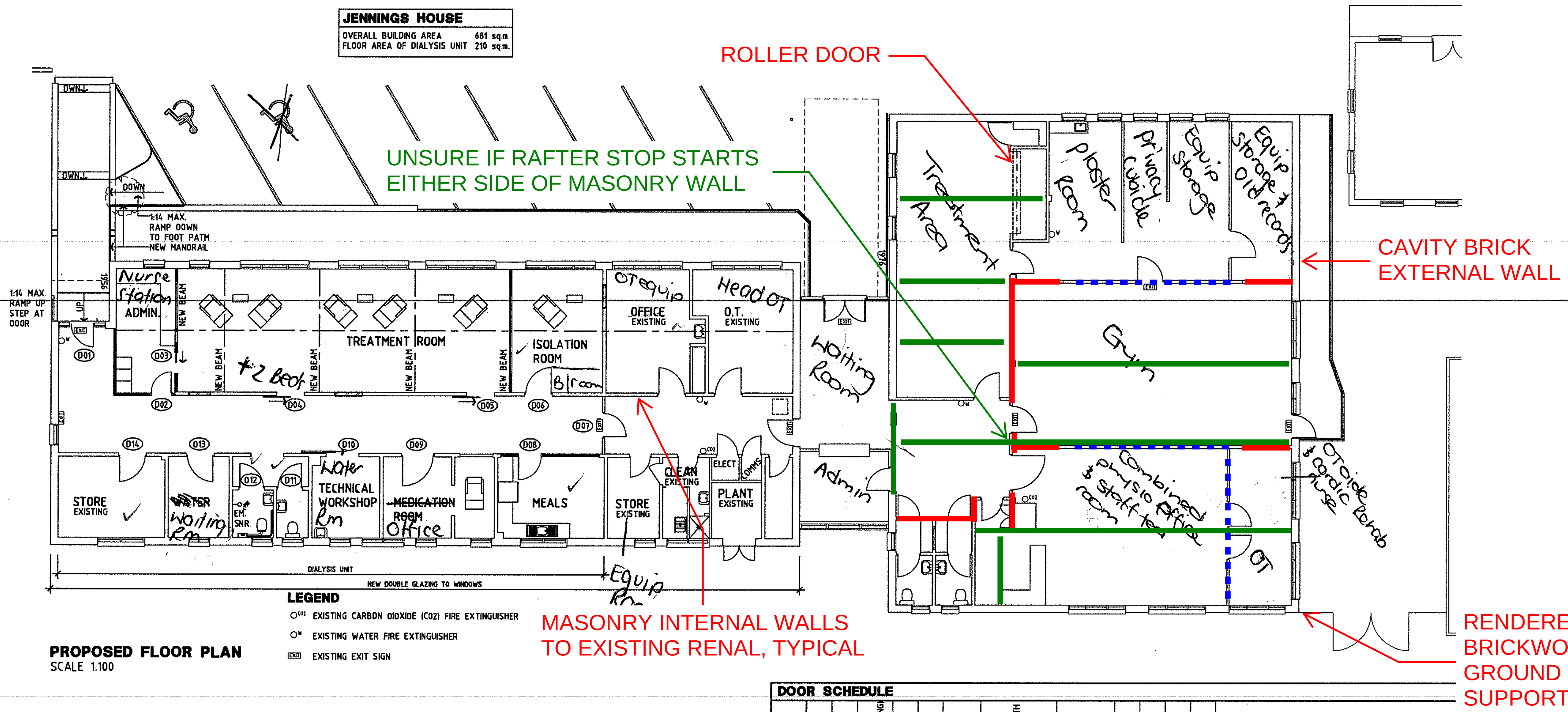
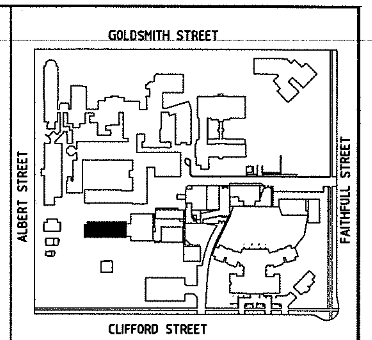
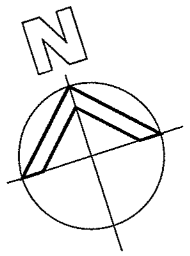




SITE MAP
NTS

THIS DRAWING HAS BEEN PRODUCED USING EXISTING DRAWINGS. ALL DIMENSIONS SUBJECT TO SITE SURVEY



MASONRY INTERNAL WALL

PLASTERBOARD INTERNAL WALL

RAFTER CONCEALED WITHIN BULKHEAD

DOOR SCHEDULE

DOOR NUMBER. REFER TO PLAN	EXISTING DOOR	NEW DOOR	DOOR CONSTRUCTION (SOLID CORE, OR EXISTING)	NEW DOOR FRAME	EXISTING DOOR FRAME	FRAME CONSTRUCTION (TIMBER OR EXISTING)	DOOR SIZE HEIGHT x WIDTH x DEPTH ALL UNITS IN mm.	HINGE TYPE & QUANTITY PER DOOR LEAF		DOOR CLOSER	NEW DOOR LOCKSETS	NEW DOOR FURNITURE	DOOR STOPS	DOOR SCHEDULE NOTES
								TYPE	QNTY					
D1	●		EX.	●	●	--	--	--	--	●	●	●	●	NOTE -
D2		●	S.C.	●	●	--	--	--	--	●	●	●	●	NOTE -
D3		●	S.C.	●	●	--	--	--	--	●	●	●	●	NOTE -
D4		●	S.C.	●	●	--	--	--	--	●	●	●	●	NOTE -
D5		●	S.C.	●	●	--	--	--	--	●	●	●	●	NOTE -
D6	●		EX.	●	●	--	--	--	--	●	●	●	●	NOTE -
D7	●		EX.	●	●	--	--	--	--	●	●	●	●	NOTE -
D8		●	S.C.	●	●	--	--	--	--	●	●	●	●	NOTE -
D9	●		EX.	●	●	--	--	--	--	●	●	●	●	NOTE -
D10	●		EX.	●	●	--	--	--	--	●	●	●	●	NOTE -
D11	●		EX.	●	●	--	--	--	--	●	●	●	●	NOTE -
D12	●		EX.	●	●	--	--	--	--	●	●	●	●	NOTE -
D13	●		EX.	●	●	--	--	--	--	●	●	●	●	NOTE -
D14	●		EX.	●	●	--	--	--	--	●	●	●	●	NOTE -

DOOR SCHEDULE NOTES:

1. USE "DORMA" DOOR CLOSER PRODUCT NUMBER TS-83 WITH HOLD OPEN OPTION. SILVER FINISH. MOUNT INTERNALLY SUPPLY AND INSTALL ALL REQUIRED FITTINGS AND FIXTURES REQUIRED FOR A COMPLETE INSTALLATION
2. USE "BOYD" DOOR STOP PRODUCT NUMBER 05-111 WITH SATIN CHROME FINISH. INSTALL TO MANUFACTURERS CURRENT PRINTED INSTRUCTIONS.
3. USE "CS CAVITY SLIDERS TIMBERFORM" DOOR WITH CAVITY SLIDERS No. 1031 PRIVACY LOCK. INSTALL TO MANUFACTURERS CURRNET PRINTED INSTRUCTIONS.
4. USE "LOCKWOOD" PASSAGE SET AND DOOR FURNITURE TO MATCH EXISTING DOORS
5. USE "LOCKWOOD" OFFICE SET AND DOOR FURNITURE TO MATCH EXISTING DOORS
6. SURFACE MOUNTED SLIDING DOOR WITN APPROVED TRACKS. INSTALL TIMBER SKIRTING TO DOOR HAD AROUND TRACKS. INSTALL ALL FITTINGS FIXTURES AND DOOR FURNITURE FOR A COMPLETE INSTALLATION AS REQUIRED. DOOR FURINTURE TO MATCH EXISTING
7. DOOR FININSHES TO MATCH EXISTING

Rev No / Drawn	Revision Description	Date
A- 0B	DEMOLITION PLAN & NOTES	05.08.04

Figured dimensions shall be used in preference to scaling of drawings

Builder to check and verify all levels datum and dimensions on the job. Conflicting information shall be brought to the Architects notice before proceeding with any work

All materials and workmanship to be in strict accordance with current manufacturers printed instructions local regulations and SAA Codes

Garry Dutaillis & Associates Pty Ltd
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GOULBURN NSW 2580
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The Royal Australian Institute of Architects

JENNINGS HOUSE

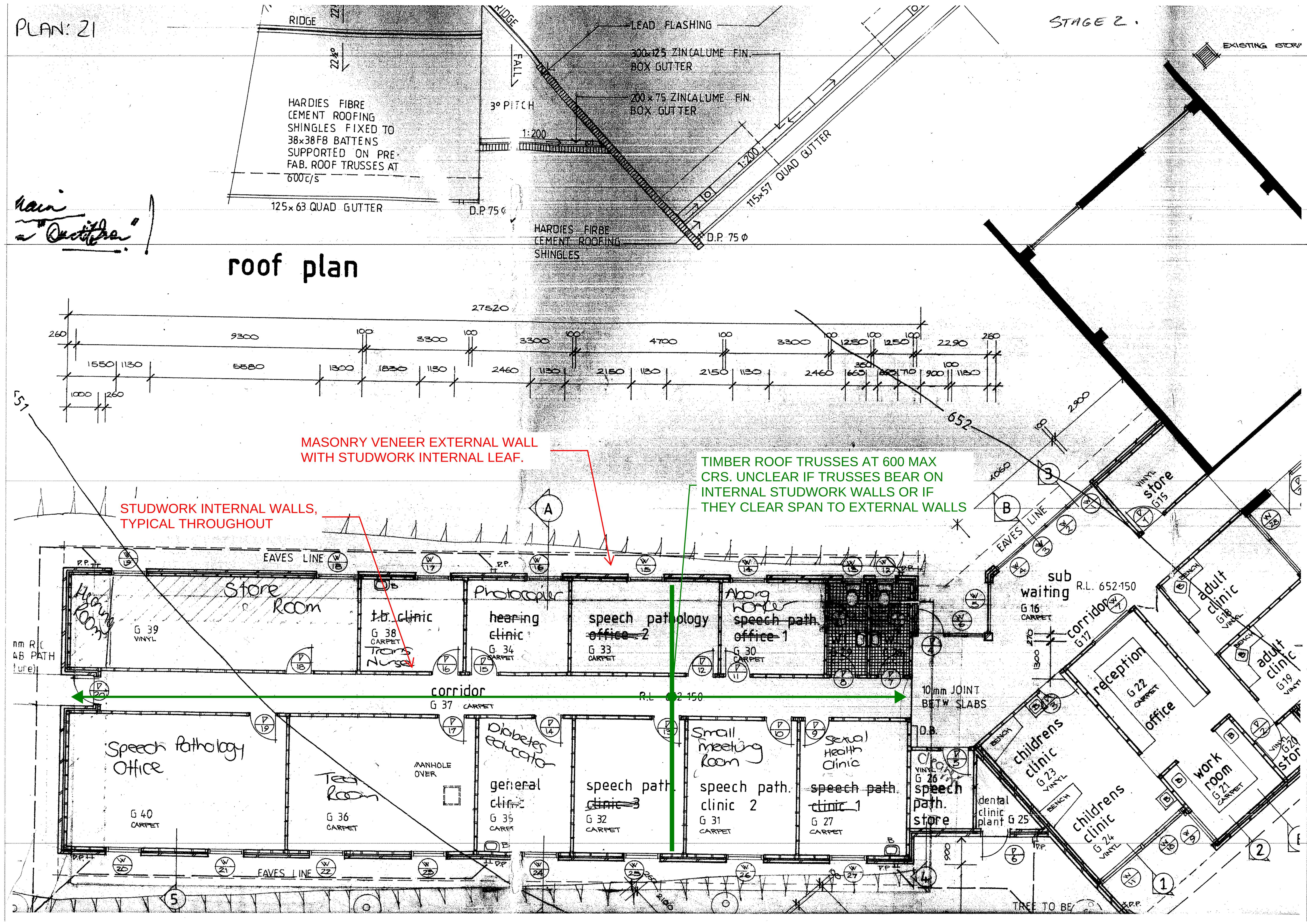
DIALYSIS UNIT

DEMOLITION PLANS

GOULBURN BASE HOSPITAL
GOULBURN NSW 2580

Drawn	Checked	Date
DB		08/04
Scale	Drawing Number	Rev No
SAS	3791-001	A

PLAN: 20



PLAN: 21
STAGE 2.
EXISTING STOREY
RIDGE
22½°
LEAD FLASHING
300x125 ZINCALUME FIN. BOX GUTTER
200 x 75 ZINCALUME FIN. BOX GUTTER
FALL
3° PITCH
1:200
HARDIES FIBRE CEMENT ROOFING SHINGLES FIXED TO 38x38F8 BATTENS SUPPORTED ON PRE-FAB. ROOF TRUSSES AT 600 c/s
125 x 63 QUAD GUTTER
D.P. 75 φ
115x57 QUAD GUTTER
HARDIES FIBRE CEMENT ROOFING SHINGLES
roof plan
27520
9300
3300
4700
2290
652
MASONRY VENEER EXTERNAL WALL WITH STUDWORK INTERNAL LEAF.
TIMBER ROOF TRUSSES AT 600 MAX CRS. UNCLEAR IF TRUSSES BEAR ON INTERNAL STUDWORK WALLS OR IF THEY CLEAR SPAN TO EXTERNAL WALLS
STUDWORK INTERNAL WALLS, TYPICAL THROUGHOUT
EAVES LINE
Store Room
G 39 VINYL
t.b. clinic
G 38 CARPET
Trans Nurse
Photocopier
hearing clinic
G 34 CARPET
speech pathology ~~office 2~~
G 33 CARPET
Abong Worker
~~speech path office 1~~
G 30 CARPET
sub waiting
G 16 CARPET
R.L. 652·150
corridor G 17
store G 15
adult clinic G 18
adult clinic G 19
reception G 22
office
corridor
G 37 CARPET
10mm JOINT BETW SLABS
Speech Pathology Office
G 40 CARPET
Tea Room
G 36 CARPET
MANHOLE OVER
Diabetes Educator
general clinic
G 35 CARPET
speech path. ~~clinic 3~~
G 32 CARPET
Small meeting Room
speech path. clinic 2
G 31 CARPET
Sexual Health Clinic
~~speech path. clinic 1~~
G 27 CARPET
Cleaner
G 26 VINYL
speech path. store
dental clinic plant G 25
childrens clinic G 23 VINYL
childrens clinic G 24 VINYL
work room G 21 CARPET
EAVES LINE
TREE TO BE

APPENDIX D. PROVISION FOR VERTICAL EXPANSION

Goulburn Hospital

Structural allowance for potential vertical expansion

Executive summary

The structural provisions required to allow future vertical expansion of one additional floor have been examined assuming the base case is a 4 storey building with a lightweight metal deck roof For discussion purposes cost rates have been assigned from previous projects. These need to be confirmed prior to insertion into a budget.

Item	Description	Quantity	Approx Cost of additional structure
Upgrade of capacity of foundations supporting columns	Total Increase in socket length of 1m diam piers	120m	$48,000
Upgrade of capacity of foundations for shear walls , lift shafts and stair shafts	Total Increase in socket length of 1m diam piers	95m	$60,000
Columns	Increase in structural capacity – average 50mm increase in size	40 cum concrete 10 tonnes reinf	$38,000
Shear walls	. Increase in thickness and reinforcement at lower levels	30 cum concrete 9 tonnes reinf	$35,000
		Subtotal	**$181,000**
Future floor slab	Post tensioned concrete slab at roof level (250 thick with 450x2400wide bands)	2400 sqm	$960,000

If in the base case a concrete roof slab is to be used rather than lightweight steel then the extra cost to allow it to be used as a floor slab in the future will be a significantly lower than the values calculated above

Possible options to be considered

Option 1 – no cost- Allow for no future vertical expansion

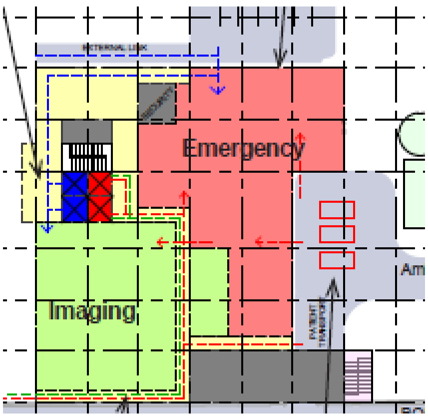
Option 2 – $181,000 - Design columns, footings and shear walls to accommodate future construction

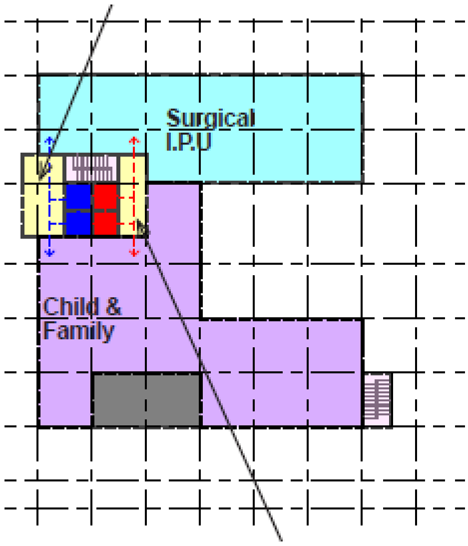
Any expansion would be very disruptive. We envision that this would be part of a major future redevelopment and basically provided to avoid the need to demolish the entire building structure.

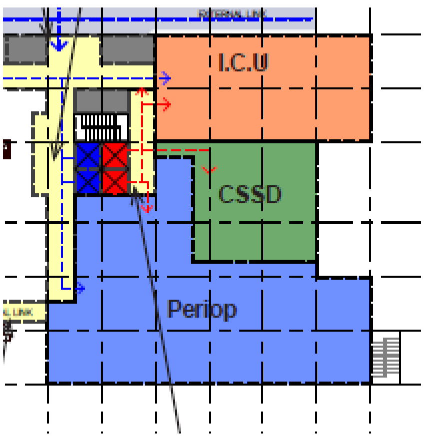
Option 3 – $1.14million- Provide a roof slab which can be converted to a floor in the future

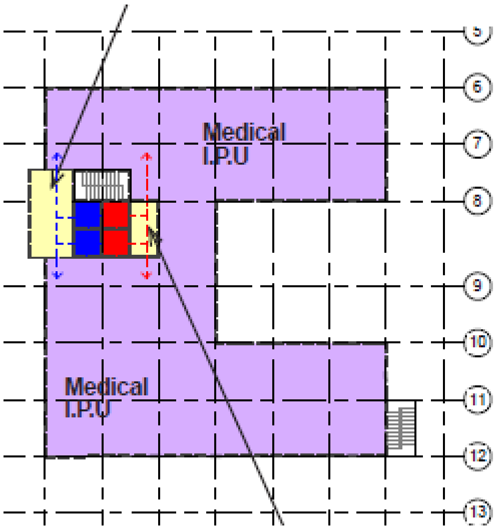
Introduction

The concept design for the main building of the hospital requires the construction of a new 4 storey building. The current planning does not predict a particular requirement for expansion, the purpose of this study is to explore the possible alternatives to accommodate future vertical expansion and the approximate associated cost of the additional structure









Geotechnical conditions

The subsurface conditions comprise shallow filling over approximately 2 to 3m of clay which in turn overlies low strength rock. The strength of rock may increase with depth. The geotechnical report recommends that the main load bearing elements be founded on rock. Pads or piers are to be designed for an allowable bearing pressure of 1000 kPa . For the purpose of this exercise, it is assumed that 1000mm diameter bored piers are to be utilised. Pad footings may also be an option.

Provisions required to vertical expansion

In order to accommodate a future expansion the following provisions could be made

1. **Design foundations of vertical load bearing elements to allow for additional floor**

The additional loading requires the socket lengths of the piers to be increased. Overall the total additional length of rock socketting of the 1000 diameter piers would be 120m

Based on a pier rate of $400/lin meter, this would equate to a cost of $48,000

2. **–Design columns for additional floor**

The effect of increasing the load per column (by up to 1200kN) would require an increase in column size by on average 50mm in each direction. The total additional volume of concrete resulting from this is 40 cum and requiring an additional 10 tonnes of reinforcement

Based on a cost of concrete of $250/cum and $2800/tonne of reinforcement this results in a cost increase of $38,000

3. **– Upgrade Shear walls and associated foundations for additional vertical loads , seismic and wind forces**

The building is classed as Importance Level 4 under the BCA. Seismic forces are the dominant lateral action to be considered

Based on the assumption that generally the shear elements will be approximately 8m in length (lift cores, stair shafts, individual walls) we estimate the following effects on the structure

Shear walls – lower 2 levels will increase in thickness by 50mm requiring 30 cum additional concrete and 9 tonnes of reinforcement This equates to a cost increase of approx $35,000

The increase in seismic load and its height of application has a significant effect on the tension piles of the cores and shear walls. On the basis that the piles act in groups with approximate lever arms of 8.0m we estimate that this will require a total increase in pile socket length of 95m at a cost of $60,000 allowing for additional tension reinforcement in piles.

4. Construction of future Floor slab

In lieu of a lightweight metal deck roof, construct a roof slab with the capacity to be topped and converted to support future accommodation. It is assumed that a metal deck roof would still need to be constructed to waterproof the slab , therefore the only offset in cost would be the deletion of the primary roof steelwork.

This structure will need to be a 250 thick post tensioned slab supported on 450 deep by 2400 wide post tensioned bands. The total area of slab will be in the order of 2400 sqm at a cost of approx $960,000

APPENDIX E. GEOTECHNICAL REPORT

REPORT

TO
NSW HEALTH INFRASTRUCTURE

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED NEW ACUTE SERVICES BUILDING

AT
GOULBURN BASE HOSPITAL
130 GOLDSMITH STREET, GOULBURN, NSW

26 April 2017
Ref: 30116V1rpt Goulburn

dlcs AS/NZS ISO 9001 certified
AS/NZS 4801 certified

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

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Jeffery & Katauskas Pty Ltd, trading as JK Geotechnics ABN 17 003 550 801

Date: 26 April 2017
Report No: 30116V1rpt Goulburn
Revision No: 1

Report prepared by:

Arthur Billingham
Geotechnical Engineer

Report reviewed by:

Fernando Vega
Senior Associate I Geotechnical Engineer

For and on behalf of
JK GEOTECHNICS
PO Box 976
NORTH RYDE BC NSW 1670

TABLE OF CONTENTS

1 INTRODUCTION

This report presents the results of a limited scope geotechnical investigation for the proposed new Acute Services building at Goulburn Base Hospital, 130 Goldsmith Street, Goulburn, NSW. The investigation was commissioned, in consultation with the project manager, Taya Kirris of TSA Management, by NSW Health Infrastructure Consultancy Agreement, Contract No. HI16591 dated 13 January 2017, following our fee proposal of 9 December 2016, Ref: P44003V – Goulburn. This report is the final version that supersedes the report that had been issued on 2 March 2017.

A summary of the principal geotechnical issues, based on findings of this investigation, is provided in Section 4.1.

The proposed works at the hospital are at preliminary planning stage. Based on the BLP's Concept Design Drawings (SK-B2-00, SK-B2-01, SK-B2-02, SK-B2-03), we understand that following demolition of some of the existing site buildings that a new building, with four levels, will be built to house the Acute Services unit of the Hospital. The proposed development area is shown on attached Figures 1 and 2. Development details, including finished levels, had not been finalised at the time of this investigation but it was indicated that due to the prevailing site slope, that excavation ranging from zero to 3m in depth is likely to be required to achieve development levels. Recommendations on earthworks including retention, and engineered fill specification has been provided. Structural loads have not been supplied and therefore, light to moderate column loads have been assumed for this type of development.

The scope of the investigation was limited to obtaining geotechnical information on subsurface conditions at four nominated and agreed test borehole locations as shown on attached Figure 2. Based on the test results, this report essentially provides the following geotechnical information, comments and recommendations on the following main items:

- Detailed logs of the boreholes with penetration test results and groundwater observations;
- Interpretation of Subsurface Profile;
- AS2870-2011 Site Classification;
- Suitable Footings Systems and Options;
- Foundation strata and depth;
- Allowable Bearing Pressures;
- Allowable Shaft Adhesions;

- Batter Slopes;
- Excavation Conditions;
- Retaining Wall Parameters;
- Subgrade Preparation;
- Engineered Fill Specification;
- CBR and modulus of subgrade reaction values for Pavement Thickness Design;
- Earthquake Design Factors;
- Inspections during Construction.

A preliminary environmental site screening of the in-situ soil materials has been carried out by our specialist environmental consulting division, Environmental Investigation Services (EIS), based on samples obtained from the geotechnical test boreholes. EIS will submit a separate report with the results, Ref: E30116Krpt, which should be read in conjunction with this geotechnical report. A hazardous building materials survey was also undertaken by EIS on only the buildings within the site and as specified by TSA Management on behalf of the client. The results of the survey are presented in a separate EIS report (Ref: E30116Krpt_HAZ).

2 INVESTIGATION PROCEDURE

Four boreholes, BHs 1 to 4, were drilled and tested on 31 January 2017, at the locations shown on the attached Figure 2, which is based on survey plans by Cardno (Drawing No. 117806501 Revision 1 dated 11/12/15). The borehole locations were set out by taped measurements from existing surface features and inferred site boundaries. The approximate reduced level (RL) at each borehole location, as shown on the borehole logs, was interpolated from spot heights and contours from the survey plan. The height datum is Australian Height Datum. Should any of these features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques.

Prior to commencement of the fieldwork, the investigation locations were electromagnetically scanned by a specialist subcontractor so that all borehole locations could be located clear of buried services. Reference to 'Dial Before You Dig' plans was also completed. Safe work measures and procedures were implemented during the course of the fieldwork.

The boreholes were drilled using our track-mounted JK305 rig with spiral auger techniques and a tungsten carbide (TC) drill bit to depths in the range of 6.0m and 7.5m below existing surface levels.

The apparent compaction of the fill and strength of cohesive soils were assessed from Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer tests carried out on cohesive samples recovered by the SPT split tube sampler. The strength of the bedrock was assessed from observation of the drilling resistance of a twin pronged TC drill bit attached to the augers, tactile examination of rock cuttings and correlation with the results of subsequent laboratory moisture content tests. It should be noted that strengths assessed in this way are approximate and variances of one strength order should not be unexpected.

Groundwater observations were recorded in all boreholes during and on completion of drilling. No longer-term groundwater monitoring was carried out. All boreholes were backfilled upon completion with concrete capping layers in those boreholes drilled through existing pavements.

Selected samples were returned to Soil Test Services (STS) Pty Ltd, a NATA accredited laboratory, to determine moisture contents, Atterberg Limits and linear shrinkages and four-day soaked CBR values. The results of the laboratory testing are provided in the attached STS Table A and B. Additionally, samples were also sent for soil aggression testing to Envirolab Services Pty Ltd, a NATA registered laboratory, the results of this testing are provided in the attached Certificate of Analysis: 161246. Selected samples collected by EIS were returned for testing and the results of which are detailed in their report, Ref: E30116Krpt.

Our geotechnical engineer, Mr Arthur Billingham, set out the borehole locations, nominated the sampling and testing and prepared logs of subsurface condition encountered. The borehole logs, which include groundwater observations, are attached, together with a set of explanatory notes which describe the investigation techniques, and their limitations and define the logging terms and symbols used.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The Goulburn Base Hospital property is located mid-slope on the north-eastern side of a local rise that rises to a crest approximately 2.5km from the site along Ridge Street. The Hospital has northern and eastern frontage with Goldsmith Street and Faithfull Street, respectively. Goldsmith Street slopes down to the east at approximately 2°. Faithfull Street slopes down to the south at approximately 1°.

The site itself, as shown on attached Figures 1 and 2, consists of an area in the north-eastern corner of Hospital currently occupied by a ‘C-plan’ two-storey brick building, surrounding car parking and environs. The brick building, which appears to be over 50 years old, appeared in good condition upon cursory visual observation. To the north and east of the building was asphalt paved car parking which appeared in poor condition with potholing and loosened material. To the south of the building is an asphalt driveway that slopes down at 2° to Faithfull Street. In the south-east corner between the car park and driveway are a couple of concrete block retaining walls with heights up to 1m.

To the west and south of the site are a number of brick hospital buildings. Set back between 3.8m and 6.0m to the west from the ‘C-plan’ building is a two storey brick building (Pathology) which sits between 0.9m and 1.3m below adjacent levels on site. To the southwest of the site is a three-storey brick building (Theatres). To the south, beyond the asphalt driveway, is a one-storey workshop building and two-storey asset management building.

3.2 Geology and Subsurface Conditions

The 1:250,000 Geological Series Sheet SI/55-12 ‘Goulburn’ indicates that the site is underlain by the Rhyanna Formation which consists of “varicoloured very thin- to thick-bedded, generally fine- to medium-grained, normally graded, ash rich, volcaniclastic sandstone and siltstone, and massive, dark brown, volcaniclastic mudstone; blue-grey to off-white, generally massive to planar laminated, tin- to thick-bedded silicified, volcanic mudstone; minor fine-grained, lithic-quartz sandstone”.

The boreholes encountered pavement and fill covering a natural profile of residual clays that grade into extremely to distinctly weathered bedrock typical of the aforementioned Rhyanna Formation consisting of tuffaceous sandstone in three boreholes and mudstone in one (BH4). A graphical borehole summary in section is shown on attached Figure 3.

A summary of the subsurface profile encountered in the test boreholes is given below; however, for a more detailed description reference should be made to the attached borehole logs.

Pavements and Fill

Asphalt pavement of 20mm and 45mm thickness was penetrated at surface in BH1 and BH2 respectively. Fill was encountered in all boreholes; from surface in BHs 3 and 4 down to 0.2m-0.3m; and below the pavements, the fill extended deeper in BHs 1 and 2 down to 1.3m and 0.7m, respectively.

Below the pavements a sandy gravel fill extended to approximately 0.4m depth. Underlying this gravelly fill, and at surface in BH3 and BH4, variable fill consisting of silty sand and silty clay was typical across the site. ‘

The fill was assessed as being variably compacted with varying inclusions including igneous gravel and slag; the fill below pavements as in BHs 1 and 2 appeared well compacted but the fill from surface as in BHs 3 and 4 appeared poorly compacted. This compaction assessment is based on in-situ SPT readings and/or our observations during drilling, which do not give a precise determination of in-situ densities since they are affected by friction during driving/pushing, the presence of gravel within the fill and the moisture content of the clay fill. Nonetheless, they provide a qualitative guide.

Residual Clays

Residual silty and gravelly clays were encountered below the fill at all locations and extended to depths ranging from 1.2m (BH4) to 2.5m (BH3). The clays were assessed as being high plasticity and of very stiff to hard strength. Gravels within the clays were ferruginous and generally fine- to medium-grained in size.

Weathered Bedrock

In boreholes 1 to 3 tuffaceous sandstone bedrock was encountered at depths in the range of 2m-2.5m, whilst mudstone bedrock was encountered in BH4 at 1.2m.

The bedrocks were assessed as being extremely weathered and extremely low to very low strength on first rock contact, improving in strength with depth to very low to low and medium to high strengths (the latter in BH4 only). The very low to low strength rock was encountered at depths of 3.5m-4.4m in BHs 1 to 3.

In BH4 mudstone of low to medium strength was encountered at 1.7m and became medium to high strength below 4m. The mudstone appeared to be heavily iron indurated particularly in the upper 3m.

Groundwater

No groundwater was observed during or on completion of drilling. No longer-term groundwater monitoring was allowed for as part of this investigation.

3.3 Laboratory Test Results

The results of moisture content tests on selected samples of the bedrock correlate reasonably well with the field strength assessment.

The results of the Atterberg Limits and Linear shrinkage tests confirmed that the residual clays to be of high plasticity and indicate these clays to be have a high shrink/swell reactive potential with moisture content change.

The four day soaked CBR test on a sample of the residual clay from BH3 resulted in a high CBR value of 10% at a compaction ratio of 98% of its Standard Maximum Dry Density (SMDD). We attribute the high CBR value for the residual clay sample due to the presence of numerous gravel.

The soil pH values ranged from 6.8 to 7.6, indicating slightly acidic to alkaline soil conditions. The sulphate contents and chloride contents were found to be low. Based on these results the soils would be classified as 'non-aggressive' exposure classification for concrete piles in accordance with Table 6.4.2(C) of AS2159-2009 'Piling – Design and Installation'.

4 COMMENTS AND RECOMMENDATIONS

4.1 Summary of Main Findings and Geotechnical Issues

As discussed in more detail in Section 3.2, the four boreholes penetrated a variable subsurface profile, essentially comprising variably compacted fill to depths in the range of 0.2m to 1.3m, underlain by residual silty and gravelly clays of high plasticity that grade into extremely weathered bedrocks of sandstone and mudstone at depths in the range of 1.2m to 2.5m below existing site surface. The strength of the bedrock improves with increasing depth. Groundwater was not encountered in the boreholes.

Based on the known borehole and test results, our observations and the proposed development concept (refer to Section 1), we consider the following to be the principal geotechnical issues to be considered in the planning, design and construction of the development:

1. The presence of fill of variable compaction and depth across the site makes it a Class P in accordance with AS2870-2011. The fill is deemed unsuitable as a bearing stratum for footings. The fill is subject to proof rolling and further assessment to decide if it is suitable to act as a supporting subgrade under ground floor slabs and pavements.

2. The residual clays have a high potential for shrink/swell with changes in moisture content, i.e. Class H2 in terms of AS2870-2011.
3. The most competent foundation stratum at the site is the sandstone and mudstone bedrocks. In view of the moderate depths to bedrock and the high reactivity of the residual clays, we recommend that the new structure is supported on a combination of deep pad footings and/or bored pier footings founded into the bedrock.
4. All cut and/or fill faces that might be incorporated in the proposed development must be rendered stable by appropriate batters and/or retention systems. All fill placed on site to replace existing or unstable areas must be engineered, controlled fill.
5. In regards to waste and contamination classification issues are likely to be required in order to classify the material to be excavated for waste disposal purposes in accordance with the NSW DEC (formerly EPA) guidelines as inert, solid, industrial or hazardous waste, reference should be made to the separate EIS report (refer to Section 1).

Further comments on the above issues are provided in the subsequent sections of this report.

4.2 Further Geotechnical Work

Only a limited subsurface investigation was completed with limited site coverage. Nevertheless, depths to bedrock was not highly variable (in the range of 1.2m-2.5m) and the rock was consistent type (sandstone) in three out of the four boreholes, whilst the fourth borehole disclosed mudstone which was of better quality than the sandstone in terms of strength and weathering. Hence, the main reasons to consider the completion of additional boreholes, if elected, would be to attain greater coverage of the proposed building area, and perhaps, to assess the possibility of greater bearing pressures in the bedrock than recommended in this report. Any additional boreholes, if elected, should be completed once demolition and site clearance has finished and details of the development are better known such as layout, levels and loads. It is also advisable to complete the boreholes using rock coring methods to assess defects and allow strength testing of recovered rock cores, for greater assessment of bearing capacity values, if possible. If additional test boreholes are elected then we also recommend the further CBR testing of the subgrade for any new pavements.

Notwithstanding, it will be essential during earthworks, foundation and pavement works that geotechnical inspections be commissioned to check initial assumptions about excavation, foundation conditions and possible variations that may occur between inspected and tested locations and to provide further relevant geotechnical advice. Irregular or ‘milestone’ inspections by a geotechnical engineer are often not adequate for earthworks and foundation works. It is recommended that the Client be made aware of the need to commission a geotechnical engineer for regular frequent inspections. The comments provided in this report should be reviewed following these inspections.

The recommendations provided in this report should be reviewed once details of the proposed development are known, as mentioned above and also, following any additional geotechnical work. It is likely that further advice/input will be required during the structural design to address issues that may not have been addressed in this report. To some degree, this is an “iterative” process between evaluation of the geotechnical site conditions and the structural design.

4.3 Site Classification

The site classification for this site in accordance with Section 2 of AS2870-2011 *‘Residential Slabs and Footings’* is ‘Class P’ due to the presence of fill of variable depth and compaction across the site but particularly at the location of BH1. Based on the field assessment of plasticity and linear shrinkage test results, the natural soils (residual clays) on this site are likely to exhibit characteristic surface movements in the order of a Class ‘H2’ site. We advise that in the strict sense AS2870 site classification does not apply to the proposed buildings or structures larger than specified in the code, but it is a useful guide in highlighting the potential foundation problems of this site. The standard footing designs in AS2870-2011 are unlikely to be relevant to this project and therefore, the footing design must be carried out using engineering principles.

4.4 Footings

The most competent foundation stratum beneath the site for the development is the sandstone and mudstone bedrocks, as identified by the test boreholes. In view of the presence of fill and the high reactivity potential of the residual clays, we recommend that the building structure is supported on footings uniformly founded within the bedrock profile. In view of the depth variations of the bedrock, we expect a combination of deep pads and bored piers could be used to reach a suitable rock foundation stratum.

The bedrock is the recommended foundation stratum for any proposed structure with movement sensitive finishes or items and/or for structures that have columns or walls with moderate or heavy loads. Notwithstanding, if required after further development details are known, we can provide the option of using spread footings on the residual clay provided the structure is designed to withstand potential reactive clay movements. However, the use of hybrid foundations for the building is to be avoided.

The following criteria are recommended for design and construction of the footings:

1. The deep pads (where rock is 1.2m or less deep), conventional bored piers or CFA piles may be used and proportioned for an allowable bearing pressure (ABP) of 600kPa when founded with a nominal socket of about 0.3m into the extremely weathered tuffaceous sandstone or mudstone of extremely low strength.
2. The ABP may be increased to 1000kPa if the piers are drilled deeper to reach the distinctly weathered sandstone or mudstone of at least very low to low strength (VL-L on logs). Even greater bearing pressures (e.g. 1500kPa to 3500kPa) may be possible within the sandstone or mudstone of low to medium strength (L-M on logs) but this would have to be proven by drilling at least two rock cored boreholes with strength testing of the rock cores.
3. An allowable shaft adhesion (ASA) of equivalent to 10% of the above ABP values may be adopted for design of pier sockets, in compression, through the mudstone and sandstone. For uplift or tension, the aforementioned ASA value should be halved. The shaft adhesion values are recommended on condition that cleanliness and roughness of pier sockets and bases are achieved.
4. For piers socketed into the bedrock, large capacity drilling rigs with rock augers would have to be used.
5. All loose or softened debris should be cleared from the base of all footings prior to concreting. All footings should be poured immediately after excavation/drilling, removal of water (if any), cleaning and inspection.

The allowable bearing pressures and shaft adhesions have been estimated on the basis of our augered borehole data. It should be noted that strengths assessed in this way are approximate and variances of one strength order should not be unexpected. If higher bearing pressures than those given in this report are required, this may be possible within L-M rock after conducting further geotechnical investigation involving cored boreholes so that a better assessment of the bedrock strength and defects can be made.

The ABP values given above is based on a serviceability criteria of deflections at the pile toe of less than or equal to 1% of the pile diameter. Footings on rock can also be designed using ‘Limit State Design’ principles. For limit state design, higher ultimate bearing capacities could be adopted provided that settlements of 5% of the pile diameter can be tolerated and there is an extensive pile test program undertaken. It should be noted that such ultimate bearing pressures must be used in conjunction with an appropriate geotechnical strength reduction factor (Φ_g). It must be understood that the use of limit state design to adopt relatively high bearing pressures (above the serviceability criteria described above) is not currently standard practice, and there is an increased risk of inadequate pile performance.

If the designer wishes to adopt the Limit State Design methods of the piling code, AS2159-1995, then the ultimate values of end bearing pressures and shaft adhesions may be estimated by multiplying the above recommended allowable values by Factors of Safety of 3 and 2, respectively. We recommend that a geotechnical strength reduction factor, Φ_g, of 0.5 be applied to the maximum pile load applied.

As a minimum requirement, the initial stages of footing excavation should be inspected by a Geotechnical Engineer to ascertain that the recommended foundation has been reached and to check initial assumptions about foundation conditions and possible variations that may occur between borehole locations. The need for further inspections can be assessed following the initial visit.

Suitable pile types are considered to be conventional bored piers and augered, grout injected (CFA) piles. For grout piles, during installation of piles it is recommended that the initial piles be installed as close as practical to our borehole locations to calibrate the equipment and operator to the subsurface conditions by direction comparison of the installation performance and readings to the borehole results. These initial readings can then be used to assist with installation of piles away from the borehole locations to assess that the appropriate foundation material has been reached.

The on-grade floor slab should be isolated from the walls, columns and footings to allow for potential shrink-swell movements in the underlying residual clays. Joints in the concrete on-grade floor slab should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints.

4.5 Earthquake Design Parameters

The following parameters can be adopted for earthquake design in accordance with AS1170.4-2007 'Structural Design Actions, Part 4: Earthquake Actions in Australia':

- Hazard Factor (Z) = 0.09
- Site Subsoil Class = Class C_e

4.6 Pavements

We understand that some pavements will be required for car parking and access roads for the new development. After stripping to nominal subgrade or formation level, the exposed subgrade in the proposed pavement area should be prepared as outlined in the above Section 4.8, with respect to proof rolling and compaction.

The design of new pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of fill excavated or imported to the site, as well as vehicle loadings and use. Various alternative types of construction could be used for the pavements. Concrete construction would undoubtedly be the best in areas where heavy vehicles manoeuvre such as bus turning and manoeuvring. Flexible pavements may have a lower initial cost but maintenance will be higher. These factors should be considered when making the final choice.

Once details of the pavement area layout, levels and traffic loading are determined, we recommend that thickness design of the pavement on residual clay subgrade of high plasticity be carried out on the basis of a lower CBR value of 3% than was obtained from the laboratory testing (Table B; reason for the higher CBR obtained during testing are discussed in Section 3.3). An equivalent modulus of subgrade reaction of 20kPa/mm (based on 750mm plate) may be adopted for concrete pavement design.

Concrete pavements should be supported on at least a 100mm thick sub-base of good quality fine crushed rock such as RTA Specification 3051 unbound base (e.g. DGB20) or similar quality, and compacted to a minimum density of 98% of MMDD. The sub-base material will provide more uniform slab support and would reduce “pumping” of subgrade “fines” at joints. Slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints.

Where the pavement edge is unconstrained by landscaping strips, we recommend that an edge thickening be provided for protection against erosion and the effects of possible future topsoil stripping.

Density tests should be regularly carried out on the sub-base layer to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 1000m^2, or three tests per layer, or three tests per visit, whichever requires the most tests. Level 2 testing of fill compaction is the minimum permissible in AS3798-2007.

Subsoil drains should be provided along the perimeter of all proposed external pavement areas, with invert levels of at least 300mm below subgrade level. The drainage trenches should be excavated with a uniform longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system.

4.7 Earthworks: Subgrade Preparation for Floor Slabs and Pavements

The main geotechnical issue with earthworks, including subgrade preparation, under ground floor slabs of the development and external pavement areas, is to do with any areas of existing fill that appears to be variably compacted. Another issue is to do with the reactivity of the residual clay. We suggest that generous time and budget allowances be provided for subgrade improvement works.

In the area of the slabs, we recommend that the compaction of any remaining existing clay fill be further improved to achieve minimum engineered fill standards as detailed below. Notwithstanding, the floor slabs must be designed and constructed to withstand potential shrink/swell movements of the underlying residual clay profile. The clay is considered to be highly reactive and may be subject to free surface movements, equivalent to those quoted in AS2870-2011 for Class H1 clay subgrades; reference to the code is essential.

Alternatively, the slab can be fully suspended on the footings bearing on the bedrock. With the latter option, if the slab is to be fully suspended on bored pier footings then it would be unnecessary to complete any particular subgrade preparation other than stripping any top layer of root affected soil. The suspended slab option is considered the appropriate solution if differential movements (e.g. from reactive clay movements) are unacceptable for the habitable spaces. Void formers of 100mm thickness should be installed below the slabs and beams to allow for potential swell of the silty clays. The void formers should extend at least over the zone where significant potential changes in moisture content of the clay subsoils may occur that would trigger shrink/swell behavior. Given the thickness of the clays above the rock are limited, in the range of about 1m-2.5m, then we suggest that the void formers extend for a distance from the inside of external walls of at least 2m-3m below the interior section of the floor slab. In addition, as mentioned further below, an external concrete apron or paving should extend at least over the 2m strip around the building and preferably further; the paving should slope to shed water away from the footings.

As a minimum, if the slab is to be supported on existing subgrade and if some differential movements can be tolerated, the subgrade for the ground floor slab and for external pavements, should undergo the following basic treatment. Earthworks recommendations provided in this report should be complemented by reference to AS3798.

1. Strip the upper layer of fill that contains deleterious materials or organics (roots), and stockpile this separately since these materials are not suitable for re-use as engineered fill. This upper layer is irregular in depth and occurrence but for budgeting purposes allow for an average stripping depth of about 0.2m-0.3m but deeper as deemed necessary after inspection by a geotechnician or geotechnical engineer.
2. The exposed subgrade at the base of the excavation should be proof rolled with at least 8 passes of a heavy (not less than 12 tonne) smooth drum vibratory roller. The purpose of the proof rolling is to detect any soft or heaving areas.
3. The final pass should be undertaken in the presence of a geotechnician or geotechnical engineer, to detect any unstable or soft subgrade areas, and to allow for some further improvement in existing fill compaction.
4. Unstable subgrade detected during proof rolling should be locally excavated down to a stiff or sound base and replaced with engineered fill or further advice should be sought. Any fill placed to raise site levels should also be engineered fill.

Particular attention must be paid to the provision of adequate site drainage. The main objective of the drainage should be to minimise the risk of water entering the clay subgrade and to prevent water from ponding next to the slabs. To minimise the possibility of moisture ingress into the clay subgrade, we recommend that surface and subsurface drains be constructed at an early stage in the program to intercept water from the higher areas of the site. The surface drain should discharge well away from the building area, say, to adjoining stormwater drainage systems. Furthermore, we recommend that close attention be paid to the design and construction of the water and sewage services beneath the proposed buildings, to minimise the risk of breakage, leakage and subsequent moisture ingress into the fill subgrade. These services should be designed with flexible joints and they should be repaired promptly when ruptured.

The clays at the site are highly reactive. The shrink/swell of the clays are triggered by changes in moisture content. Hence, for stabilizing moisture conditions of the clay next to the slabs and external walls can be assisted by paving the surface of this area with concrete and avoid planting gardens and lawns next to the walls and slabs. Concrete apron or paving should extend at least over the 2m strip around the building and preferably further; the paving should slope to shed water away from the footings. Other site, vegetation, drainage and subgrade/foundation maintenance requirements given in AS2870-2011 for highly reactive clay sites should be implemented as appropriate. As noted in previous page, in the case of suspended slabs on bored piers founded on rock, void formers of 100mm thickness should be used under the slabs.

4.8 Engineered Fill Specification

Any fill used to backfill unstable subgrade areas, raise surface levels or backfill service trenches should be engineered fill. Materials preferred for use as engineered fill are well-graded granular materials, such as ripped or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in layers not greater than 200mm loose thickness, to a minimum density of 98% of Standard Maximum Dry Density (SMDD).

The existing fill at the site are acceptable for re-use on condition that the soils used are clean (i.e. free of organics and inclusions greater than 75mm size), free of contaminants. We suggest against using the residual clay due its high plasticity and high reactivity potential. The existing fill soils should be compacted in maximum 200mm loose layers to a density strictly between 98% and 102% of SMDD and at moisture content within 2% of Standard Optimum. All clay fill should preferably be used in the lower fill layers. Thus, the use of clay materials for engineered fill will entail more rigorous earthwork supervision and compaction control, time for drying out the soils and hence, possibly a greater eventual cost for earthworks.

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 500m^2 or three tests per visit, whichever requires the most tests. We recommend that at least Level 2 control of fill compaction, as defined in AS3798-2007, be adhered to on this site. However, if a floor slab is to rely on engineered fill for support then we recommend a higher compaction control of Level 1. We can complete the abovementioned testing and supervision if required. Preferably, the geotechnical testing authority (GTA) should be engaged directly on behalf of the client and not by the earthworks subcontractor.

During construction of the fill platform runoff should be enhanced by providing suitable falls to reduce ponding of water on the surface of the fill. Ponding of water may lead to softening of the fill and subsequent delays in the earthworks program.

4.9 Excavation Conditions, Batters and Retention

We do not anticipate significant earthworks or excavation on this site; however, any earthworks e.g. ground preparation for any new pavements, and earthworks (such as footing excavations) carried out must be completed in accordance with the Code of Practice ‘Excavation Work’, Cat No. 312 prepared by WorkCover NSW and AS3798-2007.

We do not have final details whether or not retaining walls or batter slopes are proposed as part of the development. However, it has been indicated that due to the prevailing site slope, that excavation ranging from zero to 3m in depth is likely to be required to achieve development levels Also, it is likely some trench excavations are expected for installation of service conduits. Hence, we provide some recommendations below on these matters.

It is seen from the boreholes that excavations deeper than 1.2m-2m will encounter sandstone or mudstone bedrocks, which will represent hard rock excavation conditions if they are taken deep through rock of low to medium strengths or greater.

The fill and residual clays and upper layers of extremely weathered bedrock should be able to be excavated using conventional excavation equipment such as bucket attachments on large hydraulic excavators.

The sandstone/mudstone of low or greater strengths will present hard rock excavation conditions requiring the use of rock excavation equipment, such as hydraulic rock hammers. The use of excavators with hydraulic impact hammer attachments, albeit small and lightweight, for any sandstone excavation should be approached with considerable caution, as there will likely be direct transmission of ground vibrations to nearby structures and buildings. Guideline levels of vibration velocity for evaluating the effects of vibration in structures are given in the attached Vibration Emission Design Goals sheet. We recommend that the acceptable limit for transmitted vibrations be set at quite a low peak particle velocity of 5mm/s for frequencies of less than 10Hz at foundation level. To fall within these limits, we recommend that the size of rock hammers initially used during the trial not exceed medium sized rock hammer say 900kg such as Krupp 580. If it is found that transmitted vibrations are unacceptable, then it may then be necessary to change to a smaller excavator with a smaller rock hammer, or to a rotary grinder, saws, or jackhammers. If rock hammers are to be used, we recommend that the initial excavation in rock should preferably be commenced away from likely critical areas and instrument vibration monitoring undertaken. The monitoring program should be confirmed when details of the contractor's excavation methods and sequence are known. By monitoring vibrations in this way, it will allow some freedom to the excavation contractor in the equipment he adopts, so that a balance can be made between productivity and vibration reduction.

Unsupported excavations should not be attempted at this site through the variably compacted fill, residual clays and any extremely weathered rock of extremely low to very low strengths. All the latter materials require support by either formation of cut batter sides of the excavations or installation of shoring walls (e.g. shoring boxes for conduit trenches or semi-contiguous bored piers) is vertical excavations is to be used due to lack of space to accommodate battered excavation sides. Short term batters should not be steeper than 1 Vertical (V) to 1 Horizontal (H) may be adopted; but this batter must be flattened to 1V in 2H for long term stability. The batters may be steepened to 1V:0.5H in the sandstone of low or greater strength bedrock subject to inspection by geotechnical engineer. Surcharge loads should be kept well away from the edges of the temporary batters. As a guide, surcharge loadings should be no closer than 2H from the top of any batter or the face of any excavation, where H is the vertical height of the batter. The temporary batter slope would be applicable for short term situations during construction of various building structures (e.g. retaining walls). If the batter slope is to be left exposed for a period of say more than one month, or is affected by heavy rainfall, we recommend that the batter slope be flattened to 1V to 2H. If permanent batter slopes of soil cuts or of fill platforms are proposed, we recommend that they be graded at no steeper than 1 Vertical (V) on 2 Horizontal (H). Surface erosion protection, for example, quick establishing grass or proprietary systems (such as those provided by Geofabrics Australasia Pty Ltd), must be provided to the permanent batter slopes. Dish drains should be provided along the crest and toe of all permanent batter slopes to intercept surface water run-off. Discharge should be piped to the stormwater system.

Any free standing or cantilevered retaining or shoring walls that are required at the site should be designed for a coefficient of active earth pressure of at least 0.35 and a bulk unit weight of 20kN/m3, assuming a horizontal backfill surface. For the above profile areas which are highly sensitive to lateral movement, consideration should be given to the use of a greater lateral pressure coefficient, K, of at least 0.6. For sandstone or mudstone rock of low to medium strength or greater strength the Ka value may be taken to be negligible. The retaining wall should be designed to withstand full hydrostatic pressure unless measures are taken to introduce complete and permanent drainage of the ground behind the walls. All surcharge loads, including footing loads, which affect the walls should be considered in their design. The aforementioned earth pressure parameters apply to a horizontal backfill surface and, if inclined backfill surfaces are to be designed, then the above factors would have to be increased or the inclined section of backfill should be taken as a surcharge load in the design. For the design of walls socketed into the sandstone of at least low strength, it is recommended that a maximum allowable toe resistance of 200kPa may be used below the base of the excavation, including footing and service excavations, to resist the lateral pressure.

During the excavations, care must be taken to not undermine or render unstable any remaining adjoining building footings within the zone of influence of the excavations. The zone of influence is defined by line radiating up from the base of the excavation (including footing/services trenches) at a gradient of 1V in 2H. It may not be practicable to prevent significant vertical and lateral ground displacements immediately beyond the limits of the excavation, so the effects of the inevitable movements on the adjoining building have to be considered. The magnitude of lateral movement is directly related to the stiffness of the retaining system. It may be necessary to underpin adjoining footings before commencing the excavation, in order to protect them from ground displacements. Further comment on this aspect would require details of adjacent building footings to be provided. This may require additional test pit investigation to expose footings if plans are not available.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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SOIL TEST SERVICES

ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed New Acute Services Building
Location: Goulburn Base Hospital, 130 Goldsmith Street, Goulburn, NSW

Ref No: 30116V
Report: A
Report Date: 8/02/2017
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
1	1.50-1.95	26.9	63	30	33	16.0
1	5.70-6.00	21.2				
2	3.50-4.00	10.4				
2	4.30-4.50	10.7				
3	3.00-3.25	16.7				
3	4.20-4.50	14.7				
4	0.50-0.95	29.1	66	29	37	17.5
4	2.60-3.00	5.7				
4	4.00-4.50	4.4				

Notes:

- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 25/01/2017

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Ref No:**	30116V
Project:	Proposed New Acute Services Building	**Report:**	B
Location:	Goulburn Base Hospital, 130 Goldsmith Street, Goulburn, NSW	**Report Date:**	8/02/2017
		Page 1 of 1	

BOREHOLE NUMBER	3
DEPTH (m)	0.30 - 1.00
Surcharge (kg)	4.5
Maximum Dry Density (t/m^3)	1.86 STD
Optimum Moisture Content (%)	13.6
Moulded Dry Density (t/m^3)	1.83
Sample Density Ratio (%)	98
Sample Moisture Ratio (%)	99
Moisture Contents	
Insitu (%)	15.6
Moulded (%)	13.4
After soaking and	
After Test, Top 30mm(%)	18.6
Remaining Depth (%)	16.5
Material Retained on 19mm Sieve (%)	0
Swell (%)	0.0
C.B.R. value: @2.5mm penetration	10

NOTES:

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods :
  - (a) Soaked C.B.R. : AS 1289 6.1.1
  - (b) Standard Compaction : AS 1289 5.1.1
  - (c) Moisture Content : AS 1289 2.1.1
- Date of receipt of sample: 25/01/2017

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Authorised Signature / Date
(A. Tatikonda) 8/2/17

12 Ashley Street, Chatswood, NSW 2067
tel: +61 2 9910 6200

email: sydney@envirolab.com.au
envirolab.com.au

Envirolab Services Pty Ltd - Sydney | ABN 37 112 535 645

CERTIFICATE OF ANALYSIS **161246**

Client:
JK Geotechnics
PO Box 976
North Ryde BC
NSW 1670

Attention: Arthur Billingham

Sample log in details:

Your Reference:	**30116V, Goulburn**
No. of samples:	3 Soils
Date samples received / completed instructions received	02/02/17 / 02/02/17

Analysis Details:
Please refer to the following pages for results, methodology summary and quality control data.
Samples were analysed as received from the client. Results relate specifically to the samples as received.
Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details:

Date results requested by: / Issue Date:	9/02/17 / 7/02/17
Date of Preliminary Report:	Not Issued

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Accredited for compliance with ISO/IEC 17025 - Testing **Tests not covered by NATA are denoted with *.**

Results Approved By:

David Springer
General Manager

Misc Inorg - Soil				
Our Reference:	UNITS	161246-1	161246-2	161246-3
Your Reference	-------------	2	3	4
Depth	------------	0.7-0.95	1.5-1.95	1.5-1.95
Date Sampled		31/01/2017	31/01/2017	31/01/2017
Type of sample		Soil	Soil	Soil
Date prepared	-	06/02/2017	06/02/2017	06/02/2017
Date analysed	-	06/02/2017	06/02/2017	06/02/2017
pH 1:5 soil:water	pH Units	6.8	6.8	7.6
Chloride, Cl 1:5 soil:water	mg/kg	50	20	32
Sulphate, SO4 1:5 soil:water	mg/kg	83	82	40
Resistivity in soil*	ohm m	65	110	120

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity.

QUALITY CONTROL Misc Inorg - Soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base ll Duplicate ll %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			06/02/2017	161246-1	06/02/2017 || 06/02/2017	LCS-1	06/02/2017
Date analysed	-			06/02/2017	161246-1	06/02/2017 || 06/02/2017	LCS-1	06/02/2017
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	161246-1	6.8 || 6.8 || RPD: 0	LCS-1	99%
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	161246-1	50 || 54 || RPD: 8	LCS-1	103%
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	161246-1	83 || 82 || RPD: 1	LCS-1	113%
Resistivity in soil*	ohm m	1	Inorg-002	<1.0	161246-1	65 || 67 || RPD: 3	[NR]	[NR]

Report Comments:

Asbestos ID was analysed by Approved Identifier:	Not applicable for this job
Asbestos ID was authorised by Approved Signatory:	Not applicable for this job

INS: Insufficient sample for this test	PQL: Practical Quantitation Limit	NT: Not tested
NR: Test not required	RPD: Relative Percent Difference	NA: Test not required
<: Less than	>: Greater than	LCS: Laboratory Control Sample

Quality Control Definitions

Blank: This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.

Duplicate: This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.

Matrix Spike : A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.

LCS (Laboratory Control Sample) : This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.

Surrogate Spike: Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.
Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.
Spikes for Physical and Aggregate Tests are not applicable.
For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: <5xPQL - any RPD is acceptable; >5xPQL - 0-50% RPD is acceptable.
Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
1/2

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED NEW ACUTE SERVICES BUILDING
Location: GOULBURN HOSPITAL, 130 GOLDSMITH STREET, GOULBURN, NSW

Job No. 30116V
Method: SPIRAL AUGER JK305
R.L. Surface: ≈ 652.2m
Date: 31/1/17
Datum: AHD
Logged/Checked by: A.B./F.V.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION		0	-	ASPHALTIC CONCRETE: 20mm.t	D			
				FILL: Sandy gravel, fine to coarse grained igneous gravel, dark grey, fine to medium grained sand, light grey.				APPEARS WELL COMPACTED
	N = 18 6,10,8			FILL: Silty sand, fine to coarse grained, dark grey, with fine to coarse grained igneous gravel, orange and red, trace of slag.	M			
	N = 10 5,4,6	1	CH	SILTY CLAY: high plasticity, light brown.	MC<PL	H	400 410 520	RESIDUAL
	N >39 10,18, 21/90mm REFUSAL	2	-	SANDSTONE: tuffaceous, fine to medium grained, yellow brown, with clay seams.	XW	EL		
		4			DW	VL-L		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
		7						

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED NEW ACUTE SERVICES BUILDING
Location: GOULBURN HOSPITAL, 130 GOLDSMITH STREET, GOULBURN, NSW

Job No. 30116V
Date: 31/1/17
Method: SPIRAL AUGER JK305
R.L. Surface: ≈ 652.2m
Datum: AHD
Logged/Checked by: A.B./F.V.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									SANDSTONE: tuffaceous, fine to medium grained, yellow brown, with clay se	DW	VL-L		LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
						8							
						9							
						10							
						11							
						12							
						13							
						14							

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
2
1/1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED NEW ACUTE SERVICES BUILDING
Location: GOULBURN HOSPITAL, 130 GOLDSMITH STREET, GOULBURN, NSW

Job No. 30116V | **Method:** SPIRAL AUGER JK305 | **R.L. Surface:** ≈ 653.3m
Date: 31/1/17 | **Datum:** AHD
Logged/Checked by: A.B./F.V.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0	-	ASPHALTIC CONCRETE: 45mm.t				
				FILL: Sandy gravel, fine to coarse grained, dark grey, igneous, fine to medium grained sand, light grey.	D			APPEARS WELL COMPACTED
	N = 18 7,9,9			FILL: Silty clay, low to medium plasticity, brown, with fine grained sand, trace of fine grained igneous gravel.	MC<PL			
		1	CH	GRAVELLY CLAY: high plasticity, light brown, fine to medium grained, black and red brown, ferrginous gravel.	MC<PL	H	>600 >600	RESIDUAL
	N = 13 7,6,7	2	CH	SILTY CLAY: high plasticity, light brown, with fine to medium grained ferrginous gravel, black and red brown.	MC<PL		>600 580 450	
	N = SPT 15/70mm REFUSAL	3	-	SANDSTONE: tuffaceous, fine to coarse grained, green grey.	XW-DW	EL-VL		LOW 'TC' BIT RESISTANCE
		4			DW	VL		
		5		as above, but orange brown.		L		LOW RESISTANCE WITH MODERATE BANDS
						M		MODERATE RESISTANCE
		6		END OF BOREHOLE AT 6.0m				
		7						

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No. 3
1/1

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED NEW ACUTE SERVICES BUILDING
Location: GOULBURN HOSPITAL, 130 GOLDSMITH STREET, GOULBURN, NSW

Job No. 30116V
Method: SPIRAL AUGER JK305
R.L. Surface: ≈ 654.7m
Date: 31/1/17
Datum: AHD
Logged/Checked by: A.B./F.V.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Silty clay, low plasticity, dark brown, with fine to medium grained igneous and ironstone gravel.	MC<PL			GRASS COVER
	N = 10 3,5,5		CH	GRAVELLY CLAY: high plasticity, light brown and orange, fine to medium grained, black and red brown, ferruginous gravel.	MC>PL	VSt	240 260	RESIDUAL
	N = 24 6,8,16	1		GRAVELLY CLAY: high plasticity, light grey, fine to coarse grained, orange brown, siliceous gravel.	MC<PL	H	470 430	
	N > 15 12,15/ 100mm REFUSAL	2	-	SANDSTONE: tuffaceous, fine to coarse grained, yellow brown.	XW	EL		
		3			XW-DW	EL-VL		
		4		as above, but iron indurated bands.	DW	L		LOW 'TC' BIT RESISTANCE
		6		END OF BOREHOLE AT 6.0m				

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GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
4

Client: HEALTH INFRASTRUCTURE
Project: PROPOSED NEW ACUTE SERVICES BUILDING
Location: GOULBURN HOSPITAL, 130 GOLDSMITH STREET, GOULBURN, NSW

Job No. 30116V
Method: SPIRAL AUGER JK305
R.L. Surface: ≈ 652.8m
Date: 31/1/17
Datum: AHD
Logged/Checked by: A.B./F.V.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Silty sand, fine to medium grained, dark brown, with fine to medium grained igneous gravel.	M			
	N = 11 3,6,5		CH	SILTY CLAY: high plasticity, light brown.	MC>PL	VSt	370 330 260	RESIDUAL
	N > 15 7,15/ 100mm REFUSAL	1	-	MUDSTONE: grey, with sandstone, fine grained, grey seams and clay seams.	XW	EL		
		2		MUDSTONE: grey, with iron indurated bands.	DW	L-M		LOW 'TC' BIT RESISTANCE
		3		MUDSTONE: brown and grey, with iron indurated bands.		M		MODERATE RESISTACE WITH HIGH BANDS
		4				M-H		MODERATE RESISTANCE
								HIGH RESISTANCE
		5						LOW TO MODERATE RESISTANCE
		6		END OF BOREHOLE AT 6.0m				
		7						

Samples: ES, U50, DB, DS

PLOT DATE: 6/02/2017 10:07:33 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30116V GOULBURN\CAD\30116V.DWG

AERIAL IMAGE SOURCE: GOOGLE EARTH PRO 7.1.5.1557
AERIAL IMAGE ©: 2015 GOOGLE INC.

This plan should be read in conjunction with the JK Geotechnics report.

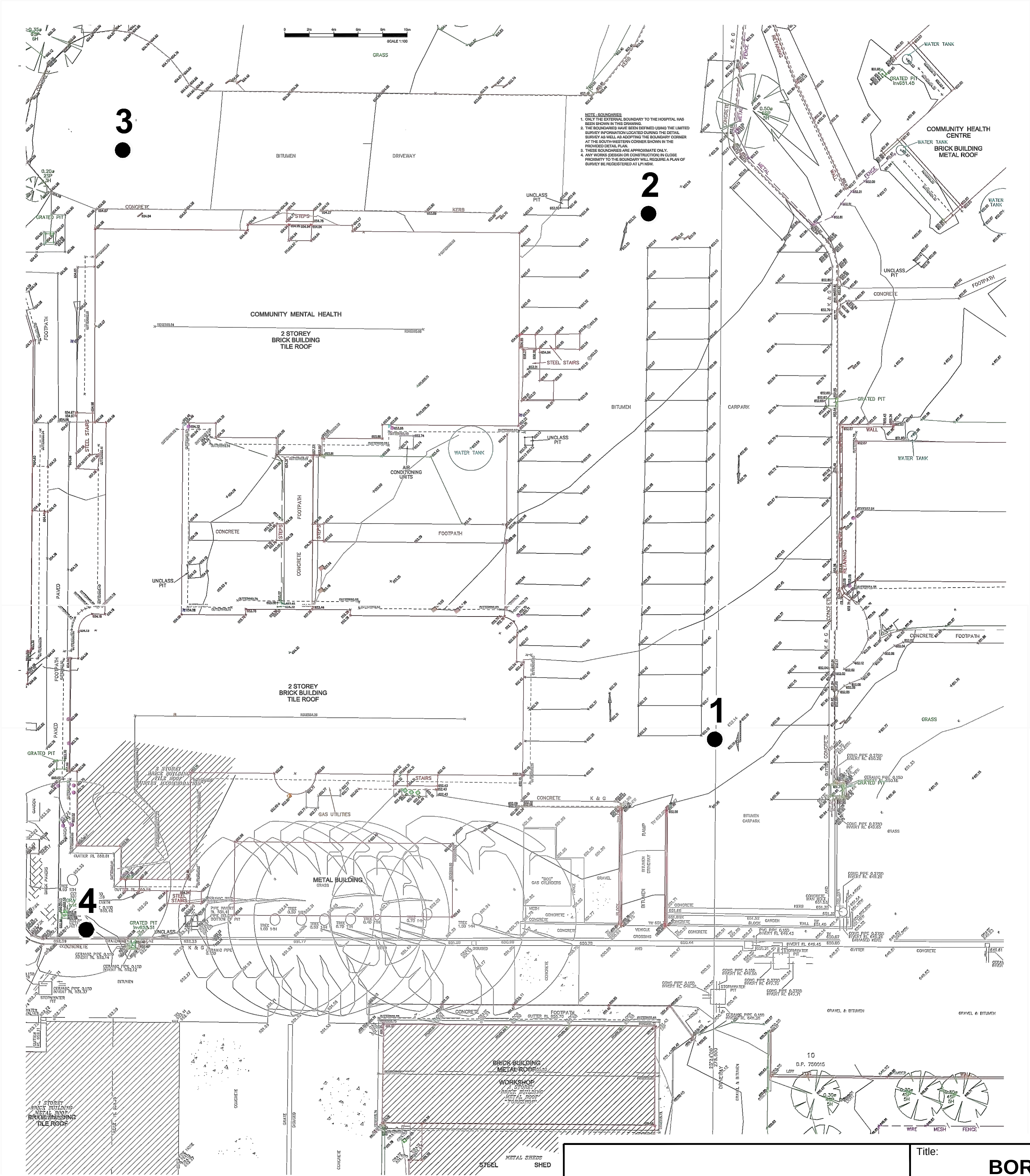
Title: **SITE LOCATION PLAN**

Location: GOULBURN HOSPITAL
130 GOLDSMITH STREET, GOULBURN, NSW

Report No: 30116V

Figure No: 1

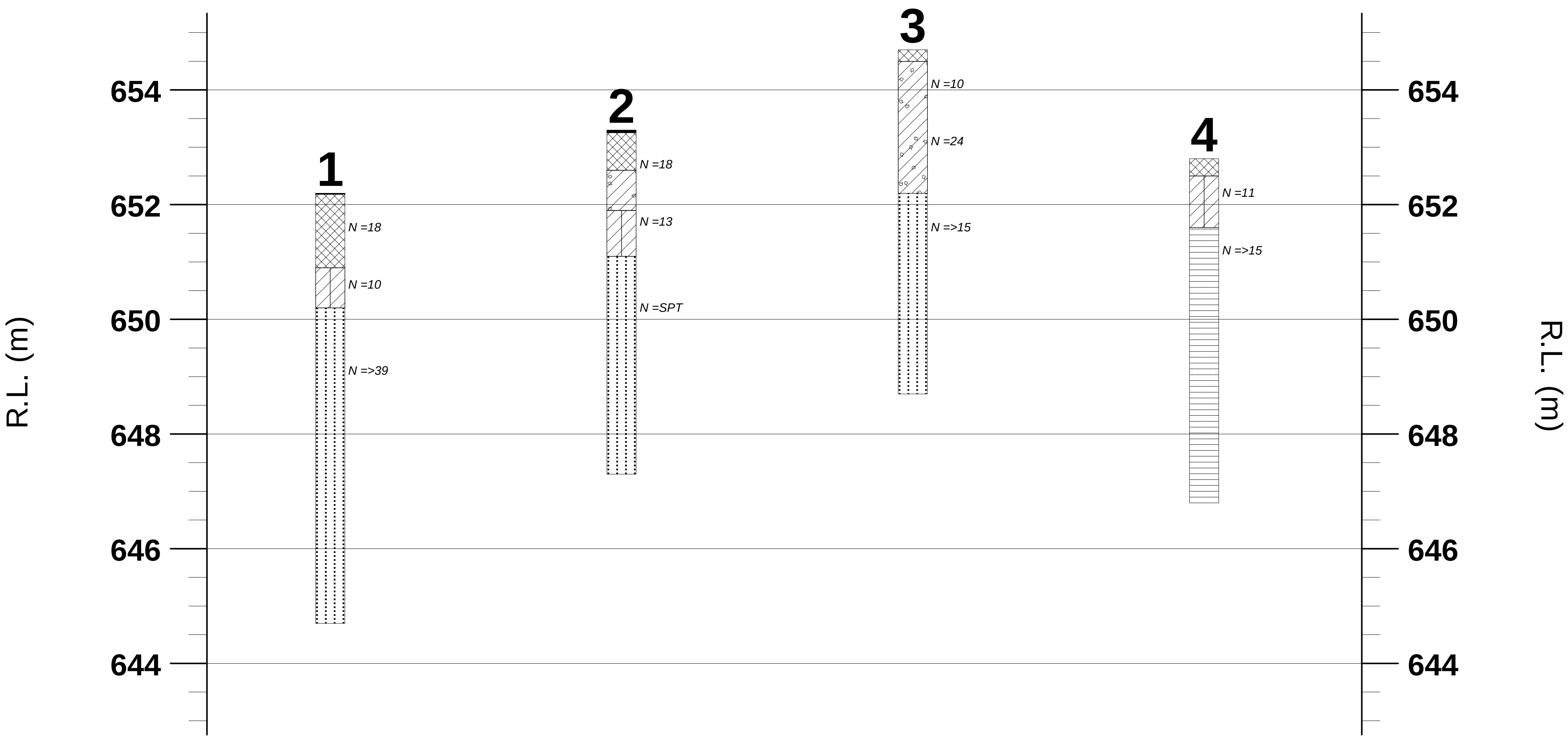
JK **Geotechnics**



0 4 8 12 16 20 SCALE 1:400 @A3 METRES	Title: **BOREHOLE LOCATION PLAN**		
	Location: GOULBURN HOSPITAL 130 GOLDSMITH STREET, GOULBURN, NSW		
	Report No: 30116V	Figure No: 2	
This plan should be read in conjunction with the JK Geotechnics report.	JK **Geotechnics**		

PLOT DATE: 6/02/2017 10:02:18 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30116V GOULBURN\CAD\30116V.DWG

GRAPHICAL BOREHOLE SUMMARY



Asphaltic/ Bituminous Paving or Coal

Fill

Silty Clay

Sandstone/ Greywacke

Gravelly clay

Siltstone, Mudstone, Claystone

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 100 (vert) ; NTS (horiz)

JK **Geotechnics**

Job No.: 30116V

Figure No.: 3

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1986 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

REPORT EXPLANATION NOTES

INTRODUCTION

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

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, the SAA Site Investigation Code. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached Unified Soil Classification Table qualified by the grading of other particles present (e.g. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2mm
Gravel	2 to 60mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

Cohesive soils are classified on the basis of strength (consistency) either by use of hand penetrometer, laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	Strength not attainable – soil crumbles

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'Shale' is used to describe thinly bedded to laminated siltstone.

SAMPLING

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

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock fragments. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers such as Revert or Biogel. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test F3.1.

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as "N_c" on the borehole logs, together with the number of blows per 150mm penetration.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

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

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

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

The attached explanatory notes define the terms and symbols used in preparation of the logs.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than "straight line" variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

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

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

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg bricks, steel etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably determine the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.

If these occur, the company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer.

SITE INSPECTION

The company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or

iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CH)
- SILTY CLAY (CL, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML)
- PEAT AND ORGANIC SOILS

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE
- SILTSTONE, MUDSTONE, CLAYSTONE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

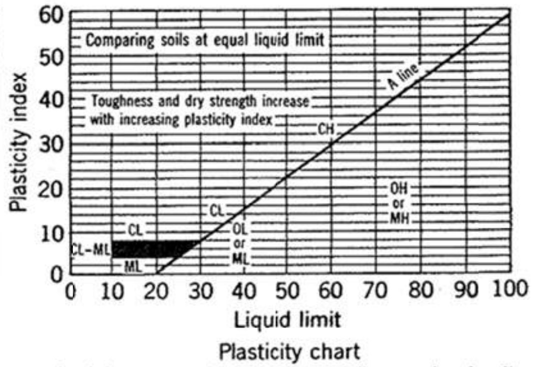
Identification Procedures larger than 75 μm and basing fractions on estimated weights)		Group Symbols a	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria	
(little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*	
(appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
	Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7	
(little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*	
(appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
	Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7	

Use grain size curve in identifying the fractions as given under field identification

Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows:
Less than 5% — *GW, GP, SW, SP*
More than 12% — *GM, GC, SM, SC*
5% to 12% — *Borderline* cases requiring use of dual symbols

dures on Fraction Smaller than 380 μm Sieve Size

	Dry Strength (crushing character-istics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
	Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
	Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
	High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
	Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
	Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

Note: 1 Soils possessing characteristics of two groups are designated by combinations of group symbols (eg. GW-GC, well graded gravel-sand mixture with clay fines).
2 Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	 C 	Standing water level. Time delay following completion of drilling may be shown. Extent of borehole collapse shortly after drilling. Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	Soil sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos screening. Soil sample taken over depth indicated, for acid sulfate soil analysis. Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25 PID = 100	Vane shear reading in kPa of Undrained Shear Strength. Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils) (Cohesionless Soils)	MC>PL MC≈PL MC<PL D M W	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. DRY – Runs freely through fingers. MOIST – Does not run freely but no free water visible on soil surface. WET – Free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt H ( )	VERY SOFT – Unconfined compressive strength less than 25kPa SOFT – Unconfined compressive strength 25-50kPa FIRM – Unconfined compressive strength 50-100kPa STIFF – Unconfined compressive strength 100-200kPa VERY STIFF – Unconfined compressive strength 200-400kPa HARD – Unconfined compressive strength greater than 400kPa Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)	 VL L MD D VD ( )	**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)** Very Loose <15 — 0-4 Loose 15-35 — 4-10 Medium Dense 35-65 — 10-30 Dense 65-85 — 30-50 Very Dense >85 — >50 Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit 'TC' bit T_{60}	Hardened steel 'V' shaped bit. Tungsten carbide wing bit. Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
----------------------	-------------	0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
----------------------	-------------	0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
----------------------	-------------	0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
----------------------	-------------	1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
----------------------	--------------	3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
----------------------	--------------	10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	