

Geotechnical inspections and testing should be undertaken during construction in accordance with AS 3798-2007 (Ref 10).

9.16 Quarry Slope Stability

The principle batter associated with the former quarry is orientated in a roughly east-west alignment and is shown on Drawing 2.

This batter has been formed at approximately 20° to 25° below the horizontal and exposed extremely low to low strength siltstone and sandstone, with very low strength coal within the lower 1 m to 2 m at the eastern end of the batter (refer Figures 15 and 16 of Section 3.4 and Figures 20 and 21 of Section 6.

The western cut batter associated with the former quarry lies to the west of the main pond (refer Drawing 2). This batter has been formed at a range of slopes, generally less than about 2.5H:1V. Where clear of vegetation, the materials exposed within the batter included very low to low strength sandstone with some pebbly sandstone in parts (refer Figures 30 and 32).

Figure 30: View across former quarry batter and pond to principal former quarry batter

Figure 31: Exposed sandstone in former quarry excavation batter

No signs of gross instability were observed within either of these batters.

Towards the upper section of the former quarry excavation batter, filling has been placed. Inspection of local erosion indicates that the filling is generally shallow (less than about 1 m) and is underlain by sandstone bedrock (refer Figure 32).

Figure 32: Localised filling underlain by sandstone in quarry batter

It is not anticipated that stabilisation measures, beyond removal of the existing filling and provision of erosion protection will be required for these batters.

It is recommended that in the event that structures are to be placed within 5 m of the crest of the existing batters, additional analysis should be undertaken to assess any additional, localised stabilisation measures that may be required.

9.17 Sediment Control

A preliminary assessment of soil erodibility was undertaken during the previous assessment by DP using the available soil test results and with reference to the Landcom "Blue Book". The results are shown in Table 14.

Table 14: Soil Erodability Assessment

Soil Type	K Value	Erodibility Category
Clay	0.07	Very Low
Silty Clay	0.08	Low
Sandy Clay	0.16	Moderate

During the present investigation, there was some evidence of turbid water in existing surface water ponds which suggests that site soils have a tendency for dispersion, even though the limited available laboratory testing reviewed during the previous assessment indicated a moderate dispersive potential. Accordingly, allowance should be made to implement standard sediment control measures to manage potential sediment. During construction activities this could include silt fences and hay bales, while long-term measures could include suitable drainage, vegetating slopes, erosion mats, hydro-mulch, rock blankets and scour protection. It is noted that pH values of up to 9 pH units were recorded in some soil samples tested from the site.

9.18 Seismic Design

The earthquake code (AS1170.4-2007, Ref 14) provides design factors based on location (earthquake risk) geotechnical conditions.

The Hazard Factor (Z) for Maitland is 0.10 as given in Table 3.2 of AS1170.4. This is the bedrock acceleration coefficient with an annual probability of exceedance of 1 in 500.

The Site sub-soil class is assessed to be Class C_e – “shallow soil site”, with reference to Table 4.1 of AS1170.4.

9.19 Acid Sulfate Soils

The whole of the site is outside the mapped areas of potential acid sulfate soils.

Coal and other materials with high carbon content can also contain sulphide ores which may lead to acid generation upon oxidation. Generally this is not an issue when the carbonaceous materials are thoroughly blended with other soils.

9.20 Mine Subsidence

The site is not within a mine subsidence district and hence proposed developments do not require the approval of the Mine Subsidence Board. The nearest proclaimed mine subsidence district is “East Maitland” located approximately 800 m to the south-west of the site.

The available information suggests that the site lies near the base of the Tomago Coal Measures and it is unlikely that coal mining has occurred or will occur beneath the site.

10. References

1. Douglas Partners Pty Ltd, "Report on Geotechnical Assessment, New Maitland Hospital, Metford", Report 81719.00.R.001.Rev1, dated 15 July 2015.
2. Australian Standard 2159-2009, "Piling – Design and Installation", Standards Australia.
3. Wollongong City Council, "Wollongong Development Control Plan 2009, Part E: General Controls – Environmental Controls, Chapter E19: Earthworks (Land Reshaping Works)".
4. Australian Standard AS 1289.5.1.1-2003, "Methods of testing soils for engineering purposes", Standards Australia.
5. Australian Standard AS2870-2011, 'Residential Slabs and Footings', April 2011, Standards Australia.
6. Cement Concrete Aggregates Australia, Technical Note 61 "Articulated Walling".
7. Austroads, "Guide to Pavement Technology, Part 2: Pavement Structural Design", 2012.
8. Australian Road Research Board, Special Report No.41, "Into a New Age of Pavement Design, A Structural Design Guide for Flexible Residential Street Pavements", dated 1989.
9. Australian Standard AS 1289.5.2.1-2003, "Methods of testing soils for engineering purposes", Standards Australia.
10. Australian Standard AS 3798-2007 "Guidelines on Earthworks for Commercial and Residential Developments", Standards Association of Australia.
11. Australian Standard AS 1289.5.4.1-2007, "Methods of testing soils for engineering purposes", Standards Australia.
12. Australian Standard AS 1289.2.11-2005, "Methods of testing soils for engineering purposes", Standards Australia.
13. P.J.N.Pells, G. Mostyn and B.F.Walker, "Foundations on Sandstone and Shale in the Sydney Region", Australian Geomechanics – December 1998.
14. Australian Standard 1170.4-2005, "Structural design actions, Part 4: Earthquake actions in Australia", Standards Australia.
15. Douglas Partners Pty Ltd, "Report on Geotechnical Investigation, Proposed New Maitland Hospital, Metford Road, Metford", Report 81719.01.R.001.Rev0, dated 14 December 2015.

11. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at New Maitland Hospital, Metford in accordance with DP's proposal NCL170350 dated 4 July 2017 and acceptance received from Rebecca Willott of Health Infrastructure dated 5 July 2017. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Health Infrastructure for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

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

Douglas Partners Pty Ltd