REPORT

TO
CITY WEST HOUSING PTY LTD

ON
GEOTECHNICAL INVESTIGATION

FOR
PROPOSED BUILDING D4 AFFORDABLE HOUSING

AT
NORTH EVELEIGH PRECINCT,
OFF WILSON STREET, EVELEIGH, NSW

13 March 2013
Ref: 26366SBrpt

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed Building D4 Affordable Housing within the North Eveleigh Precinct, off Wilson Street, Eveleigh, NSW. The investigation was commissioned by Ms Janelle Goulding of City West Housing Pty Ltd by returned of Acceptance of Proposal, Ref: P36591SB1.

The proposed Building D4 is located within the North Eveleigh Precinct, which will be developed into a residential precinct containing several residential unit buildings. The location of the site is shown on Figure 1. As shown on the supplied concept architectural drawings by Architectus (Project No. 120325, Drawing Nos A-SD100/2 A-SD101/3, A-SD102/4, A-SD103/4, A-SD104/4, A-SD105/4, A-SD106/4, A-SD107/4, A-SD108/4, A-SD114/1 and A-SD115/1) Building D4 will comprise a residential unit building with six and seven above ground levels over one basement level. The majority of the proposed basement will be at RL23.1m, with the eastern end at RL22.5m. This will require excavations for the basement to depths ranging from about 2.3m to 2.8m. The structural engineer for the project, Enstruct Group Pty Ltd, have advised working column loads of the order of 3500kN to 4500kN. Access to the building will be via a new internal road network, which is being constructed separately to the proposed building.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions as a basis for comments and recommendations on excavation, groundwater, retention and footings.

2 INVESTIGATION PROCEDURE

We have previously completed geotechnical investigations within the North Eveleigh precinct between 1998 and 2008. As part of the 2008 investigations, one borehole, BH302, was drilled within the footprint of the proposed Building D4. The results of BH302 have been used herein for this geotechnical investigation. The current fieldwork involved the drilling of four additional boreholes, BH501 to BH504.

BH302 and BH501 to BH504 were drilled to total depths ranging from 6.0m to 12.07m below the existing ground surface level, using our truck mounted JK350 and our track mounted JK300 drill rigs. BH502 and BH504 were drilled using spiral auger drilling techniques for their full depths of 7.5m and 6.0m. BH302, BH501 and BH503 were auger drilled to depths ranging from 2.94m to

8.93m and were then continued by diamond coring techniques using a NMLC core barrel with water flush to their final depths ranging from 6.04m to 12.07m.

Prior to drilling, the borehole locations were electromagnetically scanned by a specialist subcontractor to check for buried services. The borehole locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features. The locations of BH501 to BH504 were dictated by construction work being carried out on site at the time of the fieldwork. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels shown on the supplied survey plans by Cardno Young Pty Ltd (Drawing No 602083CD-02, dated April 2006). The datum of the levels is Australian Height Datum (AHD).

The strength of the subsurface soils were assessed from Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer readings on clayey samples recovered in the SPT split tube sampler. Within the augered portions of the boreholes, the strength of the weathered shale was assessed from observations of the penetration resistance of a tungsten carbide (‘TC’) bit attached to the augers, together with examination of the recovered rock cuttings, and subsequent correlations with laboratory moisture content test results. We note that rock strengths estimated in this way are indicative and variations of at least one strength order should not be unexpected.

The strength of the cored shale was assessed with reference to Point Load Strength Index (I_{s50}) test results. The point load strength test results are summarised on the cored borehole logs and in the attached STS Table B.

Groundwater observations were made within the boreholes both during auger drilling and on completion of coring. We note that water is introduced into the boreholes during coring and therefore the water levels measured at completion of coring will likely be artificially high as the water level has not had time to stabilise. PVC standpipes were installed in BH501 and BH503 on completion to allow longer term monitoring of groundwater levels. The coring water within these standpipes was removed following installation and groundwater reading taken during the fieldwork and on 11 March 2103. However, the standpipe that was installed in BH503 on 28 February 2013 was buried below a stockpile of excavated concrete on 4 and 5 March 2013 and could not be found during our last site visit on 11 March 2103. Therefore, groundwater readings could not be made within that standpipe. We suspect that the top of the standpipe was destroyed by the civil contractor on site during stockpiling of concrete and removal of the concrete pavements following installation of the standpipe. Another previously installed standpipe was also discovered on site, as shown on Figure 2, and groundwater levels within that standpipe were also measured.

Our geotechnical engineers were present on a full-time basis during the fieldwork, to direct the electromagnetic scanning, set out the borehole locations, nominate testing and sampling locations, and prepare the borehole logs. The borehole logs, which include field test results and groundwater observations, are attached to this report together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab services Pty Ltd, both NATA accredited laboratories, for testing to determine moisture contents, Atterberg limits, linear shrinkages, point load strength indices, soil pH, sulphate content and chloride content. The results of the laboratory testing are summarised in the attached STS Tables A and B and Envirolab Report No. 86743. Contamination testing of the site soils was outside the scope of this investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The North Eveleigh Rail Yards cover an area of about 9 hectares, being about 130m wide (north-south) and 950m long (east-west). The rail yards are bounded by Wilson Street to the north, Iverys Lane to the west, the main western rail lines to the south and Little Eveleigh Street to the east. The rail yards are dominated by two large double storey brick former workshop buildings within the central portion, known as the Carriageworks. These buildings are predominantly used as theatres and studios. The site of the proposed Building D4 is located within the western portion of the rail yards as shown on Figure 1.

The rail yards are relatively flat in comparison to the surrounding topography indicating that the existing rail yard site has been cut into the slope prior to the construction. The surrounding topography indicates that the rail yards would have been sloping down to the south-east prior to the cut and fill earthworks. Along the northern boundary of the rail yards, to the north of the proposed Building D4 site, are retaining walls and batter slopes supporting Wilson Street, with the street level approximately 5.5m higher than the surface levels of the rail yards and the subject site. Opposite the eastern end of the subject site is a steel shed and a concrete block structure is located to the north of this shed. The rear wall of the concrete block structure acts as a retaining wall supporting Wilson Street, but the structure was boarded up at the time of the fieldwork and inspection of the retaining wall itself was not possible. However, the exposed walls of the structure appeared to be in good condition. To the west of the concrete block structure is a grass covered batter sloping down from Wilson Street at approximately 25°; there were a few small trees on this batter. To the east of the batter is a highly corroded steel frame of a previous structure, with brick retaining wall forming three sides of this structure supporting the street to the north and batters to the east and west. The brick retaining walls appear to be in good condition. Further to the west of this structure is a grass covered batter sloping down from Wilson Street at approximately 25°.

At the time of the current 2013 fieldwork, demolition works were being carried out within and surrounding the proposed Building D4 site. The building that previously occupied part of the proposed building footprint, as shown on Figure 1, had been demolished with demolition of the pavements underway. About half the site was still concrete paved, with exposed clayey fill visible over the remainder. We were informed on site that the civil contractor was about to commence construction of a temporary roadway through the Building D4 site to allow construction of the permanent road on the northern side of Building D4.

The former rail yards extend in all directions around the Building D4 site and are primarily covered with concrete and Asphaltic Concrete (AC) pavements. Three buildings surround the subject site, but the closest of these is located approximately 20m away. These buildings are a metal clad shed to the north, a large two storey brick warehouse (Carriageworks) to the east and a two storey brick building to the west. The two buildings shown on the survey plan (Figure 1) to the south have been demolished and only concrete slabs remain. The steel shed appeared to be in good condition with no corrosion observed from a cursory inspection of the façade. The two brick structures on either side of the site appeared to be in good condition, however minor brick patch work was observed on both buildings.

3.2 Subsurface Conditions

Reference to the Sydney 1:100 000 Geological Series Sheet indicates that the site is located in an area underlain by the Ashfield Shale of the Wianamatta Group, which overlies Hawkesbury Sandstone at depth.

In summary, the boreholes encountered surface fill covering residual silty clays that grade into weathered shale bedrock. Further comments on the subsurface conditions encountered are provided below. Reference should be made to the borehole logs for detailed descriptions of the subsurface conditions encountered.

Concrete

Concrete was encountered at the surface of BH501, BH503 and BH504 and was 200mm to 220mm thick. In BH502, sandy gravel fill was encountered at the surface, but concrete was encountered at a depth of 0.2m and was 100mm thick.

Fill

Fill was encountered to depths ranging from 0.5m to 1.0m and comprised silty sandy gravel, sandy gravel, silty clay, clayey sand and clayey gravel. Generally, the fill was assessed to be poorly compacted.

Residual Silty Clays

Residual silty clays were encountered in BH302 and BH501. The silty clay was assessed to be of high plasticity and of hard strength.

Weathered Shale

Weathered shale was encountered at depths ranging from 0.8m to 2.4m. In BH302, BH502 and BH503 clayey shale was initially encountered that was extremely weathered and of extremely low strength to depths of 5.3m, 2.3m and 1.2m, respectively. The upper shale was of very variable quality, but mainly extremely weathered to distinctly weathered and of extremely low to very low strength. The deeper weathered shale profile was encountered within BH302, towards the south-eastern corner of the site. Shale that was assessed to be distinctly weathered and of at least low strength was encountered in BH302 at a depth of 7.2m and in BH501 to BH504 at depths ranging from 2.0m to 3.6m.

The cored shale in BH302, BH501 and BH503 was assessed to be slightly weathered and of medium strength. In BH502 and BH504, where coring was not carried out, and in the augered portions of BH501 and BH503, such medium strength shale was encountered at depths ranging from 2.7m to 4.8m. In BH302, medium strength shale was encountered during auger drilling at a depth of 8.4m.

Defects within the cored shale were widely spaced and comprised extremely weathered seams of up to 6mm thickness, and joints inclined at up to 90°. Sections of core loss were encountered in

BH501 and BH503 of 0.04m and 0.15m thickness and these may represent extremely weathered or clay seams.

Groundwater

Groundwater seepage was observed during auger drilling of BH302 at a depth of 8.0m. In BH501, groundwater was observed on completion of auger drilling at a depth of 4.1m. Groundwater measurements taken within the standpipe installed in BH501 and the previously installed standpipe are summarised in the following table. As detailed in Section 2 above, the standpipe in BH503 was destroyed by the civil contractor following installation.

Standpipe	Groundwater Depth and Approximate Level Measured within Standpipes on Site					
	4 March 2103		5 March 2013		11 March 2013	
	Depth	≈RL (AHD)	Depth	≈RL (AHD)	Depth	≈RL (AHD)
Existing Standpipe	2.70m	22.5m	2.35m	22.85m	2.25m	22.95m
BH501	N/A	N/A	2.15m	23.05m	2.02m	23.18m

3.3 Laboratory Test Results

Based on the Atterberg limit and linear shrinkage test results the residual silty clay tested is of medium plasticity and is assessed to have a moderate to high potential for shrink/swell reactivity with changes in moisture content. The laboratory moisture content and point load strength index test results showed reasonably good correlation with our field assessment of rock strength.

The soil pH values indicate that the weathered shale is acidic at 4.9 to 5.3. The sulphate and chloride contents were found to be low. These materials would represent exposure classification of A2 in accordance with Table 4.8.1 of AS3600-2009 'Concrete Structures'. In accordance with Table 6.4.2(C) of AS2159-2009 'Piling – Design and Installation', these materials would be classified as 'mild' exposure classification for concrete piles or 'non-aggressive' for steel piles, in accordance with Table 6.5.2(A).

4 COMMENTS AND RECOMMENDATIONS

4.1 Effect of Proposed Development on RailCorp Assets

As part of this geotechnical investigation we have been asked to comment on the effect of the proposed development on the existing and proposed RailCorp assets. In order to assess this we have been provided within the following drawings:

- Survey plans by Cardno Young Pty Ltd, Drawing No 602083CD-02, dated April 2006, showing existing surface features.
- Drawing by GHD, Ref: 21-22056-E200, Amendment 4, dated 20/12/12, showing proposed relocation of the North Eveleigh 11kV aerial line, which is located to the north of the Building D4 site. This line is an aerial power line supported on timber poles, with the line at heights of ranging from about 6.7m to 16m.
- Drawings by Connell Wagner, Project No. 27551.001, Drawing Nos SK-100, Rev. 2 and SK-101 to 105, Rev 1, SK-110 to 112, Rev 1, SK-120 to 124, Rev 1, and SK-130 to 134, Rev 1, dated 2/10/07, showing the concept design for the proposed North Eveleigh Dive and Tunnel Alignment. This proposed tunnel will be located below the existing main western railway line on the southern side of the North Eveleigh Rail Yards.

Based on these drawings, Figure 1 shows the location of the proposed Building D4 basement, the existing main western rail line, the existing and proposed 11kV aerial line, and the proposed tunnel protection zone for the North Eveleigh Dive and Tunnel Alignment. We note that the Connell Wagner drawings show two possible options for the proposed tunnel, and Figure 1 shows the widest of the possible tunnel protection zones. Figure 3 is an indicative section showing the proposed Building D4 basement and the existing and proposed RailCorp assets.

The proposed Building D basement will involve excavation to depths ranging from about 2.3m to 2.8m.

We understand that the existing 11kV aerial lines will be relocated in March 2013 prior to the construction of Building D4. At the time of drilling of our boreholes (28/2/13 to 5/3/13) the lines to the west of the proposed Building D4 were present, but during our return visit to site on 11 March 2013 these lines had been removed. The relocation of the lines will occur prior to construction of the proposed Building D4 and the closest line to subject site will be about 15m from the outline of the proposed basement. Considering this offset, the limited depth of basement excavation and the nature of the aerial 11kV line, we consider that construction of the proposed Building D4 basement should not have any effect on the proposed 11kV aerial line.

The existing main western rail line corridor is located about 70m to 75m from the proposed basement and the proposed tunnel protection zone is located about 57m to 63m from the proposed basement. From the supplied drawings (Drawing No. SK-123, Section 7) the proposed tunnel will be formed within a trough that will be about 13.3m deep. Given the offset of the proposed Building D4 basement from the existing surface rail line and proposed rail tunnel, and the limited depth of the proposed basement excavation, we consider that the construction of the proposed basement should not have any effect on the existing and proposed rail lines and tunnel.

4.2 Excavation

Excavation for the proposed basement will be required to depths ranging from about 2.3m to 2.8m. Excavations to such depths will encounter fill, residual clays and weathered shale. Towards the south-eastern corner of the site we expect that soils and shale of extremely low strength will be encountered, but elsewhere shale of low to medium strength may be encountered within the base of the excavation.

Excavation of the soils will be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. The upper extremely weathered shale should also be able to be excavated using such equipment, but some assistance with rock excavation equipment may be required if higher strength shale bands are encountered. Shale of low or greater strength will require assistance with rock excavation equipment, such as hydraulic rock hammers, ripping hooks, rotary grinders or rock saws.

The use of hydraulic rock hammers would be possible for the excavation given the offset of the existing buildings from the site. The existing building to the west is likely to be demolished prior to excavation, with the only nearby building located to the east (Carriageworks), about 25m from the basement outline. However, given the heritage nature of the Carriageworks building it may be prudent to monitor the transmitted vibrations to the building during any excavation using a rock hammer to confirm that the transmitted vibrations are within acceptable limits. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations.

Where the transmitted vibrations are considered excessive it would be necessary to use alternate excavation techniques that results in much lower vibrations, such as ripping hooks, rotary grinders or rock saws.

4.3 Groundwater

The groundwater levels measured within the standpipes were at RL22.5m to RL23.2m, which is at or just above the proposed basement level of RL22.5m to RL23.1m. Therefore, allowance should be made for seepage into the excavation and this will tend to occur along the soil/rock interface or through joints within the shale. Given the subsurface profile of residual silty clays and weathered shale, the seepage that does occur should be able to be adequately controlled using conventional sump and pump techniques.

In the long term, drainage should be provided as part of the basement retaining walls and below the basement slab. The completed excavation should be inspected by the hydraulic engineer to confirm that the designed drainage system is adequate for the actual water flows. Drainage below the slab will need to be connected to fail-safe pumps to prevent basement flooding. Alternatively, the basement may be designed to resist hydrostatic uplift forces, i.e. a tanked basement.

Given the subsurface conditions of residual silty clays and weathered shale and the limited extent of the basement below the groundwater levels, we do not consider that the proposed basement will be adversely affected by groundwater provided engineer designed drainage systems are constructed. Similarly, it is not expected that the basement will have an adverse effect on the regional groundwater flows given its limited extent into the groundwater.

4.4 Retention

Given the limited depth of the proposed excavations and the space available temporary batters could be adopted, with the permanent basement retaining walls constructed at the toe of the batters and backfilled. However, since the area immediately to the east of the site is being used as a car park for the Carriageworks, and other internal roads may be constructed prior to excavation, batters may not be possible on some sides of the site. Where batters cannot be accommodated, or are not preferred, the excavations will need to be supported by full depth retention systems installed prior to the start of the excavation, such as soldier pile retaining walls, with shotcrete infill panels.

Temporary batters no more than 3.5m high, within the soils and weathered shale, should be no steeper than 1 Vertical in 1 Horizontal (1V:1H). Such batters should remain stable in the short term provided all surcharge loads, including construction loads, are kept well clear of the crest of the batters, by a distance at least equal to the batter height from the crest. Permanent batters, if required, should be no steeper than 1V:2H, but flatter batters of the order of 1V:3H may be preferred to allow access for maintenance of vegetation.

All permanent batters should be covered with topsoil and planted with a deep rooted runner grass, or other suitable coverings, following construction to reduce erosion. All stormwater run-off should be directed away from all temporary and permanent slopes to also reduce erosion.

Permanent cantilevered retaining walls may be designed based on a triangular earth pressure distribution using an active earth pressure coefficient, K_a, of 0.33 and a bulk unit weight of $20kN/m^3$, where some resulting ground movements are acceptable. Where walls are restrained from some lateral movements, such as by other structural elements in front of the wall, design should be based on an 'at rest' earth pressure coefficient, K_0, of 0.5.

Where soldier pile walls are adopted bored piers may be used and should be socket below the base of the proposed excavations, including footing and service excavations. However, allowance for some groundwater seepage should be made and the piers should be poured as soon as possible after drilling to limit seepage. Such walls may also be designed as cantilevered walls given their limited height of less than 3m to 3.5m. Alternatively, these walls could be restrained using external anchors, if ground movements are to be kept low.

Propped or anchored retaining walls may be designed based on a trapezoidal earth pressure distribution of 6H kPa, where H is the retained height in metres. This assumes that adjacent structures and movement sensitive services are located beyond a horizontal distance of 2H from the wall, which we expect will be the case. If prior to construction structures and services are located within 2H of the wall a higher earth pressure of 8H kPa should be used. These maximum pressures should be kept constant for the central 50% of the distribution.

The above coefficients and pressures assume horizontal backfill surfaces and where inclined backfill is proposed the coefficients or pressures should be increased or the inclined backfill taken as a surcharge load. All surcharge loads, e.g. sloping backfill, traffic loads, etc, should be allowed for in the design, plus appropriate hydrostatic pressures, unless measures are taken to provide complete and permanent drainage behind the walls. The groundwater levels measured within the standpipes may be used to assess the design hydrostatic loads and further measurements could be made at the time of design to provide additional information. However, we recommend that the design be based on groundwater levels at least 0.5m higher than those measured within the standpipes.

Anchors may be provisionally designed based on an allowable bond stress of 100kPa within shale of extremely low strength, 150kPa within shale of very low strength or 250kPa within shale of low or higher strength. Anchors should have a free length of at least 3m and a minimum bond length of 3m formed beyond a line drawn up at 45° from the base of the bulk excavation level. All anchors should be proof loaded to at least 1.3 times their design working load before locking off at about 85% of the design load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to confirm that the anchors are holding their load. Design and construct packages are generally preferred for ground anchors to balance the risk of efficient design against the possibility of anchor failure and the difficulty of determining whether failure is due to poor construction or optimistic design parameters.

Where piles extend below the base of the excavation a lateral resistance of the pile toes of 100kPa within shale of extremely low to very low strength, 150kPa within shale of very low strength, of 250kPa within shale of low or higher strength may be used below the base of the all excavations, including footing and service excavations. We recommend that the passive resistance be ignored for a depth of at least 0.5m below the base of the excavation.

Where batters are used, the space between the batters and the permanent retaining walls will need to be carefully backfilled to reduce future settlement of the backfill. Only light compaction equipment should be used for compaction behind retaining walls so that excessive lateral pressures are not placed on the walls. This will require the backfill to be placed in thin layers, say 100mm loose thickness, appropriate to the compaction equipment being used. The excavated clay and shale will be difficult to properly compact within the limited space available behind the walls and consideration should be given to the use of more readily compactable materials, such as ripped or crushed rock or concrete (with a maximum particle size of no more than about 40mm). The compaction specification for the backfill will depend on whether paving or structures are to be supported on the fill. If the fill is to support paved areas it should be compacted to a density of at least 98% of Standard Maximum Dry Density (SMDD) for granular fill materials, but if it is only to support landscaped areas a lower compaction specification, say 95% of SMDD, may be appropriate, provided the risk of future settlement and maintenance can be accepted. An alternative, and our preferred option, for backfill would be to use a uniformly graded granular material, such as crushed concrete of 30mm to 70mm in size, surrounded in a geofabric, with a clay capping layer to reduce surface water infiltration.

4.5 Footings

Based on the subsurface conditions encountered, the site would be classified as Class P in accordance with AS2870-2011 due to the presence of fill. However, due to the basement excavation, the building will be supported on footings founded within the shale or clayey shale, as discussed below.

Following completion of the bulk excavation, we expect that shale or clayey shale of at least extremely low strength will be encountered and therefore, the structure should be supported entirely on footings founded within the shale to provide uniform support and reduce the risk of differential movements. Pad or strip footings could be used, or bored piers in order to reach higher strength shale. However, in the south-eastern corner of the site, near BH302, piers would be several metres deep due to the more deeply weathered nature of the shale.

Footings for the proposed building may be designed based on the allowable bearing pressures given in the table below. Where piers are adopted these should be socketed at least 0.3m into the appropriate quality shale.

Borehole	Depth and Approximate RL of Shale Adequate for Allowable Bearing Pressure							
	700kPa		1000kPa		1500kPa		3000kPa	
	Depth	≈RL (AHD)	Depth	≈RL (AHD)	Depth	≈RL (AHD)	Depth	≈RL (AHD)
302	1.4m	23.8m	5.3m	19.9m	7.2m	18.0m	8.4m	16.8m
501	2.4m	22.8m	2.4m	22.8m	2.4m	22.8m	4.5m	20.7m
502	1.0m	24.2m	2.3m	22.9m	3.7m	21.5m	4.8m	20.4m
503	0.8m	24.4m	1.2m	24.0m	2.0m	23.2m	2.7m	22.5m
504	0.9m	24.2m	0.9m	24.2m	2.0m	23.1m	3.5m	21.6m

Allowable adhesions of the rock sockets equivalent to 10% of the above allowable bearing pressures, may be used for design of piles in compression, below the 0.3m nominal socket and provided socket cleanliness and roughness are maintained.

If any of the above ground portions of the building extend past the footprint of the basement, these should be supported on piles founded within the shale below the zone of influence of the basement. This zone of influence may be taken as a line drawn at 1V:1H up from the base the excavations.

At least the initial stages of footing excavation and/or pile drilling 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. Where an allowable bearing pressure of 3000kPa is adopted, we recommend that all pile drilling be inspected by a geotechnical engineer due to the variability in the depth of such shale.

Allowance should be made for groundwater seepage into the footing excavations and bored piers. Any seepage that does occur must be pumped out and any water softened material removed immediately prior to the placement of concrete. In this regard, the footing excavations and piers should be poured as soon as possible following drilling, cleaning and inspection to reduce the risk of water seepage and base softening, but at least on the same day as excavation/drilling.

Based on the subsurface conditions encountered, the site would be classified for earthquake design as Class C_e in accordance with Section 4 of AS1170.4-2007.

4.6 Basement Floor Slabs

The basement slab will be cast on weathered shale or clayey shale. The exposed subgrade should be inspected by a geotechnical engineer, who may require proof rolling of the subgrade if soil areas are exposed (possibly in the south-eastern corner) or softening occurs. If any weak subgrade areas are exposed they should be treated as recommended by the geotechnical engineer, which may comprise excavation to a sound base and replacement with engineered fill. We do not expect that significant weak subgrade areas will be encountered based on the expected subgrade conditions. However, there is a likelihood that the subgrade will be weakened by water seepage into the excavation and we recommend that a working platform be included in the specification, which should be placed as soon as possible following excavation.

As discussed in Section 4.3 above, drainage will be required below the basement slab, but the final extent of such drainage should be assessed following inspection of the completed excavation. Alternatively, the basement could be designed to resist hydrostatic uplift forces, i.e. a tanked basement.

The basement slab should have a granular subbase layer of at least 100mm thickness below the concrete to separate the concrete from the weathered shale subgrade. The extent to which the working platform will fulfil this function will depend upon the thickness of this layer and the extent to which it is damaged by construction activities. We suggest allowance be made to ‘top up’ the working platform with at least 100mm of clean material. To assist with drainage, a single sized gravel could be used as this granular layer to act as both a drainage and separation layer. The concrete slabs should be designed with an effective shear transmission at all joints by way of either keyed or dowelled joints.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. 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.

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. If the natural soil has been stockpiled, classification of this soil as Excavated Natural Material (ENM) can also be undertaken, if requested. However, the criteria for ENM are more stringent and the cost associated with attempting to meet these criteria may be significant. 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.

If there is any change in the proposed development described in this report then all recommendations should be reviewed.

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, BC 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

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

Client:	JK Geotechnics	**Ref No:**	26366SB
Project:	Proposed Building D4 Affordable Housing	**Report:**	A
Location:	North Eveleigh Precinct, Off Wilson Street, Eveleigh, NSW	**Report Date:**	11/03/2013
		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 %
501	1.50-1.95	17.7	49	19	30	13.5
501	3.80-4.20	8.0				
502	2.30-2.50	9.7				
502	7.30-7.50	3.3				
503	2.00-2.30	6.1				
504	1.20-1.50	13.8				
504	2.70-3.00	8.2				

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: 05/03/2013

115 Wicks Road
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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	26366SB
Project:	Proposed Building D4 Affordable Housing	**Report:**	B
		Report Date:	6/03/2013
Location:	North Eveleigh Precinct, Off Wilson Street, Eveleigh, NSW	**Page 1 of 1**	

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
501	4.78-4.82	0.5	10
	5.21-5.24	0.3	6
	5.59-5.61	0.5	10
	5.96-6.00	0.4	8
	6.21-6.25	0.5	10
	6.63-6.67	0.1	2
	6.88-6.91	0.4	8
503	3.22-3.26	0.6	12
	3.64-3.68	0.8	16
	4.22-4.24	0.8	16
	4.97-5.00	0.7	14
	5.22-5.26	0.6	12
	5.59-5.63	1.0	20
	6.01-6.04	0.8	16

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RTA T223.
4. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S(50)}$

ENVIROLAB SERVICES

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
enquiries@envirolabservices.com.au
www.envirolabservices.com.au

CERTIFICATE OF ANALYSIS 86743

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

Attention: P Chuszno

Sample log in details:

Your Reference:	**26366SB, Eveleigh**
No. of samples:	3 Soils
Date samples received / completed instructions received	05/03/13 / 05/03/13

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:	12/03/13 / 11/03/13
Date of Preliminary Report:	Not issued

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

Results Approved By:

Nick Sarlamis
Inorganics Supervisor

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

Miscellaneous Inorg - soil				
Our Reference:	UNITS	86743-1	86743-2	86743-3
Your Reference	-------------	BH501	BH502	BH503
Depth	------------	2.8-3.4	1.5-1.6	1.0-1.18
Date Sampled		04/03/2013	28/02/2013	28/02/2013
Type of sample		Soil	Soil	Soil
Date prepared	-	09/03/2013	09/03/2013	09/03/2013
Date analysed	-	09/03/2013	09/03/2013	09/03/2013
pH 1:5 soil:water	pH Units	5.3	5.2	4.9
Chloride, Cl 1:5 soil:water	mg/kg	30	130	5
Sulphate, SO4 1:5 soil:water	mg/kg	110	130	42

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA 22nd ED, 4500-H+.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA 22nd ED, 4110 -B.

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	PQL	METHOD	Blank	Duplicate Sm#	Duplicate results Base II Duplicate II %RPD	Spike Sm#	Spike % Recovery
Date prepared	-			09/03/2013	86743-1	09/03/2013 || 09/03/2013	LCS-1	09/03/2013
Date analysed	-			09/03/2013	86743-1	09/03/2013 || 09/03/2013	LCS-1	09/03/2013
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	86743-1	5.3 || 5.3 || RPD: 0	LCS-1	101%
Chloride, Cl 1:5 soil:water	mg/kg	2	Inorg-081	<2	86743-1	30 || 26 || RPD: 14	LCS-1	118%
Sulphate, SO4 1:5 soil:water	mg/kg	2	Inorg-081	<2	86743-1	110 || 110 || RPD: 0	LCS-1	120%

QUALITY CONTROL Miscellaneous Inorg - soil	UNITS	Dup. Sm#	Duplicate Base + Duplicate + %RPD	Spike Sm#	Spike % Recovery
Date prepared	-	[NT]	[NT]	86743-2	09/03/2013
Date analysed	-	[NT]	[NT]	86743-2	09/03/2013
pH 1:5 soil:water	pH Units	[NT]	[NT]	[NR]	[NR]
Chloride, Cl 1:5 soil:water	mg/kg	[NT]	[NT]	86743-2	94%
Sulphate, SO4 1:5 soil:water	mg/kg	[NT]	[NT]	86743-2	120%

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
NA: 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 batched 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 and 10-140% for SVOC and speciated phenols is acceptable.

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
302
1/3

Client: CITY WEST HOUSING PTY LTD
Project: PROPOSED BUILDING D4 AFFORDABLE HOUSING
Location: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH, NSW

Job No. 26366SB
Date: 29-5-08
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 25.2m
Datum: AHD
Logged/Checked by: T.M./D.B.

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
			0			FILL: Silty sandy gravel, fine to coarse grained angular igneous, fine to medium grained sand, grey brown.	M			
					CH	SILTY CLAY: high plasticity, light grey mottled red brown, with a trace of fine to medium grained angular ironstone gravel.	MC≈PL	(H)		
			1							
					-	CLAYEY SHALE: light grey mottled red brown, with iron indurated bands.	XW	EL		
	DB	N > 14 16,14/ 150mm END								
			2							
			3							
	DB	N > 14 10,14/ 150mm END								
						as above, but grey.				
			4							
	DB									
			5							
					-	SHALE: dark grey, with iron indurated bands, L-M strength bands and extremely weathered bands.	DW	VL-L		VERY LOW 'TC' BIT RESISTANCE
	DB		6							
			7							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
302

Client: CITY WEST HOUSING PTY LTD

Project: PROPOSED BUILDING D4 AFFORDABLE HOUSING

Location: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH, NSW

Job No. 26366SB | **Method:** SPIRAL AUGER JK300 | **R.L. Surface:** ≈ 25.2m

Date: 29-5-08 | **Datum:** AHD

Logged/Checked by: T.M./D.B.

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
						SHALE: dark grey, with iron indurated bands, L-M strength seams and extremely weathered bands.	DW	VL-L		
	DS					SHALE: dark grey, with iron indurated bands, L-M strength bands and extremely weathered bands.	SW	L		LOW RESISTANCE WITH VERY LOW BANDS
▶			8							
	DS					SHALE: dark grey, with L-M and M-H strength bands.		M		MODERATE TO HIGH RESISTANCE
			9			REFER TO CORED BOREHOLE LOG				
			10							
			11							
			12							
			13							
			14							


Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
JOB NO. 21823SB1 BH302 START CORING AT 8·93m
9
10
11
12
END OF BOREHOLE AT 12.07m

JK Geotechnics

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
302
3/3

Client: CITY WEST HOUSING PTY LTD

Project: PROPOSED BUILDING D4 AFFORDABLE HOUSING

Location: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH, NSW

Job No. 26366SB
Core Size: NMLC
R.L. Surface: ≈ 25.2m

Date: 29-5-08
Inclination: VERTICAL
Datum: AHD

Drill Type: JK300
Bearing:
Logged/Checked by: T.M./D.B.

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		8		START CORING AT 8.93m						
FULL RET-URN		9		SHALE: dark grey, with light grey laminae, bedded at 0-5°.	SW-FR	M				
		10								
		11				H				
									- J, 45-90°, Un, R	
		12		END OF BOREHOLE AT 12.07m						
		13								
		14								

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
501
1/2

Client: CITY WEST HOUSING PTY LTD
Project: PROPOSED BUILDING D4 AFFORDABLE HOUSING
Location: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH, NSW

Job No. 26366SB
Date: 4-3-13
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 25.2m
Datum: AHD
Logged/Checked by: P.C./D.B.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		CONCRETE: 220mm.t				NO OBSERVED REINFORCEMENT
			-	FILL: Gravel, medium to coarse grained, sub angular igneous, dark grey, with fine to medium grained sand and clay.	M			APPEARS POORLY COMPACTED
	N = 8 2,3,5		CL	SILTY CLAY: medium plasticity, light grey, trace of fine to medium grained angular ironstone gravel.	MC>PL	H	>600 >600 >600	
ON COMPLETION OF CORING	N = 20 4,9,11	1–2			MC<PL		>600 >600 >600	
ON COMPLETION OF AUGERING			-	SHALE: dark grey and brown, with medium strength iron indurated bands.	DW	L		LOW 'TC' BIT RESISTANCE
		4		SHALE: dark grey, trace of iron indurated bands.	DW-SW	L-M		MODERATE RESISTANCE
		5–7		REFER TO CORED BOREHOLE LOG				

SAMPLES: ES, U50, DB, DS


JK Geotechnics
CLIENT: CITY WEST HOUSING PTY LTD
PROJECT: PROPOSED BUILDING D4 AFFORDABLE HOUSING
LOCATION: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH
Kodak Color Control Patches
Kodak Gray Scale
Job No. 26366 SB BH501 START CORING AT: 4.35m
4
CORELOSS 0.15m
5
6
7
END OF BH AT 7.02

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
501
2/2

Client: CITY WEST HOUSING PTY LTD
Project: PROPOSED BUILDING D4 AFFORDABLE HOUSING
Location: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH, NSW

Job No. 26366SB | **Core Size:** NMLC | **R.L. Surface:** ≈ 25.2m
Date: 4-3-13 | **Inclination:** VERTICAL | **Datum:** AHD
Drill Type: JK500 | **Bearing:** - | **Logged/Checked by:** P.C./D.B.

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (EL VL L M H VH EH)	DEFECT SPACING (mm) (500 300 100 50 30 10)	DEFECT DETAILS – DESCRIPTION: Type, inclination, thickness, planarity, roughness, coating. Specific	General
		4		START CORING AT 4.35m						
				CORE LOSS 0.15m						
50% RET-URN		5		SHALE: dark grey, with frequent grey laminae.	SW	M			- J, 80-90°, Un, S	
		6								
		7							- J, 60°, P, S	
				END OF BOREHOLE AT 7.02m						
		8								
		9								
		10								

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
502
1/2

Client: CITY WEST HOUSING PTY LTD
Project: PROPOSED BUILDING D4 AFFORDABLE HOUSING
Location: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH, NSW

Job No. 26366SB
Date: 28-2-13
Method: SPIRAL AUGER JK350
Logged/Checked by: P.C./D.B.
R.L. Surface: ≈ 25.2m
Datum: AHD

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: Sandy gravel, medium to coarse grained, sub angular, igneous, dark grey, fine to medium grained sand.	D			GRAVEL COVER APPEARS POORLY COMPACTED
			-	CONCRETE: 100mm.t				7mm DIA. REINFORCEMENT, 70mm TOP COVER
				FILL: Silty clay, medium to high plasticity, light grey mottled red brown, trace of fine to medium grained ironstone gravel.	MC>PL			APPEARS POORLY COMPACTED
	SPT 5/50mm REFUSAL	1	-	CLAYEY SHALE: light grey, with iron indurated bands.	XW	EL		
	SPT 22/100mm REFUSAL	2						
				SHALE: dark grey and brown, with iron indurated bands.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
		3						
		4				L-M		LOW TO MODERATE RESISTANCE
		5		SHALE: dark grey.	SW	M		MODERATE TO HIGH RESISTANCE
		6						
		7						

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
502
2/2

Client: CITY WEST HOUSING PTY LTD
Project: PROPOSED BUILDING D4 AFFORDABLE HOUSING
Location: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH, NSW

Job No. 26366SB
Date: 28-2-13
Method: SPIRAL AUGER JK350
R.L. Surface: ≈ 25.2m
Datum: AHD
Logged/Checked by: P.C./D.B.

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
									SHALE: dark grey.	SW	M		
									END OF BOREHOLE AT 7.5m				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
503
1/2

Client: CITY WEST HOUSING PTY LTD
Project: PROPOSED BUILDING D4 AFFORDABLE HOUSING
Location: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH, NSW

Job No. 26366SB
Date: 28-2-13
Method: SPIRAL AUGER JK350
R.L. Surface: ≈ 25.2m
Datum: AHD
Logged/Checked by: P.C./D.B.

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
DRY ON COMPLETION OF AUGERING			0			CONCRETE: 200mm.t				9mm DIA. REINFORCEMENT, 100mm TOP COVER
					-	FILL: Clayey sand, fine to medium grained, dark brown, with timber fibres.				APPEARS POORLY COMPACTED
					-	CLAYEY SHALE: grey and brown.	XW	EL		
	DS	N > 7 12,7/30mm REFUSAL	1							
						SHALE: dark grey and dark brown, with iron indurated bands.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
	DS		2					L-M		LOW TO MODERATE RESISTANCE
	DS					SHALE: dark grey.	SW	M		MODERATE TO HIGH RESISTANCE
			3			REFER TO CORED BOREHOLE LOG				
			4							
			5							
			6							
			7							


JK Geotechnics
CLIENT: CITY WEST HOUSING PTY LTD
PROJECT: PROPOSED BUILDING D4 AFFORDABLE HOUSING
LOCATION: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH
Kodak Color Control Patches
Kodak Gray Scale
SCALE (CM)
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90 95 100
Job No. 26366SB BH503 START CORING AT 2.94m
3
4
CL 0.04
5
6
END OF BH AT 6.04m

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
503

Client: CITY WEST HOUSING PTY LTD
Project: PROPOSED BUILDING D4 AFFORDABLE HOUSING
Location: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH, NSW

Job No. 26366SB
Core Size: NMLC
R.L. Surface: ≈ 25.2m
Date: 28-2-13
Inclination: VERTICAL
Datum: AHD
Drill Type: JK500
Bearing: -
Logged/Checked by: P.C./D.B.

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS – DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		2		START CORING AT 2.94m						
▼ ON COMPLETION		3		SHALE: dark grey.	SW	M			- J, 30°, P, R	
		4							- XWS, 0°, 6mm.t	
FULL RETURN									- XWS, 0°, 3mm.t	
									- XWS, 0°, 5mm.t	
				CORE LOSS 0.04m					- J, 70-90°, Un, S	
		5		SHALE: dark grey.	SW	M			- J, 20-25°, Un, R	
									- J, 80-90°, Un, S	
		6		END OF BOREHOLE AT 6.04m						
		7								
		8								
		9								

JK Geotechnics
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

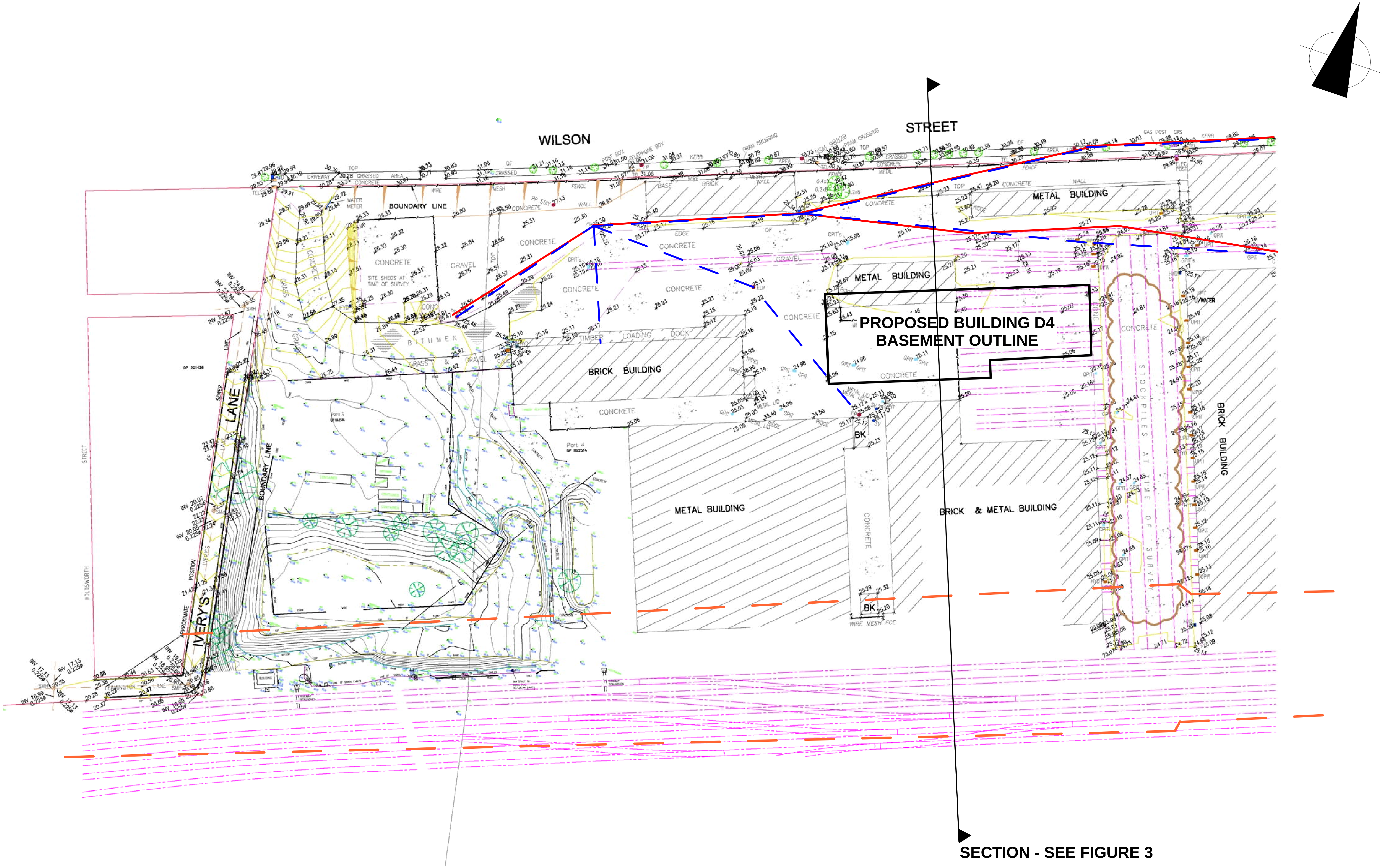
BOREHOLE LOG

Borehole No.
504
1/1

Client: CITY WEST HOUSING PTY LTD
Project: PROPOSED BUILDING D4 AFFORDABLE HOUSING
Location: NORTH EVELEIGH PRECINCT, OFF WILSON STREET, EVELEIGH, NSW

Job No. 26366SB
Date: 5-3-13
Method: SPIRAL AUGER JK300
R.L. Surface: ≈ 25.1m
Datum: AHD
Logged/Checked by: P.C./D.B.

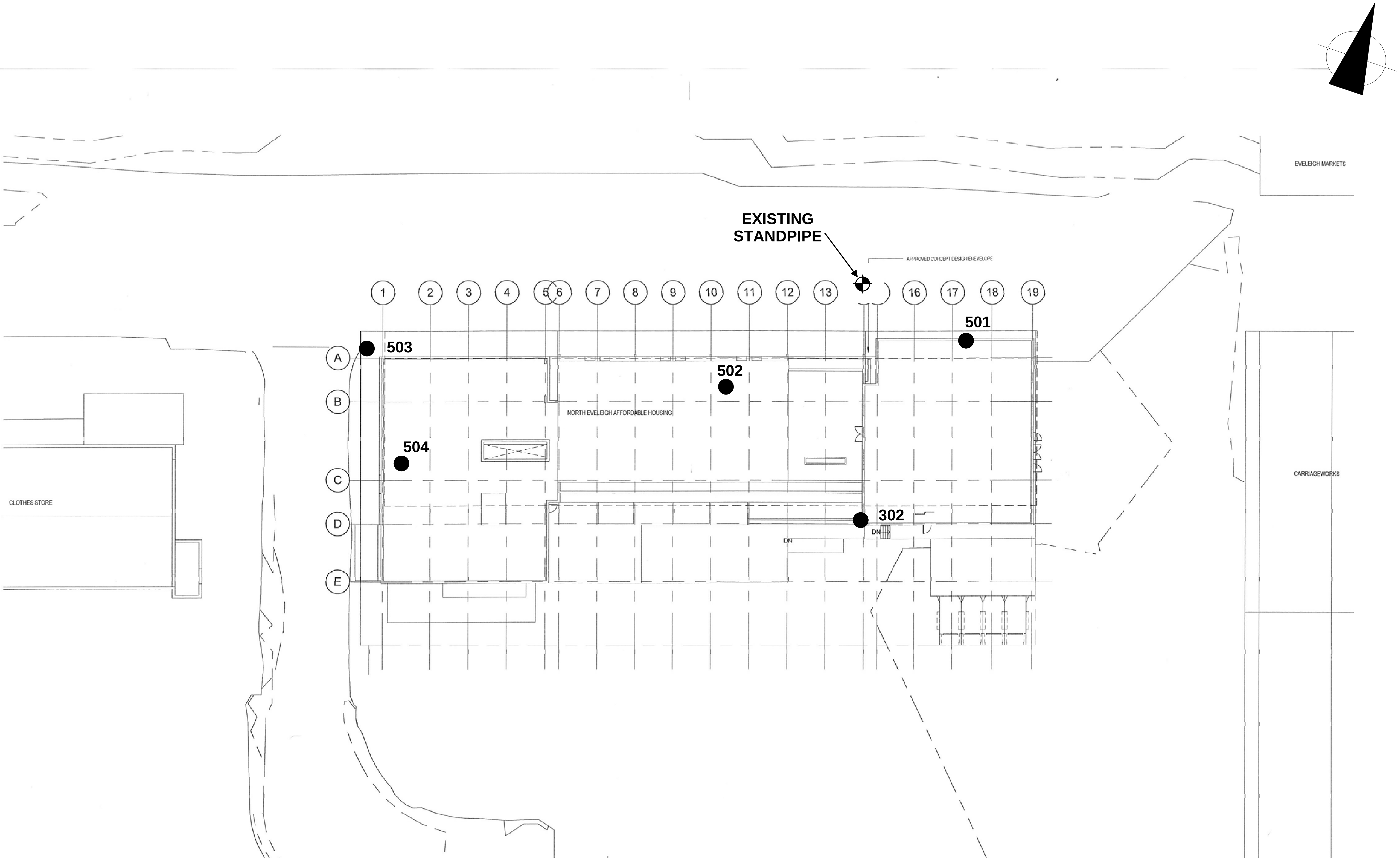
Groundwater Record	SAMPLES ES U50 DB DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			CONCRETE: 200mm.t				9mm DIA. REINFORCEMENT, 80mm & 140mm TOP COVER
		SPT 8/120mm REFUSAL			-	FILL: Clayey gravel, fine to coarse grained, sub angular, igneous and quartz, with fine to medium grained sand.	M			APPEARS POORLY COMPACTED
			1		-	SHALE: dark grey and brown, trace of iron indurated bands.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
			2			SHALE: dark grey.	DW-SW	L-M		MODERATE RESISTANCE
			3				SW	M		MODERATE TO HIGH RESISTANCE
			4							
			5							HIGH RESISTANCE
			6			END OF BOREHOLE AT 6.0m				
			7							



LEGEND

- EXISTING AERIAL 11kV LINE
- PROPOSED AERIAL 11kV LINE
- PROPOSED TUNNEL PROTECTION ZONE

Scale (m): 0 – 50	JK Geotechnics GEOTECHNICAL & ENVIRONMENTAL ENGINEERS	
Title: **PLAN SHOWING SITE LOCATION AND LOCATION OF RAILCORP ASSETS**	Report Number: 26366SB	Figure Number: 1



Scale (m): 0 20

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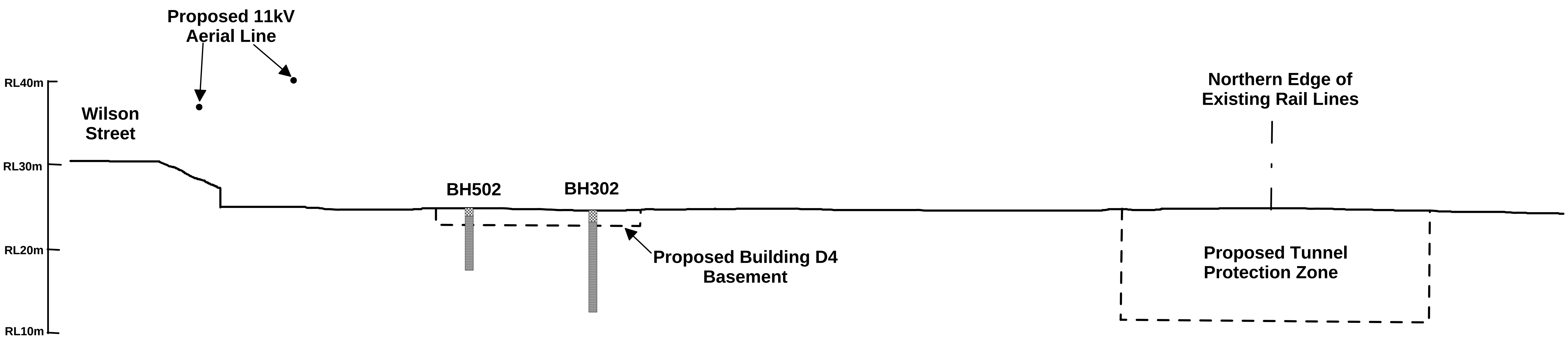
Title:	Report Number:	Figure Number:
INVESTIGATION LOCATION PLAN	26366SB	2



LEGEND

FILL

WEATHERED SHALE

Scale (m): 0 25

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Title:	Report Number:	Figure Number:
GRAPHICAL SKETCH SECTION	26366SB	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 (eg sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 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 sub-surface 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

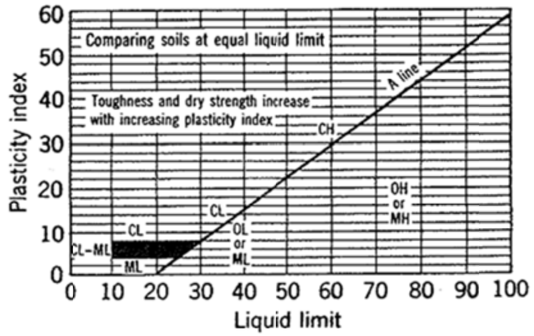
- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

Major Divisions			Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)	Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria	
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (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	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	$C_U = \frac{D_{60}}{D_{10}}$ 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*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures			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	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics		Atterberg limits above "A" line, with *PI* greater than 7
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (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*
		Sands with fines (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

Major Divisions		Identification Procedures on Fraction Smaller than 380 μm Sieve Size: Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Silts and clays liquid limit less than 50	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
	Silts and clays liquid limit greater than 50	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	
Highly Organic Soils		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		Standing water level. Time delay following completion of drilling may be shown.
	C	Extent of borehole collapse shortly after drilling.
		Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos screening.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	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	Vane shear reading in kPa of Undrained Shear Strength.
	PID = 100	Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless Soils)	D	DRY – Runs freely through fingers.
	M	MOIST – Does not run freely but no free water visible on soil surface.
	W	WET – Free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – Unconfined compressive strength less than 25kPa
	S	SOFT – Unconfined compressive strength 25-50kPa
	F	FIRM – Unconfined compressive strength 50-100kPa
	St	STIFF – Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF – Unconfined compressive strength 200-400kPa
	H	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)		**Density Index (I_D) Range (%)** — **SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose <15 — 0-4
	L	Loose 15-35 — 4-10
	MD	Medium Dense 35-65 — 10-30
	D	Dense 65-85 — 30-50
	VD	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	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T_{60}	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	