Douglas Partners
Geotechnics • Environment • Groundwater

REPORT
on
PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED GATTON CORRECTIONAL PRECINCT
KRUGERS ROAD, SPRING CREEK

prepared for
PROJECT SERVICES
(Project Services No 48370)

DP Project 47276
9 August 2007

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1.0 INTRODUCTION

This report details the results of a preliminary geotechnical investigation for the proposed Gatton Correctional Precinct to be located off Krugers Road, Spring Creek. The work was carried out for Project Services, in consultation with Ken White Consulting Engineers (KWCE), consulting civil and structural engineers for the project.

The aim of the investigation was to provide information on the following:

- subsurface conditions including groundwater (if encountered);
- excavation conditions, earthworks and site preparation, unsuitable soils, reuse of cut for fill, and workability;
- stable temporary and permanent slope batters;
- shrink-swell movements, settlements, site reactivity/classification to AS2870-1996[1] ;
- geotechnical retaining wall design parameters;
- suitable upper level footing options and allowable bearing pressures;
- ultimate end bearing and shaft adhesion pressures for bored piles;
- subgrade California bearing ratio (CBR) values for pavement thickness design; and
- topsoil suitability.

The initial scope of work was to undertake a total of thirty-two (32) test bores for the Stage 1 and Stage 2 developments. This scope of work was reduced to eleven (11) bores for the Stage 1 development only following access restrictions within the Stage 1 development area due to dense vegetation, and restricted access into the Stage 2 area.

This investigation comprised the drilling of eleven test bores, followed by geotechnical and analytical laboratory testing, engineering analysis and reporting for the Stage 1 area only. It is understood that investigation and reporting for Stage 2 will be undertaken at a later date and will be reported under separate cover.

An investigation bore was undertaken to identify the presence or otherwise of a suitable water supply at the site. The results of this work are also reported under separate cover.

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[1] *Australian Standard AS 2870 – 1996 “Residential Slabs and Footings – Construction”, Standards Association of Australia.*

2.0 SITE DESCRIPTION

The proposed Stage 1 development site is located south and west of Krugers Road, Spring Creek, approximately 12 km north of Gatton, along the Gatton-Esk Road. The site has a plan area of approximately 259 hectares and comprises Lot 240 on Plan CA31519. The final location of the proposed Stage 1 facility has not yet been fixed, and will be determined by the results of this preliminary investigation and earthworks cut and fill volumes undertaken by others.

The site is bounded by rural and forestry land on all boundaries and Krugers Road to the north-east, and is generally covered by sparse to dense timber, with localised areas of very dense timber and low grass or shrub vegetation. Some areas of surficial sandy and gravelly soils were observed throughout the site. An unsealed access track enters the site from Krugers Road and many other unsealed tracks are located throughout the site including a forestry/fire track which runs in an approximate east-west direction. Several more rutted tracks were observed to exit the main unsealed track at several locations and traverse the site. It is understood that these tracks have been formed by personnel leasing the site for logging purposes.

The topography around the site tends to slope gently down towards the south-east away from the ridge line to the west. Site levels range from RL 166m AHD at the western boundary of the Stage 1 development site to RL 123m AHD at the south-eastern boundary of the site. The ground surface slopes gently away to the east and south-east at approximately 2 degrees.

Some dry watercourses (re-entrants between localised spur lines) were observed to slope towards the south-east through the centre of the Stage 1 site and through the Stage 2 site. Localised height variations of up to 2m were observed between the base of the watercourse and the adjacent bank with near vertical to 1H:1V batter slopes.

3.0 REGIONAL GEOLOGY

The Geological Survey of Queensland's 1:100,000 Series 'Esk' Geological Sheet indicates that the site is underlain by the Triassic to Jurassic age Helidon Sandstone from the Bundamba Group.

The Helidon Sandstone is indicated as typically comprising *"quartzose sandstone, minor conglomerate, shale and siltstone".*

The residual sandy soils and weathered sandstone encountered during the field work (refer Section 5.0 below) are considered typical of the Helidon Sandstone.

4.0 FIELD WORK METHODS

The field work was undertaken between 12 June 2007 and 4 July 2007 and comprised the drilling of eleven bores (designated Bores 1, 8 to 14 and 33 to 35), as instructed by the client.

Bores 9, 10 and 13 were drilled using a trailer-mounted ID3300 drill rig equipped with continuous flight augers and a tungsten carbide (TC) drill bit. The bores were drilled to depths of between 4.62m and 4.8m with standard penetration tests (SPTs) generally undertaken at 1.5m depth intervals to assess soil type and strength.

The remaining bores were drilled using a truck-mounted EVH 210 drill rig equipped with 100mm diameter continuous spiral augers, followed by rotary washbore and NMLC coring techniques to recover rock core samples. SPTs were generally undertaken at 1.5m depth intervals to assess the relative density/strength consistency of the residual soils and weathered rock. Point load strength tests were carried out in the field on the rock core samples to assess rock strength. The bores were drilled to maximum depths of between 10.05m and 20.08m.

The test bore locations were initially set out by Douglas Partners' (DP) personnel using a hand-held GPS. After clearing of an access track along the northern boundary and relocating some of the bore locations due to accessibility problems, the actual test locations were recorded using a hand-held Garmin GPS (accurate to ±5m) in AGD 1994 co-ordinates and are given on the test bore report sheets. The test locations are indicated on Drawing 1 attached. Bore surface levels were provided by KWCE and are also recorded on the bore report sheets to AHD. Sections A-A and B-B have been drawn through the northern and southern areas of the Stage 1 site, and are presented as attached Drawings 2 and 3.

All field work was undertaken in the presence of a geotechnical engineer from DP who logged the bores, collected samples for visual and tactile assessment, and for laboratory testing.

5.0 FIELD WORK RESULTS

The subsurface conditions observed in the bores are described in detail on the test report sheets in Appendix A. Notes defining the classification methods and descriptive terms used are also included in Appendix A. Photographs of recovered core from each of the cored test bores are presented in Appendix A behind the relevant bore.

In summary, the subsurface conditions were generally similar at all test locations and comprised **residual soils** underlain by **weathered rock**. The subsurface conditions encountered are further described below:

- **Residual Soils:** Residual soils were encountered from the ground surface at all locations and generally comprised loose, grading medium dense, silty or gravelly sand overlying localised areas of very stiff, grading hard, sandy clay. The soils were generally brown to orange-brown in colour and were encountered to depths of between 1m and 4.76m.

- **Weathered Rock:** Sandstone was encountered beneath the residual soil at all locations to the termination of the bores. It was generally extremely to highly weathered, grading moderately weathered at depth, and initially extremely low strength to very low strength, grading low to medium and high strength at depth. The sandstone was brown to orange-brown in colour, fine to medium grained with some coarse grained zones.

  The rock was generally fractured (ie. defect spacing of 30mm to 100mm) in the extremely low to very low strength rock and thinly bedded. It was fractured to slightly fractured in the low to high strength rock (ie. defect spacing from 30mm to 500mm) with thin to medium bedding. The bedding was generally 5 to 15 degrees above horizontal and the jointing varied from 40 to 80 degrees above the horizontal. Some clay seams were observed in the weaker rock, whilst the partings in the stronger rock were generally planar and clean.

No free groundwater was encountered in any of the bores during auger drilling. It should be noted, however, that groundwater depths and ground moistures are affected by climatic conditions and soil permeability, and will therefore vary with time.

6.0 LABORATORY TESTING

6.1 Geotechnical Testing

Geotechnical laboratory testing comprised Emerson class dispersion tests, plasticity testing, particle size distributions, standard compaction and subgrade CBR tests. Point load strength index tests were undertaken and the results are given on the test bore report sheets. Detailed test report sheets are attached in Appendix B, and the results are summarised in the following subsections.

6.1.1 Dispersion Potential Tests

Emerson class dispersion tests were conducted on three disturbed samples, including residual soil and extremely low strength sandstone recovered from three locations across the site, to assess dispersion potential of exposed soils and weathered rock. The results are summarised in Table 1 below.

Table 1 – Summary of Erosion Potential Results

Bore No	Depth (m)	Description	Emerson Class No.
1	0.70 – 1.00	Silty sand - brown	4
1	1.50 – 1.85	Sandstone – extremely low strength	4
8	4.80 – 5.60	Sandstone – extremely low strength	4

The results indicate the residual silty sand and extremely low strength sandstone have low dispersivity under acidic conditions.

6.1.2 Classification Tests

Atterberg limits, natural moisture content and linear shrinkage tests were conducted on seven samples and particle size distribution tests conducted on six samples to assist with material classification. The results are summarised in Table 2 below.

Table 2 – Summary of Plasticity and Particle Size Distribution Test Results

Bore No	Depth (m)	Description	NMC (%)	LL (%)	PL (%)	PI (%)	LS (%)	Gravel Content (% 2mm to 60mm)	Sand Content (% 75μm to 2mm)	Silt/Clay Fines Content (% <75μm)
1	0.70 – 1.00	Silty sand with a trace of gravel	8	NP	NP	NP	NP	1	77	22
1	1.50 – 1.86	Sandstone-extremely low strength	5.2	25.3	15.3	10.1	8	30	52	18
8	4.80 – 5.60	Silty gravelly sand	2.5	25	12	13	8	13	66	21
11	0.70 – 1.00	Clayey sand	7.4	NP	NP	NP	NP	-	-	-
12	3.00 – 3.45	Clayey sand with a trace of gravel	12.6	24	12	12	8	5	56	39
14	0.20 – 1.00	Gravelly sandy clay	-	-	-	-	-	27	24	49
33	1.00 -1.45	Sandy clay with a trace of gravel	6.4	42	14	28	15.5	3	40	57
34	0.70 – 1.00	Silty sand	6.8	NP	NP	NP	NP	-	-	-

Notes: NMC = Natural Moisture Content; LL = Liquid limit; PL = Plastic Limit; PI = Plasticity Index
LS = Linear Shrinkage, NP = Non Plastic

6.1.3 Compaction and CBR Testing

Standard compaction and CBR tests were undertaken on five samples to assess their performance as pavement subgrade materials. The samples were first screened over the 19mm sieve, as required by the test standard, and were then compacted to a Standard dry density ratio of 98% at close to optimum moisture content (OMC) and soaked for four days under a 4.5kg surcharge. The results of the testing are summarised in Table 3 below.

Table 3 – Summary of Compaction and CBR test Results

Bore No	Depth (m)	Description	Standard Dry Density (t/m^3)	OMC (%)	CBR (%)
10	0.20 – 1.00	Sandy clay	1.59	23.4	7
11	0.20 – 1.00	Clayey sand	1.85	11.5	20
12	0.20 – 1.00	Silty sand	1.81	15.8	14
13	0.20 – 1.00	Gravelly sand	1.79	17.0	13
14	0.20 – 1.00	Silty sand	1.81	16.3	10

The difference in results reflects the variation in particle size and plasticity.

6.2 Topsoil Testing

Topsoil suitability testing was undertaken in accordance with the requirements of Project Services. The test results and agronomist's report are presented in Appendix C. A summary of the results are presented in Section 7.12.

7.0 COMMENTS

7.1 Proposed Development

The brief provided by Project Services indicates that the proposed development may comprise various structures up to two-storeys in height founded at different subgrade platform levels and potentially involving large volumes of cut and fill across the site. The buildings may comprise concrete framed structures with masonry or precast infill. The development may also include the construction of internal access roads and carparks.

Dependent upon the final design of the development, deep excavations may be required to facilitate visual screening of the proposed correctional facility site from the surrounding areas, to create a relatively level platform on a competent foundation, to provide a readily available source of suitable filling, and to balance cut to fill volumes.

No indication of structural loadings or structural layouts was provided prior to preparation of this preliminary investigation report.

7.2 Excavatability and Rippability Assessment

It is understood that bulk excavation of the site may be required to form a single platform level for the entire development or a number of benched platforms across the site. Site levels range from RL 66m in the west to RL 123m in the east.

An assessment of excavatability and rippability has been undertaken using the results of the point load tests (as presented on the test bore report sheets), defect spacings and joint roughness.

The results of the analyses indicate the following estimates of rippability:

- residual soil and extremely low strength to very low strength sandstone are excavatable using large hydraulic excavators (such as 25-30 tonne) or bladed using a D9N (or larger) bulldozer;
- low to medium strength rock which is thickly bedded (0.6m to 2m) and medium jointed (0.2m to 0.6m) to widely jointed (0.6m to 2m) may be excavated using a D10T (or larger) dozer with easy to medium to hard ripping;
- high strength rock which is thickly bedded (not encountered in the bores) and widely to very widely jointed (>2m) will endure very difficult ripping using a D10T dozer, and may be assisted by blasting.

Heavy rock breakers will probably be required to trim final batter slopes and the floor of any excavation where they comprise medium strength or stronger rock. Heavy rock breakers will be required for confined excavations such as trenches in medium strength or stronger rock.

Rock breakage by hydraulic rock hammers in high strength, widely jointed rock, will yield low rock hammer productivity. Hydraulic rock hammers fracture intact rock by repeated blows of the pick or moil of the hammer on the rock. The impact energy required to cause failure is proportional to:

- the strength of the rock squared (ie. with a doubling of the unconfined compressive strength, there is a four-fold reduction in productivity);
- joint spacing squared.

It should be recognised that the above excavatability estimates are based on materials encountered at test locations only and that conditions may prove more or less difficult for excavatability between and beyond these test locations.

Ground vibration and airblast over-pressure and noise will be generated as a result of blasting which will be required on parts of this site to fracture the high strength rock and allow excavation to continue. The blasting process will need to be carefully designed and controlled to minimise the impact on any structures close to the site and on any buried inground services.

Typically adverse effects and damage caused due to poorly designed and executed blasts are:

- damage of adjacent structures and the rock mass itself due to ground vibrations caused as a result of the blast;
- damage due to flyrock or boulders ejected from the blast area;
- damage due to airblast overpressure; and
- discomfort due to noise.

All of the abovementioned factors will need to be controlled to minimise the impact on any nearby structures and inground services.

7.3 Safe Batter Angles for Cutting and Filling

It is understood that extensive excavation could be undertaken at the site to achieve a uniform platform level.

Batter slopes cut into the various residual soil and rock profiles encountered during the field work may be designed for temporary and long term conditions as presented in Table 4 below:

Table 4 – Cut Batter Slopes

Material	**Safe Batter Slope (H:V)**	
	Short Term	Long Term
Stiff to hard silty clay or sandy clay, loose to medium dense clayey sand or extremely low strength rock	1.1:1	2:1(1)
Engineered clay or weathered sandstone fill	1.1:1	2:1(1)
Very low strength rock	0.75:1	1:1(2)
Low strength, fractured rock cutting	0.5:1(2)	1:1(2)
Medium strength (or better), slightly fractured rock cutting	Near vertical (2)	0.25:1(2)

Notes:

(1) Long term slopes in engineered filling or stiff to hard clay may require surface protection to reduce the risk of erosion potential (refer Section 7.4 below). It is recommended that such permanent batter slopes be limited to an average of 2H:1V, preferably with a minimum 1m wide berm at approximately 4m vertical intervals, or 3m wide berm at 7m vertical intervals.

(2) Long term cuts in very low strength or better sandstone are dependent upon the joint orientation within the rock mass. These values are contingent upon geotechnical inspection being undertaken during construction to verify that no adverse jointing is present, such as might lead to wedge or toppling failure of the rock mass, and require localised bolting or anchoring. For medium strength rock or better, intermediate berms are also recommended at approximately 7m high vertical intervals.

It is recommended that geotechnical inspection of cut faces be included during batter excavation to identify any weaker zones or potential wedge failures, which may require the use of shotcrete and mesh and rock bolts or anchors.

For both soil and rock batters, it is recommended that intermediate benches be graded to divert surface runoff away from the crests, thus allowing benches to drain sideways, and reduce the risk of small rock falls being caused by erosion. These intermediate bench drains are in addition to the use of crest drains and toe drains to remove surface water from the batters. Leading runoff into concrete lined longitudinal drains is commonly used in significant cut slopes to reduce the risk of erosion of the batters.

Where large slopes are adopted in soil or extremely low strength to very low strength rock, the use of mesh reinforced shotcrete with dowel support is recommended to protect the slope. The shotcrete will require weepholes through and strip drains behind.

Soil slopes may need to be flattened to 3H:1V or less, in order to allow vehicular access for maintenance of slopes or mowing of grass etc.

7.4 Erosion Potential of Site Soils and Weathered Rock

The results of a limited number of laboratory Emerson Class dispersivity tests (Table 1 above) on selected near surface samples of residual soil and extremely weathered rock indicate there is a low dispersion potential under acidic conditions.

It should be recognised, however, that there is a relatively high proportion of silty sands across the site. These are prone to scour under concentrated water flow conditions. It is therefore recommended that site works, including excavation and filling, be carefully planned to reduce the risk of high concentrated flows of surface water runoff or construction wastewater over steep bare slopes. This will generally require the use of sediment and erosion control measures such as surface erosion mats, mulching or vegetation on slopes; silt fences, straw bales and/or sand bags through gullies or watercourses during construction. It is recommended that the use of up-slope diversion channels, check dams and level spreaders be incorporated into the design of earthen batters, combined with a careful use of vegetation and/or shotcrete protection to reduce the risk of batter/slope erosion.

7.5 Re-Use of Cut Materials and Crushability

From the results of the test bores, it is concluded that most of the materials won from excavation of any deep cut areas will predominantly comprise an upper layer of residual sand soils and extremely low to very low strength weathered sandstone; and then low to medium strength rock generally below 3m to 5m depth with high strength rock at depth. Any fill cut from rock excavation will also consist of silty, clayey or gravelly sand and sandy clay residual soils. Dependent upon excavation depths, the rock fill may also include some localised zones of high strength rock. Such material will generally be suitable for re-use as bulk filling (ie. for platform subgrade construction). Such re-use is contingent upon particle size distribution being controlled along with moisture content, and upon minimum placement and compaction requirements being met, all as indicated in Section 7.7 below.

The crushability of such potential filling materials won from cuts at the site is dependent upon the size of material won from excavation. Several factors will affect the size of the material won from excavation including:

- the depth of tyne penetration and ripping run lengths;
- the spacing between ripping runs;
- the spacing between shot holes;
- the depth of shot holes; and
- the strength, fracture and bedding spacing of the rock.

Once the bulk excavation work methods are confirmed, trial excavations should be undertaken to determine the crushability of the excavated rock. It is anticipated that high strength rock or greater will require breakdown by rock hammer, screening and crushing, or should be put aside for slope protection measures.

7.6 Site Classification

The predominantly sandy nature and hard strength consistency of the limited zones of clay soils encountered prohibited push-tube penetration to recover ‘undisturbed’ samples for shrink-swell testing. The sample quantity recovered from the SPT was insufficient to undertake remoulded shrink-swell tests.

Atterberg limits, natural moisture content and linear shrinkage tests performed on recovered SPT samples were compared with in-house correlations to approximate shrink-swell index values for the materials tested. The approximate shrink-swell values were input into DP's in-house program *REACTIVE*, to calculate the characteristic surface movement value (y_s) in general accordance with AS 2870. It should be noted that AS 2870 provides recommended values of change in suction (Δu) and depth of suction (H_s) for major and regional centres throughout Australia. Based on published data by Fox[2], relating climatic conditions to suction, a value of 1.2 pF was adopted for Δu and 2.3m for H_s in the *REACTIVE* calculations. A cracking depth of 1.15m based on 0.5Hs was used in the analysis where no fill is placed, and a zero crack factor was used where filling is placed over existing soils.

The results of the analysis indicate that, provided 'abnormal' soil moisture conditions are not experienced, y_s values are estimated to be in the order of 10mm to 20mm for existing silty and clayey sand soils and up to say 30mm where nominally 1m of 'cut-to-fill' on site using sandy clay soil is placed over the existing soils. The predicted movements for the existing soils are consistent with a site classification of 'Class S' (slightly reactive) and 'Class M' (moderately reactive) for the clay filling example.

Reactive movements would be minimal (typically 1mm or 2mm or less) where structures and footings are founded in rock conditions where 'Class A' would apply.

If 'abnormal' soil moisture conditions are experienced at the site, the site classification would change to 'Class P' (problem site) which would require more extensive foundation works or could result in adverse foundation performance. Abnormal soil moisture conditions are defined in AS 2870 (Clause 1.3.3) and, in summary, comprise:

- Recent removal of building or structures likely to have soil moisture conditions;
- Unusual moisture caused by drains, channels, ponds, dams or tanks;
- Recent removal of large trees;
- Growth of trees too close to a structure;
- Excessive or regular watering of gardens adjacent to the structure;
- Lack of maintenance of site drainage;
- Failure to repair plumbing leaks.

The above results indicate good practice in design, construction and management of the site will be required to accommodate the potential site movements. In particular, good surface and subsurface drainage will be required, as well as limits on landscaping and adequate moisture preparation and prompt overlay sealing of clay subgrades with non-reactive granular fill (eg. CBR 15). CSIRO Guidelines (BTF18)[3] on site management for homeowners is also attached to this report (Appendix D), and provides useful advice for this site.

7.7 Subgrade Preparation and Fill Placement

It is recommended that the following site preparation be carried out for pavement subgrade and fill placement beneath slab footings using the predominantly residual silty sand and sandy clay soils and broken up sandstone rock:

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[2] *Fox E, "A Climate-Based Design Depth of Moisture Change Map of Queensland and the Use of Such Maps to Classify Sites Under AS 2870-1996"' Australian Geomechanics, Vol 35, No 4, December 2000.*

[3] *CSIRO Guidelines (BTF 18) "Foundation Maintenance and Footing Performance: A Homeowner's Guide, 2003*

(i) Bulk Earth Filling Over Low Lying Areas (Residual Soil and Extremely Low to Low Strength Rock)

- Remove any ‘uncontrolled’ or deleterious, soft, wet or highly compressible material or topsoil material rich in organics or root matter.

- Assess moisture contents of the bulk excavated sands/clays and weathered rock. For compaction of any materials other than free draining sands, the moisture content should be in the range OMC -2% (dry) to OMC +2% (wet), where OMC is the optimum moisture content at Standard compaction.

- Test roll the complete surface of the subgrade beneath the proposed embankment pad or pavement, in order to detect the presence of any soft or loose zones, which should be excavated out and replaced with approved filling. Test rolling should be carried out with a smooth drum roller with a minimum static weight of 8-tonne.

- For pavements, compact the tyned natural foundation soil to a minimum dry density ratio of 98% Standard for clay soils or a minimum density index of 75% for sands.

- For pavements, approved filling won from site, should be placed in layers not exceeding 250mm loose thickness, with each layer compacted to a minimum dry density ratio of 98% Standard or a minimum density index of 75% for filling greater than 0.5m below top of finished subgrade level. It is recommended that the final upper 0.5m of filling for the proposed platform subgrade be compacted to a minimum dry density ratio of 100% Standard or 80% density index. Where filling has a clay content, moisture content within the filling should be maintained within OMC -2% (dry) to OMC +2% (wet) during and after compaction.

- All filling beneath structures and footings should be compacted to a dry density ratio of at least 100% Standard or relative density index of at least 80%. This compaction should apply to all filling extending from a nominal horizontal distance of 2m at the edge of each structure with a nominal zone of influence of 1H:1V down and away from the proposed platform subgrade level.

- Any compacted silty or sandy clay foundation soil at or close to footing formation level should be sealed or covered as soon as practicable, to reduce the opportunity for occurrence of desiccation and cracking.

- ‘Level 1’ testing and supervision of filling, in accordance with AS 3798[4], is recommended where the filling is to be used for support of building loads, and within the 2m horizontal distance and spread from structures as outlined above.

- All weathered sandstone, won from site for re-use beneath structures and as pavement sub-grade filling, should be processed so that individual particles are no coarser than 100mm and the mean particle size is no coarser than 50mm.

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[4] *AS 3798–2007 Australian Standard “Guidelines on earthworks for commercial and residential developments”, Standards Australia*

(ii) Bulk Rock Fill Over Low Lying Areas (Medium to High Strength Rock)

For general bulk rock filling placed outside the area of influence of the various structures (refer Section 7.7(i) above), it is recommended that the following site preparation be carried out for subgrade preparation and rock fill placement:

- Remove any ‘uncontrolled’ or deleterious, soft, wet or highly compressible material or topsoil material rich in organics or root matter.
- Assess moisture contents of the bulk excavated silty and sandy clays. For compaction, the moisture content should be in the range OMC -2% (dry) to OMC +2% (wet), where OMC is the optimum moisture content at Standard compaction.
- Test roll the complete surface of the subgrade beneath the proposed embankment pad or pavement, in order to detect the presence of any soft or loose zones, which should be excavated out and replaced with approved filling. Test rolling should be carried out with a smooth drum roller with a minimum static weight of 8-tonne.
- All weathered sandstone, won from site for re-use as bulk embankment rock fill outside the structures’ zone of influence, should be processed so that individual particles are no coarser than half of the layer placement thickness and the mean particle size is no coarser than half the individual particle size.
- Approved rock filling won from site should be placed in layers not exceeding 300mm loose thickness with care taken to minimise the occurrence of voids. Fine sands and dispersive clays should not be included in the fill due to the susceptibility to erosion.

It must be recognised that it will not be possible to measure the density of the bulk rock fill layer using conventional earthworks testing equipment (ie. nuclear densometer and laboratory compaction testing) and that it will be necessary to establish a suitable roller routine so that ‘acceptable’ compaction is achieved. It follows that, where strict settlement criteria are imposed on the proposed development, there is a higher risk of settlement under bulk rock filling due to the potential of void creation during placement and due to the lack of conventional earthworks testing to confirm density levels.

(iii) Pavements Over Bulk Rock Fill

- Where pavements are proposed over bulk rock fill placed in accordance with 7.7 (ii) above, it is recommended that the rock fill be covered with a non-woven, needle punched, continuous filament polyester geofabric of sufficient strength to avoid punching failure.
- Place a minimum 0.5m thick cover of granular bridging on the geofabric in two layers of 250mm loose thickness, to provide subgrade support for the pavement. The bridging layers should be compacted to a minimum dry density ratio of 100% Standard or 80% density index.
- Granular bridging or subgrade filling should comprise ‘earth fill’ material supplied and placed in accordance with Section 7.7 (i) above.

(iv) Platform Preparation Beneath Bulk Excavation

- For rock cuts, remove all loose material from the excavated rock platform, airblast the platform surface and visually inspect prior to any platform fill placement. This is of paramount importance to verify that blasting (if undertaken) has not disturbed the rock foundation.

- Where additional filling is required to reach platform construction levels, it is recommended that filling be undertaken in accordance with the recommendations given in Section 7.7 (i) above.

The base of all soil trench excavations for any upper level strip and small pad footings should be compacted by hand-guided compaction equipment such as ‘frog’ rammer or plate vibrator, to the dry density ratio as above, in order to negate the loosening effects of the excavating equipment. Protection against desiccation and cracking should be applied, as for slab excavations. Where rock is exposed at the base of the trench excavation, this should be cleaned by air blasting to remove all loose debris prior to pouring concrete.

The above procedures will require geotechnical inspection and testing services during construction. DP is suitably qualified to conduct earthworks testing and supervision services, as well as engineering inspections of cuttings, batters and footing excavations, as may be required during the development.

It is noted that the investigation was performed during mixed weather conditions and that the near surface sand materials may cause trafficability problems for two-wheel drive vehicles after periods of rainfall or increases in moisture content.

(v) Settlement of Bulk Filling

Where bulk filling is placed under controlled conditions, there is potential for ‘creep’ of the filling material as the filling settles over time under self weight.

Potential movements for such filling are estimated as a percentage of the layer thickness, over a log cycle of time. Such settlement may be in the order of 0.1% to 0.5% of the fill thickness. This range is indicative only and may vary more extensively dependent upon the nature of the filling. Where the filling predominantly comprises granular rock materials, the lower end of the range will apply, and where the filling predominantly comprises clayey material, a higher value will apply.

It follows that the nature of the filling can only be determined after excavation and will depend upon crushability and particle size of the overburden and rock materials won from site.

Estimates of bulk filling creep settlement under self weight will vary in accordance with the depth of filling. This may lead to differential settlements where filling thicknesses are varied, such as over existing sloping ground.

The creep settlement estimates of bulk filling under self weight must be added to the settlement estimates of upper level footings in engineered filling or natural soils, and compared with settlement estimates for footings founded in rock. The designer must be aware of the potential for large differential settlements where footings in the same structure are founded in materials of differing compressibility, such as engineered bulk filling and cut rock.

7.8 Geotechnical Retaining Wall Design Parameters

At the time of the investigation, it was not known where and to what height retaining walls were proposed to be constructed. In general, however, it is recommended that all retaining structures be engineer-designed and constructed to adhere to the following loads and procedures:

- ‘At rest’ conditions (K_o) should be adopted for soil lateral pressures where rotational movement or flexing of the top of the wall is not possible or not desirable, and hence ‘active’ conditions (K_a) cannot develop.

- Depending upon the material to be retained behind the wall, the lateral pressure coefficients given in Table 5 are recommended for design.

Table 5 – Earth Pressure Coefficients (non sloping crest backfill)

Material	Unit Weight (kN/m^3)	K_o (braced structure)	K_a (cantilever structure)	Ultimate Passive Pressure (kPa)
Bulk filling comprising sandy clay/clayey sand or reworked sandstone graded and compacted as recommended in Section 7.7(i)	19 to 21	0.45	0.30	200
Very stiff or better sandy clay or loose to medium dense clayey sand	19 to 21	0.45	0.30	300
Extremely low to very low strength sandstone (1)	20 to 22	0.30	0.30	400
Low strength or better sandstone (1)	20 to 22	0.25	0.25	2000

(1) The pressure coefficients presented in Table 5 for intact rock assume a low bedding angle and no adverse jointing. Higher lateral pressures may be appropriate if potential areas of wedge or block failure are identified on-site during excavation.

- Due allowance should be made for surcharge loadings (over and above the lateral earth pressure coefficients presented above) where the finished ground level above retaining walls is above horizontal and where additional loading is likely to be applied from existing or future upslope structures, or from traffic.

- Allowance should be made for wall loading caused by flooding or inundation, as appropriate. Such flooding may penetrate up to 1.15m depth (ie. approx $0.5H_s$ as defined in AS 2870) into cracks behind the wall and will result in a triangular distribution of load near the top of the wall equal to overall depth of water times density of water.

- It is recommended that provision be made in design for build-up of hydrostatic pressure behind the wall, as described above, unless full wall height drainage is installed behind the wall.

- Drainage material behind the wall, as above, should be installed for the full height of the wall, for a width of at least 0.3m, be free draining and granular, and have a perforated or slotted drainage pipe at the heel of the wall to rapidly remove the water into the stormwater system.

- Allowable bearing pressures for retaining wall strip footings should be limited to the following:
  - 150kPa where founded in loose to medium dense clayey sand or very stiff (or stronger) clays or engineered controlled filling placed in accordance with the recommendations presented in Section 7.7 above;
  - 400kPa where founded on weathered sandstone of extremely low strength or better.

7.9 Geotechnical Design Parameters

This section provides the geotechnical parameters adopted for the calculation of allowable bearing pressures and settlement response in this report, and for use in pile lateral response design by others. The parameters are presented in Table 6 below.

It should be noted that filling materials have been assumed to comprise either 'controlled' sandy and clayey soil or weathered sandstone filling won from excavation at site, and hence would generally be partly cohesive after reworking. Furthermore, the amount of induced settlement will vary with the size and depth of the footing.

Table 6 – Geotechnical Design Parameters

Material	Undrained Shear Strength (1) (kPa)	Elastic Modulus (1) (MPa)	Bulk Density (1) (kN/m^3)
Embankment filling comprising silty or sandy clay or reworked sandstone graded and compacted as recommended in Section 7.7	100 to 150(2)	15 to 30(2)	19 to 21(2)
Very stiff residual silty clay or sandy clay	100 to 200	15 to 40	19 to 21
Extremely low to very low strength sandstone	200 to 400	50 to 75	20 to 22
Low strength sandstone	750 to 2000	100 to 500	20 to 22
Medium strength (or better) sandstone	2000 to 4000	500 to 1000	20 to 22

Notes: (1) These parameters have been estimated from SPT values, laboratory tests and published data.
(2) These parameters are based on earthworks being conducted in accordance with Section 7.7 above.

The range of parameters in Table 6 reflects the variation and localised differences encountered at the test locations. The range is provided to enable sensitivity checks to be performed.

These values are for site conditions at the time of investigation and may change if the subgrade is subjected to prolonged soaking prior to footing construction, in which case additional geotechnical advice should be sought.

7.10 Footing Options

The proposed development at the site comprises large volumes of cut and large volumes of engineered controlled replaced filling. Based on the conditions encountered in the test bores, it is considered that footing options for the development may comprise:

- upper level footings founded in controlled filling placed in accordance with the recommendations of Section 7.7;
- upper level footings founded in exposed rock at close to platform subgrade levels; or
- bored pile footings founded where competent rock is generally in excess of 2m below the platform subgrade levels (ie. in any bulk fill areas).

Where footings are founded in materials of differing compressibility, there is potential for differential settlement across the structure. This must be accounted for in design through careful articulation and choice of construction materials for use in the structure. If the structure is susceptible to differential movements, then it is recommended that all footings be founded in the same material, whether through the use of deep piled footings in rock, or a combination of upper level footings and piles founded in rock of similar compressibility.

7.10.1 Upper Level Footings

Provided that site preparation is carried out in accordance with the recommendations in Section 7.7 above, it is considered that upper level footings, founded in the engineered controlled filling (‘Level 1’ testing and supervision), may be designed using maximum allowable bearing pressures as follows:

- 150 kPa for small pad footings to 2m width or for strip footings or load support slab thickenings to 0.5m width;
- 20 kPa for slab panels.

These values are based on a factor of safety of 2.5 to 3.0 against bearing capacity failure. Footings loaded to the above maximum allowable bearing pressures are not expected to undergo settlements greater than 1% to 2% of the footing width. Hence, acceptance of these allowable bearing pressures is also contingent upon these estimated settlements being tolerable.

Maximum allowable bearing pressures for separate pad footings or strip footings founded in sandstone rock may be designed using maximum allowable pressures for the various rock strengths as follows:

- 400 kPa for extremely low to very low strength sandstone;
- 1000 kPa for low strength sandstone;
- 2000 kPa for low to medium strength sandstone; and
- 3000 kPa for medium strength sandstone.

The above allowable bearing pressures for pad or strip footings founded in rock are contingent upon successful ‘spoon testing’ in drilled holes to a minimum depth of 1.5 times the footing width.

Should the above maximum allowable bearing pressures presented above prove too low for the development loads, then piling could be considered (refer Section 7.10.2 below).

7.10.2 Bored Piles Founded in Rock

Conventional bored piles may be used should the recommended bearing pressures and associated settlements estimated in Section 7.10.1 above prove too inhibitive for upper level footings.

For design of bored piles penetrating at least 0.5m into the sandstone rock of variable strength, ultimate values may be adopted as presented in Table 7 below. It is recommended that the contribution of skin friction from the upper 1m of soil be ignored due to the potential for soil movements caused by changes in seasonal moisture content.

Table 7 – Pile Design Parameters

Material	Ultimate Unfactored Shaft Adhesion (kPa)	Ultimate Unfactored End Bearing Pressure (kPa)
Very stiff to hard silty clay or sandy clay	120	N/A
Extremely low strength sandstone	150	1500
Very low strength sandstone	200	2000
Low strength sandstone	300	3000
Low to medium strength sandstone	500	5000
Medium strength or stronger sandstone	700	7000

Where limit state methods are used to design the piles, the ultimate geotechnical strength (R_{ug}) can be taken as the unfactored ultimate shaft adhesion and unfactored ultimate end bearing values given above for the appropriate pile type and size. The R_{ug} values will need to be multiplied by a suitable geotechnical strength reduction factor (ϕ_g) to obtain the design geotechnical strength (R^*_g). Where no pile testing is carried out, and for the limited data available at this stage, a ϕ_g value of 0.45 is suggested.

Where working stress methods are used to design piles and no pile testing is carried out, the above ultimate values should be divided by a factor of safety of at least 3.0.

It is recommended that pile excavations be inspected to ensure that the above preliminary assumptions are valid and to ensure that there is no soft or loose material remaining at the base of the excavations, or smear on the side walls.

Experience indicates that bored piles founded in rock (according to the particular rock strength) and loaded to no more than the design values calculated from the ultimate unfactored values in Table 7, are unlikely to undergo settlements in excess of approximately 1% to 2% of the pile diameter.

7.11 Pavement Subgrade

The results of limited soaked CBR tests conducted on selected subgrade samples of residual sandy clay, and sandy or silty or gravelly sand, indicate CBR values of between 7% and 20%.

Based on experience with similar subgrades, it is recommended that a CBR value of 5% be adopted for subgrade materials with a high clay content (such as the sandy clay sample tested), and a CBR value of 10% adopted for predominantly granular subgrade materials in the design of either flexible sealed, unsealed granular or rigid concrete pavements, subjected to vehicular trafficking.

These values are estimated to be close to a lower bound value for these materials and are based on the assumption that the topsoil will be stripped prior to pavement construction. It is also contingent upon adequate site preparation by proof rolling (to detect any unsuitable soft or loose material) and subgrade compaction as recommended in Section 7.7 above.

Higher values may be achievable where subgrade materials comprise a high proportion of granular and rock materials won from excavation. Such values can only be determined after a representative sample comprising similar plasticity content and particle size, as proposed to be used, is subjected to additional CBR testing.

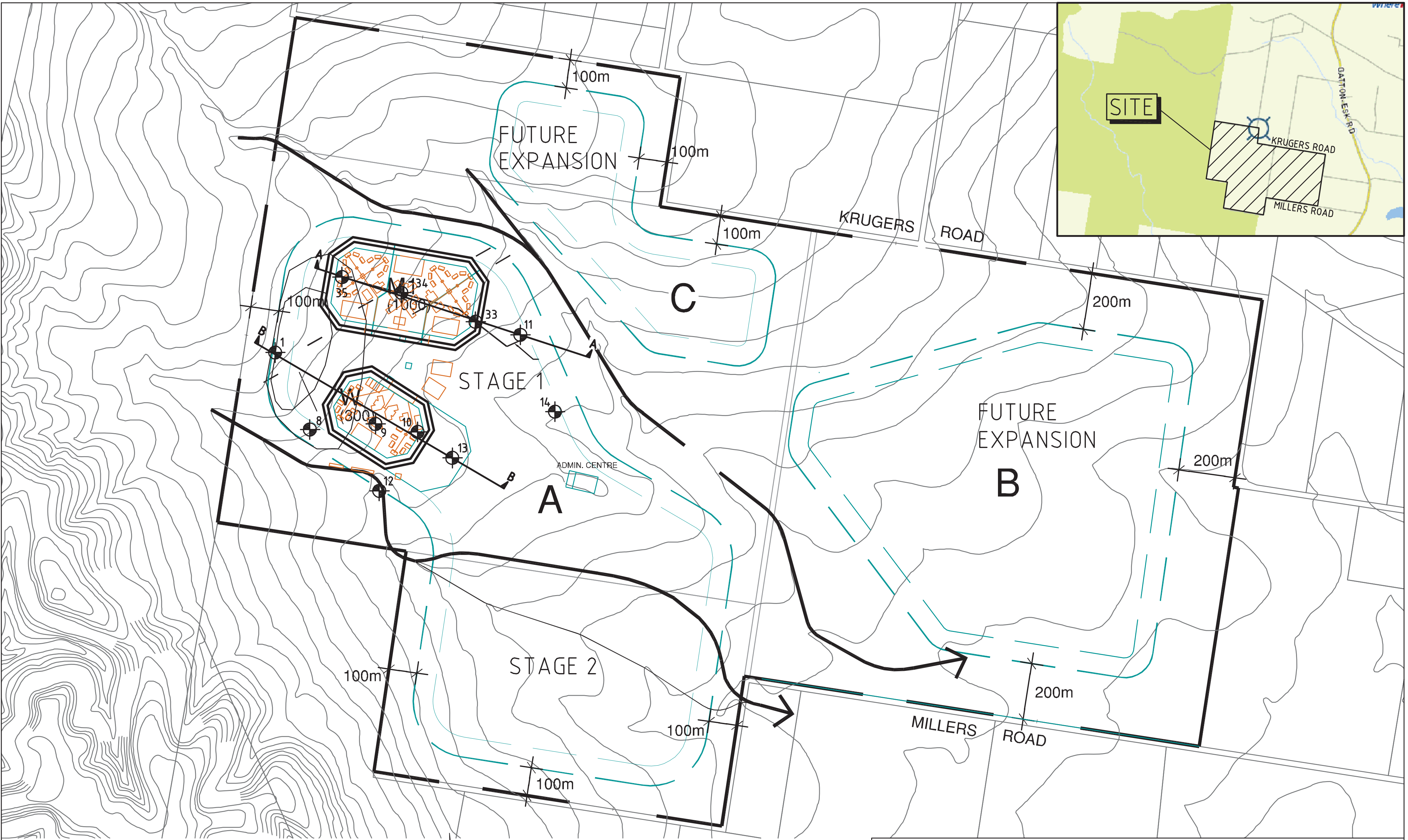
The above recommendations are based on the provision and maintenance of adequate surface and subsurface drainage.

7.12 Topsoil Suitability

The results of agronomy tests on four samples of topsoil are attached in Appendix C along with the agronomist's recommendations. In summary, the topsoils have strong acidic pH, very low nitrogen, phosphorus, potassium and trace fertility (especially Boron) and non adequate organic matter. The recommendations are to apply dolomite and slow (controlled release) fertiliser.

DOUGLAS PARTNERS PTY LTD	Reviewed by:
Chris Bell Associate Geotechnical Engineer	**Ken Boddie** Principal Geotechnical Engineer

DRAWINGS



NOTES:-

1. ADAPTED FROM DRAWING 07-1010-SK-C003 PROVIDED BY KEN WHITE CONSULTING ENGINEERS PTY LTD.
2. TEST LOCATIONS ARE APPROXIMATE ONLY AND ARE SHOWN WITH REFERENCE TO EXISTING SITE FEATURES.

LEGEND

⊕ TEST BORE LOCATION AND NUMBER

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Sydney, Newcastle,
Brisbane, Melbourne,
Perth, Darwin

Campbelltown,
Townsville, Cairns,
Wollongong, Wyong

TITLE: TEST LOCATION PLAN
Proposed Gatton Correctional Facility
Krugers Road, Spring Creek

CLIENT:	Project Services					OFFICE:	BRISBANE
DRAWN BY:	SM	SCALE:	1:12500 (A3)	PROJECT No:	47276	DRAWING No:	1
APPROVED BY:				DATE:	AUGUST 2007		

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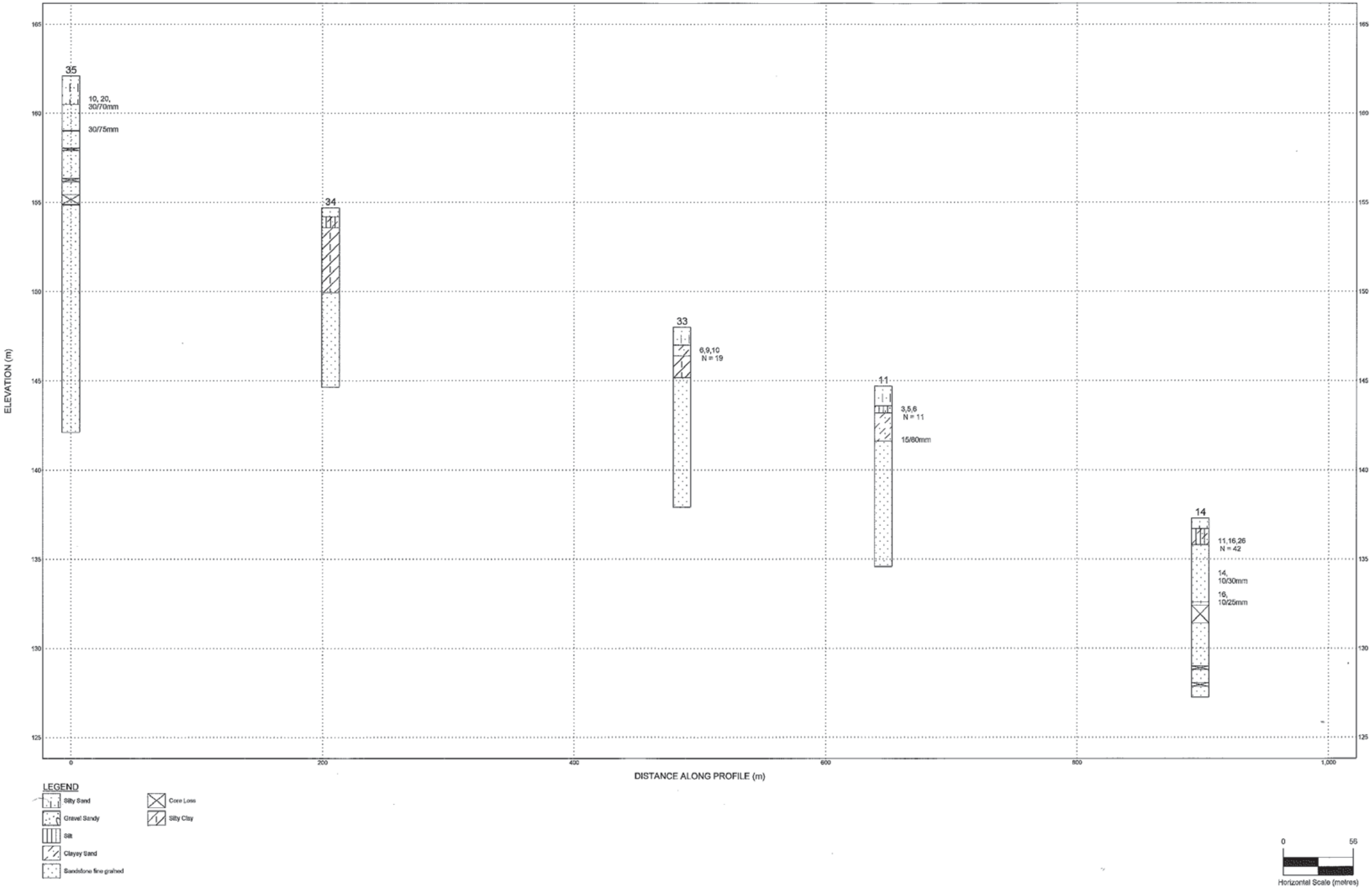

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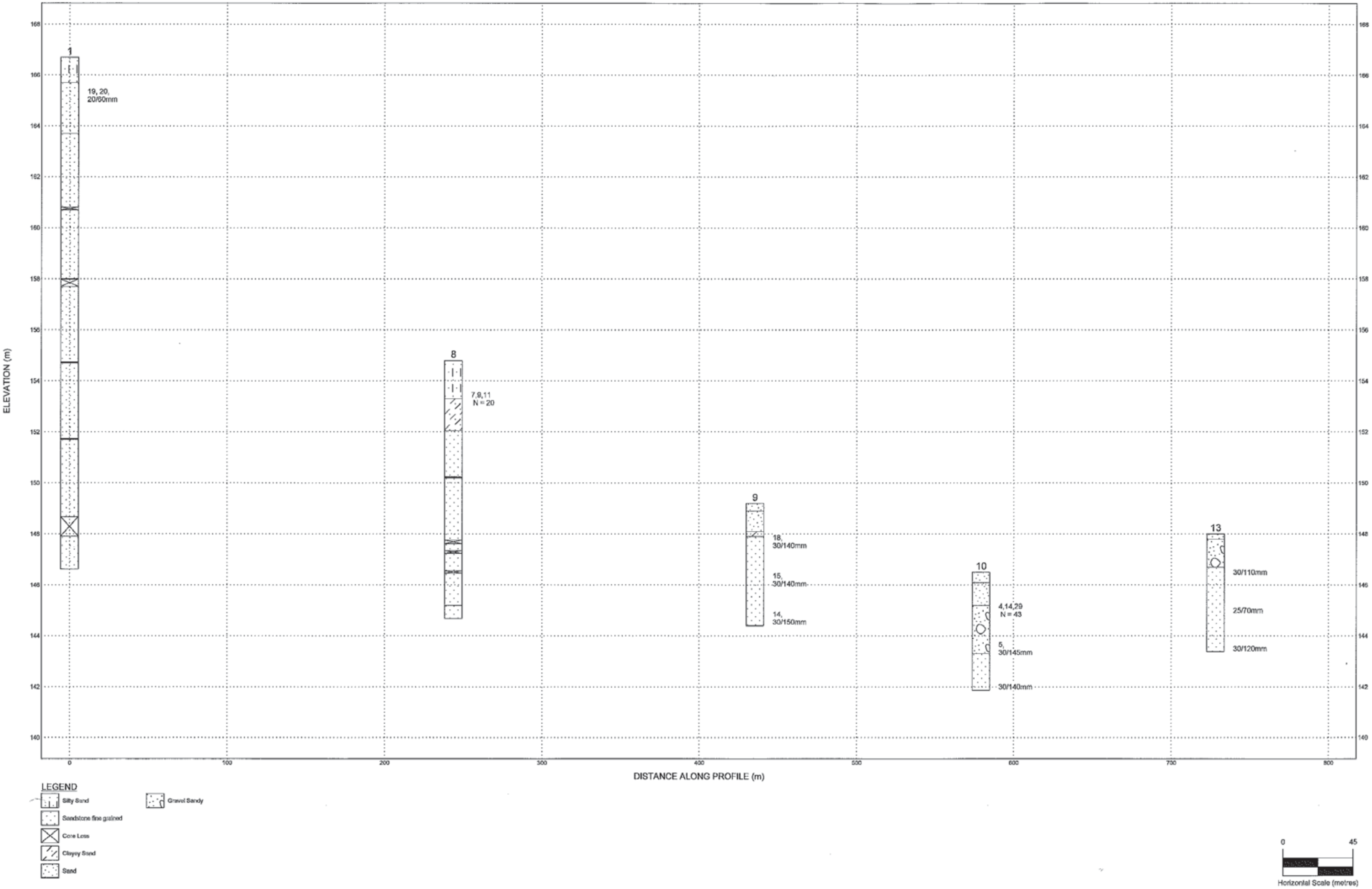
Sydney, Newcastle, Brisbane,
Melbourne, Perth, Wyong,
Campbelltown, Townsville,
Cairns, Wollongong, Darwin

TITLE: Section A-A
Proposed Gatton Corectional Precinct
Krugers Road, Spring Creek

CLIENT: Project Services
DRAWN BY:
SCALE: 1: 2788
PROJECT No: 47276
DRAWING No: 2



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Sydney, Newcastle, Brisbane,
Melbourne, Perth, Wyong,
Campbelltown, Townsville,
Cairns, Wollongong, Darwin

TITLE: Section B-B
Proposed Gatton Correctional Precinct
Krugers Road, Spring Creek

CLIENT: Project Services
DRAWN BY:
SCALE: 1: 2228
PROJECT No: 47276
DRAWING No: 3

APPENDIX A

Test Bore Report Sheets
(Nos 1, 8, 9, 10, 11, 12, 13, 14, 33, 34 and 35)

Core Photographs

Notes Relating to This Report

TEST BORE REPORT

BORE No: 1

CLIENT: Project Services
PROJECT No: 47276
DATE: 03.07.07

PROJECT: Proposed Gatton Correctional Precinct
SURFACE LEVEL: 166.7
SHEET 1 **OF** 2

LOCATION: Krugers Road, Spring Creek
DIP OF HOLE: 90°
AZIMUTH: --

Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00)	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0.0	SILTY SAND - loose grading medium dense with depth, brown silty fine to medium sand, dry - grading orange-brown									
1.0	SANDSTONE - extremely low strength, extremely weathered fine to medium sandstone, dry - grading very low strength						S			19, 20, 20/60mm
3.0	SANDSTONE - low strength, highly weathered orange-brown and grey fine to medium grained sandstone - extremely low to very low strength				3.67m: B, 5° 4.01m: B, 5° 4.22m: B, 10°, clay infill 4.3m: 4.35 & 4.4: B, 5° 5.23m: sm, clay filled <3mm 5.43m: sm, clay filled <3mm 5.51m: sm, 40mm clay fill 5.6m: 5.87: B, 5° at 20-30mm spacings		C	96	67	PL(A) = 0.22MPa PL(D) = 0.7MPa PL(A) = 0.04MPa PL(D) = 0MPa PL(A) = 0.07MPa PL(D) = 0MPa
5.89	CORE LOSS 110mm				5.89m: CORE LOSS: 110mm					
6.0	SANDSTONE - extremely low to very low strength, orange-brown and grey fine to medium grained sandstone - medium strength - extremely low strength - medium strength				6.23m: 6.78 & 6.94: B 7.22m: 7.26 & 7.3: B 7.38m: sm, 10mm clay infill 7.59m: 8.88 & 8.97: B		C	100	61	PL(D) = 0MPa PL(A) = 0.63MPa
	- low strength - extremely low to very low strength						C	82	46	PL(A) = 0.55MPa PL(D) = 0.12MPa PL(A) = 0.12MPa PL(D) = 0.02MPa
8.71	CORE LOSS 290mm				8.71m: CORE LOSS: 290mm					
9.0	SANDSTONE - very low strength, highly weathered, orange-brown and grey fine sandstone - low strength - extremely low to very low strength				9m: B at 0.1m spacings 10.2m: B at 0.3m spacings		C	98	82	PL(A) = 0.09MPa PL(D) = 0.15MPa PL(A) = 0.23MPa PL(D) = 0.31MPa PL(A) = 0.03MPa PL(D) = 0.03MPa

RIG: EVH210
DRILLER: Sutherland Exploration
LOGGED: ACS/CRB
CASING: Nil

TYPE OF BORING: Auger to 3.0m then NMLC Coring

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E430658 N6961379

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials:
Date:

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TEST BORE REPORT

BORE No: 1
CLIENT: Project Services
PROJECT No: 47276
DATE: 03.07.07
PROJECT: Proposed Gatton Correctional Precinct
SURFACE LEVEL: 166.7
SHEET 2 **OF** 2
LOCATION: Krugers Road, Spring Creek
DIP OF HOLE: 90°
AZIMUTH: --

Discontinuities: B - Bedding, J - Joint, S - Shear, D - Drill Break

Depth (m)	Description of Strata	Discontinuities	Sample Type	Core Rec. %	RQD %	Test Results & Comments
11.96	CORE LOSS 40mm	11.96m: CORE LOSS: 40mm				
12.0	SANDSTONE - as before - low strength	12m: B at 50mm spacings 12.35m: B at 0.1m spacings 12.8m: B at 0.2m spacings				PL(A) = 0.15MPa PL(D) = 0.27MPa
13	- medium strength		C	97	71	PL(A) = 0.59MPa PL(D) = 0.52MPa
14	- very low strength - low strength - medium strength					PL(D) = 0.05MPa PL(A) = 0.11MPa
14.96	CORE LOSS 40mm	14.96m: CORE LOSS: 40mm				
15.0	SANDSTONE - medium strength, moderately weathered sandstone	15.3m: B at 50mm spacings				PL(A) = 0.48MPa PL(D) = 0.44MPa
16		16m: B at 0.35m spacings	C	100	70	PL(A) = 0.32MPa PL(D) = 0.28MPa
17	- high strength, moderately weathered, light grey					PL(A) = 1.32MPa PL(D) = 0.88MPa
18.02	CORE LOSS 780mm	18m: B at 0.5m spacings 18.02m: CORE LOSS: 780mm				PL(A) = 1.13MPa PL(D) = 0.62MPa
18.8	SANDSTONE - high strength, moderately weathered sandstone		C	62	84	PL(A) = 1.01MPa PL(D) = 0.73MPa
20						PL(A) = 1.34MPa PL(D) = 0.92MPa
20.08	TEST BORE DISCONTINUED AT 20.08m					

RIG: EVH210
DRILLER: Sutherland Exploration
LOGGED: ACS/CRB
CASING: Nil

TYPE OF BORING: Auger to 3.0m then NMLC Coring

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E430658 N6961379

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials:
Date:

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Bore 1: 3.0 – 9.0m

Bore 1: 9.0 – 14.93m

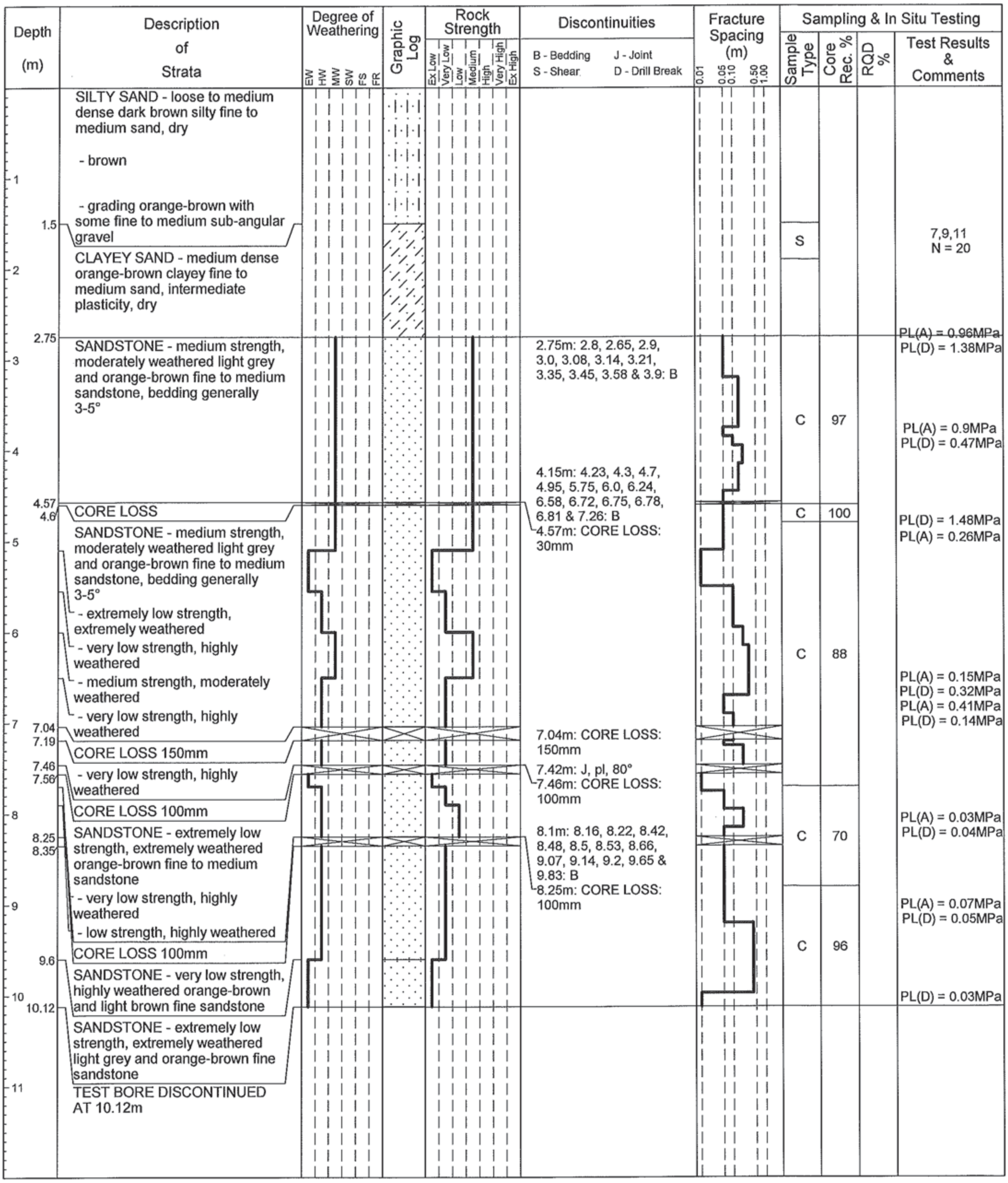
Bore 1: 14.93 – 20.08m

CORE PHOTOGRAPHS
PROPOSED GATTON CORRECTIONAL PRECINCT, SPRING CREEK

August 2007	Project 47276	Douglas Partners Geotechnics · Environment · Groundwater

TEST BORE REPORT

BORE No: 8
CLIENT: Project Services
PROJECT: Proposed Gatton Correctional Precinct
LOCATION: Krugers Road, Spring Creek
PROJECT No: 47276
SURFACE LEVEL: 154.8
DIP OF HOLE: 90°
DATE: 02.07.07
SHEET 1 **OF** 1
AZIMUTH: --



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Sample Type	Core Rec. %	RQD %	Test Results & Comments
	SILTY SAND - loose to medium dense dark brown silty fine to medium sand, dry - brown - grading orange-brown with some fine to medium sub-angular gravel					
1.5	CLAYEY SAND - medium dense orange-brown clayey fine to medium sand, intermediate plasticity, dry		S			7,9,11 N = 20
2.75	SANDSTONE - medium strength, moderately weathered light grey and orange-brown fine to medium sandstone, bedding generally 3-5°	2.75m: 2.8, 2.65, 2.9, 3.0, 3.08, 3.14, 3.21, 3.35, 3.45, 3.58 & 3.9: B 4.15m: 4.23, 4.3, 4.7, 4.95, 5.75, 6.0, 6.24, 6.58, 6.72, 6.75, 6.78, 6.81 & 7.26: B	C	97		PL(A) = 0.96MPa PL(D) = 1.38MPa PL(A) = 0.9MPa PL(D) = 0.47MPa
4.57	CORE LOSS	4.57m: CORE LOSS: 30mm	C	100		
4.6	SANDSTONE - medium strength, moderately weathered light grey and orange-brown fine to medium sandstone, bedding generally 3-5° - extremely low strength, extremely weathered - very low strength, highly weathered - medium strength, moderately weathered - very low strength, highly weathered		C	88		PL(D) = 1.48MPa PL(A) = 0.26MPa PL(A) = 0.15MPa PL(D) = 0.32MPa PL(A) = 0.41MPa PL(D) = 0.14MPa
7.04	CORE LOSS 150mm	7.04m: CORE LOSS: 150mm				
7.19	- very low strength, highly weathered	7.42m: J, pl, 80°				
7.46	CORE LOSS 100mm	7.46m: CORE LOSS: 100mm				
7.56	SANDSTONE - extremely low strength, extremely weathered orange-brown fine to medium sandstone - very low strength, highly weathered - low strength, highly weathered	8.1m: 8.16, 8.22, 8.42, 8.48, 8.5, 8.53, 8.66, 9.07, 9.14, 9.2, 9.65 & 9.83: B	C	70		PL(A) = 0.03MPa PL(D) = 0.04MPa
8.25	CORE LOSS 100mm	8.25m: CORE LOSS: 100mm				
8.35	SANDSTONE - very low strength, highly weathered orange-brown and light brown fine sandstone		C	96		PL(A) = 0.07MPa PL(D) = 0.05MPa
9.6	SANDSTONE - extremely low strength, extremely weathered light grey and orange-brown fine sandstone					PL(D) = 0.03MPa
10.12	TEST BORE DISCONTINUED AT 10.12m					

RIG: EVH210 **DRILLER:** Sutherland Exploration **LOGGED:** ACS/CRB **CASING:** Nil

TYPE OF BORING: Auger to 2.75m then NMLC Coring

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E430781 N6961106

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials:
Date:

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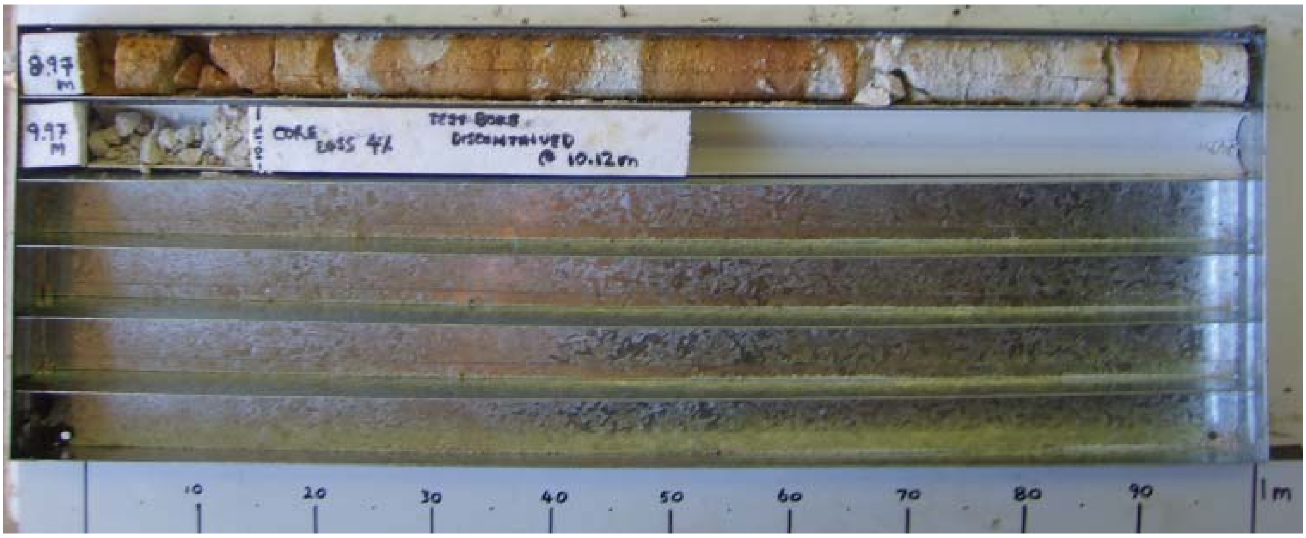
Bore 8: 2.75 – 8.97m



Bore 8: 8.97 – 10.12m

CORE PHOTOGRAPHS
PROPOSED GATTON CORRECTIONAL PRECINCT, SPRING CREEK

August 2007	Project 47276	**Douglas Partners** Geotechnics • Environment • Groundwater

TEST BORE REPORT

BORE No: 9

CLIENT: Project Services

PROJECT: Proposed Gatton Correctional Precinct

LOCATION: Krugers Road, Spring Creek

PROJECT No: 47276

SURFACE LEVEL: 149.2

DIP OF HOLE: 90°

DATE: 12.06.2007

SHEET 1 **OF** 1

AZIMUTH: --

Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00)	Sample Type	Core Rec. %	RQD %	Test Results & Comments
	SAND - loose brown fine to medium grained sand, trace of silt						A			
0.3	SAND - loose to medium dense yellow-brown fine to medium grained sand, trace of low plasticity clay									
1.1	CLAYEY SAND - medium dense to dense orange-brown clayey fine to medium grained sand									
1.3	SANDSTONE - extremely low strength, extremely weathered orange-brown and grey fine to medium sandstone, trace of clay						S			18, 30/140mm
							S			15, 30/140mm
							S			14, 30/150mm
4.8	TEST BORE DISCONTINUED AT 4.8m									

RIG: ID3300

DRILLER: All-tech

LOGGED: CRB

CASING: Nil

TYPE OF BORING: 100mm Solid Flight Auger with TC Bit

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E431014 N6961125

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED

Initials:

Date:

Douglas Partners

Geotechnics · Environment · Groundwater

TEST BORE REPORT

BORE No: 10

CLIENT: Project Services
PROJECT: Proposed Gatton Correctional Precinct
LOCATION: Krugers Road, Spring Creek

PROJECT No: 47276
SURFACE LEVEL: 146.5
DIP OF HOLE: 90°

DATE: 12.06.2007
SHEET 1 OF 1
AZIMUTH: --

Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0.2	SAND - loose brown fine to medium grained sand with a trace of fine to medium sub-angular gravel, dry									
1.3	SANDY CLAY - very stiff orange-brown fine to medium grained sandy clay with some fine to coarse sub-angular gravel, dry						B			
	GRAVELLY SAND - dense orange-brown gravelly fine to medium sand, gravel fraction is fine to medium grained sub-rounded sandstone, some clay						S			4,14,29 N = 43
3.2							S			5, 30/145mm
	SANDSTONE - extremely low strength, extremely weathered orange-brown and grey fine to medium grained sandstone with a trace of clay						S			30/140mm
4.64	TEST BORE DISCONTINUED AT 4.64m									

RIG: ID3300 **DRILLER:** All-tech **LOGGED:** CRB **CASING:** Nil

TYPE OF BORING: 100mm Solid Flight Auger

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E431164 N6961278

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials:
Date:

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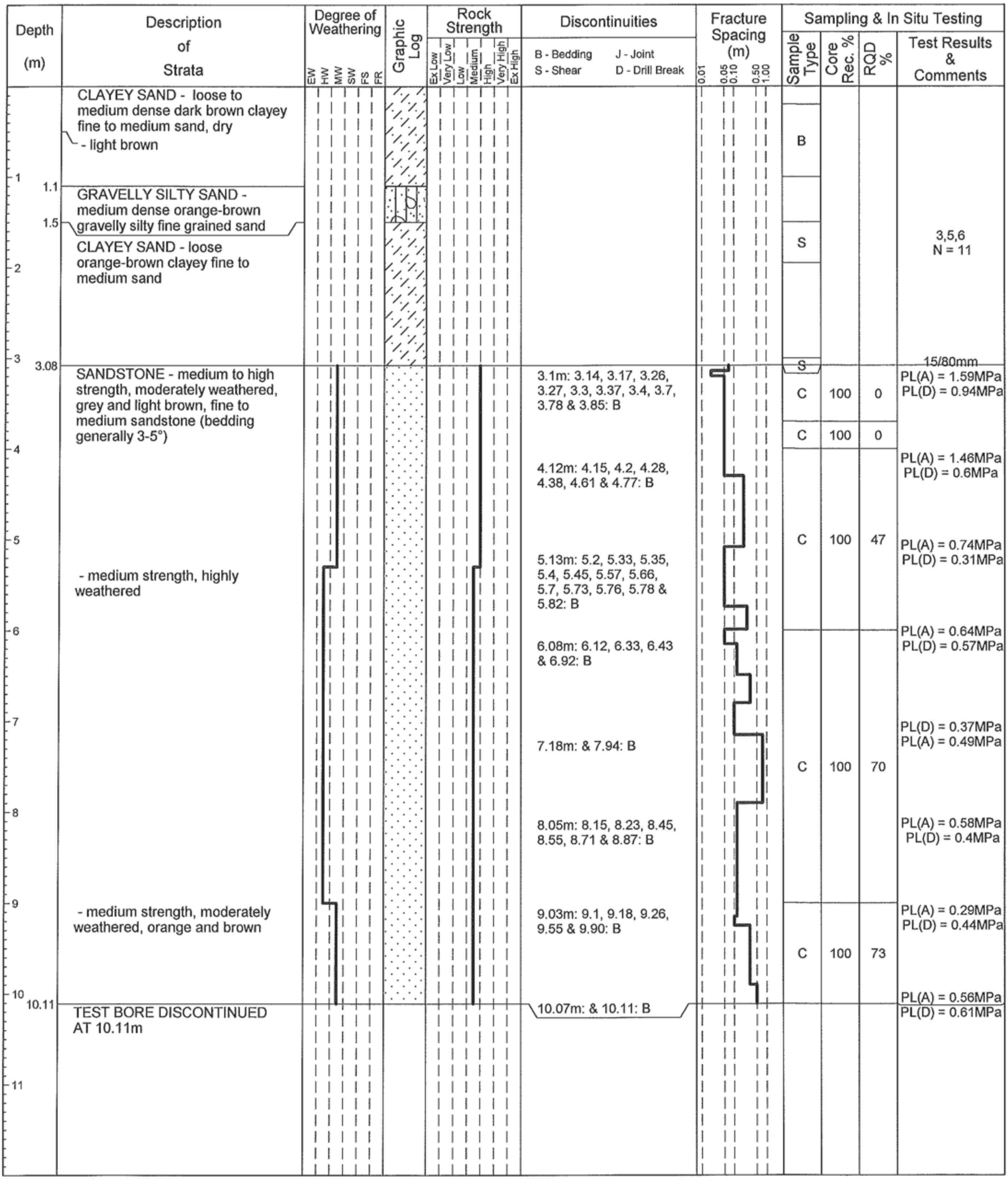
TEST BORE REPORT

BORE No: 11

CLIENT: Project Services
PROJECT: Proposed Gatton Correctional Precinct
LOCATION: Krugers Road, Spring Creek

PROJECT No: 47276
SURFACE LEVEL: 144.7
DIP OF HOLE: 90°

DATE: 25.06.07
SHEET 1 OF 1
AZIMUTH: --



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Sample Type	Core Rec. %	RQD %	Test Results & Comments
	CLAYEY SAND - loose to medium dense dark brown clayey fine to medium sand, dry - light brown		B			
1.1	GRAVELLY SILTY SAND - medium dense orange-brown gravelly silty fine grained sand					
1.5	CLAYEY SAND - loose orange-brown clayey fine to medium sand		S			3,5,6 N = 11
3.08	SANDSTONE - medium to high strength, moderately weathered, grey and light brown, fine to medium sandstone (bedding generally 3-5°)	3.1m: 3.14, 3.17, 3.26, 3.27, 3.3, 3.37, 3.4, 3.7, 3.78 & 3.85: B	S C	 100	 0	15/80mm PL(A) = 1.59MPa PL(D) = 0.94MPa
			C	100	0	
		4.12m: 4.15, 4.2, 4.28, 4.38, 4.61 & 4.77: B	C	100	47	PL(A) = 1.46MPa PL(D) = 0.6MPa PL(A) = 0.74MPa PL(D) = 0.31MPa
	- medium strength, highly weathered	5.13m: 5.2, 5.33, 5.35, 5.4, 5.45, 5.57, 5.66, 5.7, 5.73, 5.76, 5.78 & 5.82: B				
		6.08m: 6.12, 6.33, 6.43 & 6.92: B 7.18m: & 7.94: B 8.05m: 8.15, 8.23, 8.45, 8.55, 8.71 & 8.87: B	C	100	70	PL(A) = 0.64MPa PL(D) = 0.57MPa PL(D) = 0.37MPa PL(A) = 0.49MPa PL(A) = 0.58MPa PL(D) = 0.4MPa
	- medium strength, moderately weathered, orange and brown	9.03m: 9.1, 9.18, 9.26, 9.55 & 9.90: B	C	100	73	PL(A) = 0.29MPa PL(D) = 0.44MPa PL(A) = 0.56MPa
10.11	TEST BORE DISCONTINUED AT 10.11m	10.07m: & 10.11: B				PL(D) = 0.61MPa

RIG: EVH210 **DRILLER:** Sutherland Exploration **LOGGED:** ACS/CRB **CASING:** Nil

TYPE OF BORING: Auger to 3.08m then NMLC Coring

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E431529 N6961442

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials:
Date:

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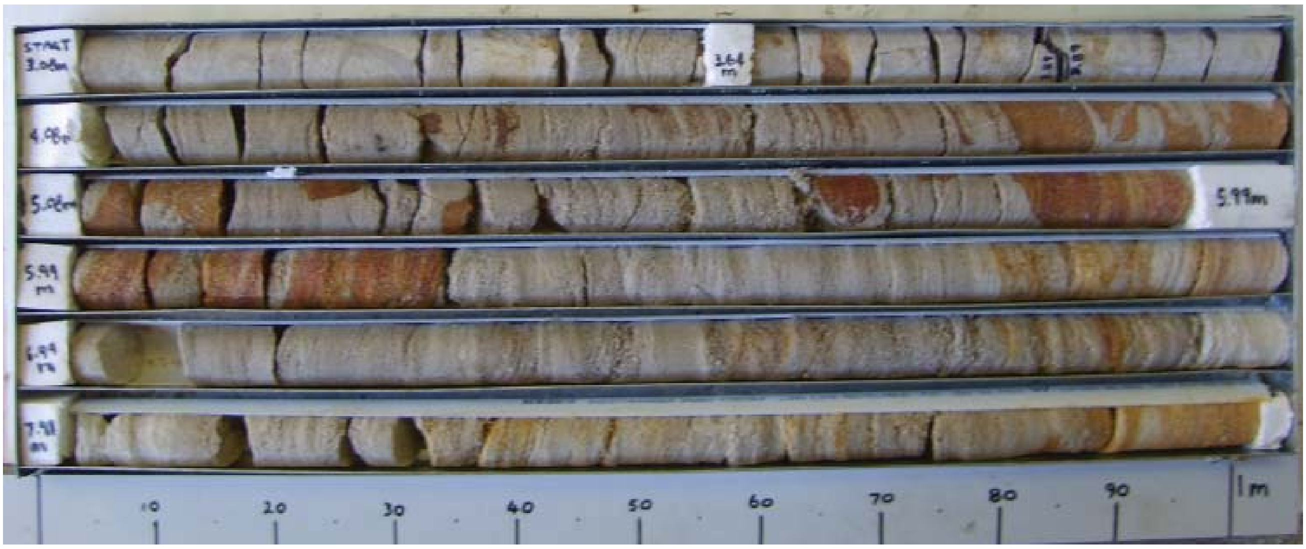
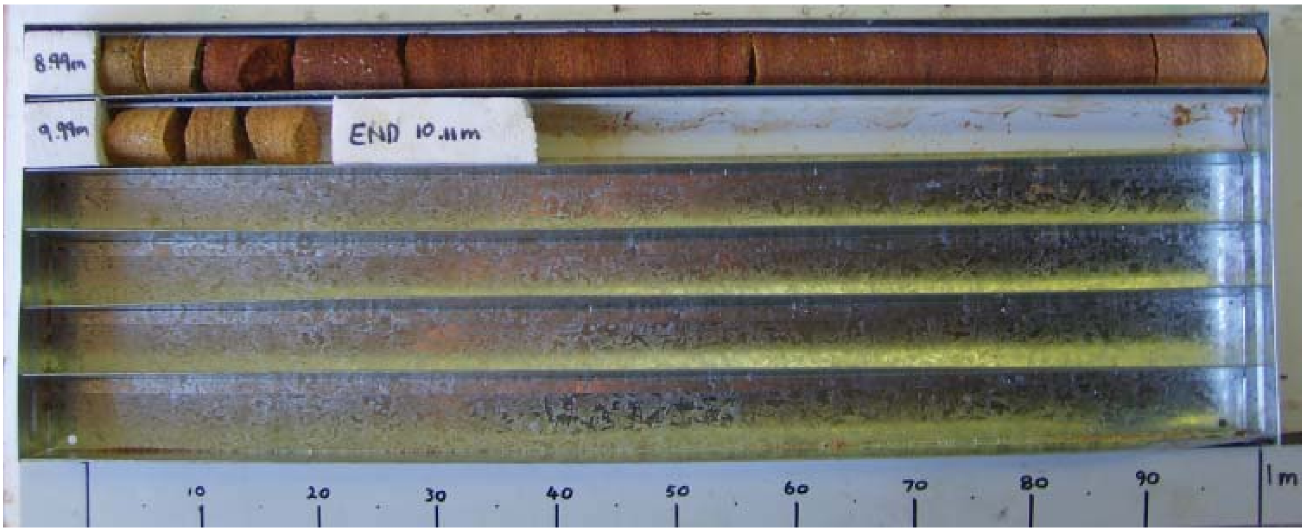


Bore 11: 3.08 – 8.99m



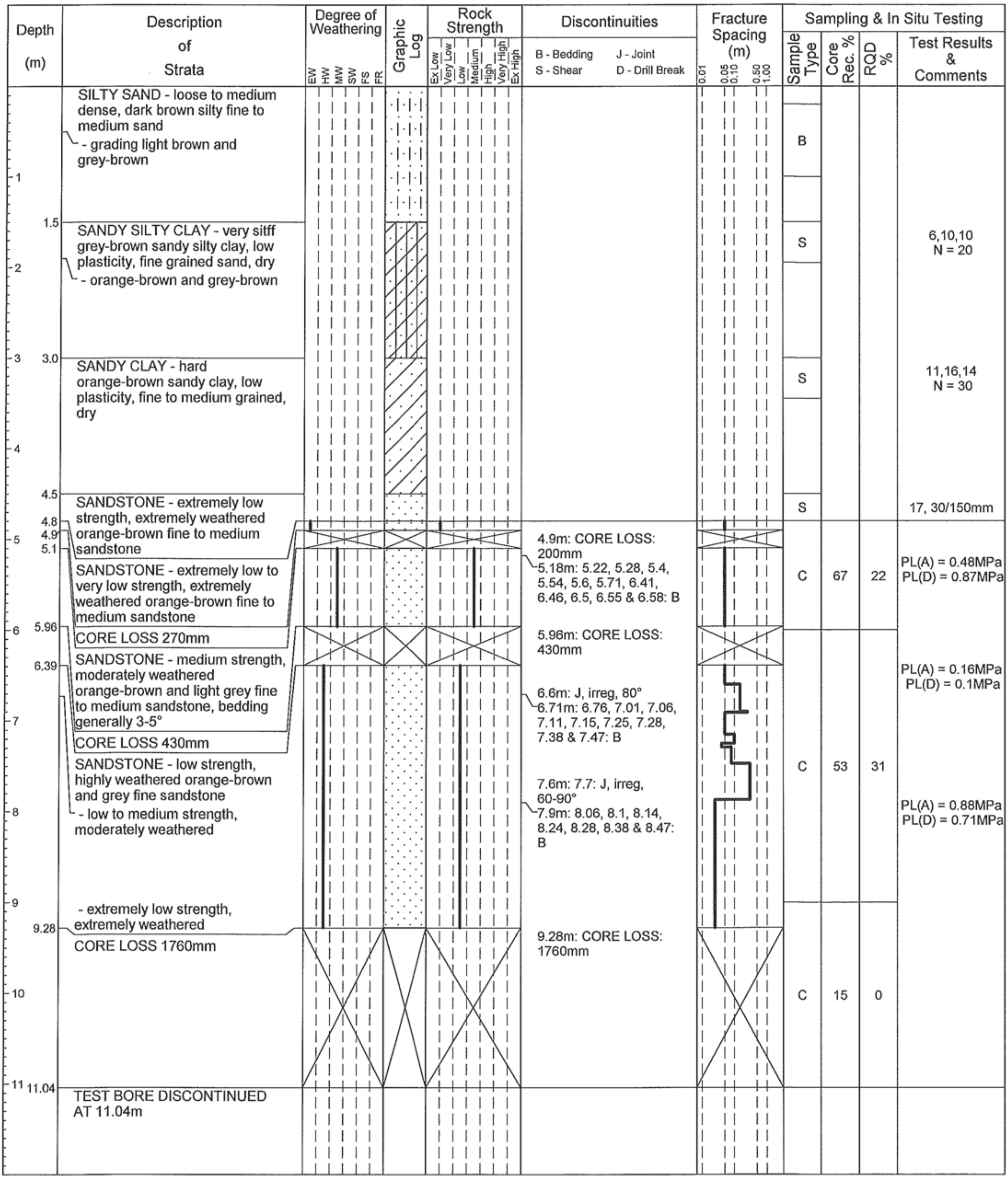
Bore 11: 8.99 – 10.11m

CORE PHOTOGRAPHS
PROPOSED GATTON CORRECTIONAL PRECINCT, SPRING CREEK

August 2007	Project 47276	Douglas Partners Geotechnics • Environment • Groundwater

TEST BORE REPORT

BORE No: 12
CLIENT: Project Services
PROJECT: Proposed Gatton Correctional Precinct
LOCATION: Krugers Road, Spring Creek
PROJECT No: 47276
SURFACE LEVEL: 144.3
DIP OF HOLE: 90°
DATE: 29.06.07
SHEET 1 **OF** 1
AZIMUTH: --



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Sample Type	Core Rec. %	RQD %	Test Results & Comments
	SILTY SAND - loose to medium dense, dark brown silty fine to medium sand - grading light brown and grey-brown		B			
1.5	SANDY SILTY CLAY - very sitff grey-brown sandy silty clay, low plasticity, fine grained sand, dry - orange-brown and grey-brown		S			6,10,10 N = 20
3.0	SANDY CLAY - hard orange-brown sandy clay, low plasticity, fine to medium grained, dry		S			11,16,14 N = 30
4.5	SANDSTONE - extremely low strength, extremely weathered orange-brown fine to medium sandstone		S			17, 30/150mm
4.8 4.9 5.1	SANDSTONE - extremely low to very low strength, extremely weathered orange-brown fine to medium sandstone	4.9m: CORE LOSS: 200mm 5.18m: 5.22, 5.28, 5.4, 5.54, 5.6, 5.71, 6.41, 6.46, 6.5, 6.55 & 6.58: B	C	67	22	PL(A) = 0.48MPa PL(D) = 0.87MPa
5.96	CORE LOSS 270mm	5.96m: CORE LOSS: 430mm				
6.39	SANDSTONE - medium strength, moderately weathered orange-brown and light grey fine to medium sandstone, bedding generally 3-5° CORE LOSS 430mm SANDSTONE - low strength, highly weathered orange-brown and grey fine sandstone - low to medium strength, moderately weathered - extremely low strength, extremely weathered	6.6m: J, irreg, 80° 6.71m: 6.76, 7.01, 7.06, 7.11, 7.15, 7.25, 7.28, 7.38 & 7.47: B 7.6m: 7.7: J, irreg, 60-90° 7.9m: 8.06, 8.1, 8.14, 8.24, 8.28, 8.38 & 8.47: B	C	53	31	PL(A) = 0.16MPa PL(D) = 0.1MPa PL(A) = 0.88MPa PL(D) = 0.71MPa
9.28	CORE LOSS 1760mm	9.28m: CORE LOSS: 1760mm	C	15	0	
11.04	TEST BORE DISCONTINUED AT 11.04m					

RIG: EVH210 **DRILLER:** Sutherland Exploration **LOGGED:** ACS/CRB **CASING:** Nil

TYPE OF BORING: Auger to 4.8m then NMLC Coring

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E431027 N6960887

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials:
Date:

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Bore 12: 4.8 – 11.04m

CORE PHOTOGRAPHS
PROPOSED GATTON CORRECTIONAL PRECINCT, SPRING CREEK

August 2007	Project 47276	**Douglas Partners** Geotechnics • Environment • Groundwater

TEST BORE REPORT

BORE No: 13
CLIENT: Project Services
PROJECT No: 47276
DATE: 12.06.2007
PROJECT: Proposed Gatton Correctional Precinct
SURFACE LEVEL: 148.0
SHEET 1 OF 1
LOCATION: Krugers Road, Spring Creek
DIP OF HOLE: 90°
AZIMUTH: --

Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Discontinuities B - Bedding, J - Joint, S - Shear, D - Drill Break	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0.2	SAND - loose brown fine to medium grained sand									
1.3	GRAVELLY SAND - loose to medium dense orange-brown gravelly fine to medium grained sand, gravel fraction is fine to medium grained and sub-angular						B			
	SANDSTONE - extremely low strength, extremely weathered orange-brown and grey fine to medium grained sandstone with a trace of clay						S			30/110mm
	- grading light grey						S			25/70mm
4.62							S			30/120mm
	TEST BORE DISCONTINUED AT 4.62m									

RIG: ID3300
DRILLER: All-tech
LOGGED: CRB
CASING: Nil

TYPE OF BORING: 100mm Solid Flight Auger

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E431287 N6961012

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials:
Date:

Douglas Partners
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TEST BORE REPORT

BORE No: 14
CLIENT: Project Services
PROJECT No: 47276
DATE: 28.06.07
PROJECT: Proposed Gatton Correctional Precinct
SURFACE LEVEL: 137.3
SHEET 1 **OF** 1
LOCATION: Krugers Road, Spring Creek
DIP OF HOLE: 90°
AZIMUTH: --

Discontinuities: B - Bedding, J - Joint, S - Shear, D - Drill Break

Depth (m)	Description of Strata	Discontinuities	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0.0	SILTY SAND - loose to medium dense brown silty fine to medium sand, dry					
0.6	CLAYEY SILTY SAND - light orange-brown clayey silty fine sand, intermediate plasticity, dry					
1.5	SANDSTONE - extremely low strength, extremely weathered orange-brown fine to medium sandstone		S			11,16,26 N = 42
	- very low strength					
			S			14, 10/30mm
	- grading grey					
			S			16, 10/25mm
4.71						
4.88	SANDSTONE - very low strength, extremely weathered grey-brown fine sandstone, bedding generally 5-10°		C	22		
	CORE LOSS 1000mm	4.88m: CORE LOSS: 1000mm				
5.88	SANDSTONE - very low strength, extremely weathered grey-brown fine sandstone, bedding generally 5-10°	6.38m: D 6.41m: D 7.56m: B 7.78m: 7.88, 7.92, 7.95 & 8.97: B	C	89		PL(D) = 0MPa PL(A) = 0.04MPa PL(D) = 0.03MPa PL(A) = 0.3MPa PL(D) = 0.07MPa PL(A) = 0MPa
8.3	CORE LOSS 200mm	8.3m: CORE LOSS: 200mm				
8.5	SANDSTONE - very low strength, highly weathered grey fine sandstone	8.85m: D	C	78		PL(A) = 0.02MPa PL(D) = 0.04MPa
9.24	CORE LOSS 200mm	9.24m: CORE LOSS: 200mm				
9.44	SANDSTONE - low strength, moderately weathered light grey-brown fine sandstone	9.5m: & 9.8: B	C	100	90	PL(A) = 0.21MPa PL(D) = 0.63MPa
10.05	TEST BORE DISCONTINUED AT 10.05m					

RIG: EVH210 **DRILLER:** Sutherland Exploration **LOGGED:** ACS/CRB **CASING:** Nil

TYPE OF BORING: Auger to 4.71m then NMLC Coring

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E431652 N6961169

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials:
Date:

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Bore 14: 4.71 – 10.05m

CORE PHOTOGRAPHS
PROPOSED GATTON CORRECTIONAL PRECINCT, SPRING CREEK

August 2007	Project 47276	**Douglas Partners** Geotechnics • Environment • Groundwater

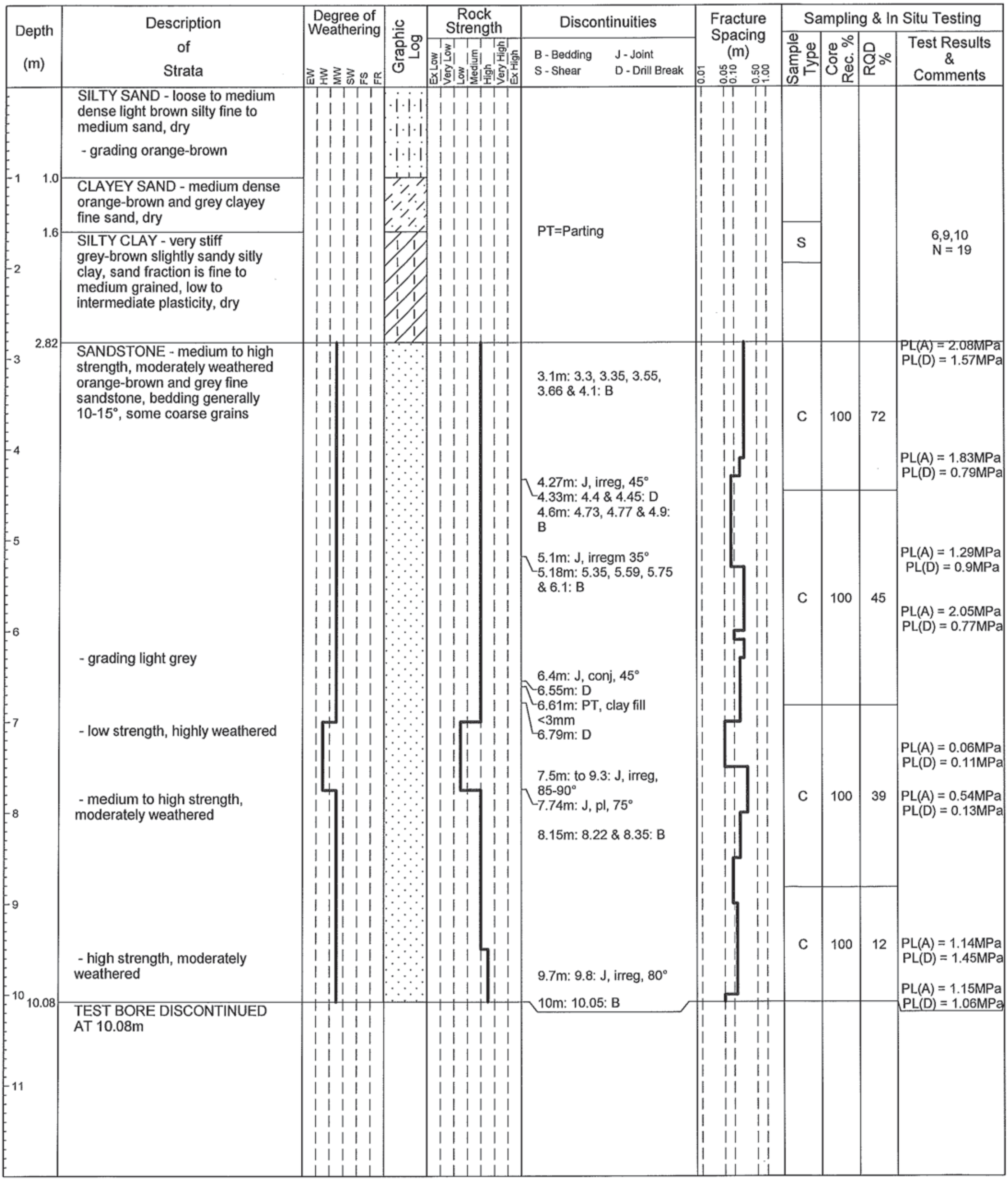
TEST BORE REPORT

BORE No: 33

CLIENT: Project Services
PROJECT: Proposed Gatton Correctional Precinct
LOCATION: Krugers Road, Spring Creek

PROJECT No: 47276
SURFACE LEVEL: 148.0
DIP OF HOLE: 90°

DATE: 26.06.07
SHEET 1 OF 1
AZIMUTH: --



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Sample Type	Core Rec. %	RQD %	Test Results & Comments
	SILTY SAND - loose to medium dense light brown silty fine to medium sand, dry - grading orange-brown					
1.0	CLAYEY SAND - medium dense orange-brown and grey clayey fine sand, dry					
1.6	SILTY CLAY - very stiff grey-brown slightly sandy sitly clay, sand fraction is fine to medium grained, low to intermediate plasticity, dry	PT=Parting	S			6,9,10 N = 19
2.82	SANDSTONE - medium to high strength, moderately weathered orange-brown and grey fine sandstone, bedding generally 10-15°, some coarse grains	3.1m: 3.3, 3.35, 3.55, 3.66 & 4.1: B	C	100	72	PL(A) = 2.08MPa PL(D) = 1.57MPa PL(A) = 1.83MPa PL(D) = 0.79MPa
		4.27m: J, irreg, 45° 4.33m: 4.4 & 4.45: D 4.6m: 4.73, 4.77 & 4.9: B 5.1m: J, irregm 35° 5.18m: 5.35, 5.59, 5.75 & 6.1: B	C	100	45	PL(A) = 1.29MPa PL(D) = 0.9MPa PL(A) = 2.05MPa PL(D) = 0.77MPa
	- grading light grey	6.4m: J, conj, 45° 6.55m: D 6.61m: PT, clay fill <3mm 6.79m: D				
	- low strength, highly weathered - medium to high strength, moderately weathered	7.5m: to 9.3: J, irreg, 85-90° 7.74m: J, pl, 75° 8.15m: 8.22 & 8.35: B	C	100	39	PL(A) = 0.06MPa PL(D) = 0.11MPa PL(A) = 0.54MPa PL(D) = 0.13MPa
	- high strength, moderately weathered	9.7m: 9.8: J, irreg, 80° 10m: 10.05: B	C	100	12	PL(A) = 1.14MPa PL(D) = 1.45MPa PL(A) = 1.15MPa PL(D) = 1.06MPa
10.08	TEST BORE DISCONTINUED AT 10.08m					

RIG: EVH210 **DRILLER:** Sutherland Exploration **LOGGED:** ACS/CRB **CASING:** Nil

TYPE OF BORING: Auger to 2.82m then NMLC Coring

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E431369 N6961488

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials:
Date:

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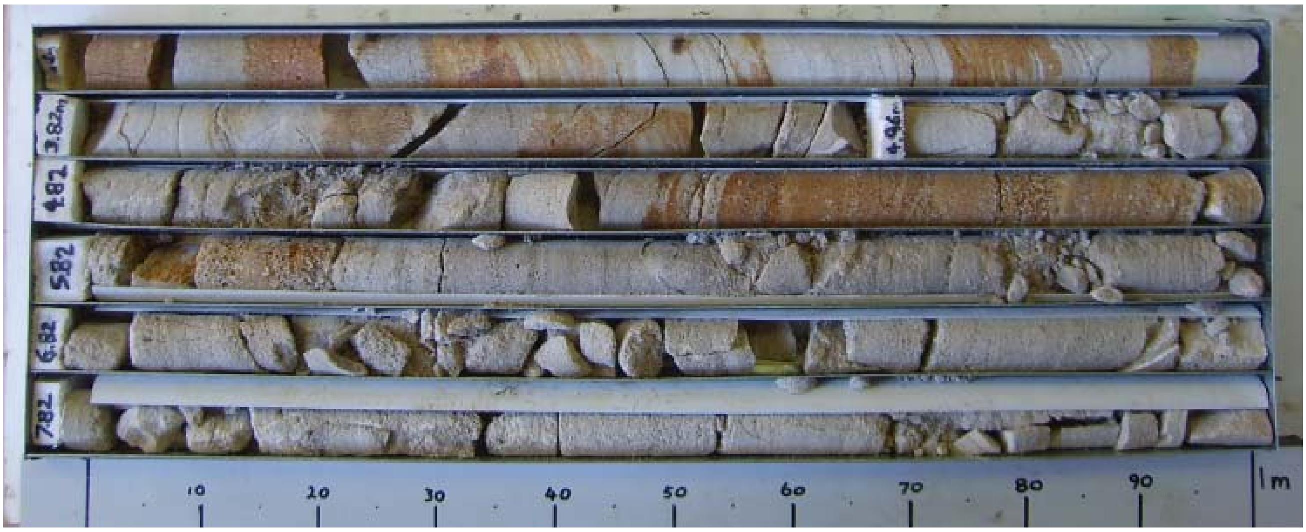
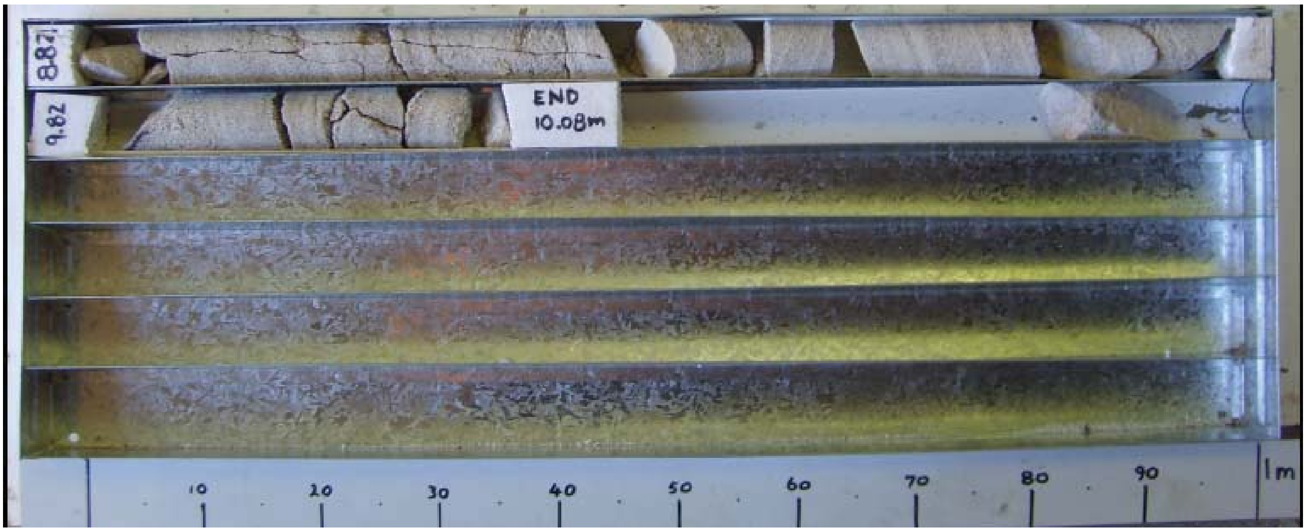


Bore 33: 2.82 – 8.82m



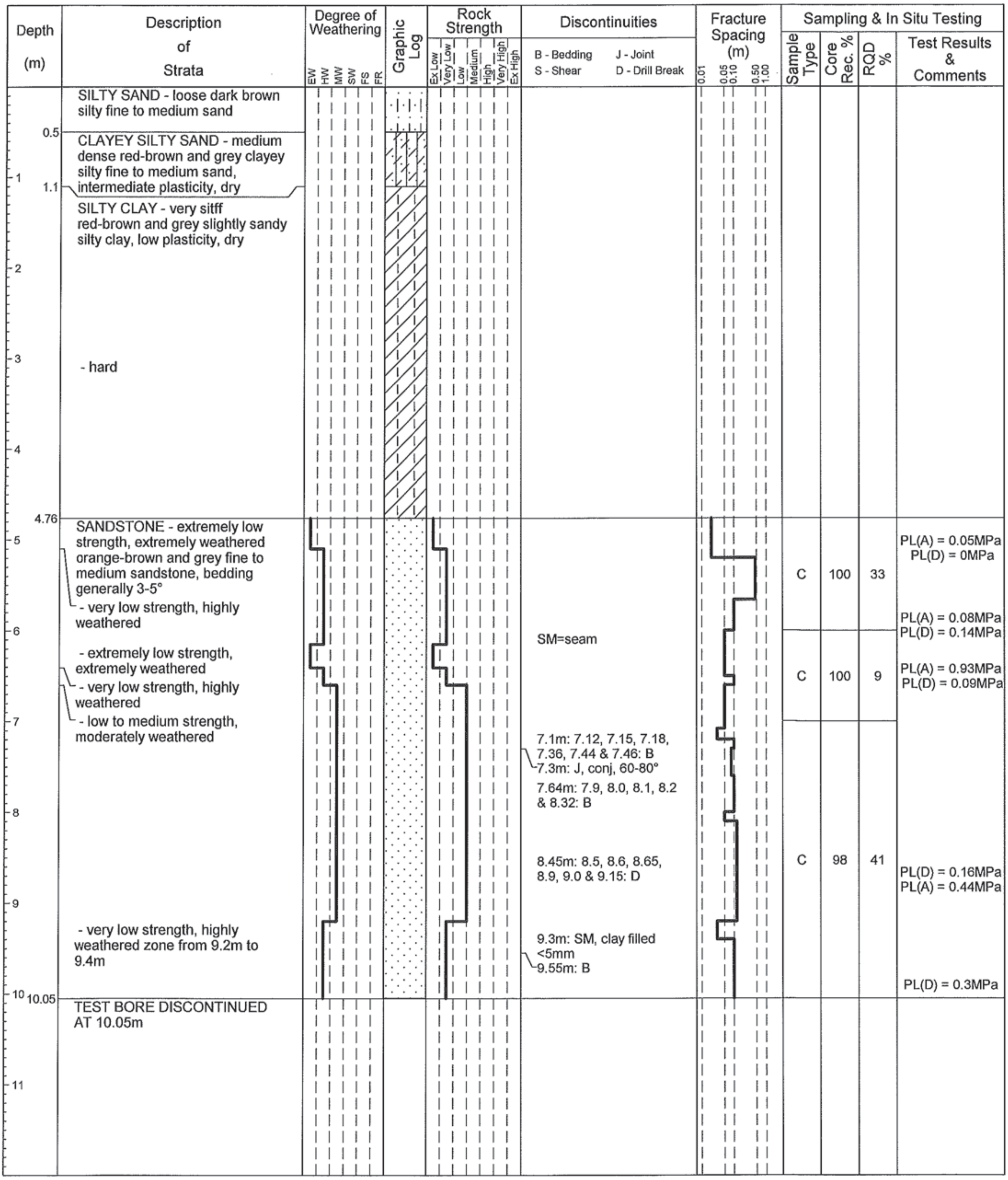
Bore 33: 8.82 – 10.08m

CORE PHOTOGRAPHS
PROPOSED GATTON CORRECTIONAL PRECINCT, SPRING CREEK

August 2007	Project 47276	Douglas Partners Geotechnics · Environment · Groundwater

TEST BORE REPORT

BORE No: 34
CLIENT: Project Services
PROJECT: Proposed Gatton Correctional Precinct
LOCATION: Krugers Road, Spring Creek
PROJECT No: 47276
SURFACE LEVEL: 154.7
DIP OF HOLE: 90°
DATE: 26.06.07
SHEET 1 OF 1
AZIMUTH: --



Depth (m)	Description of Strata	Discontinuities	Sample Type	Core Rec. %	RQD %	Test Results & Comments
	SILTY SAND - loose dark brown silty fine to medium sand					
0.5	CLAYEY SILTY SAND - medium dense red-brown and grey clayey silty fine to medium sand, intermediate plasticity, dry					
1.1	SILTY CLAY - very sitff red-brown and grey slightly sandy silty clay, low plasticity, dry - hard					
4.76	SANDSTONE - extremely low strength, extremely weathered orange-brown and grey fine to medium sandstone, bedding generally 3-5° - very low strength, highly weathered		C	100	33	PL(A) = 0.05MPa PL(D) = 0MPa PL(A) = 0.08MPa PL(D) = 0.14MPa
	- extremely low strength, extremely weathered - very low strength, highly weathered	SM=seam	C	100	9	PL(A) = 0.93MPa PL(D) = 0.09MPa
	- low to medium strength, moderately weathered - very low strength, highly weathered zone from 9.2m to 9.4m	7.1m: 7.12, 7.15, 7.18, 7.36, 7.44 & 7.46: B 7.3m: J, conj, 60-80° 7.64m: 7.9, 8.0, 8.1, 8.2 & 8.32: B 8.45m: 8.5, 8.6, 8.65, 8.9, 9.0 & 9.15: D 9.3m: SM, clay filled <5mm 9.55m: B	C	98	41	PL(D) = 0.16MPa PL(A) = 0.44MPa PL(D) = 0.3MPa
10.05	TEST BORE DISCONTINUED AT 10.05m					

RIG: EVH210 **DRILLER:** Sutherland Exploration **LOGGED:** ACS/CRB **CASING:** Nil

TYPE OF BORING: Auger to 4.76m then NMLC Coring

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E431103 N6961590

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials:
Date:

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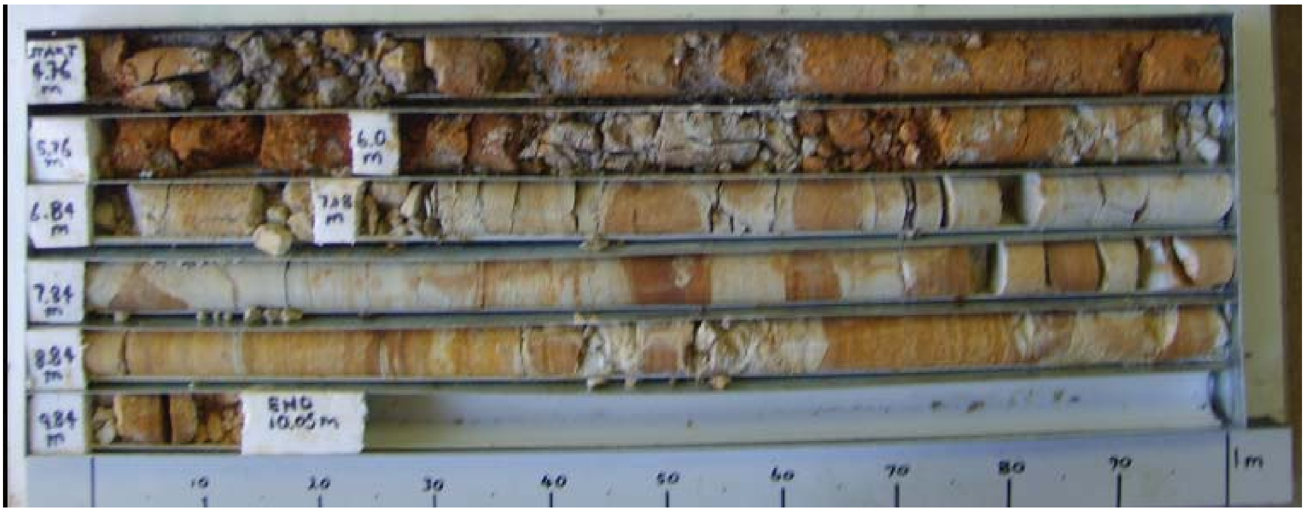


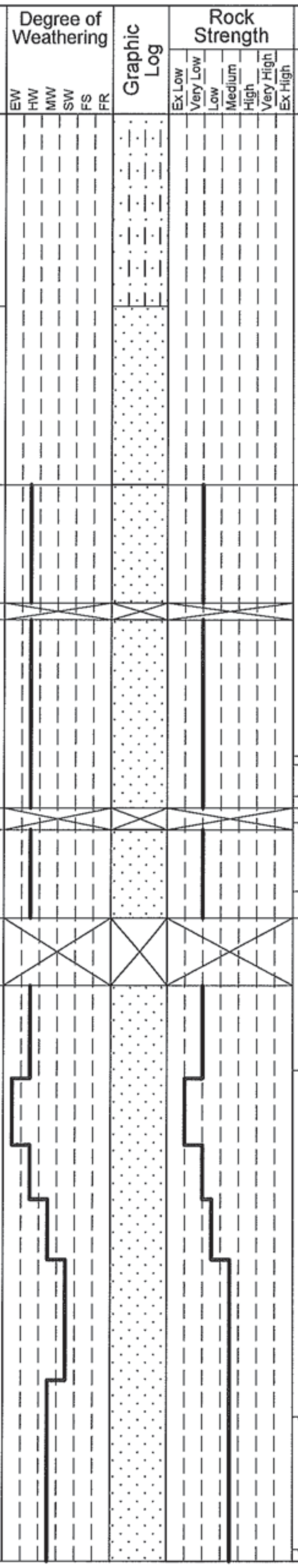
Bore 34: 4.76 – 10.05m

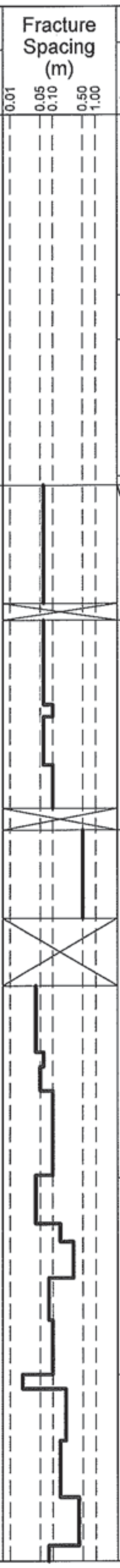
CORE PHOTOGRAPHS
PROPOSED GATTON CORRECTIONAL PRECINCT, SPRING CREEK

TEST BORE REPORT

BORE No: 35
CLIENT: Project Services
PROJECT No: 47276
DATE: 27.06.07
PROJECT: Proposed Gatton Correctional Precinct
SURFACE LEVEL: 162.1
SHEET 1 **OF** 2
LOCATION: Krugers Road, Spring Creek
DIP OF HOLE: 90°
AZIMUTH: --

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0	SILTY SAND - loose grading medium dense orange-brown silty fine to medium sand					
1.6	SANDSTONE - extremely low strength, extremely weathered orange-brown and light brown fine sandstone	SM=seam	S			10, 20, 30/70mm
3.08			S			30/75mm
3.08	SANDSTONE - very low to low strength, highly weathered orange-brown and light grey fine sandstone, bedding generally 3-5°	3.1m: B, generally 50mm spacings to 4.9	C	88	0	PL(A) = 0.12MPa PL(D) = 0.05MPa
4.06	CORE LOSS 140mm	4.06m: CORE LOSS: 140mm				
4.2	SANDSTONE - very low to low strength, highly weathered orange-brown and light grey fine sandstone	5.05m: 5.1, 5.15 & 5.2: D 5.33m: J, pl, 45° 5.4m: SM, clay infill <3mm 5.66m: B	C	90	10	PL(A) = 0.15MPa PL(D) = 0.03MPa PL(A) = 0.17MPa PL(D) = 0.06MPa
5.76	CORE LOSS 180mm	5.76m: CORE LOSS: 180mm				
5.94	SANDSTONE - very low to low strength, highly weathered orange-brown and light grey fine sandstone	5.88m: B 6.45m: 6.5 & 6.55: B	C	75	29	
6.67	CORE LOSS	6.67m: CORE LOSS: 560mm				PL(A) = 0.67MPa PL(D) = 0.17MPa
7.23	SANDSTONE - very low to low strength, highly weathered orange-brown and light grey fine sandstone	7.78m: & 7.88: D 7.93m: J, pl, 45°				PL(A) = 0.21MPa PL(D) = 0.11MPa
	- extremely low to very low strength, extremely weathered	8.4m: SM, clay infill <3mm				
	- very low to low strength, highly weathered	8.78m: 9.2: B, 40mm spacings				
	- low strength, moderately weathered light brown and grey	9.41m: D	C	100	50	PL(D) = 0.14MPa PL(A) = 0.28MPa
	- medium strength, slightly weathered	10.23m: & 10.3: B				PL(A) = 1.4MPa PL(D) = 1MPa
	- medium strength, moderately weathered	10.6m: SM, clay infill <3mm thick 10.8m: D	C	96	57	PL(A) = 0.94MPa PL(D) = 0.71MPa PL(A) = 0.7MPa





RIG: EVH210 **DRILLER:** Sutherland Exploration **LOGGED:** ACS/CRB **CASING:** Nil

TYPE OF BORING: Auger to 3.08m then NMLC Coring

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E430895 N6961647

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials:
Date:

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TEST BORE REPORT

BORE No: 35

CLIENT: Project Services
PROJECT No: 47276
DATE: 27.06.07

PROJECT: Proposed Gatton Correctional Precinct
SURFACE LEVEL: 162.1
SHEET 2 OF 2

LOCATION: Krugers Road, Spring Creek
DIP OF HOLE: 90°
AZIMUTH: --

Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low, Very Low, Low, Medium, High, Very High, Ex High	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Sample Type	Core Rec. %	RQD %	Test Results & Comments
12.0	SANDSTONE - as before - with some coarse grains to 12.8m				11.9m: J, ro, irreg, 40° 12.17m: B, ironstained					PL(D) = 0.4MPa
							C	96	57	
13	- low strength, moderately weathered				12.8m: 12.87: fractured 13.16m: B, ironstained					PL(A) = 0.29MPa PL(D) = 0.15MPa
14	- high strength, slightly weathered									PL(A) = 1.89MPa PL(D) = 1.48MPa
15							C	100	88	PL(A) = 1.66MPa PL(D) = 1MPa
16										PL(A) = 1.21MPa PL(D) = 0.86MPa
17										PL(A) = 1MPa PL(D) = 1.07MPa
18							C	98	70	PL(A) = 1.24MPa PL(D) = 1.25MPa
19										PL(A) = 1.09MPa PL(D) = 1MPa
20 20.0	TEST BORE DISCONTINUED AT 20.0m									
21										
22										
23										

RIG: EVH210 **DRILLER:** Sutherland Exploration **LOGGED:** ACS/CRB **CASING:** Nil

TYPE OF BORING: Auger to 3.08m then NMLC Coring

WATER OBSERVATIONS: No free groundwater observed

REMARKS: Co-ordinates: E430895 N6961647

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED

Initials:

Date:

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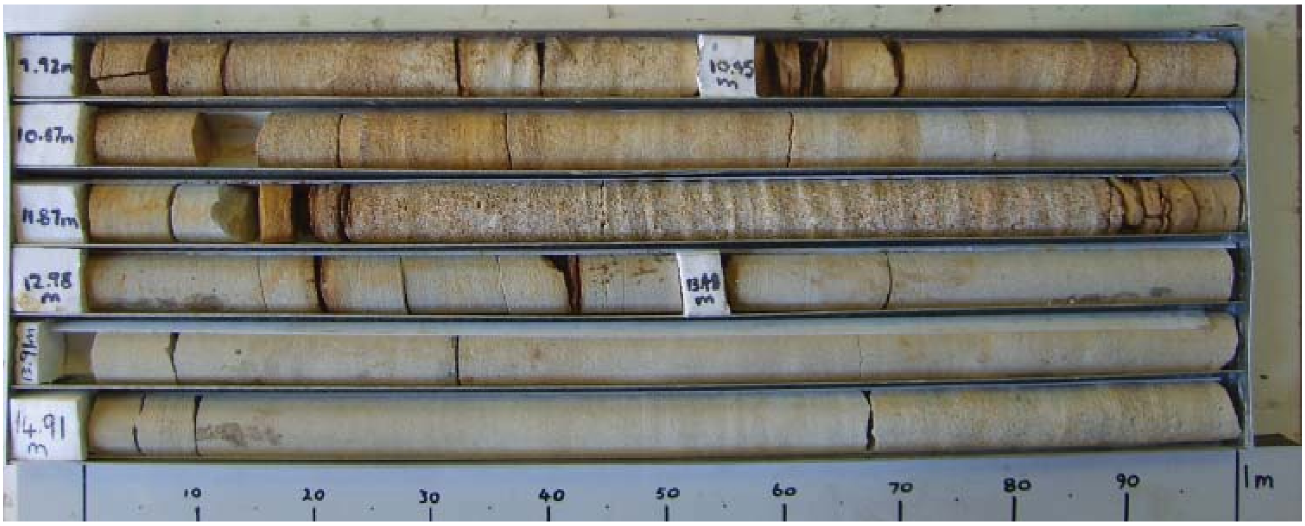
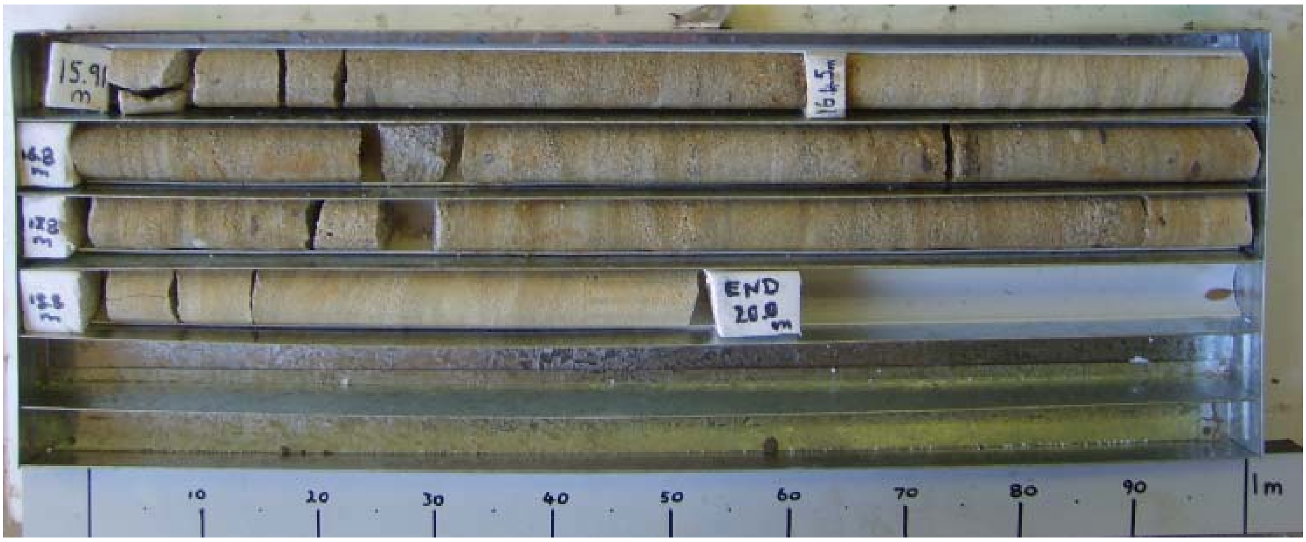
Bore 35: 3.04 – 9.92m



Bore 35: 9.92 – 15.91m



Bore 35: 15.91 – 20.0m

CORE PHOTOGRAPHS
PROPOSED GATTON CORRECTIONAL PRECINCT, SPRING CREEK

August 2007	Project 47276	Douglas Partners Geotechnics • Environment • Groundwater

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NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and Classification Methods

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

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

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

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

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

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

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

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

Sampling

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

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

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

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

Drilling Methods.

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

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

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

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

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

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

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

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

  as 4, 6, 7
  N = 13

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

  as 15, 30/40 mm.

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

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

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: —

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

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

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

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

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

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

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

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

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

Hand Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Bore Logs

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

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

Ground Water

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

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

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

Engineering Reports

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

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

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

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

Site Anomalies

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

Reproduction of Information for Contractual Purposes

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

Site Inspection

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

DESCRIPTION AND CLASSIFICATION OF ROCKS FOR ENGINEERING PURPOSES

DEGREE OF WEATHERING

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

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index [$I_{S(50)}$] and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described in Australian Standard AS4133.4.1–1993.

Term	Symbol	Field Guide*	Point Load Index [$I_{S(50)}$] MPa	Approx Unconfined Compressive Strength (q_u) MPa**
Extremely Low	EL	Easily remoulded by hand to a material with soil properties.	<0.03	<0.6
Very Low	VL	Material crumbles under firm blows with sharp end of geological pick; can be peeled with a knife; too hard to cut a triaxial sample by hand. SPT will refuse. Pieces up to 30mm thick can be broken by finger pressure.	0.03 – 0.1	0.6 – 2
Low	L	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the geological pick point; has dull sound under hammer. A piece of core 150mm long by 40mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.	0.1 – 0.3	2 – 6
Medium	M	Readily scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.	0.3 – 1	6 – 20
High	H	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken with geological pick with a single firm blow; rock rings under hammer.	1 – 3	20 – 60
Very High	VH	Hand specimen breaks with geological pick after more than one blow; rock rings under hammer.	3 – 10	60 – 200
Extremely High	EH	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.	>10	>200

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

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

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

DESCRIPTION AND CLASSIFICATION OF ROCKS FOR ENGINEERING PURPOSES

STRATIFICATION SPACING

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

DEGREE OF FRACTURING

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude known artificial fractures such as drilling breaks. The orientation of rock defects is measured as an angle relative to a plan perpendicular to the core axis.

Note the recording of actual spacing and range of spacing is preferred in place of the terms below.

Term	Description
Fragmented	The core is comprised primarily of fragments of length less than 20mm, and mostly of width less than the core diameter.
Highly fractured	Core lengths are generally less than 20mm to 40mm with occasional fragments.
Fractured	Core lengths are mainly 30mm to 100mm with occasional shorter and longer sections.
Slightly fractured	Core lengths are generally 300mm to 1000mm with occasional longer sections and occasional sections of 100mm to 300mm.
Unbroken	The core does not contain any fracture.

ROCK QUALITY DESIGNATION (RQD)

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

REFERENCE

International Society of Rock Mechanics, Suggested Method for Determining the Point Load Strength, 1985.

APPENDIX B

Laboratory Report Sheets

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DETERMINATION OF EMERSON CLASS NUMBER OF SOIL

Client: **Project Services**

Project: Proposed Correctional Precinct

Location: Krugers Road, Spring Creek

Project No: 47276
Report No: B07-1050
Report Date: 02/07/2007

Date Sampled: 20/07/2007
Date of Test: 26/07/2007
Page: 1 of 1

SAMPLE NO	DEPTH (m)	DESCRIPTION	WATER TYPE	WATER TEMP	CLASS NO.
Bore 1	0.7-1.0m	Silty Sand Brown	Dist	22 C	4
Bore 1	1.5-1.85m	Extremely low strength Sandstone	Dist	22 C	4
Bore 8	4.8-5.6m	Extremely low strength Sandstone	Dist	22 C	4

Test Method(s): AS 1289 3.8.1 - 2006

Sampling Method(s): Disturbed and SPT Samples

Remarks:

Approved Signatory:

Tested: TL
Checked: SJ

Srdjan Jajcanin
Senior Soils Technician

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RESULTS OF MOISTURE CONTENT, PLASTICITY AND LINEAR SHRINKAGE TESTS

Client : **Project Services**

Project : Proposed Correctional Development

Location : Millers Road

Project No. : 47276
Report No. : B07-1056
Report Date : 6/08/2007
Date Sampled : 20/07/2007
Date of Test: 27/07/2007
Page: 1 of 1

TEST LOCATION	DEPTH (m)	DESCRIPTION	Code	W_F %	W_L %	W_P %	PI %	*LS %
Bore 1	0.7-1.0m	Silty Sand with a trace of gravel	2,5	8	NP	NP	NP	NP
Bore 1	1.5-1.86m	Sandstone - ELS	2,5	5.2	25	15	10	8
Bore 8	4.8-5.6m	Silty Gravelly Sand	2,5	2.5	25	12	13	8
Bore 11	0.7-1.0m	Clayey Sand Brown	2,5	7.4	NP	NP	NP	NP
Bore 12	3.0-3.45m	Clayey Sand Light Brown	2,5	12.6	24	12	12	8
Bore 33	1.0-1.45m	Clayey Sand with a trace of gravel	2,5	6.4	42	14	28	15.5
Bore 34	0.7-1.0m	Clayey Sand Grey	2,5	6.8	NP	NP	NP	NP

Legend:

W_F	Field Moisture Content
W_L	Liquid limit
W_P	Plastic limit
PI	Plasticity index
LS	Linear shrinkage from liquid limit condition.

Test Methods:

Moisture Content:	AS 1289.2.1.1 - 2005
Liquid Limit:	AS 1289.3.1.2 - 1995
Plastic Limit:	AS 1289.3.3.1 - 1995
Plasticity Index:	AS 1289.3.2.1 - 1995
Linear Shrinkage:	AS 1289.3.4.1 - 1995
Sampling Method(s):	**Disturbed**

Code

Sample history for plasticity tests

1	Air dried
2	Low temperature (<50ºC) oven dried
3	Oven (105ºC) dried
4	Unknown

Method of preparation for plasticity tests

5	Dry sieved
6	Wet sieved
7	Natural

*Specify if sample crumbled CR or curled CU

Remarks

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Approved Signatory

Tested	TL
Checked	SJ

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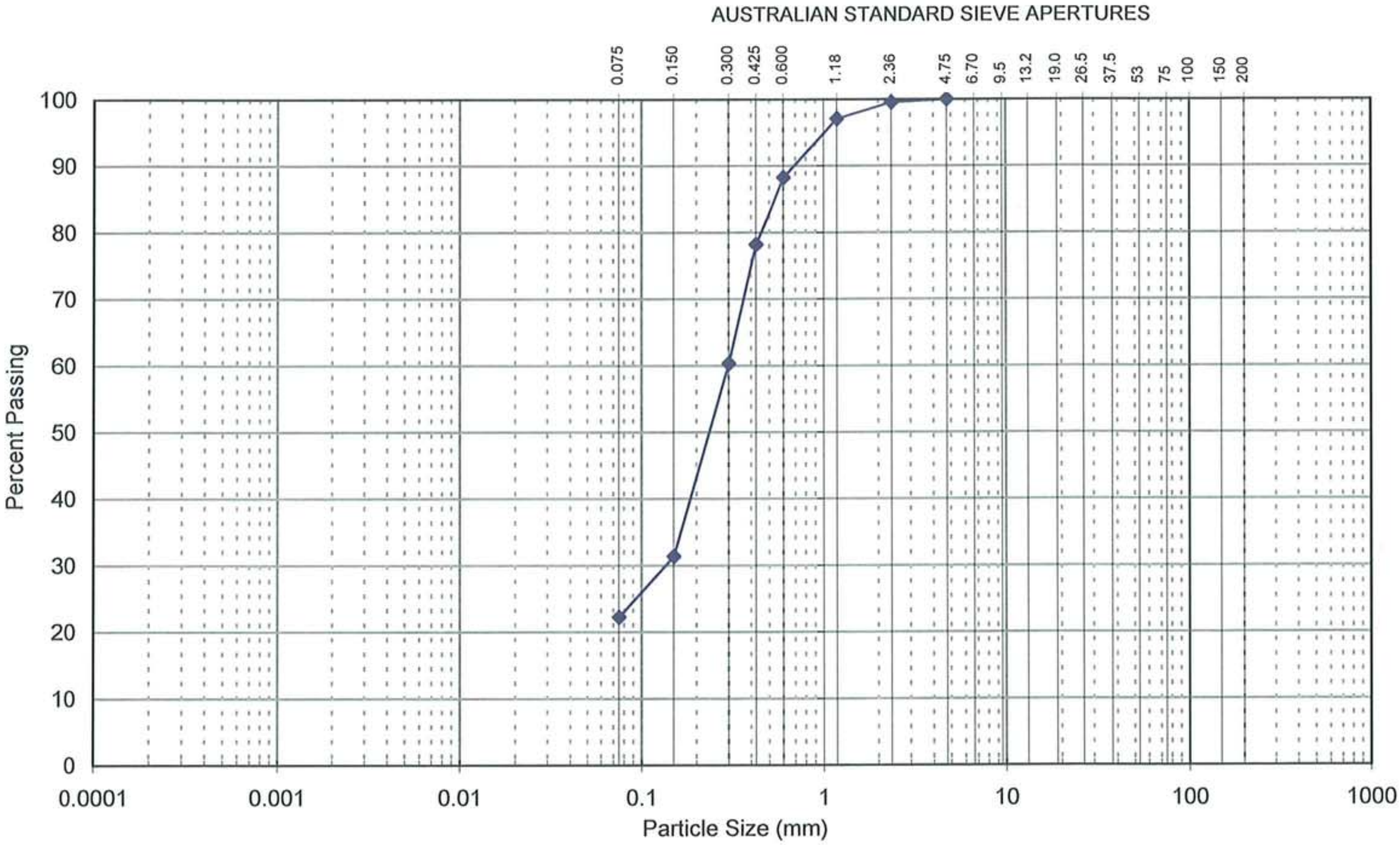
RESULTS OF PARTICLE SIZE DISTRIBUTION TEST

Client :	Project Services	**Project No. :**	47276
		Report No. :	BO7-1044
Project :	Proposed Correctional Precinct	**Report Date :**	6/08/2007
Location :	Krugers Road, Spring Creek	**Date Sampled:**	20/07/2007
Test Location :	Bore 1	**Date of Test:**	28/07/2007
Depth / Layer :	0.7-1.0m	**Page:**	1 of 1



CLAY FRACTION	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION			COBBLES
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	

0.002 0.006 0.02 0.06 0.2 0.6 2.0 6.0 20 60

Description:	Silty sand brown
Test Method(s):	AS 1289 3.6.1 - 1995, AS 1289 3.6.3 - 2003
Sampling Method(s):	Disturbed Sample
Method of Dispersion:	
Remarks:	

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COMPETENCE

NATA Accredited Laboratory Number 828

This document is issued in accordance with NATA's accreditation requirements.
Accredited for compliance with ISO/IEC 17025.

Approved Signatory:

Tested:	SG
Checked:	SJ

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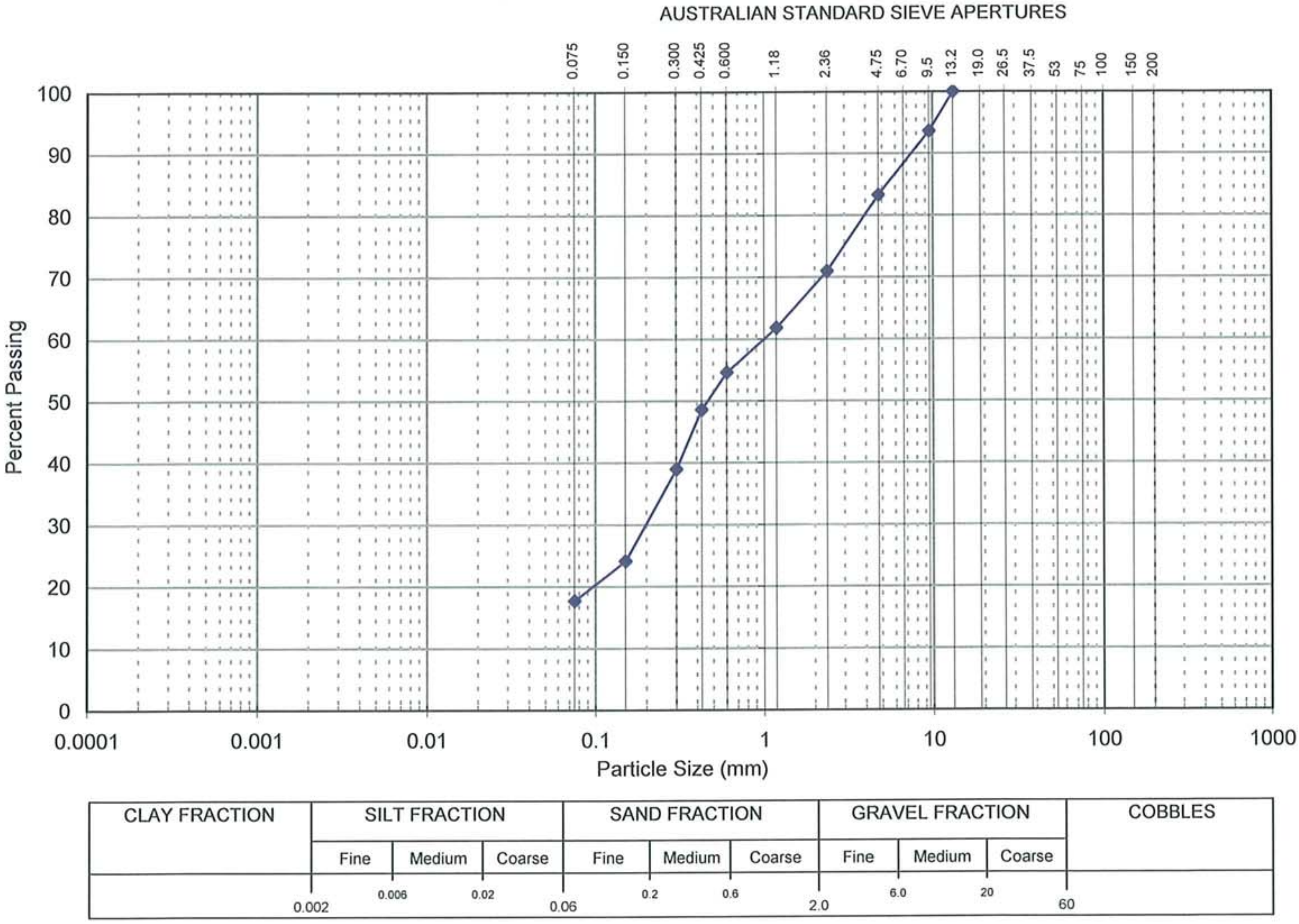
RESULTS OF PARTICLE SIZE DISTRIBUTION TEST

Client :	Project Services	**Project No. :**	47276
		Report No. :	BO7-1045
Project :	Proposed Correctional Precinct	**Report Date :**	6/08/2007
Location :	Krugers Road, Spring Creek	**Date Sampled:**	20/07/2007
Test Location :	Bore 1	**Date of Test:**	28/07/2007
Depth / Layer :	1.5-1.85m	**Page:**	1 of 1



Description: Extremely low strength sandstone red brown

Test Method(s): AS 1289 3.6.1 - 1995, AS 1289 3.6.3 - 2003

Sampling Method(s): SPT Sample

Method of Dispersion:

Remarks:

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Approved Signatory:

Tested:	SG
Checked:	SJ

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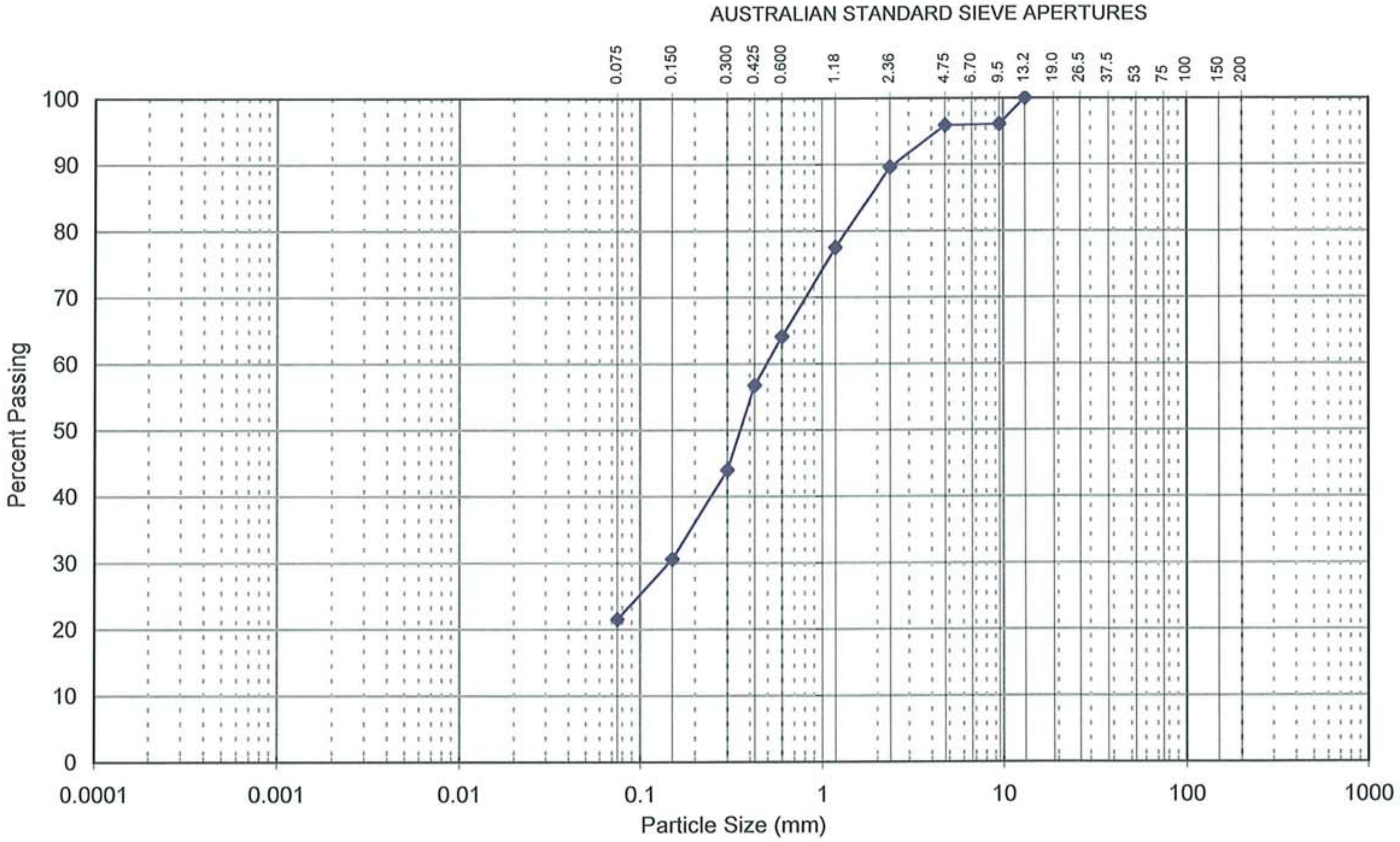
brisbane@douglaspartners.com.au

RESULTS OF PARTICLE SIZE DISTRIBUTION TEST

Client :	Project Services	**Project No. :**	47276
		Report No. :	BO7-1046
Project :	Proposed Correctional Precinct	**Report Date :**	6/08/2007
Location :	Krugers Road, Spring Creek	**Date Sampled:**	20/07/2007
Test Location :	Bore 8	**Date of Test:**	28/07/2007
Depth / Layer :	4.8-5.6m	**Page:**	1 of 1



CLAY FRACTION	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION			COBBLES
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
0.002	0.006	0.02	0.06	0.2	0.6	2.0	6.0	20	60	

Description: Silty gravelly sand grey

Test Method(s): AS 1289 3.6.1 - 1995, AS 1289 3.6.3 - 2003

Sampling Method(s): Disturbed Sample

Method of Dispersion:

Remarks:

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Approved Signatory:

Tested:	SG
Checked:	SJ

Srdjan Jajcanin
Senior Soil Technician

Douglas Partners
Geotechnics · Environment · Groundwater

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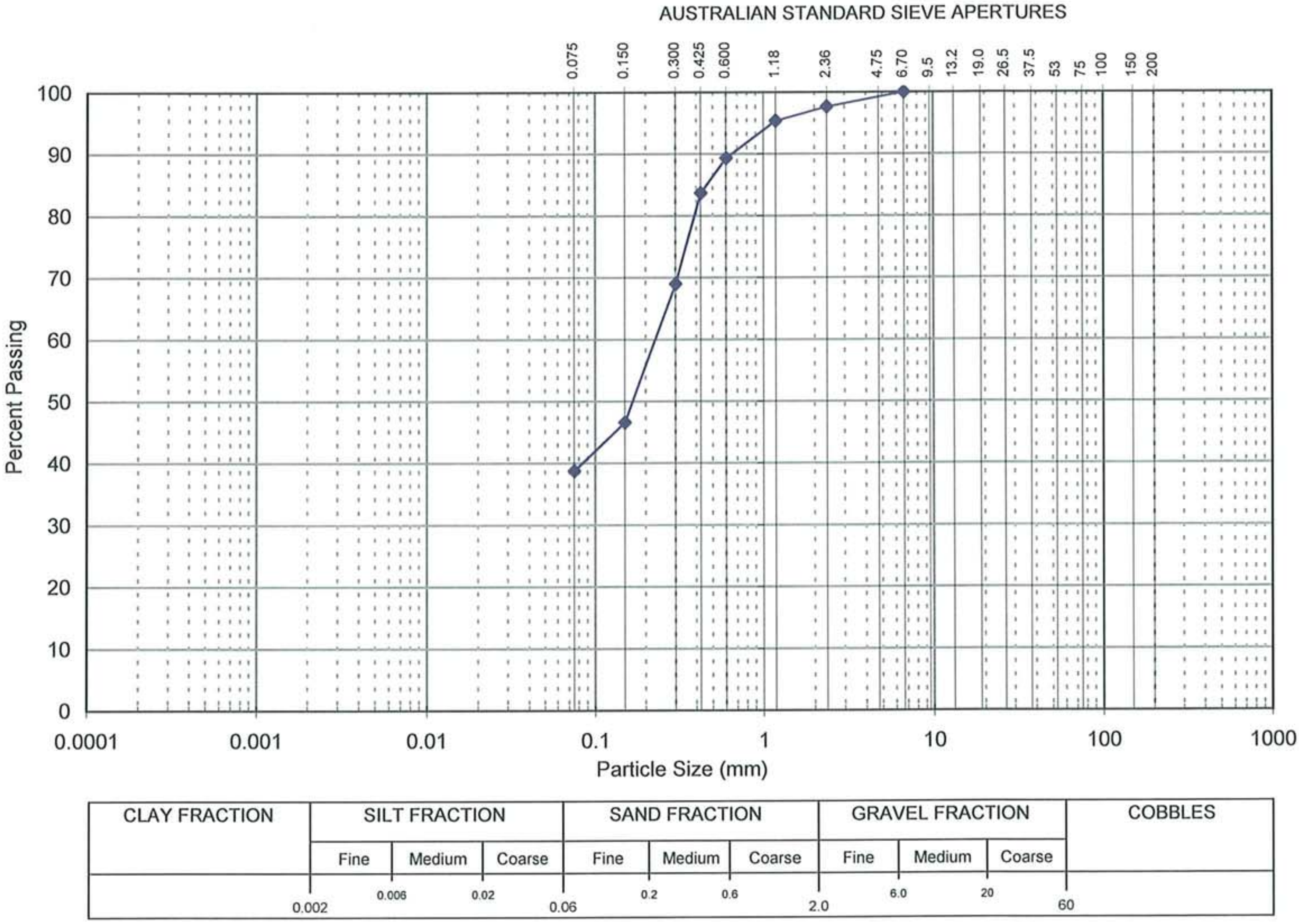
RESULTS OF PARTICLE SIZE DISTRIBUTION TEST

Client :	Project Services	**Project No. :**	47276
		Report No. :	BO7-1047
Project :	Proposed Correctional Precinct	**Report Date :**	6/08/2007
Location :	Krugers Road, Spring Creek	**Date Sampled:**	20/07/2007
Test Location :	Bore 12	**Date of Test:**	28/07/2007
Depth / Layer :	3.0-3.45m	**Page:**	1 of 1



Description: Clayey sand with a trace of gravel

Test Method(s): AS 1289 3.6.1 - 1995, AS 1289 3.6.3 - 2003

Sampling Method(s): SPT Sample

Method of Dispersion:

Remarks:

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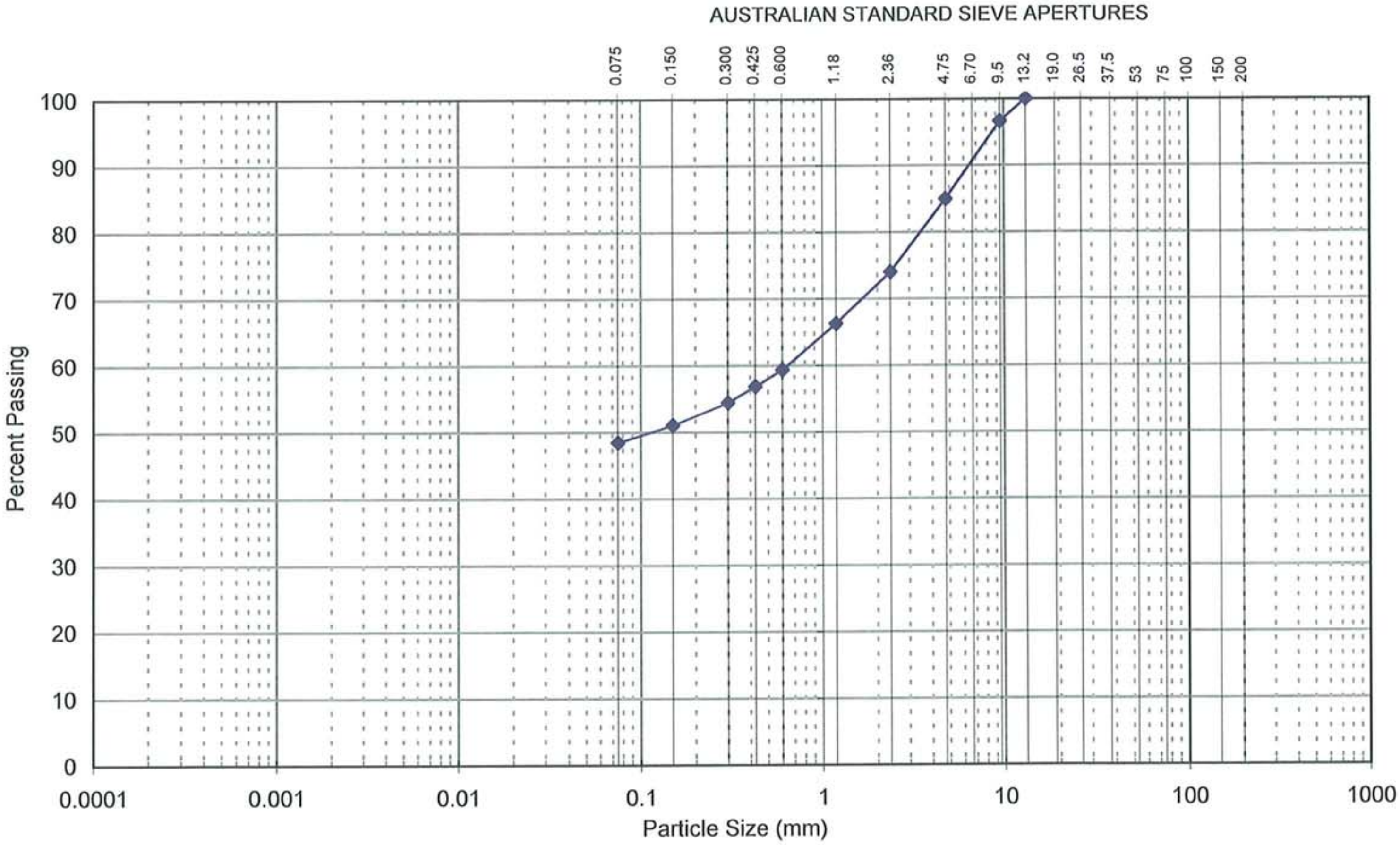
RESULTS OF PARTICLE SIZE DISTRIBUTION TEST

Client :	Project Services	**Project No. :**	47276
		Report No. :	BO7-1048
Project :	Proposed Correctional Precinct	**Report Date :**	6/08/2007
Location :	Krugers Road, Spring Creek	**Date Sampled:**	20/07/2007
Test Location :	Bore 14	**Date of Test:**	28/07/2007
Depth / Layer :	0.2-1.0m	**Page:**	1 of 1



CLAY FRACTION	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION			COBBLES
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
0.002	0.006	0.02	0.06	0.2	0.6	2.0	6.0	20	60	

Description: Gravelly sandy clay brown

Test Method(s): AS 1289 3.6.1 - 1995, AS 1289 3.6.3 - 2003

Sampling Method(s): Disturbed Sample

Method of Dispersion:

Remarks:

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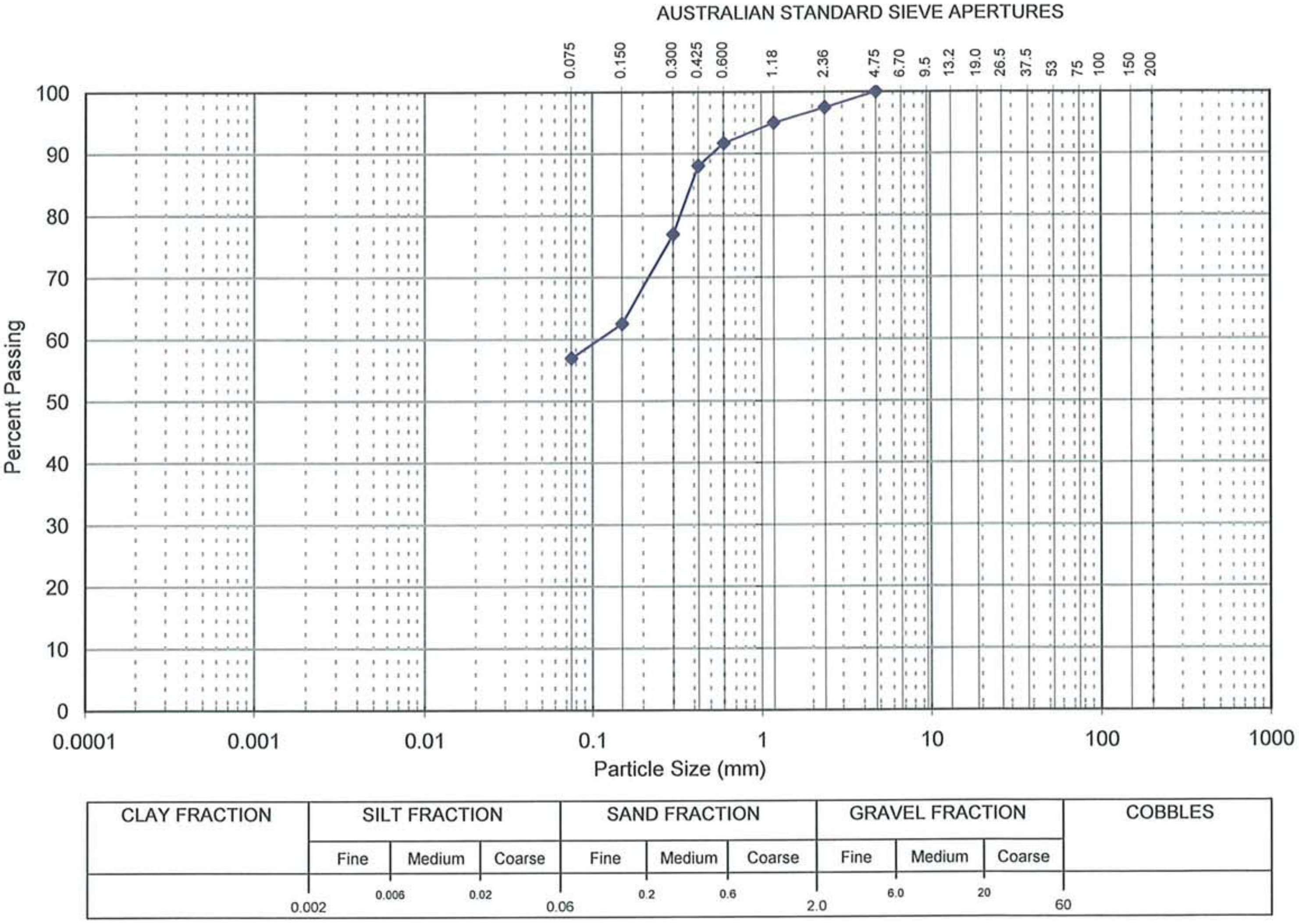
RESULTS OF PARTICLE SIZE DISTRIBUTION TEST

Client :	Project Services	**Project No. :**	47276
		Report No. :	BO7-1049
Project :	Proposed Correctional Precinct	**Report Date :**	6/08/2007
Location :	Krugers Road, Spring Creek	**Date Sampled:**	20/07/2007
Test Location :	Bore 33	**Date of Test:**	28/07/2007
Depth / Layer :	1.0-1.45m	**Page:**	1 of 1



Description: Sandy Clay with a trace of gravel

Test Method(s): AS 1289 3.6.1 - 1995, AS 1289 3.6.3 - 2003

Sampling Method(s): SPT Sample

Method of Dispersion:

Remarks:

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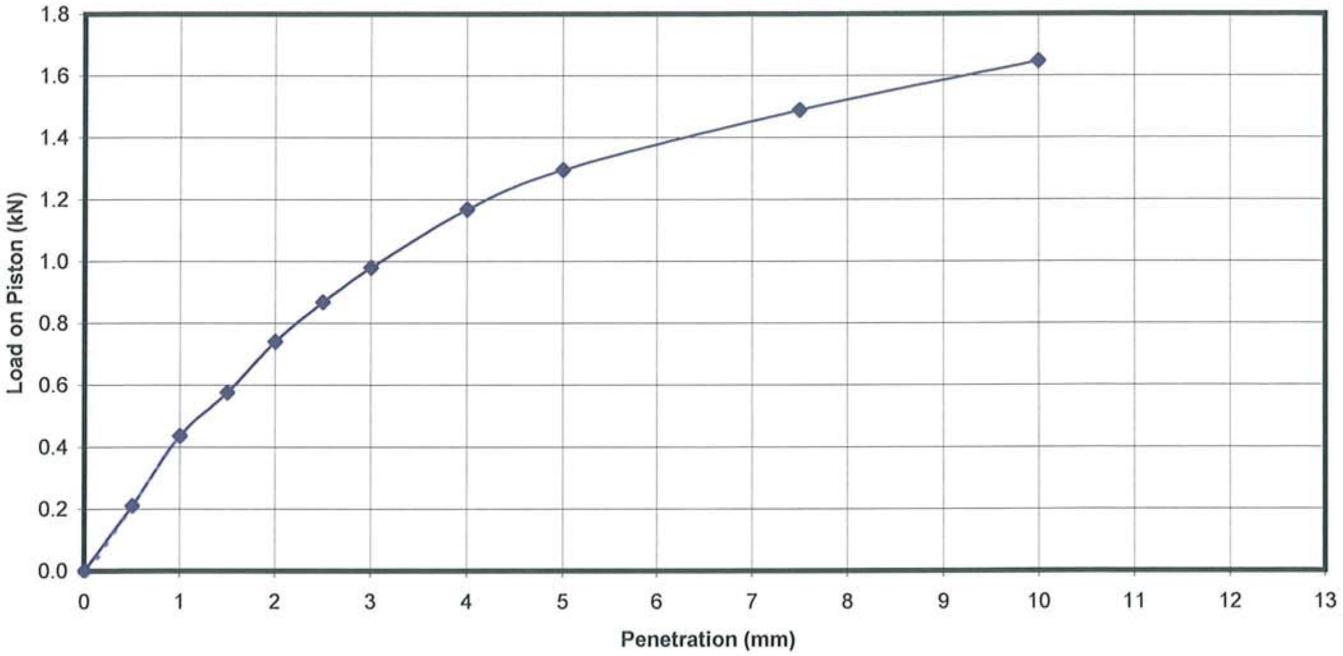
RESULT OF CALIFORNIA BEARING RATIO TEST

Client :	Project Services	**Project No. :**	47276
		Report No. :	B07- 1051
Project :	Proposed Correctional Precinct	**Report Date :**	06-Aug-07
		Date Sampled :	20-Jul-07
Location :	Krugers Road, Spring Creek	**Date of Test:**	26-Jul-07
Test Location :	Bore 10		
Depth / Layer :	0.2-1.0m	**Page:**	1 of 1



Description: Sandy Clay

Test Method(s): AS1289.6.1.1-1998, AS1289.2.1.1-1992

Sampling Method(s): AS 1289.1.2.1-1998, AS 1289.1.1-2001

Percentage > 19mm: 0.0%

LEVEL OF COMPACTION: 98% of STD MDD
MOISTURE RATIO: 102% of STD OMC
SURCHARGE: 4.5 kg
SOAKING PERIOD: 4 days
SWELL: 0.3%

CONDITION		MOISTURE CONTENT %	DRY DENSITY t/m^3
At compaction		23.8	1.55
After soaking		25.9	1.55
After test	Top 30mm of sample	26.2	-
	Remainder of sample	26.0	-
Field values		18.2	-
Standard Compaction		23.4	1.59

RESULTS

TYPE	PENETRATION	CBR (%)
TOP	**2.5 mm**	**7**
	5.0 mm	**7**

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Approved Signatory:

Tested: TF
Checked: SJ

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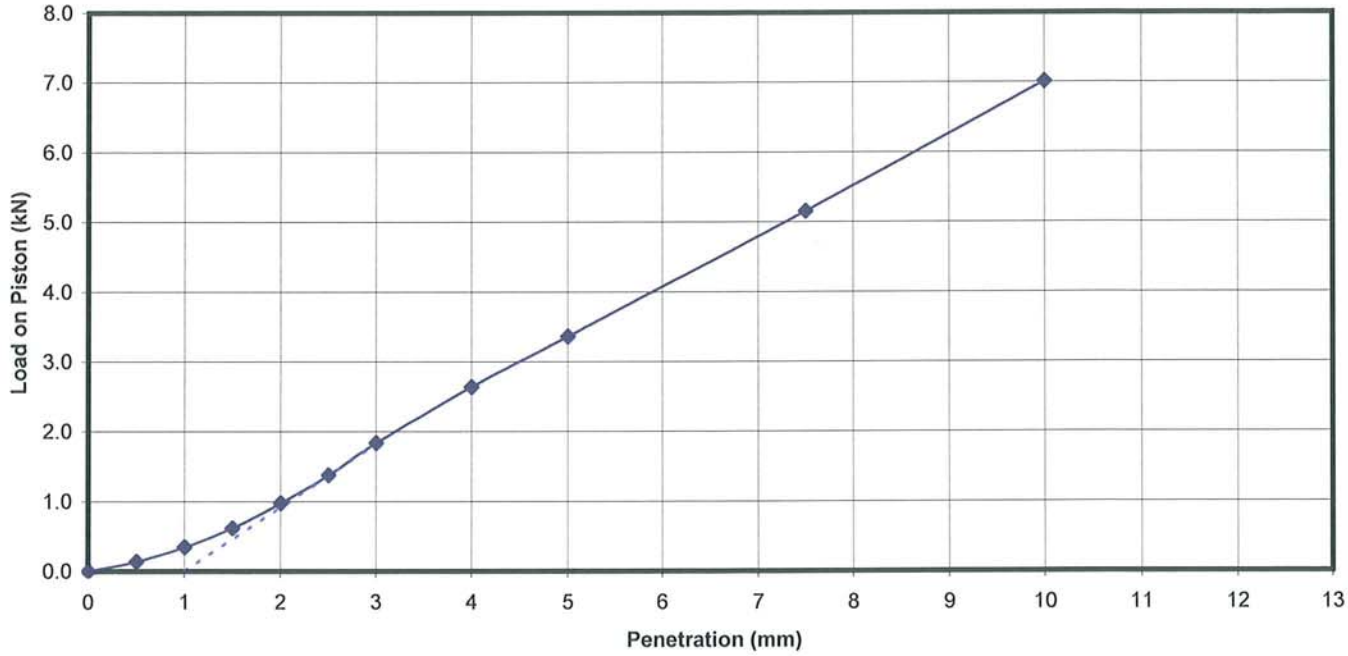
RESULT OF CALIFORNIA BEARING RATIO TEST

Client : Project Services
Project : Proposed Correctional Precinct
Location : Krugers Road, Spring Creek
Test Location : Bore 11
Depth / Layer : 0.2-1.0m

Project No. : 47276
Report No. : B07- 1052
Report Date : 06-Aug-07
Date Sampled : 20-Jul-07
Date of Test: 26-Jul-07
Page: 1 of 1



Description: Clayey Sand

Test Method(s): AS1289.6.1.1-1998, AS1289.2.1.1-1992

Sampling Method(s): AS 1289.1.2.1-1998, AS 1289.1.1-2001

Percentage > 19mm: 0.0%

LEVEL OF COMPACTION: 98% of STD MDD
MOISTURE RATIO: 99% of STD OMC
SURCHARGE: 4.5 kg
SOAKING PERIOD: 4 days
SWELL: 0.0%

CONDITION	MOISTURE CONTENT %	DRY DENSITY t/m^3
At compaction	11.3	1.82
After soaking	13.5	1.82
After test Top 30mm of sample	11.9	-
Remainder of sample	12.8	-
Field values	7.7	-
Standard Compaction	11.5	1.85

RESULTS		
TYPE	**PENETRATION**	**CBR (%)**
TOP	**2.5 mm**	**17**
	5.0 mm	**20**

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Approved Signatory:

Tested: TF
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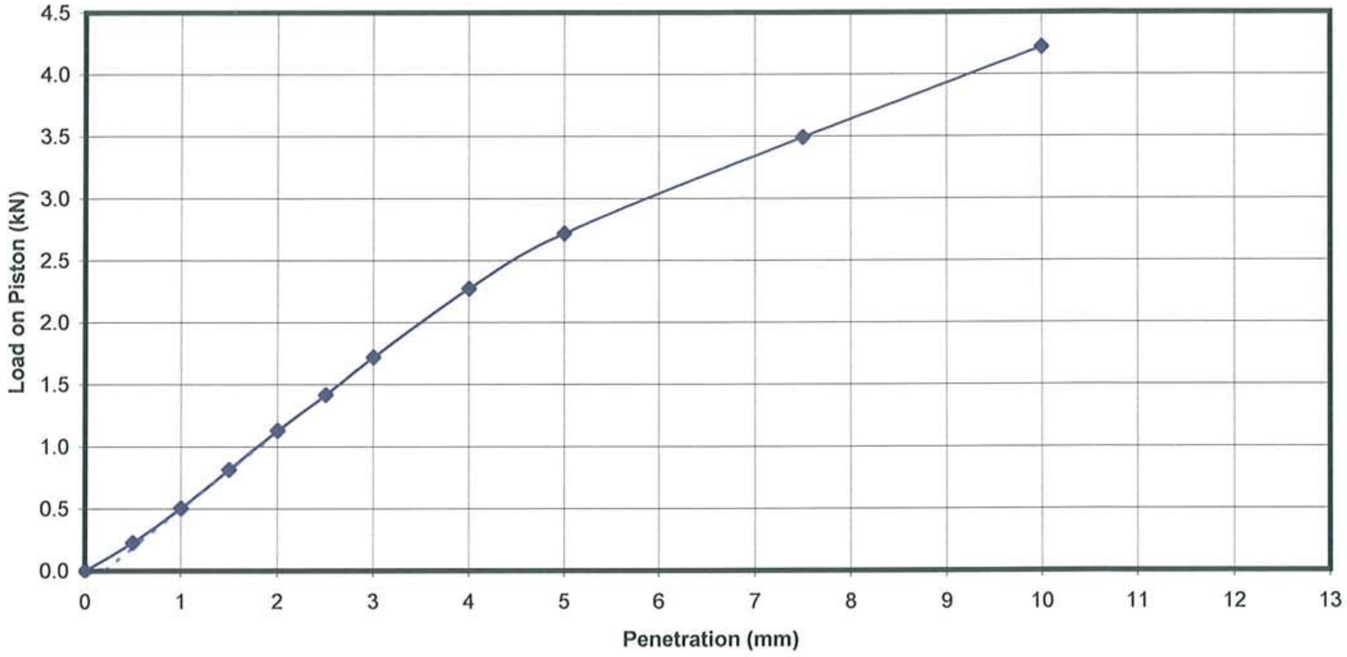
RESULT OF CALIFORNIA BEARING RATIO TEST

Client :	Project Services	**Project No. :**	47276
		Report No. :	B07- 1053
Project :	Proposed Correctional Precinct	**Report Date :**	06-Aug-07
		Date Sampled :	20-Jul-07
Location :	Krugers Road, Spring Creek	**Date of Test:**	26-Jul-07
Test Location :	Bore 12		
Depth / Layer :	0.2-1.0m	**Page:**	1 of 1



Description: Silty Sand

Test Method(s): AS1289.6.1.1-1998, AS1289.2.1.1-1992

Sampling Method(s): AS 1289.1.2.1-1998, AS 1289.1.1-2001

Percentage > 19mm: 0.0%

LEVEL OF COMPACTION: 98% of STD MDD
MOISTURE RATIO: 100% of STD OMC
SURCHARGE: 4.5 kg
SOAKING PERIOD: 4 days
SWELL: 0.1%

CONDITION		MOISTURE CONTENT %	DRY DENSITY t/m^3
At compaction		15.8	1.77
After soaking		18.8	1.77
After test	Top 30mm of sample	17.3	-
	Remainder of sample	17.6	-
Field values		12.8	-
Standard Compaction		15.8	1.81

RESULTS		
TYPE	**PENETRATION**	**CBR (%)**
TOP	**2.5 mm**	**12**
	5.0 mm	**14**

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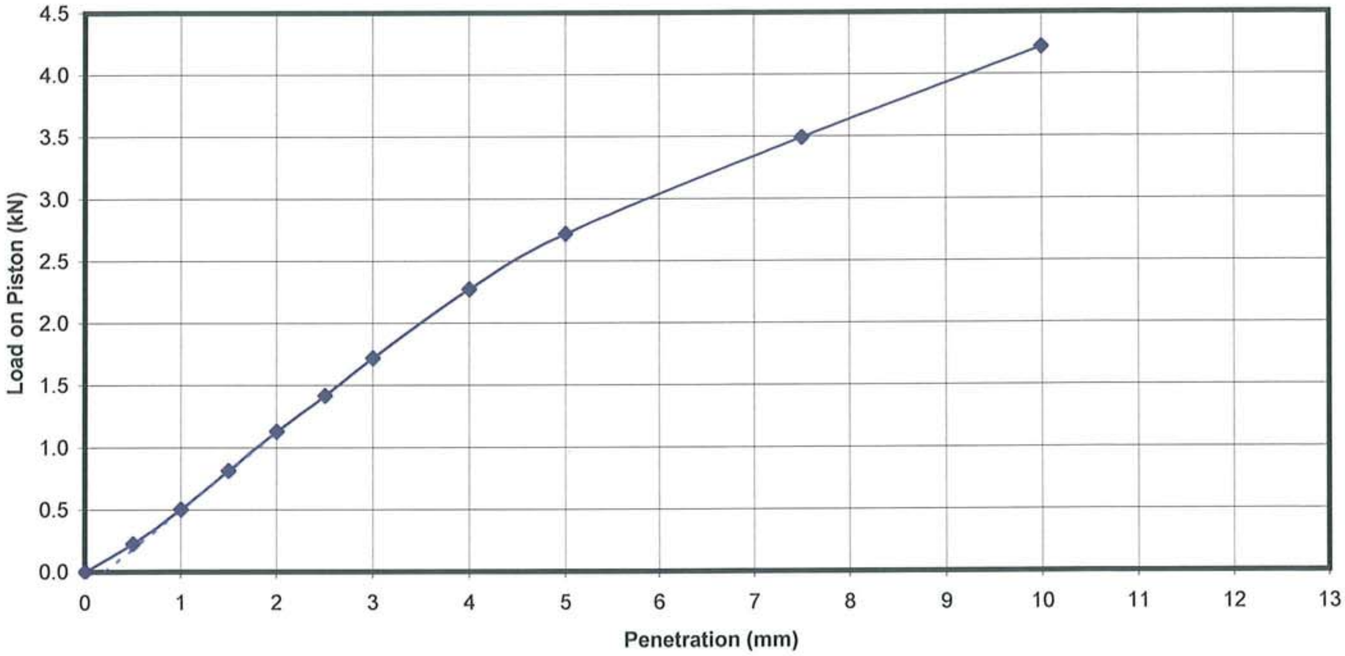
RESULT OF CALIFORNIA BEARING RATIO TEST

Client :	Project Services	**Project No. :**	47276
		Report No. :	B07- 1054
Project :	Proposed Correctional Precinct	**Report Date :**	06-Aug-07
		Date Sampled :	20-Jul-07
Location :	Krugers Road, Spring Creek	**Date of Test:**	26-Jul-07
Test Location :	Bore 13		
Depth / Layer :	0.2-1.0m	**Page:**	1 of 1



Description: Gravelly Sand

Test Method(s): AS1289.6.1.1-1998, AS1289.2.1.1-1992

Sampling Method(s): AS 1289.1.2.1-1998, AS 1289.1.1-2001

Percentage > 19mm: 0.0%

LEVEL OF COMPACTION: 98% of STD MDD
MOISTURE RATIO: 100% of STD OMC
SURCHARGE: 4.5 kg
SOAKING PERIOD: 4 days
SWELL: 0.1%

CONDITION		MOISTURE CONTENT %	DRY DENSITY t/m^3
At compaction		15.8	1.77
After soaking		18.8	1.77
After test	Top 30mm of sample	17.3	-
	Remainder of sample	17.6	-
Field values		12.8	-
Standard Compaction		15.8	1.81

RESULTS		
TYPE	**PENETRATION**	**CBR (%)**
TOP	**2.5 mm**	**12**
	5.0 mm	**14**

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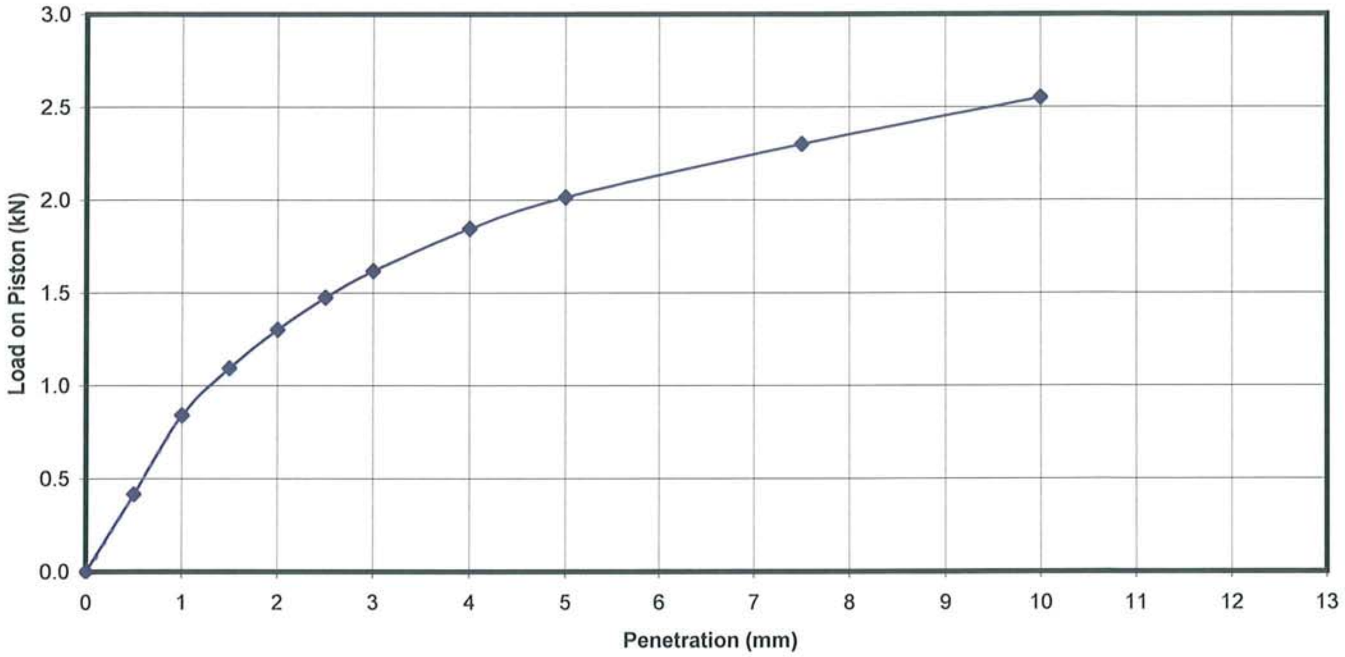
RESULT OF CALIFORNIA BEARING RATIO TEST

Client :	Project Services	**Project No. :**	47276
		Report No. :	B07- 1055
Project :	Proposed Correctional Precinct	**Report Date :**	06-Aug-07
		Date Sampled :	20-Jul-07
Location :	Krugers Road, Spring Creek	**Date of Test:**	26-Jul-07
Test Location :	Bore 14		
Depth / Layer :	0.2-1.0m	**Page:**	1 of 1



Description: Silty Sand brown

Test Method(s): AS1289.6.1.1-1998, AS1289.2.1.1-1992

Sampling Method(s): AS 1289.1.2.1-1998, AS 1289.1.1-2001

Percentage > 19mm: 0.0%

LEVEL OF COMPACTION: 98% of STD MDD **SURCHARGE:** 4.5 kg **SWELL:** -0.6%
MOISTURE RATIO: 102% of STD OMC **SOAKING PERIOD:** 4 days

CONDITION		MOISTURE CONTENT %	DRY DENSITY t/m^3
At compaction		16.5	1.77
After soaking		18.9	1.78
After test	Top 30mm of sample	19.5	-
	Remainder of sample	19.5	-
Field values		14.1	-
Standard Compaction		16.3	1.81

RESULTS		
TYPE	PENETRATION	CBR (%)
TOP	**2.5 mm**	**11**
	5.0 mm	**10**

NATA Accredited Laboratory Number 828

Approved Signatory:

Tested:	TF
Checked:	SJ

Srdjan Jajcanin
Senior Soil Technician

APPENDIX C

Topsoil Report by
Environmental Soil Solutions Australia Pty Ltd

Environmental Soil Solutions
Australia Pty Ltd

ACN: 090 697 331 ABN 61 829 722 881

PO BOX 442 SUNNYBANK 4109 PH: (07) 33458238 FAX: (07) 33451390 MOB: 0403245560 E- e.s.s.a@bigpond.net.au

August 9, 2007

Soil Report, For Douglas Partners

Soil Properties Assessment
Gatton Correctional Centre

Job No 07 – 237

Attention: Chris Bell

D Baker BSc

Soil Sample Suitability Assessment (Planting Media) Soils, Douglas Partners – Gatton Correctional Centre

Introduction

Mr. C Bell (DP) requested Environmental Soil Solutions Australia Pty/Ltd (ESSA) to assess the soil characteristics, sodicity, nutrients (including organic matter) of soil samples sent from soil assessment at Gatton Correctional Centre, Gatton Site project for preliminary assessment using the modified Q Build suite testing. The brief was to comment on soil properties as directed by Douglas Partners and comment on any soil factors, which may affect its use as planting media and any suggested amendments.

Laboratory analysis, to assess the full range of soil properties relating to the soil characteristics as potential planting media, was requested by Douglas Partners (DP). Douglas Partners sent a total of Four (4) blended bulked soil sample to ESSA for assessment.

Soil sample was received by ESSA 3 August 2007 and sent to Phosyn Analytical and ESSA on 3 August 2007 for a range of analysis to assess the soil characteristics requested by DP of the above – mentioned samples. Phosyn and ESSA Soil Reports for the 4 samples (batch B03893 A to D (Phosyn) is dated 8 August 2007. Phosyn report is No B030893 A to D / Laboratory Numbers SI1621 to SI 1624. The Laboratory numbers corresponding to Test sample sites and depths is listed in Table 1. Data sheets are attached to appendix of this report.

NOTE

- SI 1621 = BH8 Gatton Correctional Centre, Gatton
- SI 1622 = BH 11 Gatton Correctional Centre, Gatton
- SI 1623 = BH 12 Gatton Correctional Centre, Gatton
- SI 1624 = BH 33 Gatton Correctional Centre, Gatton

The soil properties (soil test parameters) assessed were:

- Soil pH and salt
- Soil Cations (calcium, magnesium, sodium, potassium & Aluminum)
- Nitrate -nitrogen
- Organic Matter,
- Phosphorus,

- Fertility (N, P, K)
- Trace Elements (Cu, Zn, Mn&Fe)
- Sulfate
- Texture
- Boron

Method:

- Site soil samples from the site were sampled by Douglas Partners to characterize the soil for laboratory analysis for a range of analytes as listed in this report and to comment on the suitability as growing media.

In addition, Calcium to Magnesium ratio ESP (% Sodium /ECEC) and ECEC as defined in Table 1.

Methods used for the assessment followed Australian Laboratory Handbook of Soil and Water Chemical Methods (1992) by Rayment & Higgenson. The methods used are recognized Australia wide as consistent methods and procedures that are used for diagnostic and research purposes.

Comment on Results

Introduction

Chemical data for the sample submitted was examined and the following comments are provided for the site. The soil properties are commented on in the report.

Soil pH

pH (1:5 Water) for Site soils is 5.3 to 4.9 for BH's 8 to 33. pH for the site soils is rated very strongly to strongly acid.

Strongly Acid pH soils such as this with sandy, loam texture are medium to strongly buffered and plant nutrient availability will be NOT be ideal for site soil. The ideal pH range for growing media for most crops/ grasses is pH 6.5-7.5.

- Agricultural Lime/dolomite amendment IS required based on the pH test. For strongly acid pH soils soil nutrient availability will be not ideal for plants. The recommendation for this soil with lime / dolomite modification is made. The recommendation for this soil with agricultural grade dolomite modification to correct soil pH & sodicity is made. Rates are given in recommendations.

Salts

EC (electrical conductivity) and chloride indicate salt content. Using general ratings salt concentration is:

- Rated very low

The very low rated soil is satisfactory for Site-soil, garden soil and under turf soil uses and will not adversely affect plant growth for most plants.

Soil Cations

Calcium to magnesium (Ca/Mg) ratios are greater than 1 for BH's 8,11 & 33 while BH 12 is 0.5and is heavier in texture. If Ca to Mg ratio less than 1 or greater than 6 they are not rated as ideal. These samples have ratios Ca/Mg of 2 for BH 8,11 &33, which is satisfactory BUT BH 12 is low at 0.5.

ESP is a soil property measurement that helps to indicate a soil's tendency to disperse and have a tendency to lose aggregation, impermeability, and surface crusting and poor aeration. Values greater than ESP (corrected for soluble chloride) of > 6 (range 11 - 13) is rated at not satisfactory, as values of ESP greater than 6 are rated sodic.

On that basis this soil would have moderate tendency to disperse, as ESP is >6 (corrected for soluble chloride). However as clay & silt percentage of especially BH's 8,11 &33 is low is not expected to be a major difficulty , however BH12 is heavier and may be of concern. The addition of agricultural dolomite as well as correcting the acid pH will reduce the effects of sodicity.

Calcium levels are low and not adequate for plant growth while Magnesium levels are low. Both cations do need boosting.

Texture

Textural analysis classes these soils as loamy sand for BH's 8,11 & 33 and sandy clay loam (brown) for BH 12.

Data assessed for the bulk sample (Ca/Mg, ESP) indicate that this soil silt and clay may be low to moderate dispersible. Dispersive effects of silt and clay in the samples are however rated low in garden and under turf applications but BH 12 especially on batters.

Fertility –

Soil Nitrogen (N), Phosphorus (P) and Potassium (K) & others have been assessed.
In summary,

- N is *very low* and will need to be applied
- Phosphorus is *very low*–Sample may suit P sensitive native plants but additional P will need to be added for general plantings and especially turf grass.
- Potassium (K), is *very low* and will need to be added for healthy plants especially turf grass
- Sulfur S, is low and would be added in applied fertilizer
- Boron B is *low*

Trace Elements

Trace elements were assessed for this soil and in summary the results are:

- Zinc (Zn) is *medium* and will not need boosting
- Copper (Cu) is rated *low*
- Manganese (Mn) is rated *medium*
- Iron (Fe) is rated *medium*
- Boron (B) *is low*

Any fertiliser amendment would need to contain trace element but also contain Nitrogen (N), Phosphorus (P), Potassium (K) and trace elements boosted for most plants.

Organic Matter

Organic Matter is not adequate at <1.1 % .The AS 4419 (2003) figure for plant growth is only satisfactory when a minimum of 3% is required.

Incorporation of well-composted organic matter is required, but would be of benefit if incorporated compost @ minimum 20% by volume in top 100mm. If organic or slow release fertilizers are applied this will ensure nutrients are not leached from the root zone. Incorporation of well composted organic matter (NDI > 0.5) at min'm is an option at 20 % by volume for all site soils assessed for turf and garden areas. This will help retain nutrients, reduce water required and increase success and sustainability of the landscaping.

Summary

- The site soils have strongly acid pH, low N, P, K and Trace (especially Boron) fertility and non adequate organic matter that are generally associated with sandy soils in this district. Application of appropriate agricultural dolomite /lime (dolomite preferred) and fertilizer (and composted organics) is recommended.
- Dolomite rate suggested is 4-5kg/m3. Dolomite should be well mixed with the soils to be amended. In addition, any subsoil is strongly recommended be tested for minimum pH, EC & cations and amended if required to ensure it is suitable for plant root development.

Application of appropriate fertilizer is suggested. Incorporation of

- Appropriate fertilizer is recommended according to the proposed plantings. For other than native low fertilizer requirement plants only starter fertilizer is required while for most other plants fertilizer rates of 300 – 400kg/ha (3 – 4 kg/m3) are suggested. Controlled release fertilizer (CRF's , such as Scotts Sierrablen)) are recommended as organic or non coated products may dissolve and enter the surface or groundwater systems.

NOTE : The soil with amendments suggested (fertilizer) and irrigation management should provide a reasonable growing medium for plants & turf.

If you have any queries please contact me.

D E Baker BSc
MASSSI , IUSS

APPENDIX

Soil Analysis Reports
PHOSYN

- B030893
- Laboratory Number SI 1621 to SI 1624, August 8 , 2007

PHOSYN
ANALYTICAL

Phosyn Analytical, 1/60 Junction Road,
Andrews, Queensland 4220, Australia
Tel: +61 7 5568 8700, Fax: +61 7 5522 0720
email: phosynanalytical@phosyn.com

Analysis Results (SOIL)

Customer : DOUGLAS PTNS
GATTON C C

Distributor : ENVIRONMENTAL SOIL SOLUTIONS
5 DUNPHY ST
SUNNYBANK HILLS
QLD

Sample Ref : BH 8

Date Received : 06/08/07

Sample No : B030893A / SI1621

Crop : DATA ONLY

Analysis	Result
pH [H2O]	5.3
pH [CaCl2]	4.5
Organic Matter (%)	1.1
CEC (meq/100g)	4.3
EC (dS/m)	0.11
NO3-N (ppm)	<1.0
Phosphorus [Olsen] (ppm)	7
Potassium (meq/100g).	0.19
Calcium (meq/100g)	2.42
Magnesium (meq/100g)	1.01
Sulphur (ppm)	8
Boron (ppm)	0.3
Copper (ppm)	0.3
Iron (ppm)	31
Manganese (ppm)	11.1
Zinc (ppm)	0.5
Aluminium (meq/100g)	0.10
Sodium (meq/100g)	0.5
Chloride (ppm)	111
Ca base saturation (%)	56.9
K base saturation (%)	4.5
Mg base saturation (%)	23.8
Na base saturation (%)	12.5
Ca:Mg Ratio	2.4
Al base saturation (%)	2.4

Date Printed : 8 Aug 2007

Analysis Results (SOIL)

Sample Ref : BH 8
Sample No : B030893A

Additional Comments

You should consult your local agronomist and/or Phosyn representative before deciding upon any course of action based on this report.

Calcium (Ca): 1 meq/100g equals 200ppm
Magnesium (Mg): 1 meq/100g equals 120ppm
Sodium (Na): 1 meq/100g equals 230 ppm
Potassium (K): 1 meq/100g equals 390 ppm
Aluminium (Al): 1 meq/100g equals 90 ppm

Date Printed : 8 Aug 2007

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Analysis Results (SOIL)

Customer : DOUGLAS PTNS
GATTON C C

Distributor : ENVIRONMENTAL SOIL SOLUTIONS
5 DUNPHY ST
SUNNYBANK HILLS
QLD

Sample Ref : BH 11

Sample No : B030893B / SI1622

Crop : DATA ONLY

Date Received : 06/08/07

Analysis	Result
pH [H2O]	5.0
pH [CaCl2]	4.3
Organic Matter (%)	0.7
CEC (meq/100g)	3.4
EC (dS/m)	0.10
NO3-N (ppm)	<1.0
Phosphorus [Olsen] (ppm)	3
Potassium (meq/100g)	0.10
Calcium (meq/100g)	1.88
Magnesium (meq/100g)	0.84
Sulphur (ppm)	6
Boron (ppm)	0.2
Copper (ppm)	0.2
Iron (ppm)	21
Manganese (ppm)	8.1
Zinc (ppm)	0.4
Aluminium (meq/100g)	0.12
Sodium (meq/100g)	0.4
Chloride (ppm)	119
Ca base saturation (%)	55.6
K base saturation (%)	3.0
Mg base saturation (%)	24.9
Na base saturation (%)	13.0
Ca:Mg Ratio	2.2
Al base saturation (%)	3.6

Date Printed : 8 Aug 2007

Analysis Results (SOIL)

Sample Ref : BH 11
Sample No : B030893B

Additional Comments

You should consult your local agronomist and/or Phosyn representative before deciding upon any course of action based on this report.

Calcium (Ca): 1 meq/100g equals 200ppm
Magnesium (Mg): 1 meq/100g equals 120ppm
Sodium (Na): 1 meq/100g equals 230 ppm
Potassium (K): 1 meq/100g equals 390 ppm
Aluminium (Al): 1 meq/100g equals 90 ppm

Date Printed : 8 Aug 2007

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Analysis Results (SOIL)

Customer : DOUGLAS PTNS
GATTON C C

Distributor : ENVIRONMENTAL SOIL SOLUTIONS
5 DUNPHY ST
SUNNYBANK HILLS
QLD

Sample Ref : BH 12

Sample No : B030893C / SI1623

Crop : DATA ONLY

Date Received : 06/08/07

Page Number
1 / 2

Analysis	Result
pH [H2O]	4.9
pH [CaCl2]	4.3
Organic Matter (%)	0.7
CEC (meq/100g)	3.5
EC (dS/m)	0.01
NO3-N (ppm)	<1.0
Phosphorus [Olsen] (ppm)	4
Potassium (meq/100g)	0.13
Calcium (meq/100g)	0.74
Magnesium (meq/100g)	1.64
Sulphur (ppm)	4
Boron (ppm)	0.2
Copper (ppm)	0.3
Iron (ppm)	56
Manganese (ppm)	1.8
Zinc (ppm)	0.7
Aluminium (meq/100g)	0.54
Sodium (meq/100g)	0.5
Chloride (ppm)	11
Ca base saturation (%)	20.9
K base saturation (%)	3.7
Mg base saturation (%)	46.3
Na base saturation (%)	13.8
Ca:Mg Ratio	0.5
Al base saturation (%)	15.3

Date Printed : 8 Aug 2007

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Analysis Results (SOIL)

Sample Ref : BH 12
Sample No : B030893C

Page Number
2 / 2

Additional Comments

You should consult your local agronomist and/or Phosyn representative before deciding upon any course of action based on this report.

Calcium (Ca): 1 meq/100g equals 200ppm
Magnesium (Mg): 1 meq/100g equals 120ppm
Sodium (Na): 1 meq/100g equals 230 ppm
Potassium (K): 1 meq/100g equals 390 ppm
Aluminium (Al): 1 meq/100g equals 90 ppm

Date Printed : 8 Aug 2007

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Analysis Results (SOIL)

Customer : DOUGLAS PTNS
GATTON C C

Distributor : ENVIRONMENTAL SOIL SOLUTIONS
5 DUNPHY ST
SUNNYBANK HILLS
QLD

Sample Ref : BH 33

Date Received : 06/08/07

Sample No : B030893D / SI1624

Page Number
1 / 2

Crop : DATA ONLY

Analysis	Result
pH [H2O]	4.9
pH [CaCl2]	4.2
Organic Matter (%)	0.9
CEC (meq/100g)	2.9
EC (dS/m)	0.05
NO3-N (ppm)	<1.0
Phosphorus [Olsen] (ppm)	6
Potassium (meq/100g)	0.10
Calcium (meq/100g)	1.52
Magnesium (meq/100g)	0.71
Sulphur (ppm)	5
Boron (ppm)	0.2
Copper (ppm)	0.2
Iron (ppm)	30
Manganese (ppm)	6.5
Zinc (ppm)	0.3
Aluminium (meq/100g)	0.20
Sodium (meq/100g)	0.4
Chloride (ppm)	42
Ca base saturation (%)	52.8
K base saturation (%)	3.5
Mg base saturation (%)	24.7
Na base saturation (%)	12.2
Ca:Mg Ratio	2.1
Al base saturation (%)	6.9

Date Printed : 8 Aug 2007

Analysis Results (SOIL)

Sample Ref : BH 33
Sample No : B030893D

Additional Comments

You should consult your local agronomist and/or Phosyn representative before deciding upon any course of action based on this report.

Calcium (Ca): 1 meq/100g equals 200ppm
Magnesium (Mg): 1 meq/100g equals 120ppm
Sodium (Na): 1 meq/100g equals 230 ppm
Potassium (K): 1 meq/100g equals 390 ppm
Aluminium (Al): 1 meq/100g equals 90 ppm

Date Printed : 8 Aug 2007

APPENDIX D

CSIRO Guidelines (BTF 18)

Foundation Maintenance and Footing Performance: A Homeowner's Guide

CSIRO

BTF 18
replaces
Information
Sheet 10/91

Buildings can and often do move. This movement can be up, down, lateral or rotational. The fundamental cause of movement in buildings can usually be related to one or more problems in the foundation soil. It is important for the homeowner to identify the soil type in order to ascertain the measures that should be put in place in order to ensure that problems in the foundation soil can be prevented, thus protecting against building movement.

This Building Technology File is designed to identify causes of soil-related building movement, and to suggest methods of prevention of resultant cracking in buildings.

Soil Types

The types of soils usually present under the topsoil in land zoned for residential buildings can be split into two approximate groups – granular and clay. Quite often, foundation soil is a mixture of both types. The general problems associated with soils having granular content are usually caused by erosion. Clay soils are subject to saturation and swell/shrink problems.

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870, the Residential Slab and Footing Code.

Causes of Movement

Settlement due to construction

There are two types of settlement that occur as a result of construction:

- Immediate settlement occurs when a building is first placed on its foundation soil, as a result of compaction of the soil under the weight of the structure. The cohesive quality of clay soil mitigates against this, but granular (particularly sandy) soil is susceptible.
- Consolidation settlement is a feature of clay soil and may take place because of the expulsion of moisture from the soil or because of the soil's lack of resistance to local compressive or shear stresses. This will usually take place during the first few months after construction, but has been known to take many years in exceptional cases.

These problems are the province of the builder and should be taken into consideration as part of the preparation of the site for construction. Building Technology File 19 (BTF 19) deals with these problems.

Erosion

All soils are prone to erosion, but sandy soil is particularly susceptible to being washed away. Even clay with a sand component of say 10% or more can suffer from erosion.

Saturation

This is particularly a problem in clay soils. Saturation creates a bog-like suspension of the soil that causes it to lose virtually all of its bearing capacity. To a lesser degree, sand is affected by saturation because saturated sand may undergo a reduction in volume – particularly imported sand fill for bedding and blinding layers. However, this usually occurs as immediate settlement and should normally be the province of the builder.

Seasonal swelling and shrinkage of soil

All clays react to the presence of water by slowly absorbing it, making the soil increase in volume (see table below). The degree of increase varies considerably between different clays, as does the degree of decrease during the subsequent drying out caused by fair weather periods. Because of the low absorption and expulsion rate, this phenomenon will not usually be noticeable unless there are prolonged rainy or dry periods, usually of weeks or months, depending on the land and soil characteristics.

The swelling of soil creates an upward force on the footings of the building, and shrinkage creates subsidence that takes away the support needed by the footing to retain equilibrium.

Shear failure

This phenomenon occurs when the foundation soil does not have sufficient strength to support the weight of the footing. There are two major post-construction causes:

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.
- In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes
H	Highly reactive clay sites, which can experience high ground movement from moisture changes
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes
A to P	Filled sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

Tree root growth
Trees and shrubs that are allowed to grow in the vicinity of footings can cause foundation soil movement in two ways:

- Roots that grow under footings may increase in cross-sectional size, exerting upward pressure on footings.
- Roots in the vicinity of footings will absorb much of the moisture in the foundation soil, causing shrinkage or subsidence.

Unevenness of Movement

The types of ground movement described above usually occur unevenly throughout the building's foundation soil. Settlement due to construction tends to be uneven because of:

- Differing compaction of foundation soil prior to construction.
- Differing moisture content of foundation soil prior to construction.

Movement due to non-construction causes is usually more uneven still. Erosion can undermine a footing that traverses the flow or can create the conditions for shear failure by eroding soil adjacent to a footing that runs in the same direction as the flow.

Saturation of clay foundation soil may occur where subfloor walls create a dam that makes water pond. It can also occur wherever there is a source of water near footings in clay soil. This leads to a severe reduction in the strength of the soil which may create local shear failure.

Seasonal swelling and shrinkage of clay soil affects the perimeter of the building first, then gradually spreads to the interior. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Swelling gradually reaches the interior soil as absorption continues. Shrinkage usually begins where the sun's heat is greatest.

Effects of Uneven Soil Movement on Structures

Erosion and saturation
Erosion removes the support from under footings, tending to create subsidence of the part of the structure under which it occurs. Brickwork walls will resist the stress created by this removal of support by bridging the gap or cantilevering until the bricks or the mortar bedding fail. Older masonry has little resistance. Evidence of failure varies according to circumstances and symptoms may include:

- Step cracking in the mortar beds in the body of the wall or above/below openings such as doors or windows.
- Vertical cracking in the bricks (usually but not necessarily in line with the vertical beds or perpends).

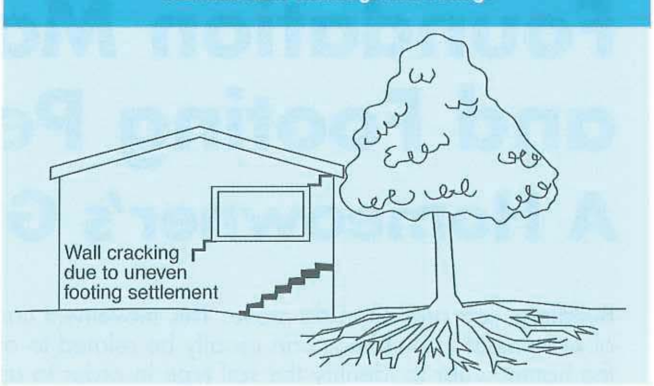
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the external footings are pushed higher than the internal ones.

The first noticeable symptom may be that the floor appears slightly dished. This is often accompanied by some doors binding on the floor or the door head, together with some cracking of cornice mitres. In buildings with timber flooring supported by bearers and joists, the floor can be bouncy. Externally there may be visible dishing of the hip or ridge lines.

As the moisture absorption process completes its journey to the innermost areas of the building, the internal footings will rise. If the spread of moisture is roughly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

Trees can cause shrinkage and damage



As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun's effect is strongest. This has the effect of lowering the external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

Doming and dishing are also affected by weather in other ways. In areas where warm, wet summers and cooler dry winters prevail, water migration tends to be toward the interior and doming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensity is toward dishing.

Movement caused by tree roots
In general, growing roots will exert an upward pressure on footings, whereas soil subject to drying because of tree or shrub roots will tend to remove support from under footings by inducing shrinkage.

Complications caused by the structure itself
Most forces that the soil causes to be exerted on structures are vertical – i.e. either up or down. However, because these forces are seldom spread evenly around the footings, and because the building resists uneven movement because of its rigidity, forces are exerted from one part of the building to another. The net result of all these forces is usually rotational. This resultant force often complicates the diagnosis because the visible symptoms do not simply reflect the original cause. A common symptom is binding of doors on the vertical member of the frame.

Effects on full masonry structures
Brickwork will resist cracking where it can. It will attempt to span areas that lose support because of subsided foundations or raised points. It is therefore usual to see cracking at weak points, such as openings for windows or doors.

In the event of construction settlement, cracking will usually remain unchanged after the process of settlement has ceased.

With local shear or erosion, cracking will usually continue to develop until the original cause has been remedied, or until the subsidence has completely neutralised the affected portion of footing and the structure has stabilised on other footings that remain effective.

In the case of swell/shrink effects, the brickwork will in some cases return to its original position after completion of a cycle, however it is more likely that the rotational effect will not be exactly reversed, and it is also usual that brickwork will settle in its new position and will resist the forces trying to return it to its original position. This means that in a case where swelling takes place after construction and cracking occurs, the cracking is likely to at least partly remain after the shrink segment of the cycle is complete. Thus, each time the cycle is repeated, the likelihood is that the cracking will become wider until the sections of brickwork become virtually independent.

With repeated cycles, once the cracking is established, if there is no other complication, it is normal for the incidence of cracking to stabilise, as the building has the articulation it needs to cope with the problem. This is by no means always the case, however, and monitoring of cracks in walls and floors should always be treated seriously.

Upheaval caused by growth of tree roots under footings is not a simple vertical shear stress. There is a tendency for the root to also exert lateral forces that attempt to separate sections of brickwork after initial cracking has occurred.

The normal structural arrangement is that the inner leaf of brickwork in the external walls and at least some of the internal walls (depending on the roof type) comprise the load-bearing structure on which any upper floors, ceilings and the roof are supported. In these cases, it is internally visible cracking that should be the main focus of attention, however there are a few examples of dwellings whose external leaf of masonry plays some supporting role, so this should be checked if there is any doubt. In any case, externally visible cracking is important as a guide to stresses on the structure generally, and it should also be remembered that the external walls must be capable of supporting themselves.

Effects on framed structures
Timber or steel framed buildings are less likely to exhibit cracking due to swell/shrink than masonry buildings because of their flexibility. Also, the doming/dishing effects tend to be lower because of the lighter weight of walls. The main risks to framed buildings are encountered because of the isolated pier footings used under walls. Where erosion or saturation cause a footing to fall away, this can double the span which a wall must bridge. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by a door or window opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage without first exhibiting some or all of the above symptoms for a considerable period. The same warning period should apply in the case of upheaval. It should be noted, however, that where framed buildings are supported by strip footings there is only one leaf of brickwork and therefore the externally visible walls are the supporting structure for the building. In this case, the subfloor masonry walls can be expected to behave as full brickwork walls.

Effects on brick veneer structures
Because the load-bearing structure of a brick veneer building is the frame that makes up the interior leaf of the external walls plus perhaps the internal walls, depending on the type of roof, the building can be expected to behave as a framed structure, except that the external masonry will behave in a similar way to the external leaf of a full masonry structure.

Water Service and Drainage

Where a water service pipe, a sewer or stormwater drainage pipe is in the vicinity of a building, a water leak can cause erosion, swelling or saturation of susceptible soil. Even a minuscule leak can be enough to saturate a clay foundation. A leaking tap near a building can have the same effect. In addition, trenches containing pipes can become watercourses even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, interstrata seepage into subfloor areas and saturation.

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem.
Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.
- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870.

AS 2870 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

Prevention/Cure

Plumbing
Where building movement is caused by water service, roof plumbing, sewer or stormwater failure, the remedy is to repair the problem. It is prudent, however, to consider also rerouting pipes away from the building where possible, and relocating taps to positions where any leakage will not direct water to the building vicinity. Even where gully traps are present, there is sometimes sufficient spill to create erosion or saturation, particularly in modern installations using smaller diameter PVC fixtures. Indeed, some gully traps are not situated directly under the taps that are installed to charge them, with the result that water from the tap may enter the backfilled trench that houses the sewer piping. If the trench has been poorly backfilled, the water will either pond or flow along the bottom of the trench. As these trenches usually run alongside the footings and can be at a similar depth, it is not hard to see how any water that is thus directed into a trench can easily affect the foundation's ability to support footings or even gain entry to the subfloor area.

Ground drainage
In all soils there is the capacity for water to travel on the surface and below it. Surface water flows can be established by inspection during and after heavy or prolonged rain. If necessary, a grated drain system connected to the stormwater collection system is usually an easy solution.

It is, however, sometimes necessary when attempting to prevent water migration that testing be carried out to establish watertable height and subsoil water flows. This subject is referred to in BTF 19 and may properly be regarded as an area for an expert consultant.

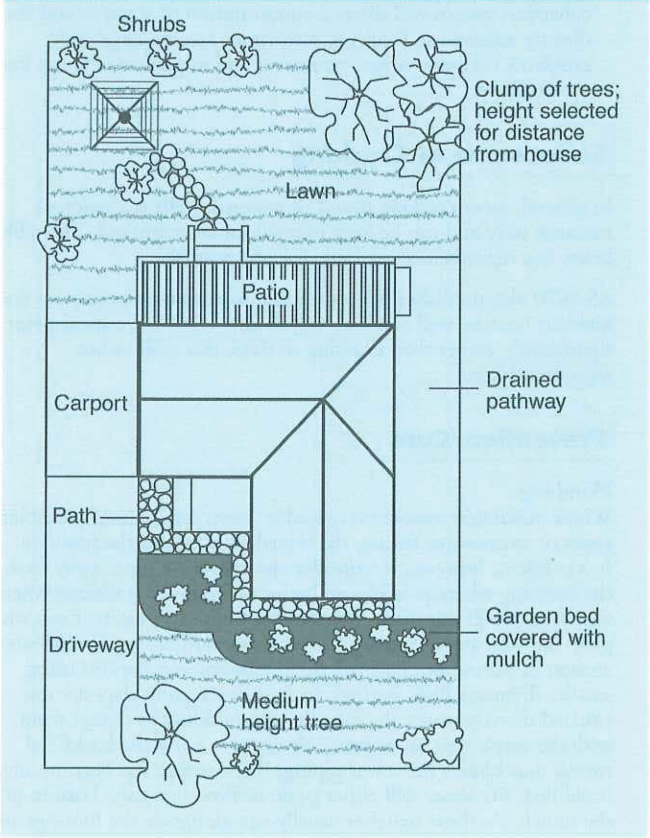
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

For this reason, particularly where problems exist or are likely to occur, it is recommended that an apron of paving be installed around as much of the building perimeter as necessary. This paving

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

Description of typical damage and required repair	Approximate crack width limit (see Note 3)	Damage category
Hairline cracks	<0.1 mm	0
Fine cracks which do not need repair	<1 mm	1
Cracks noticeable but easily filled. Doors and windows stick slightly	<5 mm	2
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weathertightness often impaired	5–15 mm (or a number of cracks 3 mm or more in one group)	3
Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted	15–25 mm but also depend on number of cracks	4

Gardens for a reactive site



should extend outwards a minimum of 900 mm (more in highly reactive soil) and should have a minimum fall away from the building of 1:60. The finished paving should be no less than 100 mm below brick vent bases.

It is prudent to relocate drainage pipes away from this paving, if possible, to avoid complications from future leakage. If this is not practical, earthenware pipes should be replaced by PVC and backfilling should be of the same soil type as the surrounding soil and compacted to the same density.

Except in areas where freezing of water is an issue, it is wise to remove taps in the building area and relocate them well away from the building – preferably not uphill from it (see BTF 19).

It may be desirable to install a grated drain at the outside edge of the paving on the uphill side of the building. If subsoil drainage is needed this can be installed under the surface drain.

Condensation

In buildings with a subfloor void such as where bearers and joists support flooring, insufficient ventilation creates ideal conditions for condensation, particularly where there is little clearance between the floor and the ground. Condensation adds to the moisture already present in the subfloor and significantly slows the process of drying out. Installation of an adequate subfloor ventilation system, either natural or mechanical, is desirable.

Warning: Although this Building Technology File deals with cracking in buildings, it should be said that subfloor moisture can result in the development of other problems, notably:

- Water that is transmitted into masonry, metal or timber building elements causes damage and/or decay to those elements.
- High subfloor humidity and moisture content create an ideal environment for various pests, including termites and spiders.
- Where high moisture levels are transmitted to the flooring and walls, an increase in the dust mite count can ensue within the living areas. Dust mites, as well as dampness in general, can be a health hazard to inhabitants, particularly those who are abnormally susceptible to respiratory ailments.

The garden

The ideal vegetation layout is to have lawn or plants that require only light watering immediately adjacent to the drainage or paving edge, then more demanding plants, shrubs and trees spread out in that order.

Overwatering due to misuse of automatic watering systems is a common cause of saturation and water migration under footings. If it is necessary to use these systems, it is important to remove garden beds to a completely safe distance from buildings.

Existing trees

Where a tree is causing a problem of soil drying or there is the existence or threat of upheaval of footings, if the offending roots are subsidiary and their removal will not significantly damage the tree, they should be severed and a concrete or metal barrier placed vertically in the soil to prevent future root growth in the direction of the building. If it is not possible to remove the relevant roots without damage to the tree, an application to remove the tree should be made to the local authority. A prudent plan is to transplant likely offenders before they become a problem.

Information on trees, plants and shrubs

State departments overseeing agriculture can give information regarding root patterns, volume of water needed and safe distance from buildings of most species. Botanic gardens are also sources of information. For information on plant roots and drains, see Building Technology File 17.

Excavation

Excavation around footings must be properly engineered. Soil supporting footings can only be safely excavated at an angle that allows the soil under the footing to remain stable. This angle is called the angle of repose (or friction) and varies significantly between soil types and conditions. Removal of soil within the angle of repose will cause subsidence.

Remediation

Where erosion has occurred that has washed away soil adjacent to footings, soil of the same classification should be introduced and compacted to the same density. Where footings have been undermined, augmentation or other specialist work may be required. Remediation of footings and foundations is generally the realm of a specialist consultant.

Where isolated footings rise and fall because of swell/shrink effect, the homeowner may be tempted to alleviate floor bounce by filling the gap that has appeared between the bearer and the pier with blocking. The danger here is that when the next swell segment of the cycle occurs, the extra blocking will push the floor up into an accentuated dome and may also cause local shear failure in the soil. If it is necessary to use blocking, it should be by a pair of fine wedges and monitoring should be carried out fortnightly.

This BTF was prepared by John Lewer FAIB, MIAMA, Partner, Construction Diagnosis.

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