Civil

TaylorThomsonWhitting

Civil Design Statement for Graythwaite Rehabilitation Centre, Ryde Hospital

for Aurora Projects Pty. Ltd.

27 May 2011

TTW Job No: 101672P

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1.0 GENERAL

The Civil & external Stormwater Engineering component of the development is to be designed and constructed to comply with:

- The relevant current Australian Standards, Building Code of Australia and Design Codes.
- The requirements of all relevant Statutory Authorities and Local Regulations including Ryde City Council requirements and development approval consent conditions.
- Relevant Natspec technical specifications modified to the requirements of this project prepared by a suitably qualified Civil Engineer.

The current Civil and stormwater engineering design drawings as well as Sediment & Erosion Control Plan for the Graythwaite Rehabilitation Centre (GRC), Ryde Hospital are as follows:

- SKC01-P2 (Siteworks Plan)
- SKC03-P1 (Sediment & Erosion Plan)

The above design drawings are to be progressively updated and are subject to change. These drawings are also subject to relevant Authorities' approval and development consent conditions.

On completion of construction the Civil Engineer shall certify that the civil works are constructed in accordance with the plans and specifications.

Staging of the works will be required to be such that the agents of erosion are minimised at any one time.

An Erosion and Sedimentation Control Plan has been prepared for the site. Refer drawing SKC03

Necessary measures will be adopted as may be necessary for erosion control, including the following where applicable:

- Staging: Staging of operations (eg. clearing, stripping, demolition);
- Restoration: Progressive restoration of disturbed areas;
- Drains: Temporary drains and catch drains;
- Dispersal: Diversion and dispersal of concentrated flows to points where the water can pass through the site without damage;
- Spreader Banks: Or other structures to disperse concentrated silt traps;
- Construction and maintenance of silt traps to prevent discharge of scoured material to downstream areas;
- Temporary grassing: Or other treatment to disturbed areas (eg. contour ploughing);
- Temporary fencing.

The builder will be required to liaise and comply with the requirements of the Landcom's Managing Urban Stormwater "Soil and Construction" (The "Blue Book") and the local Council.

The builder will be required to apply dust and noise control measures to minimise disturbance to the functioning of neighbouring properties. The contractor will be required to demonstrate the proposed works equipment to be within acceptable limits for noise and vibration as determined by a registered acoustic consultant.

2.0 BULK EARTHWORKS AND EARTHWORKS

The design of all excavation or earthworks to be fit for purpose and with drainage, siltation and sediment controls satisfying all authority requirements.

Shoring, excavation techniques, excavation support, temporary and permanent batters shall be in accordance with TTW's specification and the recommendations of a qualified practicing Geotechnical Engineer – Refer to Jeffery and Katauskas Pty. Ltd.'s Geotechnical Investigation Ref.24595ZArpt dated 21 March 2011 in **Appendix B**

Bulk earthworks will be designed to minimise impact on the environment and provide control measures during construction to this effect.

To maintain the water quality during the construction stage, soil and erosion control measures will need to be installed. These measures include silt fences around the site, hay bales upstream of culverts and an appropriate 'truck shake down' facility at the exit to the site.

Bulk earthworks will be designed to minimise impact on the environment and provide control measures during construction to this effect.

Site preparation will include the following:

- Stripping of topsoil from work areas to be stockpiled for landscape areas.
- Tyne, water and roll the areas over which filling, paving or building slabs are to be placed. Six passes of a 10 tonne static roller are required. The final pass shall be a proof roll where

movement of greater than 3mm under the roller will indicate Bad Ground.

- Placement of acceptable material from cut areas shall be placed in layers of not more than 200mm to the compaction requirements.
- Filled areas and cut areas to be overlain by buildings and pavements are to be protected to maintain constant moisture content in the soil. The protection is to remain in place until construction is complete.
- Temporary Batters must be as specified by the Geotechnical Report by Jeffery & Katauskas Pty Ltd. Ref.24595ZArpt dated 21 March 2011 – refer **Appendix B**.

2.1 Compaction Of Earthworks

Location	Compaction	Allowable Variation from Optimum Moisture Content
Compaction of subgrade under pavements	98% SMDD	2%, -2%
Compaction of landscape areas	95% SMDD	+2%, -2%
Compaction of pavement base subbase	98% SMDD	+2%, -2%

Maximum fill layer thickness is 200mm.

An independent approved NATA registered testing authority will be required to perform all the compaction testing of earthworks and submit test certificates to the Superintendent.

2.2 Testing

The Contractor will be required to provide a plan of test sample locations and a schedule of testing to comply with the following minimum requirements. The following testing frequencies relate to acceptance on a “not one to fail” basis.

Location	Frequency of Tests
Building platforms	Not less than a) 1 test per layer per 100m^2 or b) 1 test per 200m^3 distributed evenly throughout full depth and area or c) 1 test per lot per layer whichever requires the most tests
Pavements	Not less than a) 1 test per 25 linear metres per layer for roadway b) 1 test per layer per 1000m^2 for carparks.
Trenches	a) 1 test per 2 layers per 40 linear metres
Filled landscape area	a) 1 test per layer per 1000m2 or b) 1 test per 200m^3 distributed reasonably through full depth and area whichever requires the most tests.

An independent approved NATA registered testing authority will be required to perform all the compaction testing of earthworks and submit test certificates to the Principal. Certification will be required that aggregates are suitable for use in roadwork and concrete.

3.0 STORMWATER DESIGN PARAMETERS

3.1 Existing Stormwater Drainage Condition

The site falls from South to North and currently the majority of the site's stormwater system consists of series of pipes & inlet pits. The proposed site is about 80% of the surface area impervious surface in its current state.

The site's existing stormwater pipe system drains towards Council's 450mm Stormwater pipe system that crosses Fourth Avenue fronting the proposed site. An existing pit (shown pit # 2 in TTW's SKC01) which is located inside the GRC's site is to be retained and is proposed to be the point of connection. The proposed development is to provide an on-site detention system (OSD) system in accordance with City of Ryde Development Control Plan 2006 – Section 8.2 Stormwater Management.

Existing stormwater pipe infrastructures owned by Ryde Hospital that are located within the proposed GRC site exist. These pipes that traverses the proposed GRC site are proposed to be diverted and that appropriate stormwater easement are to be provided as necessary in accordance with City of Ryde Development Control Plan 2006 – Section 8.2 Stormwater Management. The proposed diversions are shown in drawing SKC01 which are as follows:

(i) Stormwater line between pits 2 and 16 which is located to the west of the proposed site.

(ii) Stormwater line between pits 2 and 7 which is located to the east of the proposed site.

(iii) Stormwater easement in favour of the hospital is between pits 10 and 3.

3.2 Proposed Stormwater System

All proposed internal pipeworks which consist of pipes and access pits shall be designed in accordance with Australian Rainfall & Runoff (AR&R), Council and AS 3500.3 1998 and will be directed to the on-site detention (OSD) tank systems prior to discharge to Council's receiving stormwater network in Fourth Avenue.

The majority of the catchment is from the roof which is proposed to be captured and re-use for non-potable usage with appropriate water quality filter/treatment. All overflows from the rainwater tanks are to be diverted onto the proposed OSD tanks.

To reduce stormwater site's existing peak flowrates runoff, the proposed stormwater drainage system includes on-site detention (OSD) tanks.

The OSD tanks system shall be designed in accordance with Ryde City Council's requirements to provide temporary storage and restrict stormwater peak discharge flowrates from the site to a rate that specified by Council.

The proposed stormwater drainage system has been designed to provide both minor and major flow conveyance systems which is generally in accordance with the intent of Australian Rainfall and Runoff (1987) and the City of Ryde requirements. The proposed frontage carpark and vehicular site entry/exit act as overland flowpaths which accommodate surcharges from the proposed site stormwater system up to and including the 100-year ARI storm event.

The proposed stormwater concept plan as well as the proposed diversion of existing stormwater line is shown in **Appendix A**.

4.0 Flooding

Taylor Thomson Whitting (TTW) have reviewed the comprehensive flood study undertaken by Bewsher Consulting for Ryde City Council. A relevant extract to the proposed site that simulates the 1 in 100-year Average Recurrence Interval (ARI) storm event is shown in **Appendix C** of this report. TTW have superimposed this flood extent information over a detailed survey plan to approximate the 100-year ARI flood level and extent immediately north of the proposed site.

The lowest proposed finished habitable floor level of 89.50mRL is approximately 1650mm higher than the 100-year ARI flood level which is approximated at 87.85m, AHD.

Our investigation shows the proposed development is outside the mainstream flood extent.

Flood evacuation plan is not necessary as the proposed site is outside the flood extent and appropriate freeboard is provided in accordance with Floodplain Development Manual – 2005.

The Department of Environment, Climate Change and Water (DECCW) guidelines on sea level rise shows that the adopted benchmarks are for a rise relative to 1990 mean sea levels of 91cm by Year 2100.

Since the proposed site is away from the harbour foreshore and approximately 89 metre above sea level, it is not anticipated that the proposed development be affected by an increase in the sea level.

With respect to the increase in rainfall intensity, it is understood that at most, the rainfall for 1 day will increase by 10 percent as results show. To date, the study on shorter time frames is not completed. However, TTW's topographical analysis shows the proposed site is at the top of the catchment and an increase of rainfall intensity of 10% will have a very negligible impact on flood levels. As mentioned above, currently the proposed development has approximately 1650mm freeboard above the approximated 100-year flood level on Fourth Avenue.

Given that the proposed development:

(i) makes provision for overland flowpath for local overland flooding,

(ii) makes provision for OSD system to control stormwater site's existing peak flowrates runoff.

(iii) outside the mainstream 100-year ARI flood extent,

(iv) not adversely affected by climate change and

(v) provision of approximately 1650mm of freeboard above the approximated 100-year ARI flood level.

TTW is of the opinion that the proposed development has adequate flood protection and has no adverse impact on flood levels.

5.0 PAVEMENT DESIGN PARAMETERS

5.1 General

Pavements include roads, parking areas, paths and service vehicle access areas.

5.2 Design Parameters

Design all pavements for a 25 year life. Design all pavements trafficked by buses and trucks for a minimum 1×10^6 repetitions of a standard axle load. For other vehicular traffic areas design for 1.0×10^5 repetitions of a standard axle load. The standard axle load is as defined in AUSTROADS Pavement Design.

Flexible pavement will be based on a CBR value of 3% and a short term modulus of 25MPa for rigid pavement. This parameters are consistent with the site's Geotechnical Engineer's recommendation.

5.3 Finishes

For vehicular trafficked pavements provide a non-skid finish and for pedestrian trafficked pavements a non-slip finish.

5.4 Subsoil Drainage

Provide subsoil drainage or spoon drains at periphery of excavation to allow for seepage from the ground.

Provide sub-soil drainage to suit roads, ramps and bridge abutments and to the back face of retaining walls.

5.5 Falls for Drainage

Minimum falls for drainage on pavements shall not be less than:

Concrete -	1%	1:100
Asphaltic Concrete or Bitumen -	2%	1:50
Paving -	2%	1:50

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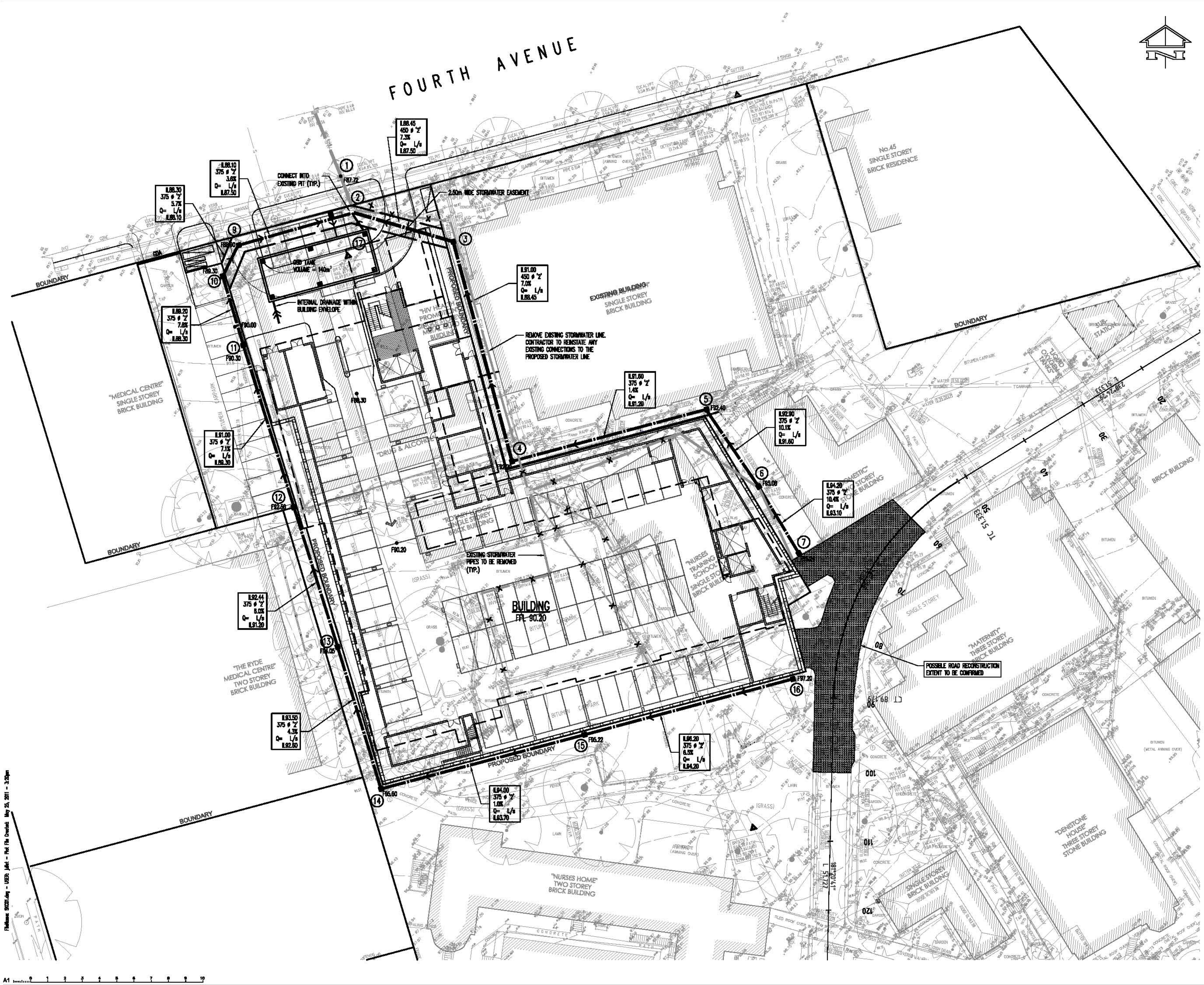
APPEDIX A

(i) Civil & External Stormwater including Diversion Works Drawings.



Rev	Description	Eng	Draft	Date
P4	ISSUE FOR APPROVAL	SB	JT	25.05.11
P3	ISSUE FOR INFORMATION	SB	JT	18.05.11
P2	ISSUE FOR INFORMATION	SB	JT	13.05.11
P1	ISSUE FOR COORDINATION	SB	JT	12.05.11

Project

GRAYTHWAITE REHABILITATION CENTRE, RYDE HOSPITAL

Sheet Subject

SITEWORKS PLAN

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Job No	Drawing No	Revision
101672	SKC01	P4

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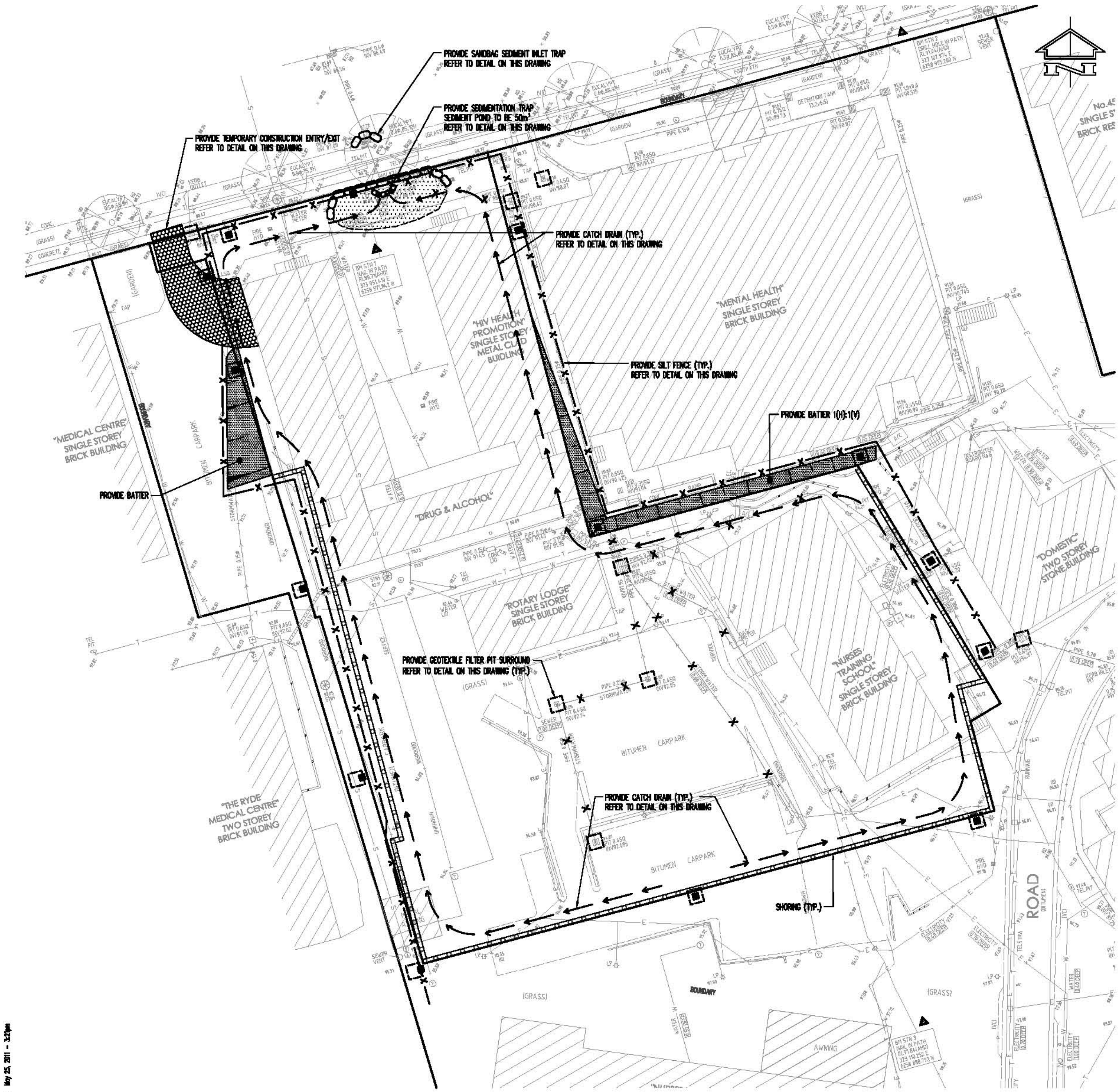

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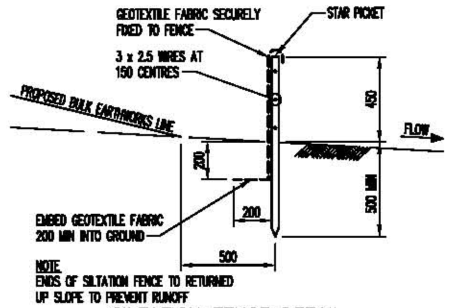

SILTATION FENCE DETAIL
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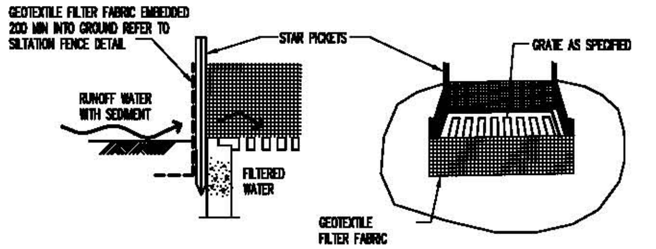
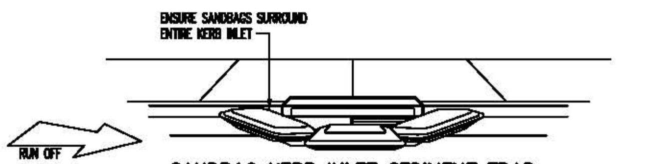
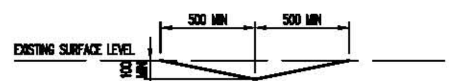


GEOTEXTILE FILTER PIT SURROUND
NTS



SANDBAG KERB INLET SEDIMENT TRAP
NTS



TYPICAL SECTION THROUGH CATCH DRAIN
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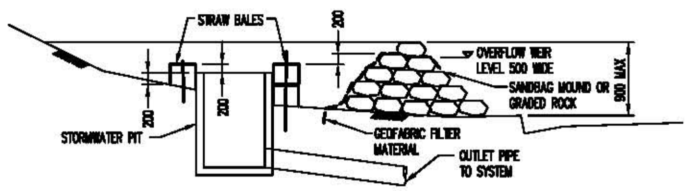
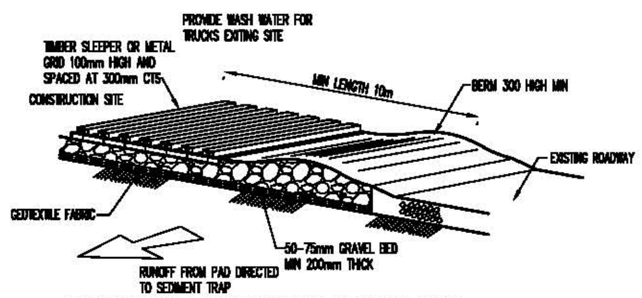


SEDIMENTATION TRAP
NTS



TEMPORARY CONSTRUCTION VEHICLE EXIT
NTS

EROSION AND SEDIMENT CONTROL NOTES

1. All work shall be generally carried out in accordance with
   (A) Local authority requirements,
   (B) EPA – Pollution control manual for urban stormwater,
2. Erosion and sediment control drawings and notes are provided for the demolition works only.
   The erosion and sediment control plan shall be implemented and adopted to meet the varying situations as work on site progresses.
3. Maintain all erosion and sediment control devices to the satisfaction of the superintendent and the local authority.
4. When stormwater pits are constructed prevent site runoff entering the pits unless silt fences are erected around pits.
5. Minimise the area of site being disturbed at any one time.
6. Protect all stockpiles of materials from scour and erosion. Do not stockpile loose material in roadways, near drainage pits or in watercourses.
7. All soil and water control measures are to be put back in place at the end of each working day, and modified to best suit site conditions.
8. Control water from upstream of the site such that it does not enter the disturbed site.
9. All construction vehicles shall enter and exit the site via the temporary construction entry/exit.
10. All vehicles leaving the site shall be cleaned and inspected before leaving.
11. Maintain all stormwater pipes and pits clear of debris and sediment. Inspect stormwater system and clean out after each storm event.
12. Clean out all erosion and sediment control devices after each storm event.

Sequence Of Works

1. Prior to commencement of excavation the following soil management devices must be installed.

1.1. Construct silt fences below the site and across all potential runoff sites.

1.2. Construct temporary construction entry/exit and divert runoff to suitable control systems.

1.3. Construct measures to divert upstream flows into existing stormwater system.

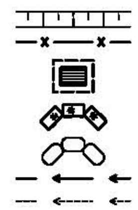
1.4. Construct sedimentation traps/basin including outlet control and overflow.

1.5. Construct turf lined swales.

1.6. Provide sandbag sediment traps upstream of existing pits.

2. Construct geotextile filter pit surround around all proposed pits as they are constructed.
3. On completion of pavement provide sand bag kerb inlet sediment traps around pits.
4. Provide and maintain a strip of turf on both sides of all roads after the construction of kerbs.

EROSION AND SEDIMENT CONTROL LEGEND



Batter
Siltation fence
Stormwater pit with Geotextile filter surround
Hay bale barriers
Sandbag sediment trap
Catch drain
Overland flow path

NOTE:

– CONTRACTOR MUST PERFORM A SERVICES SEARCH AND CONFIRM THE EXACT LOCATION, EXTENTS AND DEPTHS OF ALL SERVICES PRIOR TO CONSTRUCTION AND NOTIFY ANY CONFLICT WITH THE DRAWINGS IMMEDIATELY TO THE ENGINEERING/ SUPERINTENDENT.

– IT IS RECOMMENDED THAT EXISTING SERVICES BE TRACED AND POTHOLED TO CONFIRM EXACT LOCATIONS & DEPTHS.

–ALL EXISTING INGROUND STORMWATER PIPES ARE TO BE RETAINED.

–PROVIDE SOIL AND EROSION CONTROL AS SHOWN

Rev	Description	Eng	Draft	Date
P3	ISSUE FOR APPROVAL	SB	JT	25.05.11
P2	ISSUE FOR INFORMATION	SB	JT	18.05.11
P1	ISSUE FOR INFORMATION	SB	JT	13.05.11

Project
GRAYTHWAITE REHABILITATION CENTRE, RYDE HOSPITAL

Sheet Subject
EROSION AND SEDIMENT CONTROL PLAN

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Job No	Drawing No	Revision
101672	SKC03	P3

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APPEDIX B

(i) Geotechnical Report - Jeffery and Katauskas Pty. Ltd.'s Geotechnical Investigation Ref.24595ZArpt dated 21 March 2011

AS/NZS ISO 9001
Certified
Davis Langdon Certification Services

REPORT

TO

HEALTH INFRASTRUCTURE

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED GRAYTHWAITE REHABILITATION CENTRE

AT

RYDE HOSPITAL, FOURTH AVENUE, DENISTONE, NSW

21 March 2011

Ref: 24595ZArpt

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

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Table A:	**Summary of Laboratory Test Results**
Table B:	**Summary of Four Day Soaked CBR Test Result**
Table C:	**Summary of Point Load Strength Index Test Results**

Borehole Logs JK1 to JK7

Figure 1:	**Borehole Location Plan**
Figures 2 & 3:	**Graphical Borehole Summaries**

Appendix A:	**Envirolab Services Pty Ltd Soil pH & Soil Sulphate Test Results (Report Ref: 52449[REV R00], dated 8 March 2011)**

Report Explanation Notes

1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Graythwaite Rehabilitation Centre located at Ryde Hospital, Fourth Avenue, Denistone, NSW. The investigation was commissioned by Health Infrastructure by Contract No. HI 10121(Rev A), dated 27 January 2011. The commission was on the basis of our revised fee proposal, Ref: "P33386ZA2" dated 23 December 2010.

For the purpose of this report, we have taken Fourth Avenue to bound the site to the north, with "Site North" shown on the attached Figure 1.

From a conversation with Ms Vanessa Waller of Nettleton Tribe Partnership Pty Ltd (NTP) on 18 March 2011, we understand that the design of the proposed new building has yet to be finalised. We were then supplied with the current NTP architectural drawings for Options 1 and 2 as outlined below:

- Drawing No. 3717_SK36 "*Site Plan*";
- Drawing No. 3717_SK40 "*Option 1 Roof Plan*";
- Drawing No. 3717_SK41 "*Site Sections (Option 1)*";
- Drawing No. 3717_SK50 "*Ground Floor Plan Option 1*";
- Drawing No. 3717_SK51 "*Option 1 Level 1 Floor Plan*";
- Drawing No. 3717_SK52 "*Option 1 Level 2 Floor Plan*";
- Drawing No. 3717_SK53 "*Ground Floor Plan Option 2*";
- Drawing No. 3717_SK54 "*Option 2 Level 1 Floor Plan*";
- Drawing No. 3717_SK55 "*Option 2 Level 2Floor Plan*".

Based on the above drawings, we understand that the proposed new building will comprise two, three storey buildings overlying a common "L" plan shaped ground floor level. The proposed ground floor level will accommodate mostly car parking and will be constructed between reduced level (RL) 89.3m at its northern end, and RL 90.2m at its southern end. The survey datum is the Australian Height Datum

(AHD). Due to the sloping nature of the site, the proposed ground floor level will require excavation to a maximum depth of about 6m below existing grade. Two lift shafts are also proposed. We have assumed that the proposed lift pits will require a maximum excavation depth of 1.5m below bulk excavation level. From the supplied geotechnical investigation brief prepared by Taylor Thomson Whitting (NSW) Pty Ltd (TTW), Ref: "101035" dated 17 December 2010, we understand that the proposed new building will be a concrete framed structure with expected column loads between 750kN and 1500kN, and line loads of 50kN/m.

An external pavement area is also proposed on the northern side of the proposed new building. We have assumed that this area will require cut and fill earthworks to a maximum depth/height of about 1.5m, and that a concrete pavement will be constructed.

The purpose of the investigation was to assess the subsurface conditions at seven borehole locations and, based on the information obtained, to present our comments and recommendations on earthworks, excavation conditions, seepage, retention, footings, ground floor slab-on-grade, soil aggression and earthquake design parameters.

Environmental Investigation Services (EIS), who are the environmental consulting division of Jeffery and Katauskas Pty Ltd (J&K), were also commissioned to carry out a Stage 1 environmental site assessment for the proposed development (report Ref: "E24595KHrpt" dated March 2011). This geotechnical investigation report must be read in conjunction with the EIS report.

2 INVESTIGATION PROCEDURE

All fieldwork was carried out in accordance with our revised safe work method statement, which was submitted in our email, Ref: "24595ZA email10" dated 9 February 2011. Prior to the commencement of the fieldwork, a specialist sub-consultant electro-magnetically scanned the borehole locations for buried services.

The fieldwork for the investigation was carried out on 21 & 22 February 2011 and comprised the drilling of seven boreholes (JK1 to JK7), at the locations shown on Figure 1, to depths between 6.0m and 11.0m below existing grade. The boreholes were drilled using our track mounted JK300 drill rig.

The borehole locations were set out by tape measurements from existing surface features and apparent site boundaries. The surface reduced levels indicated on the attached borehole logs were interpolated between spot level heights and ground contour lines shown on the supplied survey plans prepared by Craig & Rhodes (Ref: 342-02, dated 23/2/09) and Norton Survey Partners (Ref: 33719, dated 8/2/11), and are therefore only approximate. The survey datum is AHD.

The boreholes were spiral flight auger drilled using a twin pronged tungsten carbide (TC) bit. The relative compaction/strength of the subsoil profile was assessed from the Standard Penetration Test (SPT) 'N' values, together with hand penetrometer readings on clayey soils recovered in the SPT split spoon sampler and by tactile examination. The strength of the underlying bedrock was assessed by observation of auger penetration resistance, together with examination of recovered auger cuttings and correlations with subsequent laboratory moisture content tests.

At depths of 8.20m in JK5 and 2.75m in JK7, the boreholes were extended into the bedrock by rotary diamond coring techniques, using an NMLC triple tube core barrel with water flush. The strength of the cored bedrock was assessed by examination of the recovered rock cores, together with correlations with subsequent laboratory

Point Load Strength Index ($I_{S(50)}$) tests. Based on the poor core recovery in JK7, due to the weathered nature of the bedrock, the borehole was advanced below 5.30m depth using the spiral flight augers. Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Groundwater observations were also made in the boreholes. In JK4 and JK6, 50mm diameter slotted PVC standpipes, with push-on end caps, were installed for groundwater level monitoring purposes. The top of each standpipe was protected by a cast-iron "Gatic" cover, which was concreted flush with the ground surface. The standpipe installation details are presented on the attached borehole logs. The groundwater levels in the standpipes were monitored during the course of the fieldwork and during a return visit on 15 March 2011.

Our geotechnical engineer (David Schwarzer) was present full-time during the fieldwork to set out the borehole locations, direct the electro-magnetic scanning, nominate testing and sampling, prepare the attached borehole logs and direct the standpipe installations. The Report Explanation Notes define the logging terms and symbols used.

Selected soil and rock chip samples were returned to a NATA registered laboratory [Soil Test Services Pty Ltd (STS)] for moisture content, Atterberg Limits, linear shrinkage, Standard compaction and four day soaked CBR testing. The results are summarised in the attached Tables A and B.

The recovered rock cores were photographed and returned to STS for Point Load Strength Index testing. The photographs are enclosed facing the relevant cored borehole logs. The Point Load Strength Index test results are plotted on the borehole logs and are also summarised in the attached Table C. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also summarised in Table C.

Selected soil samples were returned to a second NATA registered laboratory (Envirolab Services Pty Ltd) for soil pH and soil sulphate testing. The results are summarised in the attached Appendix A.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located towards the crest of a north facing, gently to moderately sloping hillside. The site itself generally grades at approximately 4° and is bound by Fourth Avenue to the north.

At the time of the fieldwork, the site was terraced into essentially three areas which were separated by retaining walls and batter slopes. However, along the western site boundary was an asphaltic concrete (AC) surfaced driveway and car park which graded down to Fourth Avenue. Boreholes JK4 and JK7 were positioned in this area. The AC surfacing was generally in fair to poor condition with some areas containing cracking up to 10mm wide.

The upper (southern) terrace was occupied by an AC surfaced driveway and car park. Borehole JK5 was positioned in the upper terrace. The AC surfacing was generally in fair to poor condition with cracking up to 5mm wide. The adjoining (higher lying) hospital grounds to the south were up to 1.6m higher in level than the upper terrace. The change in level was accommodated by a grass covered batter slope which graded between 22° and 33°. In some areas, the toe of the batter slope was supported by a timber “Koppers” log retaining wall, which was up to 0.9m high.

The central terrace was primarily occupied by an AC surfaced carpark, which was flanked on its eastern and western sides by AC surfaced driveways that descended from the upper terrace. Boreholes JK3 and JK6 were positioned in the central terrace. The AC surfacing was generally in fair to poor condition with cracking up to 5mm wide. Along the southern side of the car park, the upper terrace was supported by a concrete block retaining wall, which was up to 1.6m high and appeared to be in fair condition with some vertical cracking up to 1mm wide. At the eastern return of the retaining wall adjacent to the side driveway, was a stepped crack which ranged in thickness from hairline at its base to 3mm wide at the top of the wall. The ground surfaces beyond the outer sides of the two descending driveways were supported by a combination of "Koppers" log retaining walls and concrete block retaining walls, which were up to 0.8m high.

On the northern side of the central terrace was a single storey brick building ("Rotary Lodges"), which appeared to be in good external condition based on a cursory inspection. On the eastern side of the central terrace was a single storey brick building ("Versa Lindsay"), which also appeared to be in good external condition based on a cursory inspection.

A maximum elevation relief of about 1.9m existed between the central terrace and the lower (northern) terrace. The central terrace was supported on its northern side by a combination of batter slopes and toe walls, which were of predominantly brick construction, but flanked with "Koppers" log and concrete block walls. The primary retention feature was a 0.9m high, east-west oriented, brick toe wall which supported a batter slope that graded to a maximum of 25° and was up to 1m high (ie. above the top of the brick wall). The brick toe wall was in poor condition with outward rotation about its toe up to 10° off vertical. At the western end of the brick retaining wall was a concrete block retaining wall which was up to 1.6m high. The concrete block wall returned and diminished in height in a northerly direction. Along this return, the concrete block retaining wall supported the western driveway. The concrete block retaining wall was in good to fair condition with some vertical and stepped cracks up to 2mm wide.

The lower terrace was occupied by a single storey steel clad building ("Health Promotions Centre"). The perimeter of this building was supported on brick walls/strip footings. Due to the sloping nature of the site, the brick walls were up to 1.8m high at the northern end of the building. Towards the northern end of the westernmost wall was a vertical and stepped crack which ranged in thickness from hairline at its base to 6mm wide at the top of the wall. On the northern side of the western wing of the "Health Promotions Centre" was a second vertical and stepped crack which was up to 30mm wide. The base of the crack was obscured by a brick staircase. The top of this brick staircase, as well as the top of the similar staircase at the northern end of the eastern wing, had rotated outwards away from the building for a maximum horizontal separation of 50mm.

The remaining areas of the site contained lawns, garden beds, scattered small to large size trees, and concrete footpaths. Concrete kerbs and gutters surrounded the paved areas.

The surrounding hospital structures comprised one, two and three storey brick buildings, which appeared to be in good condition when viewed from within the site. These surrounding buildings either abutted the site or were set back a short distance from the site boundaries. Abutting the eastern side of the lower (northern) terrace, the lower ground floor level of the adjoining "Ryde Community Mental Health" building was lower than the footpath, which ran along the eastern site boundary. The footpath level was up to 1.9m higher than the adjoining lower ground floor level, assuming the lower ground floor level extends to the rear (south) of the adjoining building.

The northern site boundary was lined with a low height brick wall, which was up to 1m high. The brick wall retained a soil profile on its southern side to a maximum height of about 0.6m. The boundary wall was in poor condition with some stepped cracks up to 2mm wide and outward rotation about its toe up to 5° off vertical.

3.2 Subsurface Conditions

The 1:100,000 series geological map of Sydney indicates the site to be underlain by Ashfield Shale of the Wianamatta Group. Generally, the boreholes encountered AC pavements or fill, overlying residual silty clay, then weathered shale bedrock at relatively shallow depths. Reference should be made to the attached borehole logs for details at each specific location. Graphical borehole summaries are presented as Figures 2 and 3. A summary of the subsurface conditions encountered in the boreholes is provided below:

AC Pavement

AC surfacing was encountered in JK4 to JK7 and was 40mm thick. The AC surfacing was supported on a sandy “roadbase” which contained either igneous or sandstone gravel. The sandy “roadbase” layer ranged in thickness between 160mm (JK7) and 360mm (JK4 & JK6).

Fill

Fill, predominantly comprising clayey soils and to a lesser extent silty soils, was encountered from the surface in JK1, JK2 and JK3 to depths of 0.1m, 2.5m and 0.3m, respectively. The fill profile in JK1 and the surficial fill profile in JK2 resembled imported topsoil. Based on the SPT results and limited hand penetrometer readings, the upper fill profile in JK2 was assessed to be well compacted, however the lower fill profile was assessed to be poorly compacted.

Residual Silty Clay

Residual silty clay of variable, but predominantly high plasticity and of stiff to hard strength, was encountered below the fill in JK1 and JK3, and below the sandy “roadbase” layer in JK4 to JK7.

Weathered Shale Bedrock

Weathered shale bedrock was encountered in all boreholes at depths between 0.8m (JK6) and 3.2m (JK3). The shale bedrock was extremely variable in weathering and strength. In JK3, the shale was initially extremely to distinctly weathered and of extremely low and very low strength, improving at 7.0m depth to low strength. In JK6 however, the shale was initially distinctly weathered and of low and medium strength, improving at 6.5m depth to medium and high strength.

The lower rock cored length of JK5 encountered fresh shale of medium and high strength, however a 0.71m thick core loss zone (presumably a clay band or extremely weathered band washed out by the drill flush water) was intersected mid-length. Minor rock defects including two clay/extremely weathered seams and a sub-vertical joint were also encountered in the rock cored length of JK5.

In JK7, we commenced rock coring of the low strength shale at 2.75m depth and extended to 5.3m depth (ie. total length of 2.55m). However, only a rock core length of 0.85m was retrieved, with 1.7m length or core loss (presumably a weaker band washed out by the drill flush water). The recovered short length of rock core contained numerous defects, including clay seams and joints, at an average vertical spacing of 30mm. Based on the encountered difficulty in rock coring this material (from blockages arising from swelling of the clay fraction), the borehole was continued down to its termination depth using the spiral flight augers.

Groundwater

JK1, JK2, JK4 and JK5 were "dry" during and on completion of augering. JK7 was also "dry" during and on completion of augering down to 2.75m depth. Groundwater seepage was encountered in JK3 at 7.5m depth. Similarly, groundwater seepage was encountered in JK6 during and on completion of augering at depths of 6.5m and 6.3m, respectively. We note that the groundwater levels may not have stabilised within the limited observation period. Due to the introduced drill flush water associated with rock coring in JK5 and JK7, no meaningful final groundwater levels were obtained.

At 28 hours after completion of drilling in JK4, groundwater was measured at 4.6m depth, which corresponds to RL 87.9m (AHD). At 27 hours after completion of drilling in JK6, groundwater was measured at 3.1m depth, which corresponds to RL 90.5m (AHD).

We returned to site on 15 March 2011 to remeasure the groundwater level in each standpipe. We were unable to access the JK6 standpipe as a car was parked over the borehole. Groundwater in the JK4 standpipe was measured at 4.4m depth, which corresponds to RL 88.1m (AHD).

We suspect that the observed groundwater is flowing at a very low rate at the soil-bedrock interface and through defects in the rock mass.

3.3 Laboratory Test Results

The moisture content and Atterberg Limits test results confirmed our field classification of the site soils. The sampled clayey silt fill from JK2 was of very low plasticity such that the Atterberg Limits and linear shrinkage could not be measured. However, the Atterberg Limits and linear shrinkage test results indicated the sampled residual silty clay of medium plasticity from JK3 to have a moderate to high potential for shrink-swell reactivity with changes in moisture content.

The four day soaked CBR test carried out on a residual silty clay sample from JK1 resulted in a value of 3.0% when compacted to 96% of Standard Maximum Dry Density (SMDD) and surcharged with 9kg. The insitu moisture content of the clay sample was 3.3% "dry" of its Standard Optimum Moisture Content (SOMC). During soaking, the clay sample swelled 2.0%.

The results of the moisture content tests carried out on recovered rock chip samples correlated reasonably well with our field assessment of bedrock strength. The results of the Point Load Strength Index tests carried out on the recovered rock cores from JK5 and JK7 correlated well with our field assessment of bedrock strength. The estimated UCSs in the cored rock portions of these boreholes ranged from 2MPa to 26MPa.

The results of the soil aggression testing are tabulated below:

Borehole	**Sample Depth (m)**	**Soil Description**	**Soil pH**	**Soil Sulphate (mg/kg)**
JK1	0.5-0.95	Residual Silty Clay	4.6	84
JK3	0.5-0.95	Residual Silty Clay	4.4	65
JK5	0.5-0.95	Residual Silty Clay	4.5	100

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation

Prior to any excavation commencing, reference should be made to the WorkCover Authority of NSW's "Code of Practice – Excavation Work" dated 31 March 2000 (Cat. No. 312).

Prior to the commencement of demolition and excavation, we recommend that detailed dilapidation surveys be compiled on the nearby buildings to the east, west and south. The dilapidation reports can be used as a benchmark against which to set vibration limits and for assessing possible future claims for damage arising from the works. The respective owners should confirm that the dilapidation survey reports are a fair representation of existing conditions. We could carry out the dilapidation surveys if commissioned to do so.

4.1.1 Site Preparation

The proposed development will require demolition of existing buildings and pavements. Following this, all vegetation (including tree root balls), topsoil, root affected soils, and any deleterious or contaminated fill should be stripped and disposed appropriately off site. Reference should be made to the EIS report for guidance on the off-site disposal of the excavated materials.

Care should be taken not to undermine or remove support from the site boundaries during excavation.

4.1.2 Excavation Conditions

Based on the borehole logs, the proposed ground floor excavation will extend through the soil and weathered shale bedrock profiles.

Excavation of the soil profile and extremely low to very low strength shale bedrock could be carried out using a large hydraulic excavator and/or dozers. The rock excavation will also extend through low and medium strength shale bedrock. Due to the significant number of defects encountered in the shale profile, that would facilitate excavation, we recommend that the excavation of the bedrock be carried out using either a dozer of at least Caterpillar D7 size or equivalent, and/or a very large excavator using a ripping tyne where necessary. Excavation of the medium strength shale bedrock could present hard ripping or “hard rock” excavation conditions and may require the use of hydraulic rock hammers for the excavators.

Care should be taken when using hydraulic rock hammers so that ground borne vibrations do not adversely affect nearby buildings. We recommend that continuous quantitative ground vibration monitoring be carried out whenever hydraulic rock hammers are being used (ie. during demolition and excavation). From a structural point of view, continuous vibrations and instantaneous vibrations (ie. during demolition) on the nearby brick buildings should be limited to peak particle velocities of 5mm/s and 10mm/s, respectively, subject to review of the dilapidation reports. The operators of the nearby health facilities should however confirm the vibration limits taking equipment sensitivity and patient comfort into account. If excessive vibrations are encountered, then a smaller rock hammer should be used or further advice could be sought. Notwithstanding, we recommend that the following procedures be adopted whenever hydraulic rock hammers are being used in order to reduce vibrations:

- Maintain rock hammer oriented towards the face and enlarge excavation by breaking small wedges off face.
- Operate hammer in short burst only, to reduce amplification of vibrations.
- Use excavation contractors with appropriate experience and a competent supervisor who is aware of vibration damage risks, etc. The contractor should

have all appropriate statutory and public liability insurances and should be provided with a full copy of this report.

4.1.3 Groundwater Seepage

Groundwater inflows into the proposed ground floor excavation are expected as local seepage flows within the fill, at the fill/residual silty clay interface, through gravel bands or relic joints/fissures within the residual silty clay, at the soil/rock interface, and through joints and bedding partings within the shale bedrock profile, particularly after heavy rain. Seepage volumes into the excavation are expected to be controllable by gravity drainage systems and/or conventional sump and pump dewatering systems.

4.2 Excavation Retention

4.2.1 Temporary Cut Batter Slopes

Where space permits at the northern end of the proposed excavation, battering of soil and weathered shale cuts is feasible on condition that surcharge loads are kept well away from the crests of the batter slopes. Temporary batter slopes should be graded at no steeper than 1 Vertical (V) on 1 Horizontal (H). Conventional reinforced block retaining walls may then be constructed at the toes of the batter slopes and subsequently backfilled.

4.2.2 Cast Insitu Retention Systems

Where batter slopes cannot be accommodated or are not preferred, we recommend that the proposed vertical cuts in the soil and weathered shale bedrock profiles be supported for the full depth of the excavation by soldier pile walls with shotcrete infill panels. The soldier piles must be installed prior to excavation commencing and must be progressively shotcreted and anchored, or internally propped, as the excavation proceeds (ie. once the restraining point has been uncovered). Careful control of the construction sequence will be required to reduce potential movements.

The piles can be used as load bearing piles for the proposed new building if taken down to the appropriate founding depths; that is, they will need to be embedded below bulk excavation level (including nearby footings, service trenches and lift pits) at suitable depths to satisfy founding and stability considerations.

Due to the presence of medium and high strength shale bedrock, only high torque drilling rigs equipped with rock augers, should be brought to site. All perimeter piles should be tremie poured due to the expected depths of the pile holes. We strongly recommend that a full copy of this report be provided to the piling contractor so that appropriate drilling rigs and equipment are brought to site.

Construction of the soldier pile wall must be of high quality. The shotcrete infill panels must be completed without delay to reduce the shrinkage of clay soils immediately outside the excavation and to limit potential rock wedge failures between the soldier piles. The construction sequence should be fully specified and carefully controlled to reduce potential movements.

We expect that the proposed lift pits will be cut vertically. Due to the presence of steeply inclined joints, which could initiate rock wedge failures in vertically cut faces, we strongly recommend that the lift pit cut faces be inspected by an experienced geotechnical engineer on completion of excavation to assess the need for temporary support (eg. dowels and/or rock bolts). We have assumed that the lift pit cut faces will be supported in the long-term by retaining walls.

4.2.3 Retention Design Parameters

The major consideration in the selection of earth pressures for the design of the retention system is the need to limit deformations occurring outside the excavation. The characteristic earth pressure coefficients and subsoil parameters provided below may be adopted for the design of the retention systems.

- The details of all nearby footings located within the zone of influence of the proposed ground floor excavation (ie. above a 1V on 1H line inclined up from bulk excavation level) should be investigated to assess the possible surcharging on the retention system. We could carry out this test pit investigation at locations nominated by TTW if commissioned to do so.

- For progressively anchored or propped walls, where only minor movements can be tolerated (eg. along the southern side of the proposed new building, provided there are no movement sensitive buried services), we recommend the use of a trapezoidal earth pressure distribution and a lateral earth pressure of 6H (kPa) for the soil and weathered shale bedrock profiles, where H is the retained height in metres (ie. between surface level and bulk excavation level, including nearby footings, service trenches and lift pits). These pressures should be assumed to be uniform over the central 60% of the support system. For the shotcrete infill panel design, a trapezoidal earth pressure distribution and a lateral earth pressure of 4H (kPa) can be adopted for the soil and weathered bedrock profiles.

- For progressively anchored or propped walls located in areas which are highly sensitive to lateral movement (eg. along the eastern side of the proposed new building), we recommend the use of a trapezoidal earth pressure distribution and a lateral earth pressure of 8H (kPa) for the soil and weathered shale bedrock profiles, where H is the retained height in metres. For the shotcrete infill panel design, a trapezoidal earth pressure distribution and a lateral earth pressure of 6H (kPa) can be adopted for the soil and weathered bedrock profiles.

- For both of the above cases, it is essential that the rock faces between the soldier piles are progressively inspected by an experienced geotechnical engineer or engineering geologist as the excavation proceeds at no more than 1.5m depth intervals, so that any large wedges that could detach are identified and appropriate additional support measures implemented (eg. dowels and/or rock bolts).

- For conventional reinforced block retaining walls constructed in front of temporary cut batters and for lining lift pit excavations, we recommend that use of a triangular lateral earth pressure distribution and an "at rest" earth pressure coefficient (K_0) of 0.55 for the soil and weathered shale bedrock profiles, assuming a horizontal backfill surface. Bulk unit weights of $20kN/m^3$ and $22kN/m^3$ should be adopted for the soil and weathered shale bedrock profiles, respectively.

- Any surcharge affecting the walls (eg. traffic loading, inclined backfill, nearby retaining walls and their backfill, nearby footings, etc.) should be allowed in the design using the above K_0 value, assuming a horizontal backfill surface.

- The retaining walls should be designed as fully drained with measures undertaken to induce complete and permanent drainage of the ground behind the walls. Subsurface drains should incorporate a non-woven geotextile filter fabric such as Bidim A34 to reduce subsoil erosion. At least two equally spaced strip drains (with weep hole outlets) should be provided between soldier piles. All drainage water should be piped to the stormwater system.

- For perimeter piles embedded at least 0.5m into the weathered shale bedrock below bulk excavation level (including below nearby internal footing excavations, service trenches and lift pits), a maximum allowable lateral toe resistance of 150kPa may be adopted. This relatively conservative value is based on the deeply weathered shale profiles encountered in some of the boreholes, namely JK3. The above design value assumes excavation is not carried out within the zone of influence of the wall toe. The upper 0.2m depth of the socket should not be taken into account to allow for disturbance and tolerance effects during excavation. The quality of the toe restraint rock should be progressively inspected at bulk excavation level by an experienced geotechnical engineer to confirm that unexpected conditions do not exist.

4.2.4 Rock Anchors

Prior to the installation of rock anchors, which will extend outside the site boundaries, permission must be sought from the respective neighbouring property owners. Rock anchors bonded at least 3m into weathered shale bedrock may be designed for a maximum allowable bond stress of 150kPa.

All anchors should be proof tested to 1.3 times the working load under the supervision of an experienced engineer independent of the anchor contractor. The testing may allow an upgrading of the above bond stress. We recommend that only experienced contractors be considered for the anchor installations.

We have assumed that permanent lateral support of the lower ground floor walls will be provided by the new structure. If not, then the rock anchors will need to be designed for corrosion resistance and for long-term durability.

4.2.5 Backfilling Behind Reinforced Block Retaining Walls

Backfilling behind free-standing reinforced block retaining walls should be carried out in accordance with AS3798-2007 ("Guidelines on Earthworks for Commercial and Residential Developments") in order to reduce post-construction settlements.

Compaction of engineered fill behind retaining walls is very difficult and time consuming. The use of a single sized durable gravel, such as "blue metal" or crushed concrete gravel (free of fines), which do not require significant compactive effort is often preferred if good performance is a priority; at least in the lower layers. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in maximum 200mm thick loose layers. A non-woven geotextile filter fabric such as Bidim A34 should be placed as a separation layer immediately above the cut batter slope (prior to backfilling) to control subsoil erosion. Provided the gravel backfill is placed and compacted as recommended above, density testing of the gravel backfill would not be required. The geotextile should then be wrapped over the surface of the gravel backfill and capped with at least a 0.3m thick compacted layer of clayey engineered fill, as specified below.

As an alternative to single size gravel, backfilling can be carried out using the excavated "clean" clay soils on condition that they are free of organic matter and contain a maximum particle size of 40mm. Backfilling should be carried out in maximum 100mm thick loose layers and compacted using a portable hand operated "Wacker Packer", a roller attachment fitted to a hydraulic excavator and/or a small static trench roller. Assuming these retaining walls will be supporting paved areas, each engineered fill layer should be compacted to a minimum density ratio of 98% of SMDD and to within 2% of SOMC.

Density tests should be regularly carried out on the clayey engineered backfill to confirm the above specification is achieved. The frequency of density testing for backfill behind retaining walls should be at least one test per two layers per $50m^2$

(assumes maximum 100mm thick loose layers). Level 2 testing of fill compaction is the minimum permissible in AS3798-2007. Due to an inherent conflict of interest, the geotechnical testing authority (GTA) should be directly engaged by Health Infrastructure or their representative, and not by the earthworks contractor or sub-contractors.

4.3 Footings

Based on the results of the investigation, we expect that the proposed lower ground floor excavation will generally expose weathered shale bedrock, however, at the northern end of the proposed building footprint, the shale bedrock will not be exposed as JK1 and JK2 encountered the bedrock surface at depths of 1.8m and 2.5m below existing grade, respectively. Internal pad and strip footings founded in extremely low strength or stronger shale bedrock may be designed for a maximum allowable bearing pressure of 700kPa. This relatively low value (suitable for Class V shale) is based on the deeply weathered nature of shale as encountered in the boreholes, particularly JK3.

Where the bedrock surface is too deep to construct internal pad and strip footings (ie. at the northern end of the proposed building footprint), then conventional bored piles should be used. Conventional bored piles socketed at least 0.3m into extremely low strength or stronger shale bedrock may also be designed for a maximum allowable end bearing pressure of 700kPa. Pile rock sockets formed below the minimum 0.3m length requirement may be designed for a maximum allowable shaft adhesion value of 70kPa (compression) on condition that the pile shaft is suitably roughened using a grooving tool fitted to the side of the auger.

Conventional bored piles, used in the construction of the perimeter soldier pile walls, and founded in extremely low strength or stronger shale bedrock may also be designed for a maximum allowable end bearing pressure of 700kPa. From 0.5m depth below bulk excavation level (including footing excavations, service trenches and lift pits), the pile socket may be designed for a maximum allowable shaft adhesion value of 70kPa (compression) on condition that the pile shaft is suitably roughened.

The bearing pressures provided above are based upon serviceability criteria of deflections at the footing base of less than 1% of the minimum footing dimension/pile diameter. As outlined in Section 4.2.2, the prospective piling contractors should be provided with a full copy of this report so that appropriate drilling rigs and equipment are brought to site.

Piles on the shale bedrock may also be designed using "Limit State Design" principles as detailed in the paper "Foundations on Sandstone and Shale in the Sydney Region" by Pells, Mostyn and Walker (Australian Geomechanics, Number 33, Part 3, December 1998, Pages 17-29) and in AS2159-2009 ("Piling Design & Installation"). For ultimate limit state design, an ultimate bearing capacity of 3000kPa and an ultimate shaft adhesion value of 100kPa in compression could be adopted for the weathered shale bedrock. Settlement limitations to the structures will still need to be satisfied and can be estimated using the elastic modulus values provided below. The modulus values are only approximate due to inherent uncertainties.

Stratum	Elastic Modulus (MPa)
Existing Fill	Ignore
Residual Silty Clays of at least stiff	10 - 20
Weathered Shale Bedrock	50 - 200

It should be noted that the ultimate bearing pressures must be used in conjunction with an appropriate "*Basic Geotechnical Strength Reduction Factor*" (ϕ_{gb}). Provided there is good workmanship, quality control and performance monitoring in the piling process, we recommend that a ϕ_{gb} value of 0.6 be adopted for end bearing and shaft adhesion.

All pad and strip footings should be cleaned out, inspected by a geotechnical engineer, and poured on the same day as excavation. If delays in pouring are envisaged, then we recommend that a concrete blinding layer be provided over the bases to reduce deterioration due to weathering.

Conventional bored piles should be cleaned out, inspected and poured on the same day as drilling. Piles should only be cleaned out when concrete is ready to be poured. We recommend that the initial stages of bored pile drilling be inspected by a geotechnical engineer to confirm that a satisfactory bearing stratum has been achieved.

4.4 Lower Ground Floor Slab

Based on the borehole information, the proposed ground floor excavation will mostly expose weathered shale bedrock, however at the northern end, a soil subgrade will be exposed. As such, we provide recommendations where shale is and is not exposed.

4.4.1 Shale Subgrade

The shale bedrock may be susceptible to weathering and degradation following exposure to the elements and in particular rainfall periods. We therefore recommend that good and effective drainage be provided during construction. The principal aim of the drainage is to promote run-off towards designated sumps by cross-falls and to reduce ponding. Any softened material should be scraped off prior to floor slab (including sub-floor drainage) construction.

4.4.2 Clay Subgrade

Subgrade Preparation

Following bulk excavation, we recommend that the clay subgrade be proof rolled with at least eight passes of a static smooth drum roller of at least 12 tonnes deadweight. The vibratory mode on the roller must not be used due to the close proximity of the nearby buildings and the need to limit ground borne vibrations. The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of unstable or soft areas.

Subgrade heaving during proof-rolling may occur in areas where the clays have become "saturated". Subgrade heaving should also be expected in the vicinity of JK2 where deep "uncontrolled" fill was encountered. Heaving areas should be locally removed to a stable base and replaced with engineered fill as outlined below. Possible alternatives to stripping the full depth of the heaving areas (eg. bridging layers), such as in the vicinity of JK2, should be provided by the geotechnical engineer during the proof rolling inspection, as appropriate.

If soil softening occurs after prolonged periods of rainfall, then the subgrade should be over-excavated to below the depth of moisture softening and replaced with engineered fill. If the clay subgrade exhibits shrinkage cracking, then the surface should be watered and rolled until the shrinkage cracks are no longer evident.

Engineered Fill

Engineered fill must be used where site levels need to be raised. Engineered fill can comprise the excavated clay soils on condition that they are free of organic matter and contain a maximum particle size of 75mm. Such material should be compacted in maximum 200mm thick loose layers using a large static roller to a density ratio strictly between 98% and 102% of SMDD and at a moisture content within 2% of SOMC. Based on the CBR test result, the clay soils will most likely require "drying out" in order to conform to the above moisture specification.

It may therefore be more economical and simpler to import well graded granular materials, such as crushed sandstone to a maximum particle size of 75mm, or DGS40 (RTA QA Specification 3051) which would also provide a good working platform over the clay soils during inclement weather periods. This material would also most likely be specified where bridging layers are required. Engineered fill comprising well graded granular materials should be compacted in maximum 200mm thick loose layers using a large static roller to a minimum density ratio of 98% of SMDD.

Service Trenches

Backfilling of service trenches must be carried out using engineered fill in order to reduce post-construction settlements. Due to the reduced energy output of the rollers that can be placed in trenches, backfilling should be carried out in 100mm thick loose layers and compacted using a trench roller or a pad foot roller attachment fitted to an excavator. Due to the reduced loose layer thickness, the maximum particle size of the backfill material should also reduce to 40mm. The compaction specifications provided above are applicable.

Earthworks Testing

Density tests should be regularly carried out on the engineered fill to confirm the above specifications are achieved. The frequency of density testing for general engineered fill should be at least one test per layer per 500m^2, or three tests per visit, whichever requires the most tests. The frequency of density testing for trench backfill should be at least one test per two layers per 40 linear metres (assumes maximum 100mm thick loose layers).

Level 2 testing of fill compaction is the minimum permissible in AS3798-2007. Due to an inherent conflict of interest, the GTA should be directly engaged by Health Infrastructure or their representative.