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Third Draft

Additional Geotechnical Investigation Herston Quarter Redevelopment – Northern Car Park

Research Road, Herston

Prepared for
Herston Development Co Pty Ltd
Project No. 017-141C

29 October 2018

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geotechnical • geo-environmental • groundwater

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SECTION 1 - INTRODUCTION

1.1 ***Introduction***

It is understood that it is proposed to develop the site by the construction of a nine level Northern Car Park (NCP) structure, with the provision for a future three to six level building to be situated above the car park. The NCP structure is further understood to have a lowest finished slab level (FSL) of approximately RL6.0m, with bulk excavations proposed to RL5.35m to RL10.5m requiring excavation to approximately 20m depth into the side of an existing steep slope and that the excavated face is to be supported independently of the car park structure. Maximum building column working loads of 20MN approximately are advised. The location and extent of the site are shown approximately on Drawing No. 1, attached.

1.2 ***Proposed Scope of Work***

Based on discussions held on site on 6 February and 21 March 2018 between Butler Partners Pty Ltd (Butler Partners) and the principal contractor for the project Watpac Construction Pty Ltd (Watpac), and Watpac's emails of 7 and 23 February and 22 March 2018, additional geotechnical investigation and/or geotechnical consultancy advice was required for the following items:

Stage 1C Services Diversion Adjacent to Building B52

It was understood that diversion of the existing Back Road services over the south-east corner of the site is required to allow for construction of the NCP structure to occur. It is also understood construction options for the services diversion and retention support options for the adjacent NCP excavation is limited, due to the existing steep slope and close proximity of an existing elevated building (i.e. Building B52) located adjacent to the proposed excavation face, and geotechnical analysis and advice on the preferred retention support option for the NCP excavation in this area was subsequently required.

Future MNHHS 'Connection' Building

It was further understood that a future MNHHS 'connection' building is being considered adjacent to and south of the NCP structure and excavation, and geotechnical analysis and advice on the design of the retention support for the NCP excavation was required to accommodate and allow for future surcharge loading associated with construction and design of the MNHHS building. It was also understood that the building will be provisionally 10m in width and 70m in length and could be up to three levels in height, and have provisional column working loads of 3,000kN vertical and 300kN horizontal.

To be able to undertake geotechnical analysis and provide advice on the above items, it was proposed to undertake additional geotechnical investigation of the site by the drilling and sampling of one bore in the area of the proposed services diversion and one bore at (or close to) the centre of the proposed MNHHS building to approximately 20m depth.

NCP Structure Foundations

It was proposed to undertake additional geotechnical investigation over the northern area of the site by the drilling and sampling of two bores to approximately 20m depth in the existing car park and adjacent to a fire training area, for the purpose of providing additional geotechnical information for design of the proposed NCP structure foundations.

Building C28

It was also proposed to undertake additional geotechnical investigation of the ground conditions adjacent to the existing Building C28, by the drilling and sampling of one bore to approximately 20m depth, for the purpose of providing additional geotechnical information and undertaking geotechnical analysis for design of the proposed NCP excavation retention system to be located adjacent to the Building C28 foundations.

1.3 Commission

Based on the proposed nature of the development, the anticipated subsurface conditions and the scope of work provided, a fee to undertake the additional geotechnical investigation was presented in a proposal to Watpac dated 22 March 2018. Butler Partners was subsequently commissioned by Herston Development Co Pty Ltd to undertake the additional geotechnical investigation as proposed (based on a scope of work proposed by Watpac), which was undertaken in general consultation with Calibre Consulting Pty Ltd (Calibre), structural engineers for the development. An initial draft of this report was first issued for comment on 29 June 2018, prior to completion of excavation retention analysis for the development and a second draft of the report was issued on 17 August 2018, following completion of the analysis for Stage 1C (Services Diversion). Subsequent to the issue of the 17 August 2018 report, additional retention analysis was requested to be conducted for the proposed future MNHHS ‘Connection’ Building at Section 4 on attached Drawing No. 1. This current report supersedes the 17 August 2018 report.

SECTION 2 - THE SITE

2.1 Background

2.1.1 Geotechnical Investigation

The results of the initial geotechnical investigation, comprising the drilling and sampling of six bores (Bores 1 to 6), is given in Butler Partners's following draft report:

> *Draft Geotechnical Investigation*
> *Herston Quarter Redevelopment – Northern Car Park*
> *Research Road, Herston*
> Project No.: 017-141B Dated: 24 November 2017

For completeness, all results from the initial investigation are included herein; this current report supersedes the 24 November 2017 report.

2.1.2 Environmental Site Assessment

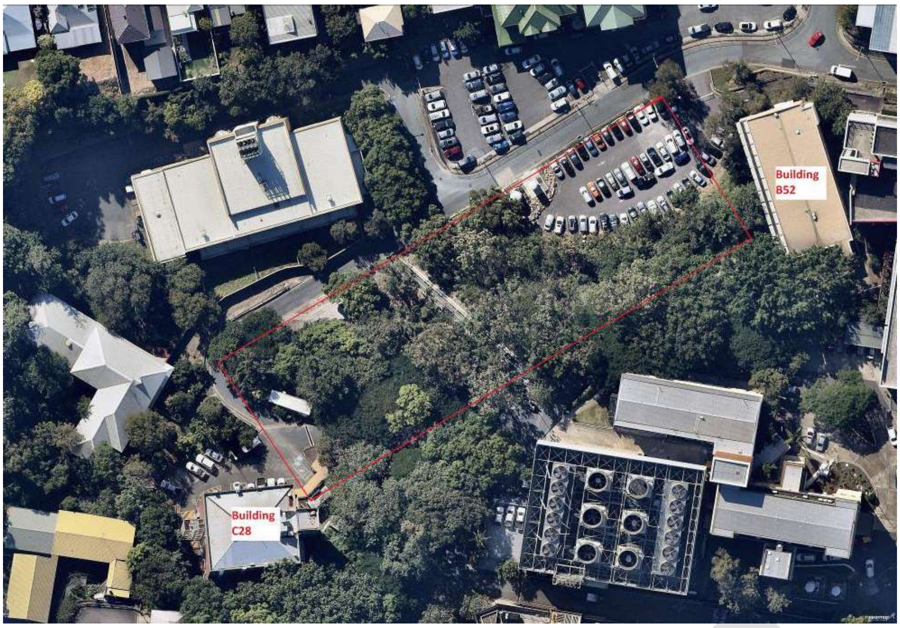
Butler Partners has also previously undertaken a contamination assessment of the overall Herston Quarter Redevelopment site (including the NCP site) and the results are given in the following report:

> *Environmental Site Assessment*
> *Herston Quarter Development*
> *300 Herston Road, Herston*
> Project No.: 017-141A Dated: 19 January 2018

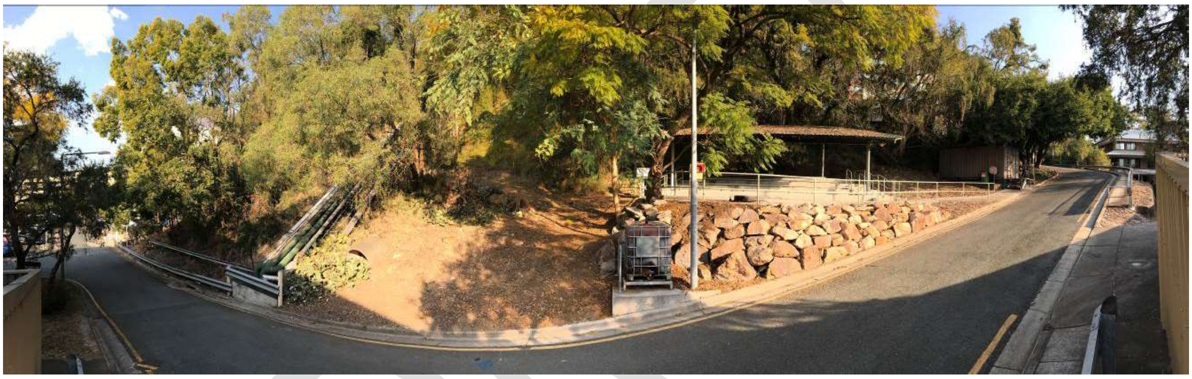
As part of the current (additional) geotechnical investigation, groundwater level was measured in a groundwater monitoring well installed in Bore 66 (previously drilled and sampled as part of the above investigation), located in the south-western corner of the site adjacent to Building C28.

2.2 Site Description

At the time of the investigation, the site was generally underdeveloped, except for a sealed car park located at the north-eastern corner of the site, a concrete covered fire training area directly south-west of the car park, and Research Road and Back Road running along the northern, western and southern boundaries of the site. Building B52 is located adjacent to the eastern boundary of the site and Building C28 is located adjacent to the south-western corner of the site. The remainder of the site was covered with trees and the ground surface level sloped non-uniformly downwards generally to the north-west and north-east from a high of approximately RL30m midway along the southern boundary down to a low of approximately RL4.5m at the north-eastern corner of the site. An aerial view of the site close to the time of investigation is given in Photograph 1, and general views of the site at the time of the investigation are given in Photograph 2 to Photograph 7.



Photograph 1: Aerial view of the site (Source Nearmap, accessed 19 May 2018)



Photograph 2: Panoramic view of the site looking south-east from near Bores 3/3A (Fire Training Area)

Photograph 3: General view of the site looking south from near Bores 1 and 9 (existing car park)

Photograph 4: General view along Back Road looking east from near Bore 7

Photograph 5: General view along Research Road looking west from near Bore 5

Photograph 6: General view of Building C28 looking south-west from near Bore 8

Photograph 7: General view of the south-east corner of the site looking east towards Building B52 from near Bore 6

2.3 Geology

Reference to the Geotechnical Survey of Queensland's 1:31,680 geological series City of Brisbane – Sheet 3 indicates that the site is located in an area mapped as Neranleigh-Fernvale 'Group' (comprising greywacke, siltstone, shale, chect, jasper and basic volcanics), close to a boundary with (overlying) Brisbane Tuff (comprising rhyolitic tuff, conglomerate, breccia with minor sandstone and shale).

SECTION 3 - FIELDWORK

3.1 Drilling and Sampling Methods

The additional investigation comprised the drilling and sampling of four bores (Bores 3A, 7 to 9) with a truck-mounted Hydrapower Scout drilling rig and a 'limited access', track mounted multi-purpose CE180 drilling rig using solid flight auger, wash bore and NMLC triple tube rock core drilling techniques. Strata identification was from the inspection of disturbed material returned to the surface on the augers and in the drilling circulation fluid, supplemented by the inspection of 'disturbed' Standard Penetration Test (SPT) samples, and inspection of the rock core recovered.

3.2 Groundwater Monitoring Wells

At the completion of the drilling of Bore 9, a standpipe groundwater monitoring well was installed. The groundwater monitoring well was constructed from Class 18 UPVC with factory slotted screen (0.5mm slot width and 4mm slot spacing). The screen was surrounded by a coarse sand pack placed to a level marginally above the screen section, with the annulus above the gravel sealed with bentonite, backfilled with spoil and concrete plugged at ground surface. A summary of the well construction details is given in Table 1, which includes a well installed in the previously drilled Bore 5 and a well installed in the previously drilled Bore 66 (as part of previous environmental assessment of the overall site; refer to Section 2.1.2). Details of the monitoring well construction for Bores 5 and 9 are also given on relevant attached Bore Report sheets.

Table 1: Summary of Monitoring Well Construction Details

Monitoring Well	Depth (m)	Ground Surface Elevation (mAHD)	Screen Depth		Screen Length (m)	Strata Screened	Completion Date
			Top (m)	Bottom (m)			
5	19.5	RL29.5	4.5	19.5	15.0	weathered rock	10 November 2017
9	20.5	RL5.3	5.5	20.5	15.0	weathered rock	28 April 2018
66*	16.1	RL21.0	7.1	16.1	9.0	weathered rock	25 July 2017

* Refer to Section 2.1.2

3.3 Bore Locations and Supervision

Bore locations were set out by direct measurement from existing site features and their locations (including the previously drilled Bores 1 to 6) are indicated approximately on Drawing No. 1, attached. The ground surface level at each bore location was determined by interpolation from Land Solution Australia Pty Ltd's Drawing *RBH_Survey_Northern Carpark 3*_LSA_171018(1).dwg.

An experienced geotechnical engineer set out the bore locations, logged the stratigraphy encountered in the bores, directed the insitu sampling and testing program and supervised the fieldwork and the construction of the groundwater monitoring well.

SECTION 4 - INVESTIGATION RESULTS

4.1 Reports

The subsurface conditions encountered in the previously drilled bores (Bores 1 to 6) and current drilled bores (Bores 3A and 7 to 9) are given on Bore Report sheets included in Appendix A and Appendix B, using classification and descriptive terms defined in the accompanying notes. Laboratory test report sheets are included in Appendix C.

4.2 Subsurface Conditions

For a description of the stratigraphy encountered in the bores, the Bore Report sheets should be consulted. The ground conditions encountered in the bores were highly variable, however broad summaries of the subsurface conditions encountered at the bore locations are given in the following sections.

As an aid to stratigraphic interpretation at the site, five sections (Sections 1-1 to 5-5) have been drawn through selected bores and the sections are presented on Drawings Nos. 2 and 4 attached.

4.2.1 Northern Area of the Site (Bores 1, 2, 3, 3A and 9)

The subsurface conditions encountered in Bores 1 to 3, 3A and 9 generally comprised a variable surface layer of bituminous concrete, pavement gravels, fill and possible fill (Bores 1, 2 and 9 only) and/or residual soils comprising silty/sandy clays and shaley clays to between approximately 1.8m and 5.7m depth (deepest in Bore 9). The fill and residual soils were underlain by variably weathered, extremely low to medium strength argillite, siltstone, conglomerate and mudstone (rock); high strength and medium to high strength rock was encountered in Bore 3A from approximately 12.7m to 17.5m depth.

4.2.2 Southern Area of the Site (Bores 4 to 8)

The subsurface conditions encountered in Bores 4 to 8, generally comprised a surface layer of bituminous concrete and pavement gravels, underlain by sand, gravel and clay fill to between approximately 0.5m and 5.9m depth in all bores (excluding Bore 6) and stiff to hard shaley clay to between approximately 3.0m and 7.3m depth (in Bores 4 and 7 only). The soils were underlain by variably weathered, extremely low to high strength tuff in Bores 5 to 7 only. The tuff and soils were generally underlain by variably weathered, extremely low to high strength argillite, conglomerate and mudstone, except in Bore 6 which terminated in high strength tuff.

4.2.3 Contact Between Tuff and Conglomerate/Argillite

A ‘contact’ zone between the tuff and the underlying conglomerate and argillite was encountered in Bore 5 below approximately RL16.5m, and Bore 7 below approximately RL7.1m which comprised sandy clay and ‘carbonaceous’ argillite (with coal seams). The ‘contact’ zone (comprising poor quality materials) is considered likely to exist below the tuff in other areas of the site.

4.2.4 Strength Inversions

'Strength inversions' (i.e. 'strong' materials underlain by 'softer' materials) were noted in several bores. For example, extremely low strength rock underlying very low to low strength rock at 14.8m depth (RL8.7m) in Bore 4, medium strength rock underlying medium to high strength rock at 27.5m depth (RL-4.5m) in Bore 3A, very low to low strength rock underlying high strength rock at 19.9m depth (RL7.1m) in Bore 7 and medium strength rock underlying high strength rock at 16.0m depth (RL5.0m) in Bore 8. Frequent 'strength inversions' were also encountered in the rock below approximately 5.7m depth (RL-0.4m) in Bore 9.

4.3 Groundwater

No free groundwater was encountered during auger drilling in the bores and could not be subsequently observed once wash bore drilling was commenced. Groundwater observations made in the monitoring wells after development, are given in Table 2. It should be noted that groundwater levels can vary seasonally, with prevailing weather and with tidal variations. If construction is to be undertaken at a significant time following this investigation and/or following significant 'wet' weather, it would be prudent to confirm groundwater levels prior to construction.

Table 2: Measured Groundwater Depths/Reduced Levels in Monitoring Wells

Well	Groundwater Observation		
	Date Measured	Depth (m)	Reduced Level (m AHD)
5	21 June 2018	12.3	RL17.2
	26 June 2018	12.7	RL16.8
9		2.8	RL2.5
66*		10.2	RL10.8

* Refer to Section 2.1.2

4.4 Laboratory Testing

Selected samples of soil and weathered rock recovered from the current and previous bores were submitted to one of Butler Partners's National Association of Testing Authorities (NATA) registered geotechnical testing laboratories for assessment of erosion and sediment control parameters, particle size distribution and plasticity. Selected rock core samples were also submitted for assessment of strength using point load strength test methods. All testing was conducted in accordance with test methods given in Australian Standard AS1289 and AS4133. The results of testing are summarised in the following sections.

It should be noted that sample descriptions provided in the laboratory results summary tables (and the laboratory test result sheets) are based on the inspection of each individual laboratory test sample only. No allowance has been made in sample descriptions for sampling, sub-sampling or test methodology in determination of the mass material properties. Estimates of mass material properties are provided on each individual Bore Report Sheet and as such, the laboratory test results should be read in conjunction with the relevant bore report sheets.

4.4.1 Erosion and Sediment Control Parameters

Seven soil/weathered rock/fill samples were selected for testing to determine Emerson Class Number, pH and electrical conductivity and the test results are presented in Table 3. The Emerson Class Number test results indicate that the samples tested had a high to low potential for dispersion in distilled water.

Table 3: Summary of Reported Emerson Class, pH and Conductivity Test Results

Bore	Depth (m)	Sample Description	Emerson Class No.	pH	Electrical Conductivity (mS/cm)
1	0.5 – 0.95	Fill - Clayey Gravelly Sand	4	7.5	0.56
2	0.5 – 0.95	Clayey Gravelly Sand	4	6.9	0.62
3	0.5 – 0.95	Shaley Clay	6	6.5	0.56
4	1.5 – 1.95	Fill - Silty Sandy Gravel	4	7.5	0.54
6	0.5 – 0.59	Tuff (XW/DW)	4	7.7	0.60
7	0.5 – 0.95	Shaley Clay	5	7.4	0.07
8	1.5 – 1.95	Fill – Sandy Clayey Gravel	2	5.8	0.06

4.4.2 Particle Size Distribution

Three soil and fill samples were tested for measurement of particle size distribution using wash sieve grading techniques and the reported results are summarised in Table 4.

Table 4: Reported Particle Size Distribution Test Results

Bore	Depth (m)	Sample Description	Sample Moisture Content (%)	Gravel Fraction⁽¹⁾ (%)	Sand Fraction⁽²⁾ (%)	Silt/Clay Fraction⁽³⁾ (%)
3A	1.5 – 1.95	Shaley Clay	10.3	15	25	60
4	3.0 – 3.45	Fill - Silty Sandy Gravel	8.4	38	26	36
8	3.0 – 3.45	Fill – Clayey Gravelly Sand	8.3	24	38	38

(1) Particle size <60mm, >2mm; (2) Particle size <2mm, >0.06mm; (3) Particle size <0.06mm

4.4.3 Plasticity

Selected samples recovered from Bores 1 to 4 and 6 to 8 were tested for plasticity using Atterberg Limit test methods and the test results are summarised in Table 5, together with each sample's classification.

Table 5: Summary of Reported Bore Plasticity Test Results

Bore	Depth (m)	Sample Description	Sample Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)	Classification*
1	3.0 – 3.45	Shaley Clay	38.6	81	29	52	25.5	XW
2	3.0 – 3.44	Argillite- extremely low strength	11.1	44	19	25	12.0	CI
3	1.5 – 1.92	Shaley Clay/Argillite	9.2	42	17	25	13.0	CI
3A	3.0 – 3.11	Argillite – extremely low strength	6.5	36	21	15	7.5	XW
4	5.8 – 6.25	Shaley Clay	12.9	38	17	21	10.5	CI
6	1.5 – 1.92	Tuff- extremely low strength	11.1	31	19	12	5.5	XW
7	1.5 – 1.95	Shaley Clay	29.1	48	19	29	13.5	CI
8	6.0 – 6.34	Argillite – extremely low strength	14.4	43	19	24	10.0	XW
	10.5 – 10.7	Argillite – very low strength	14.2	36	21	15	7.5	XW
9	4.5 – 4.95	Shaley Clay	15.5	39	20	19	9.5	CI
	9.0 – 9.37	Argillite – extremely low strength	20.0	68	16	52	18.5	XW

* In accordance with AS1726 *Geotechnical site investigations.*

4.4.4 Rock Strength

Selected samples of rock core recovered from both current and previous bores were tested for measurement of rock strength in both 'diametral' and 'axial' directions, using point load strength index [Is(50)] test methods. The test results are given on the relevant Bore Report sheets and are also tabulated with depth in Appendix C.

SECTION 5 - GEOTECHNICAL DESIGN DISCUSSION

5.1 Ground Conditions

The results of the investigation indicate that the site was generally underlain at the bore locations by a discontinuous layer of fill (including bituminous concrete and pavement gravels at some locations) to a maximum depth of 5.9m (in seven out of the ten bores) and/or residual soils comprising stiff to hard silty/sandy clays and shaley clays to a maximum depth of 7.3m (in seven bores). Below the fill and/or residual soils, variably weathered, extremely low to high strength tuff was encountered over the south-eastern area of the site in Bores 5 to 7. Variably weathered, extremely low to high strength argillite, conglomerate and mudstone was encountered below the tuff in Bore 5 and Bore 7; a ‘contact’ zone comprising poor quality materials was encountered between the tuff and conglomerate/argillite. Across the remaining areas of the site (in Bores 1 to 3, 3A, 4, 8 and 9), variably weathered, extremely low to high strength argillite, siltstone, conglomerate and mudstone was encountered below the fill and/or residual soils. Numerous ‘strength inversions’ were also encountered in a number of bores across the site. In these highly variable ground conditions, geotechnical design will need to consider (at least) the following key issues:

- variability in subsurface conditions over the site and with depth;
- quality/suitability of existing fill;
- earthworks and excavatability;
- potential presence of contamination;
- site trafficability;
- batter stability;
- movement control to site boundaries and adjacent buildings;
- suitable temporary excavation support methods;
- excavation support loads (construction and permanent);
- retaining wall pressures;
- groundwater control (construction and permanent) and disposal;
- suitable foundation types;
- suitable foundation strata and variation over site;
- presence of ‘strength inversions’ and potential to affect bearing capacity;
- bearing capacity of proposed founding strata and variability over the site;
- foundation settlement;
- the performance of proposed subgrade materials for slab and pavement design; and
- construction aspects.

Discussion of geotechnical design parameters, as well as design and construction recommendations and suggestions are detailed in the following sections.

5.2 Existing Fill

It is not known whether the existing fill material at the site is ‘controlled’ (i.e. it is not known whether the existing fill has been placed and uniformly compacted to an appropriate engineering specification). Supporting documentation should be obtained and checked to confirm that the fill has been placed in a controlled manner to a specification that is appropriate for the proposed development.

If documentation does not exist, or the specification is not appropriate for the proposed development, then it is suggested that the existing fill is assumed to be uncontrolled.

If the fill cannot be shown to be controlled, then consideration should be given to the potential for variations in both the composition and degree of compaction of the fill. The presence of voids within uncontrolled fill as well as potential soft/loose zones or inclusions of deleterious materials may lead to potentially significant future total and differential settlements, possibly occurring over relatively short distances.

To minimise the risk of potentially adverse settlement occurring, it is recommended that all uncontrolled fill present in settlement sensitive areas be removed and replaced/recompacted as controlled fill.

5.3 Earthworks

5.3.1 Bulk and Confined Excavation

It is understood that the proposed lowest bulk excavation levels for the development is RL5.35m to RL10.5m. The rock encountered in the bores ranged from extremely low to high strength, and it is considered possible that zones of 'stronger' and/or 'less jointed' rock may also exist within the proposed excavation depth. The excavatability of rock is critically dependent upon the depth of weathering, nature and distribution of rock defects (i.e. persistent joints, sheared, extremely to highly weathered or highly fractured zones) as well as strength, the 'size' of the excavation, the availability of plant and any site-specific limitations.

Excavation of fill, soils and extremely low to low strength rock should be readily achieved in bulk excavation using a large hydraulic excavator. A large tractor (D11) should be able to economically remove low strength and some medium strength rock, however supplementary rock breaking may be required to aid production in medium strength rock (subject to existing rock fracturing). It should be noted that successful ripping would require the use of a large tractor and freedom to rip in all directions. If this is not possible, it is likely that rock breakers will be required in lieu of ripping.

Bulk excavation of medium strength rock will require relatively major use of 'rock breaker' equipment unless joint spacing is moderately close (i.e. less than 0.3m). In high strength (or stronger) rock (with relatively few discontinuities), it is likely that the most economic method of excavation will involve blasting, followed by either ripping with a heavy tractor or use of a large rock breaker. Without blasting, rock breaker excavation methods only would be expected to be very slow and potentially severely damaging to equipment.

All confined and detailed excavation (e.g. pad footings/service trenches etc.) in medium strength or stronger rock will also require heavy rock breakers and very low production rates and high plant wear should be allowed for in high strength (or stronger) rock unless controlled blasting is used. Due to the anticipated nature of the rock jointing, some 'over break' should be expected.

Based on past experience in similar conditions, the maximum economic depths of bulk excavation using various methods has been preliminarily estimated at the each bore location and the estimates are given in Table 6. The depths given in the table must be read in conjunction with the Bore Report sheets, noting the maximum drilled depth of each bore.

Table 6: Preliminary Estimated Bulk Excavation Method Transition Depths

Excavation Method	**Estimated Transition Depth Below Ground Surface (m) Between One Excavation Method and Another for 'Economic Bulk Excavation'**								
	Bore								
	1	**2**	**3A**	**4**	**5**	**6**	**7**	**8**	**9**
Hydraulic Excavator ▼ **D11 Ripping***	>6.1	>6.1	12 – 13	>20.9	2 – 3	5 – 6	3 – 4	14- 15	22 – 23
D11 Ripping* ▼ **Heavy Rock Breaker**	?	?	13 – 14	?	4 – 5 (possibly)	5 – 6	4 – 5	15 – 16	?
Heavy Rock Breaker ▼ **Blasting**	–	–	13 (possibly)	–	–	5 – 6	–	–	–

* Possibly supplemented by heavy rock breaker work

Consideration should be given in selecting suitable excavation methods/plant due to the potential of encountering 'harder' rock below bore location termination depths, and at 'shallower' depth intermediate to the bore locations.

All confined excavations should be fully supported or battered/benched to a stable angle to ensure personnel safety.

5.3.2 Bored Pile Drillability

Heavy drilling equipment will be required to drill medium strength (or stronger) rock. In high strength (and stronger) rock and in all rock where significant quartz seams are present, low to very low production should be allowed for in pricing, combined with high bit wear. Use of coring buckets may be required in high strength (or stronger) rock, with core bucket diameters of 0.6m or less possibly being required in very high strength (or stronger) rock.

5.3.3 Subgrade Preparation

Following excavation to design level, the exposed subgrade should be uniformly compacted to the appropriate minimum dry density ratio nominated in Table 7. Any 'soft spots' encountered should be either tyned and dried then recompacted, or excavated and replaced with compacted select fill. However, in areas where existing fill materials are at, or close below, subgrade level, it may not be possible to obtain proper compaction and allowance is suggested for over excavation and replacement with a track rolled coarse granular trafficability layer (Section 5.3.6), placed under engineering supervision.

Table 7: Subgrade and Fill Compaction

Location	Minimum Dry Density Ratio
General floor slab support	100% (Standard compaction)
Pavement subgrade - top 500mm of subgrade - >500mm below subgrade level	100% (Standard compaction) 97% (Standard compaction)

5.3.4 Cut to Fill

The materials encountered in the bores would be expected to be suitable for reuse as fill, provided they do not contain organic or deleterious materials; are not contaminated; do not contain 'over-size' (>75mm size) fragments; are not 'overwet' and reactive behaviour can be tolerated or designed for.

5.3.5 Fill

All fill placed to support settlement sensitive structures/features should be placed in layers not greater than 250mm (loose thickness) and be uniformly compacted to the minimum dry density ratios nominated in Table 7. Reactive materials should be avoided for use as fill if possible, where future reactive movement is to be minimised and fill moisture/suction change can occur.

However, if use of reactive fill cannot be avoided, it should be placed and maintained at a moisture content of 1% wet of Standard optimum moisture content in order to reduce potential shrink-swell movements. If the reactive fill moisture content is not prevented from changing over time, the reactive fill will start to undergo reactive volume change with seasonal (or other) moisture change.

It should be noted that over-compacting reactive clay fill (particularly at a moisture content below Standard optimum) should be avoided as potentially significant expansion could occur on 'wetting up'. Due allowance must be made in design and detailing for reactive fill movements if reactive fill is used.

To assist with the achievement of adequate control over fill placement, geotechnical testing as set out in Section 8 of Australian Standard AS3798 – 2007 *Guidelines on earthworks for commercial and residential developments* would be required, and it is recommended that 'Level 1' geotechnical supervision and testing be adopted.

5.3.6 Trafficability

Trafficability for rubber-tyred plant will be difficult to impossible in the soils/fill/extremely low to very low strength rock encountered in the bores above and at bulk excavation level. Consideration should therefore be given to the placement of a coarse granular working surface. The actual layer thickness required for light construction plant must be determined, based on the actual layout and loading of the proposed traffic and subgrade conditions exposed. However, as an initial guide, a coarse granular bridging layer of not less than approximately 0.2m would generally be expected to be required to assist with site trafficability for 'lightweight' traffic over at least a 'stiff' clay subgrade. A substantially thicker layer would be required in areas where heavy construction plant is to traffic the site (e.g. piling rigs), or where existing fill materials are exposed at or close below subgrade level, and the layer thickness must be determined on a case by case basis once details of the 'heavy' plant are known.

5.3.7 Contaminated Materials

An Environmental Site Assessment (ESA) of the site has been undertaken by Butler Partners and the results of the ESA (reported separately; refer to Section 2.1.2) should be referred to as part of bulk earthworks design/procedures/pricing and stormwater/groundwater control/disposal etc.

5.3.8 Surface Water Drainage

Site earthworks will need to be properly drained so that water does not cause additional wetting up and softening of subgrade soils. Trafficing wet subgrades (without a trafficability layer) with any plant would be expected to result in significant subgrade damage.

5.3.9 Reactivity

The 'clays' and some weathered rock encountered in the bores are considered to generally have a moderate to high potential for reactive movement (i.e. have the potential for shrink and swell movements associated with wetting up and drying back). It is also anticipated that extremely low to very low strength rock may have the potential to display reactive behaviour, and would therefore be expected to readily change volume with change in moisture content, especially if disturbed and re-compacted.

5.3.10 Erosion and Sediment Control

The results of testing on selected samples given in Table 3 can be used in conjunction with relevant soil classifications as input to an assessment of site erosion and sediment control risk.

5.4 Batter Stability

If movement sensitive features/sections etc. are not located 'close' to excavations and geometry permits, battered slopes may be adopted.

Stable (cut) batter angles will need to be properly assessed once design and earthworks procedures have been finalised. As a preliminary guide, the values given in Table 8 are suggested for unsurcharged batters, up to 4m in height, where some movement behind batter crests is acceptable. It may be possible to excavate slopes in excess of 4m high in very stiff to hard clays and weathered rock at the batter slopes nominated in Table 8, provided the clays are 'dry', are not extensively fissured and that standing time is strictly limited.

Table 8: Maximum Unsurcharged Cut Batter Slopes to 4m Height

Material	Strength	Temporary Batter(1)	Permanent Batter(1)
Uncontrolled Fill	–	1V : 2H(2) (unreliable)	Not suitable
Silty/Sandy Clay and Shaley Clay	stiff very stiff/hard	1V : 1.5H 1V : 1H	1V:2.5H 1V:2.5H
Tuff/Argillite	extremely low very low to low medium (or stronger)	1V:1H 1V : 0.75H(3) 1V : 0.5H(3)	1V:2.5H 1V:1.5H(3) 1V:0.75H(3)

(1) Subject to confirmation by engineering inspection and not underlain by 'softer' material

(2) Flatter if saturated

(3) Depends on jointing and must be inspected/checked during excavation. Allowance should be made to flatten the batter, or install anchors/dowels and shotcrete if adverse jointing is encountered

The batter angles given in Table 8 are based on the assumption that batter faces are protected from erosion and that drainage is designed to keep surface water and groundwater away from the slopes. If free water is allowed to emanate from batter faces, slopes are likely to be unstable at the nominated batter slopes.

Detailed stability analysis prior to bulk earthworks design finalisation will be required to confirm stable batter slopes and associated construction limitations, batter extent etc. and detailed inspection by an experienced geotechnical engineer will be required at the time of construction to confirm the stability of temporary batter faces.

At the batter angles nominated in Table 8 there may be some localised slumping of batter slopes and it will be necessary to ensure that batter faces are protected from any surface water or groundwater seepage effects.

If insufficient space exists for the construction of cut batters at the slopes given in Table 8, or potential uncontrolled crest movement cannot be accepted, the excavation sides will need to be continuously supported in order to prevent instability.

5.4.1 Batter Stability Adjacent to Building C28

It is understood that proposed bulk excavations for the NCP development will occur directly adjacent to the existing Building C28 structure in the south-west corner of the site, and demolition of the existing structure and stabilising the excavation by permanently battering the slope is the preferred development option.

A preliminary slope stability assessment of the proposed batter slope has been carried out based on the ground conditions encountered in Bore 8 (and to a lesser extent Bore 4) and is discussed further in the following sections. **It is emphasised that the slope stability analysis detailed herein is preliminary and that significant additional analysis is expected to be required as part of the detailed design.**

An alternative (less preferred) option is to leave the existing Building C28 structure in place and design and install a retention system that will support the existing structure foundations and surrounding ground. Preliminary analysis for this option is given in Section 5.6.4.

5.4.1.1 Analysis Method

The preliminary slope stability analysis was undertaken using the commercially available geotechnical analysis software Slope/W, which uses limit equilibrium methods to assess the Factor of Safety (FOS) against slope instability. The analysis carried out was based on the following assumptions:

- slope geometry based on a bulk excavation level at the toe of the slope as RL10.5m and considering a 1V:1H batter and a 'shallower' 1V:2H batter profile;
- subsurface profiles based on the results of Bore 8 (and to a lesser extent Bore 4);
- sufficient space exists for construction of the cut batter;
- a 'general' surcharge of 16kPa was applied at and behind the crest of the slope;
- Mohr-Coulomb strength model for soils;
- strength parameters based on the results of the strata strengths encountered at the bore locations and the results of laboratory testing;
- groundwater profile modelled to include a groundwater table at approximately RL17m (considered the 'worst' case out of Bores 5 and 66; refer to Section4.3) which 'drains' out at the toe of the excavated face; and
- 'long term' analyses carried out using effective stress soil/rock strength parameters.

5.4.1.2 Soil Parameters

The 'long term', drained soil/rock strength parameters and soil/rock layering adopted for the stability analysis at the south-west corner of the development adjacent to Building C28 are given in Figure 1 and Figure 2.

5.4.1.3 Interpretation of Calculated Factor of Safety Values

In the 'long term' it is typical to adopt a minimum calculated FOS in the range of 1.4 to 1.5, depending on the level of uncertainty in input parameters. Where extensive investigation has been carried out and applied loads are well defined, a FOS at the low end of the range could be considered, however, as the degree of uncertainty in parameters, geometry, applied loads, groundwater conditions and variability increases the acceptable FOS limit from slope stability analysis should increase.

5.4.1.4 Preliminary Analysis Results

For the analysis conducted, an automated search of potential circular failure surfaces was carried out to assess the failure surface with the lowest calculated FOS. The results of the preliminary analysis considering excavated (cut) batter slopes of 1V:1H and 1V:2H is presented graphically in Figure 1 and Figure 2 respectively (showing the failure surface with the lowest calculated FOS) and is also summarised in Table 9.

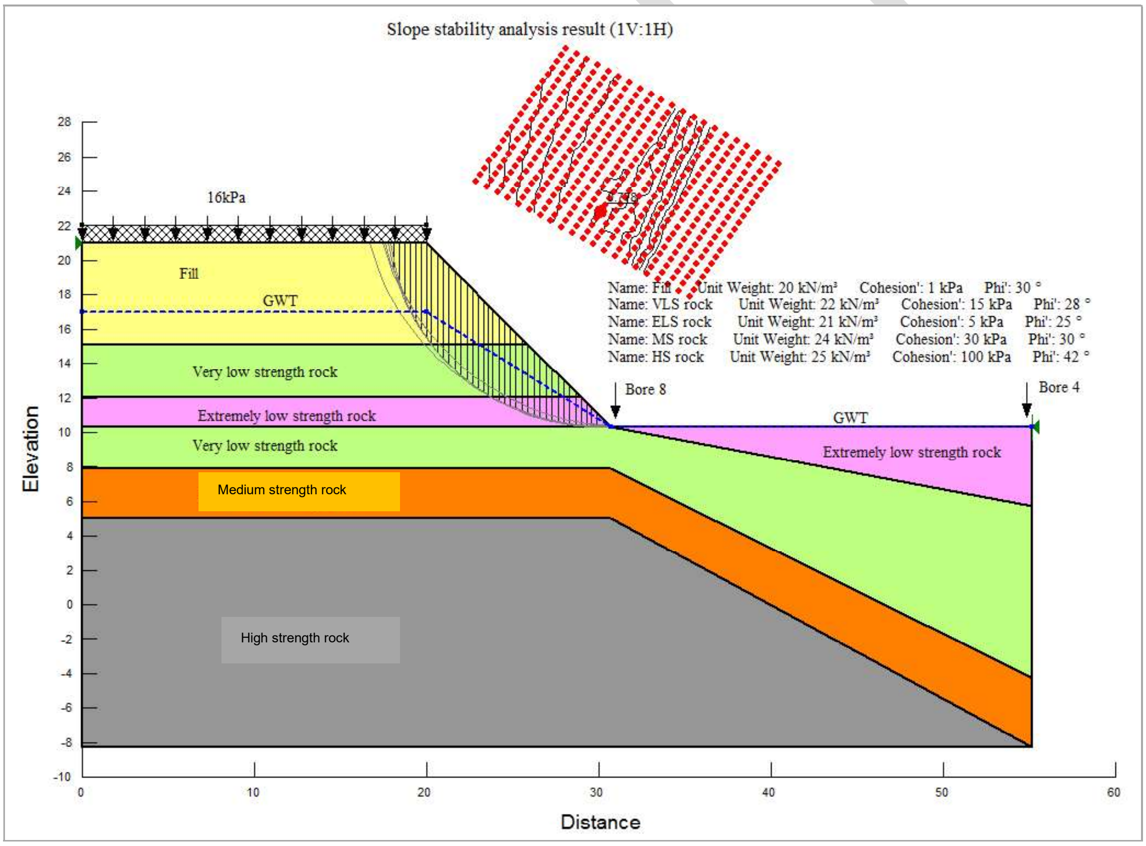


Figure 1: 'Long term' analysis for an excavated (cut) batter slope of 1V:1H at the location of Bore 8

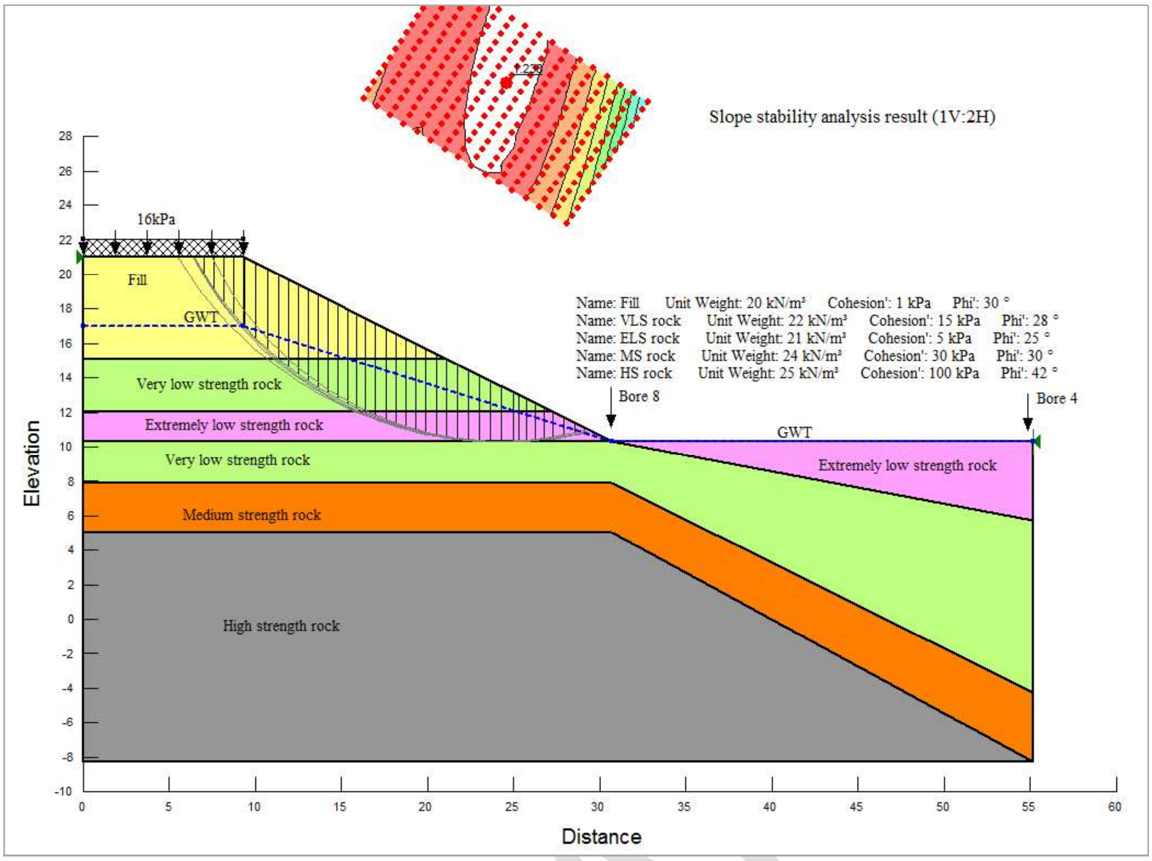


Figure 2: ‘Long term’ analysis for an excavated (cut) batter slope of 1V:2H at the location of Bore 8

Table 9: Summary of Calculated Minimum FOS Values for Excavated (Cut) Batter Slopes of 1V:1H and 1V:2H

Description	Lowest Calculated FOS
	Long Term
Excavated (cut) batter slope of 1V:1H	0.78
Excavated (cut) batter slope of 1V:2H	1.24

It is considered that the preliminary results given in Table 9 indicate ‘unacceptable’ minimum FOS values for 1V:1H and 1V:2H batter slopes. Options to improve the stability of the slope could include one or more of the following, however detailed slope stability analysis would be required to confirm if these option(s) are feasible:

- ‘flatten’ the overall fatter slope to ‘shallower’ than 1V:2H if space permits;
- stage the batter slope by one or more ‘flat’ benches (if space permits); and/or
- mechanically stabilise the face of the batter, this would also potentially allow for a ‘steeper’ batter (refer also to Section 5.5 and Section 5.6).

5.5 Temporary Excavation Support

All excavations should be temporarily supported or battered back to maintain stability. Where excavation sides cannot be battered (and/or instability cannot be tolerated), the excavation sides will need to be continuously supported to prevent instability. Anchoring or bracing may be required, and due consideration should be given in design where existing services or buildings are located close to the proposed excavation.

The following retention systems (or a combination) may be suitable for use at the site. Pre-excavation may be required to penetrate existing fill and remnant structure (if present), prior to wall installation.

It is also considered feasible that a combined excavation support system comprising part battered slopes and part mechanical retention may be feasible for use at the site where sufficient space exists to create an upper level batter. Detailed analysis will be required to determine the most cost effective combination of batter slope and mechanical support.

5.5.1 Anchored Soldier Piles and Shotcrete Lagging

Fill, soils and extremely low to low strength rock will require continuous support to prevent instability of vertical sided excavation, and a system of sheared anchor supported cast insitu soldier piles installed in predrilled holes which, in turn, retain shotcrete lagging could be considered. Such a wall system would be most suitable in areas of deep soils/fill and/or extremely low to low strength value such as in Bores 4 and 8.

5.5.2 Anchored Cast Insitu or Panel Wall

Where ‘reasonable’ quality rock is close to ground surface (e.g. Bores 5 to 7), temporary and permanent excavation support could be provided by a system of reinforced shotcrete panels supported by stressed anchors.

5.6 Rear Excavation Support Design

It is strongly recommended that design of the excavation support along the eastern, southern and western boundaries of the site consider the proposed excavation methodology, to minimise the requirement for re-design, delays and additional costs during construction (e.g. mobile crane hard-stands, excavation load-out points, etc. proposed at the crest of the excavation, which could introduce significant loads into the boundary retention if not separately pile supported).

Details of adjacent building loads/foundation levels along the site boundaries **must be confirmed as part of design finalisation, so that the relevant estimated excavation support loads can be assessed**.

Based on the results of the bores, it is considered that the excavation could be supported by an anchored, soldier pile wall and an anchored shotcrete panel wall and preliminary analyses of such wall systems were carried out considering the following:

- anchored soldier pile wall using the wall analysis computer program WALLAP for ground conditions based on an idealised stratigraphy at Bore 4, with no significant additional loads/surcharges at or behind the crest of the wall (refer to Section 5.6.1);
- anchored shotcrete panel wall using the widely used PLAXIS 2D geotechnical analysis package for ground conditions based on an idealised stratigraphy at Bore 5 and considering:
  - no significant additional loads/surcharges; and
  - future building/surcharge loads at and behind the crest of the wall from a future MNHHS ‘Connection’ Building (refer to Section 5.6.2); and
- anchored shotcrete panel wall at the location of the proposed Stage 1C services diversion adjacent to Building B52, using PLAXIS 2D for ground conditions based on an idealised stratigraphy at Bore 7 and considering building/surcharge loads from the adjacent Building B52 foundations (refer to Section 5.6.3; and

- anchored soldier pile wall at the south-west corner of the site adjacent to Building C28, using PLAXIS 2D for ground conditions based on an idealised stratigraphy at Bore 8 and considering building/surcharge loads from the adjacent Building C28 foundations (refer to Section 5.6.4).

It is emphasised that the WALLAP/PLAXIS analyses detailed herein are preliminary and that significant additional analysis are expected be required as part of the detailed design of the basement retention systems.

5.6.1 Soldier Pile Retention System with no Significant Surcharge

Preliminary analysis of the proposed excavation retention system considering a soldier pile wall has been undertaken through Section 5-5 (refer Drawing No. 4 attached) near the location of Bore 4 using the wall analysis computer program WALLAP, for a lowest bulk excavation level of RL10.5m and assuming no significant surcharge loading. The program allows staged construction to be analysed, and provides wall bending moments and lateral deflections, computed using finite element analysis methods, following each construction stage.

5.6.1.1 Ground Conditions

The ground conditions adopted for the WALLAP analysis were based on the strata encountered in Bore 4. A summary of the simplified stratigraphic layers adopted for the analysis is given in Table 10.

Table 10: Ground Profile Adopted in Analysis for Section A Using Bore 4

Material	Top of Layer (m)
Fill	RL24.6 (ground surface)
Shaley Clay (very stiff)	RL19.6
Extremely low strength Argillite	RL16.3
Very low strength Argillite	RL14.6
Extremely low strength Mudstone	RL8.8
Very low strength Argillite	RL5.8

The existing ground surface behind the retention wall slopes upwards and the additional load due to the sloping ground was taken into account in the form of a linearly varying surcharge.

5.6.1.2 Retention System

The retention system was designed as a soldier pile wall with shotcrete lagging stabilized by multiple rows of pre-stressed ground anchors. Details of the soldier pile wall configuration adopted for the preliminary WALLAP analysis are given in Table 11.

Table 11: Soldier Pile Configuration for Section A

Wall Details	Value
Wall Type	Soldier pile wall
Soldier Pile Diameter (m)	0.75
Soldier Pile Horizontal c/c Spacing (m)	2.0
Soldier Pile Toe Level (m)	RL1.7
Soldier Pile Young's Modulus (GPa)	28

Multiple rows of pre-stressed ground anchors were adopted to maintain wall stability and to minimise wall deflection and bending moment. The anchor properties used for the analysis are given in Table 12.

Table 12: Adopted Anchor Properties for Section A

Parameter	**Anchor**				
	A1	**A2**	**A3**	**A4**	**A5**
Anchor Reduced Level (m)	RL23.1	RL20.0	RL17.0	RL14.0	RL12.2
Anchor Depth (m)	1.5	4.4	7.6	10.6	12.4
Cross Section Area of Steel Strand (m^2)	0.000556	0.000695	0.000695	0.000973	0.001112
Young's Modulus of Strand (GPa)	200	200	200	200	200
Horizontal Spacing (m)	2.0	2.0	2.0	2.0	2.0
Anchor Declination Angle (°)	30	30	30	30	30
Assumed Free Length (m)	35.0	29.0	23.4	17.6	13.0
Preload (kN)	450	600	600	900	1050

5.6.1.3 Groundwater and Surcharge

The groundwater level was assumed to be 2m below the ground surface behind the wall and to be lowered down to 0.3m below the excavation level in front of the wall during each excavation stage. A uniform pressure of 16kPa was applied behind the wall to represent a general surcharge.

5.6.1.4 Construction Stages

The analysis comprised a repetition of two calculation phases for each excavation stage: excavation to 0.5m at maximum below the anchor level and installation of the anchor, which was preloaded to the specified load.

5.6.1.5 Analysis Results

The analysis results are summarised in Table 13, Table 14 and Figure 3, with the detailed WALLAP input/output file attached to Appendix D.

Table 13: Maximum Calculated Wall Deflection and Structural Forces

Parameter	**Calculated Maximum Value**
	Section A Using Bore 4
Bending Moment (kNm/pile)	553
Shear Force (kN/pile)	616
Deflection (mm)	43

Table 14: Calculated Maximum Anchor Loads

Anchor	**Calculated Maximum Load (kN/anchor)**
	Section A Using Bore 4
A1	489
A2	685
A3	704
A4	976
A5	1125

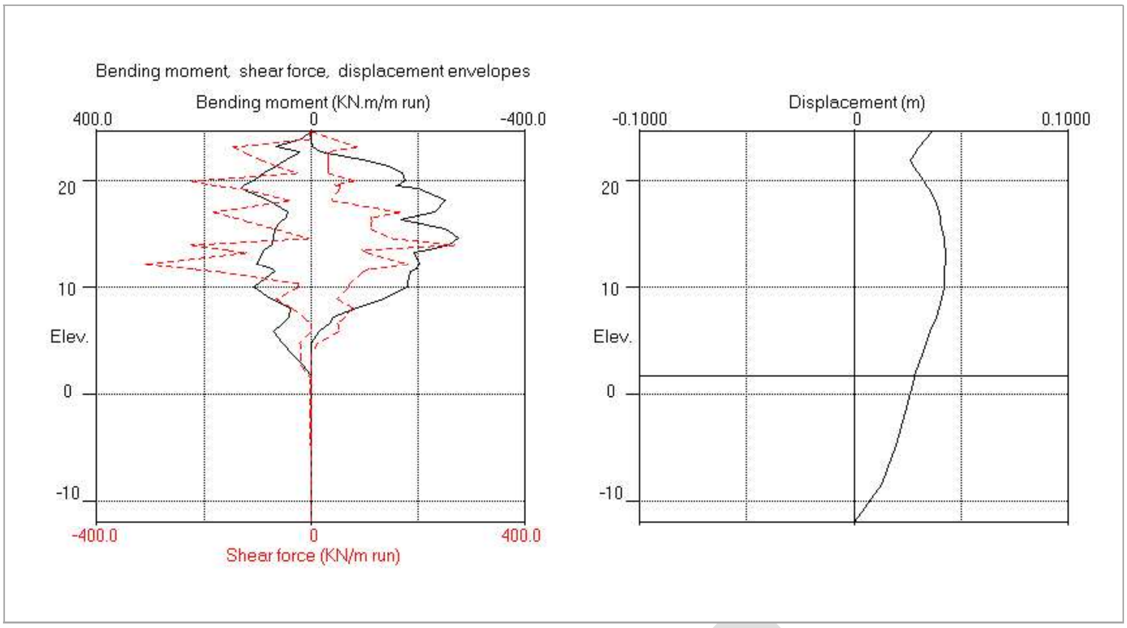


Figure 3: WALLAP calculated bending moment, shear force and displacement envelopes for Section 5-5

5.6.2 Anchored Shotcrete Panel Wall adjacent to Future MNHHS 'Connection' Building

Preliminary analysis of the proposed excavation retention system considering an anchored shotcrete panel wall through Section 4-4 (refer Drawing No. 3 attached) adjacent to the future MNHHS 'connection' building has been undertaken using the widely used PLAXIS2D geotechnical analysis package, for a lowest bulk excavation level of RL5.35m and a horizontal anchor spacing of 3m. Two conditions were analysed, as follows:

- Condition 1 - no significant surcharge loading at and behind the crest of the wall; and
- Condition 2 - future building/surcharge loads at and behind the crest of the wall from the 'connection' building.

5.6.2.1 Ground Condition and Material Properties

The PLAXIS2D model used in the analysis was based on the idealised ground conditions encountered in Bore 5, which was considered the 'worst' ground conditions compared to Bore 6 and Bore 7, through Section 4-4 (east and downslope from Bore 5). A summary of the simplified stratigraphic layers and associated material properties adopted in the model is given in Table 15 and Table 16.

Table 15: Adopted Ground Profile

Material	Top of Layer (m)
Fill	RL26.0 (ground surface)
Extremely low strength tuff	RL25.4
Medium strength tuff	RL23.7
Low strength tuff	RL17.0
Medium strength tuff	RL16.0
Extremely low strength argillite/coal seams	RL13.5
Low strength conglomerate	RL12.0

Table 16: Adopted Material Properties

Material	E' (MPa)	c' (kPa)	ϕ' (°)	γ (kN/m^3)
Fill	50	0	36	20
Extremely low strength tuff	90	5	28	22
Medium strength tuff	1000	60	40	25
Low strength tuff	150	25	27	24
Extremely low strength argillite/coal seams	70	5	27	22
Low strength conglomerate	250	30	31	24
Rock joint	-	10	30	-

Furthermore, a major rock joint was assumed to exist behind the excavation face, 'daylighting' at RL5.35m on the excavation face and rising upwards at 45^0 behind the face. The strength parameters of rock joints are also given in Table 16.

5.6.2.2 Adopted Retention System

The retention system adopted for the analysis model comprised an incrementally installed reinforced shotcrete wall stabilized by multiple rows of pre-loaded ground anchors. The wall properties adopted for the analysis are given in Table 17.

Table 17: Adopted Wall Properties

Parameter	Value
Wall Type	Shotcrete Wall
Wall Thickness (mm)	300
Toe Reduced Level (m)	RL5.35
Young's Modulus - E(GPa)	28

Seven rows of ground anchors were adopted to minimise wall deflection and bending moment, and the anchor properties used for the analyses is given in Table 18.

Table 18: Adopted Anchor Properties

Parameter	Anchor						
	A1	A2	A3	A4	A5	A6	A7
Anchor Reduced Level (m)	RL25.0	RL22.3	RL19.3	RL16.5	RL13.5	RL10.5	RL8.0
Anchor Depth (m)	1.0	3.7	6.7	9.5	12.5	15.5	18
Cross Section Area of Steel Strand (m^2)	0.001112	0.001112	0.00139	0.00139	0.00139	0.001529	0.001529
Young's Modulus of Strand (GPa)	200	200	200	200	200	200	200
Horizontal Spacing (m)	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Anchor Declination Angle (°)	15	15	15	15	15	15	15
Assumed Free Length (m)	17.9	15.7	13.3	11.0	8.5	6.1	4.1
Preload (kN)	540	1080	1440	1440	1440	1440	1440

The ground slab was assumed to be 200mm thick.

5.6.2.3 Groundwater Condition

The groundwater table was assumed to be 2m below the initial ground surface.

5.6.2.4 Surcharge

5.6.2.4.1 Condition 1

The analysis undertaken for Condition 1 was based on (as advised by Calibre) a 50kPa uniformly distributed foundation pressure applied at the ground surface under the Centre for Clinical Nursing Building and a uniform surcharge pressure of 16kPa applied elsewhere, from approximately 1m behind the wall during excavation.

5.6.2.4.2 Condition 2 – MNHHS 'Connection' Building

It is understood that the proposed future MNHHS 'connection' building adjacent the NCP development may be approximately 10m in width and 70m in length and up to three levels in height, and may comprise the following:

- a concrete framed structure with concrete slabs including a roof slab;
- structure loading is likely to be supported off columns for the three levels, comprising two rows of columns (one row 'close' to the outer extremity of the building and one row 'close' to the crest of the retention wall), with column working loads of 3,000kN vertical and 300kN horizontal; and
- a column grid along the length of the new building could correspond with column positions to the NCP structure.

Based on the above assumptions, a piled foundation system to support the 'connection' building structure was assumed in the analysis to carry the anticipated high column loads. Two rows of piles were used at the assumed column locations – i.e. approximately 1.5m from the crest of the retention wall and approximately 7m behind the retention wall, respectively. The centre-to-centre spacing between adjacent piles in a row was assumed to be approximately 8m.

The piles were 'treated' in the 2D analysis model as embedded pile rows, representing an approximate estimate of the overall effect of the pile and pile loading on the wall stability.

Each pile was assumed to be 750mm in diameter and carrying the proposed 3,000kN vertical and 300kN horizontal loads. Two pile lengths were considered for the analysis; a 'short' pile terminating at approximately RL15.0m and a 'long' pile terminating below the proposal bulk excavation level at approximately RL4.0m.

Figure 4 shows an overview of the PLAXIS model considering two rows of 'short' piles after the MNHHS 'connection' building foundation load was applied.

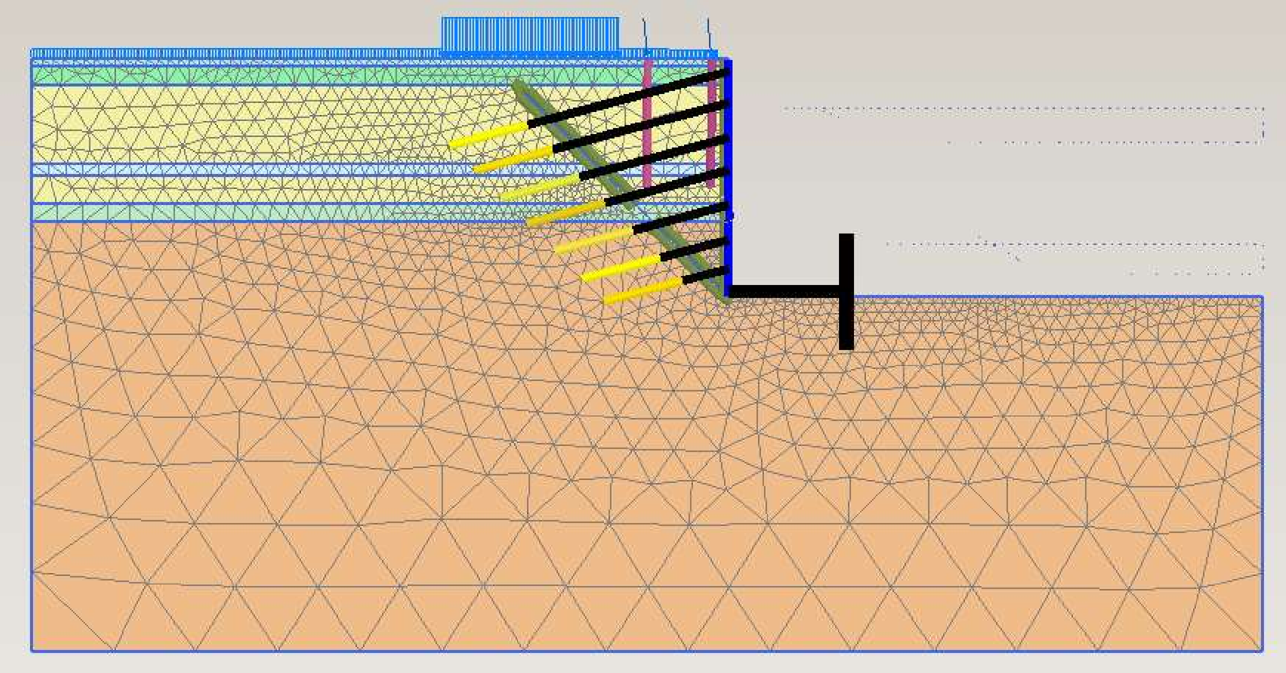


Figure 4: PLAXIS Model Analysed with two rows of 'short' piles

5.6.2.5 Construction Stages

Each excavation stage comprised two calculation phases: excavation to a maximum 0.5m below the anchor level and installation of the anchor and shotcrete. The anchor is preloaded to the specified load. The above process was repeated until the final bulk excavation level was reached. The detailed calculation phases are given in Table 19.

For the Condition 2 analysis, the MNHHS foundation piles were activated and the pile load applied after the ground slab was installed.

Table 19: Detailed Calculation Phases

Phase	Description
1	Form initial ground surface and activate surcharge
2	Excavate to RL24.5m
3	Install shotcrete, install and stress anchor A1
4	Excavate to RL21.8m
5	Install shotcrete, install and stress anchor A2
6	Excavate to RL18.8m
7	Install shotcrete, install and stress anchor A3
8	Excavate to RL16.0m
9	Install shotcrete, install and stress anchor A4
10	Excavate to RL13.0m
11	Install shotcrete install and stress anchor A5
12	Excavate to RL10.0m
13	Install shotcrete, install and stress anchor A6
14	Excavate to RL7.5m
15	Install shotcrete, install and stress anchor A7
16	Excavate to RL5.5m
17	Install shotcrete to RL5.35m
18	Install ground slab
19	Activate MNHHS pile and column loads (Condition 2 analysis only)

5.6.2.6 Analysis Results

The analysis results for Condition 1 and Condition 2 are summarised in Table 20 to Table 22 and graphical output from PLAXIS are given in Figure 5 to Figure 7 for calculated bending movement and shear force envelopes respectively.

Table 20: Maximum Calculated Wall Deflection and Structural Forces

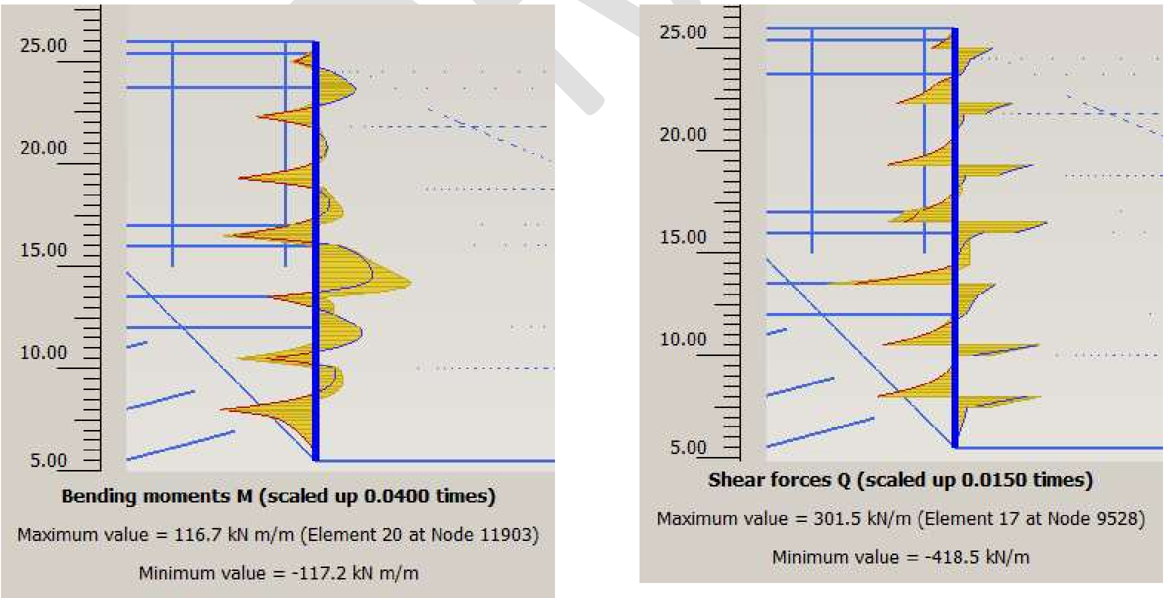
Parameter	Calculated Maximum Value		
	Condition 1	Condition 2	
		Short Pile	Long Pile
Bending Moment (kNm/m)	117	117	120
Shear Force (kN/m)	419	419	412
Deflection at Wall Top (mm)	42	53	48

Table 21: Calculated Maximum Anchor Load

Anchor	Calculated Maximum Load (kN)		
	Condition 1	Condition 2	
		Short Pile	Long Pile
A1	627	676	669
A2	1155	1183	1175
A3	1469	1496	1488
A4	1451	1482	1463
A5	1447	1447	1449
A6	1573	1631	1579
A7	1515	1561	1523

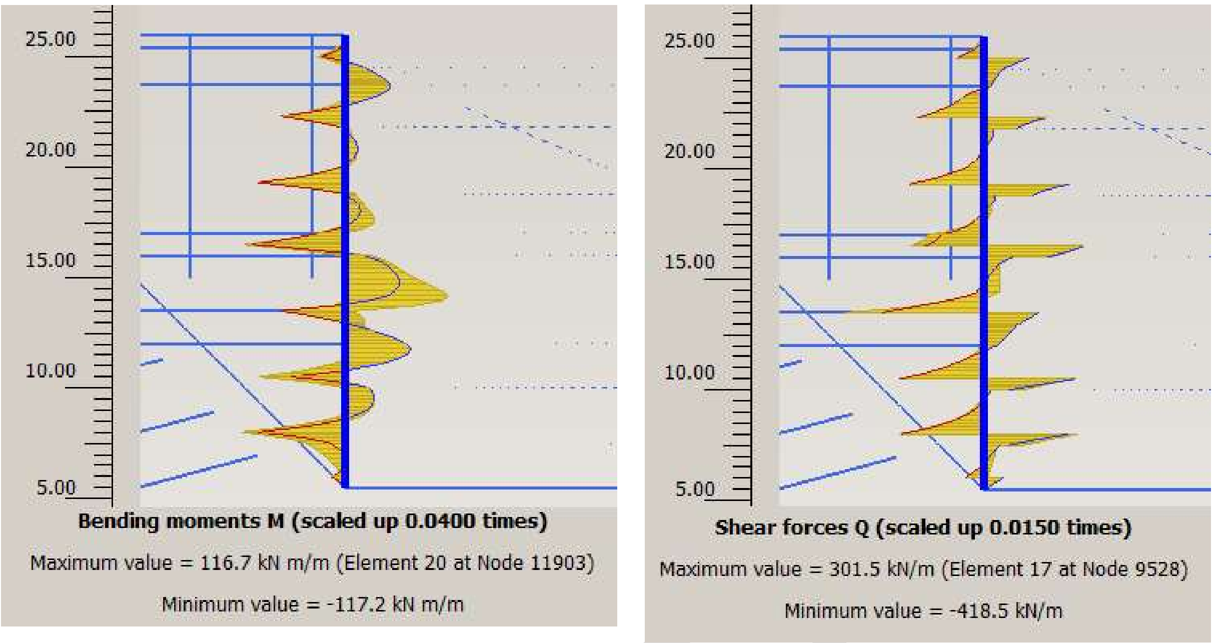
Table 22: Calculated Pile Head Movement Due To MNHHS Loading (Condition 2)

Pile Length	Away From Retention Wall		'Close' to Retention Wall	
	Horizontal (mm)	Vertical (mm)	Horizontal (mm)	Vertical (mm)
Short pile	17	12	19	14
Long pile	14	9	14	11



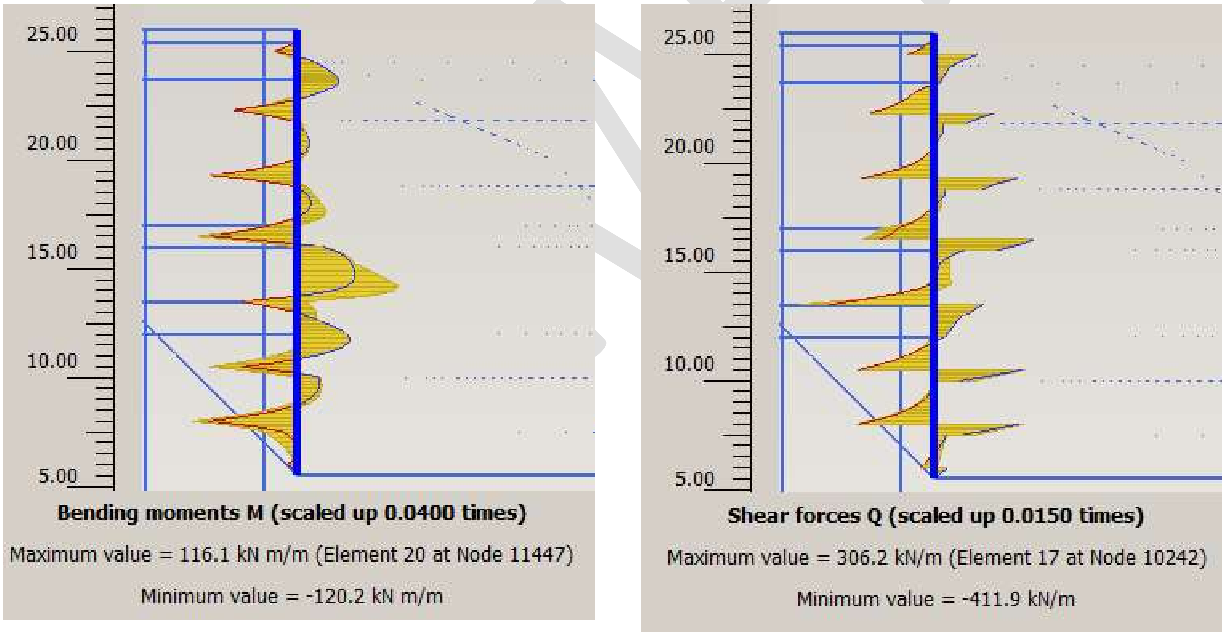
(a) Bending Moment Envelope *(b) Shear Force Envelope*

Figure 5: Calculated wall bending moment and shear force plots (Condition 1)



(a) Bending Moment Envelope *(b) Shear Force Envelope*

Figure 6: Calculated wall bending moment and shear force plots (Condition 2 with 'short' piles)



(a) Bending Moment Envelope *(b) Shear Force Envelope*

Figure 7: Calculated wall bending moment and shear force plots (Condition 2 with 'long' piles)

5.6.3 Anchored Shotcrete Panel Wall (Stage 1C - Services Diversion)

Preliminary analysis has been undertaken of a potential excavation retention system comprising an anchored shotcrete panel wall located adjacent to the existing HADS (B52) Building. The analysis was undertaken at Section X-X (refer Drawing No. 1 attached) using the widely used PLAXIS2D geotechnical analysis package, for a lowest bulk excavation level of RL5.5m.

5.6.3.1 Ground Condition and Material Properties

The PLAXIS2D model used in the analysis was primarily based on the ground conditions encountered in Bore 7, which was considered ‘worse’ than the ground conditions encountered at Bore 6. Minor adjustment was made to the Bore 7 ground condition to suit the ground surface level at the analysed Section (X-X). A summary of the adopted stratigraphic layers and associated material properties is given in Table 23 and Table 24.

Table 23: Adopted Ground Profile

Material	**Top of Layer (m)**
Shaley clay (very stiff to hard)	RL19.5 (ground surface)
Medium strength tuff	RL18.0
High strength tuff 1	RL16.5
Clay seam	RL7.8
High strength tuff 1	RL7.5
High strength tuff 2	RL5.0
High strength tuff 3	RL3.5
Clay seam	RL2.3
Very low strength Carbonaceous argillite	RL2.0
Low strength conglomerate	RL1.0

Table 24: Adopted Material Properties

Material	**E’ (MPa)**	**c’ (kPa)**	**ϕ’ (°)**	**γ (kN/m^3)**
Shaley clay (very stiff to hard)	35	5	26	20
Medium strength tuff	1100	70	41	25
High strength tuff 1	2500	116	45	25
Clay seam	35	5	26	20
High strength tuff 2	500	55	40	25
High strength tuff 3	800	70	41	25
Very low strength Carbonaceous argillite	70	5	27	22
Low strength conglomerate	800	55	40	24
Rock joint	-	10	30	-

Furthermore, a major rock joint was assumed to exist behind the excavation face, ‘daylighting’ at RL5.5m on the excavation face and rising upwards at 45^0 behind the face. The strength parameters of rock joints are also given in Table 24.

5.6.3.2 Retention System

The retention system adopted for the analysis model comprised an incrementally installed reinforced shotcrete wall stabilized by multiple rows of pre-loaded ground anchors. The wall properties adopted for the analysis are given in Table 25.

Table 25: Adopted Wall Properties

Parameter	**Value**
Wall Type	Shotcrete Wall
Wall Thickness (mm)	300
Toe Reduced Level (m)	RL5.5
Young’s Modulus – E (GPa)	28

Four rows of ground anchors were adopted to minimise wall deflection and bending moment, and the anchor properties used for the analyses is given in Table 26.

Table 26: Adopted Anchor Properties

Parameter	Anchor			
	A1	A2	A3	A4
Anchor Reduced Level (m)	RL18.0	RL15.0	RL11.5	RL8.7
Anchor Depth (m)	1.5	4.5	8.0	10.8
Cross Section Area of Steel Strand (m^2)	0.000556	0.000695	0.000834	0.000973
Young's Modulus of Strand (GPa)	200	200	200	200
Horizontal Spacing (m)	2.5	2.5	2.5	2.5
Anchor Declination Angle (°)	15	15	15	15
Assumed Free Length (m)	12.2	9.7	6.9	5.2
Preload (kN)	450	600	750	900

5.6.3.3 Groundwater Condition

The initial ground water table was assumed to be 2m below the initial ground surface. The ground water table was subsequently assumed to be 2m below the external ground surface level (behind the wall) in the 'far field' and was lowered down to the excavation level in front of the wall. A drained effective stress analysis was conducted.

5.6.3.4 General Surcharge and Assumed Building Loads

A uniform pressure of 16kPa was applied as a general surcharge behind the wall at the ground surface level. To facilitate the analysis, a uniform pressure of 400kPa was also applied in a 2.5m wide strip, located 4m behind the wall to simulate surcharge load from the HADS building footings. **It is considered that the assumed HADS building foundation load is a simplified (and conservative) treatment, which should be reviewed and confirmed by the development structural consultant.**

Figure 8 shows an overview of the PLAXIS model with HADS footing pressure applied.

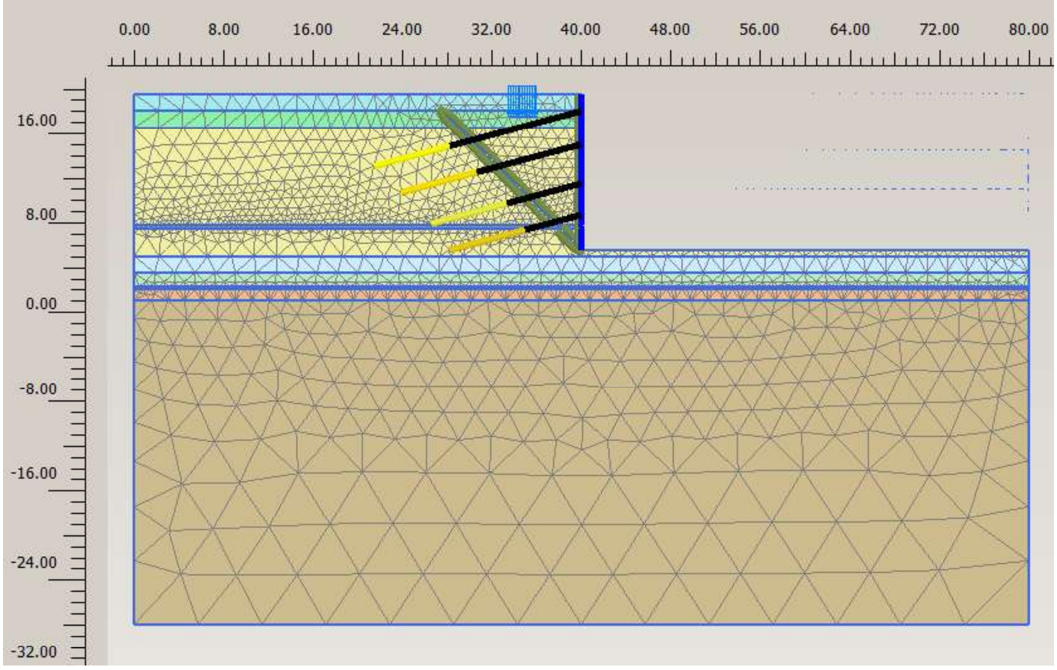


Figure 8: PLAXIS Model Analysed with HADS footing pressure applied

5.6.3.5 Construction Stages

Each excavation stage comprised two calculation phases: excavation to a maximum 0.5m below the anchor level and installation of the anchor and shotcrete. The anchor is preloaded to the specified load. The above process was repeated until the final bulk excavation level was reached. The detailed calculation phases are given in Table 27.

Table 27: Detailed Calculation Phases

Phase	Description
1	Activate HADS footing pressure
2	Activate general surcharge (zero displacement at beginning)
3	Excavate to RL17.5m
4	Install shotcrete, install and stress anchor A1
5	Excavate to RL14.5m
6	Install shotcrete, install and stress anchor A2
7	Excavate to RL11.0m
8	Install shotcrete, install and stress anchor A3
9	Excavate to RL8.2m
10	Install shotcrete install and stress anchor A4
11	Excavate to RL5.5m
12	Install shotcrete to RL5.5m

5.6.3.6 Analysis Results

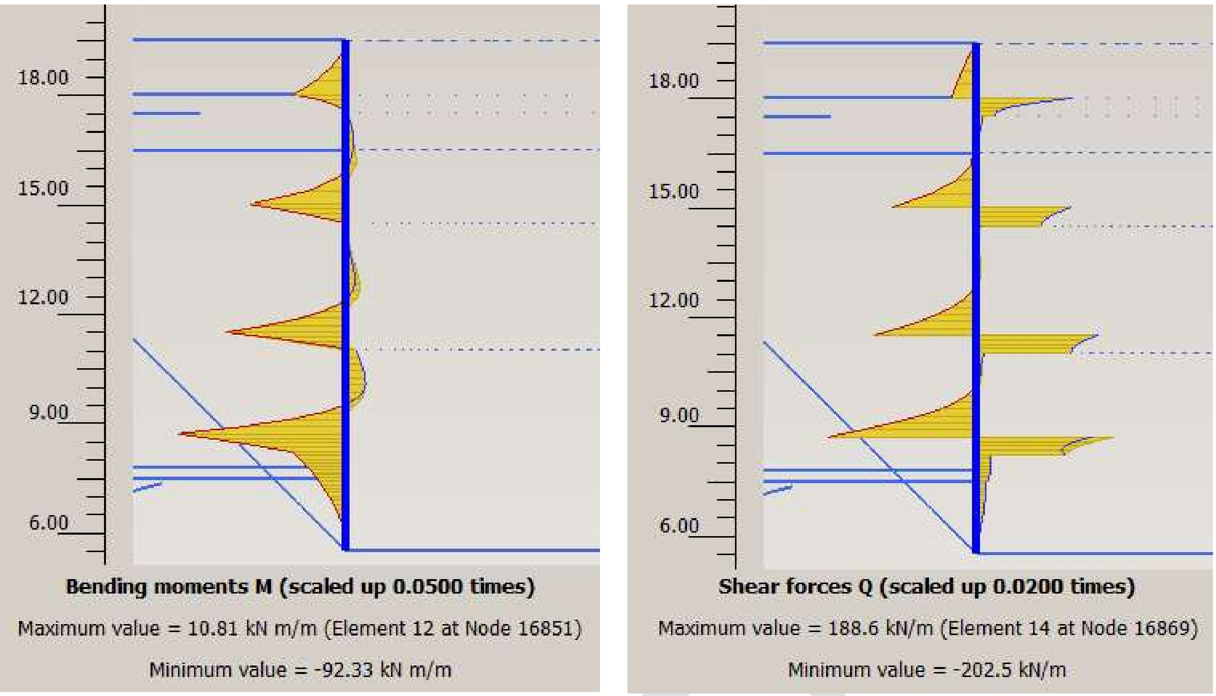
The analysis results are summarised in Table 28 and Table 29 and graphical output from PLAXIS are given in Figure 9 for calculated bending movement and shear force envelopes respectively.

Table 28: Maximum Calculated Wall Deflection and Structural Forces

Parameter	Calculated Maximum Value
Bending Moment (kNm/m)	92
Shear Force (kN/m)	203
Deflection (mm)	9

Table 29: Calculated Maximum Anchor Load

Anchor	Calculated Maximum Load (kN)
A1	472
A2	627
A3	787
A4	933



(a) Bending Moment Envelope *(b) Shear Force Envelope*

Figure 9: Calculated wall bending moment and shear force plots (vertical coordinate is RLm)

5.6.4 Soldier Pile Retention System adjacent to Existing Building C28

Preliminary analysis of the proposed excavation retention system considering a soldier pile wall adjacent to the existing Building C28 structure has been undertaken using PLAXIS2D, for a lowest bulk excavation level of RL10.5m and considering building / surcharge loads at and behind the crest of the wall from the Building C28 structure.

5.6.4.1 Ground Condition and Material Properties

The PLAXIS2D model used in the analysis was based on the idealised ground conditions encountered in Bore 8 (and to a lesser extent Bore 4), through Section 2-2 (refer Drawing No.2 attached) approximately 4m upslope from Bore 8. A summary of the simplified stratigraphic layers and associated material properties adopted in the model is given in Table 30 and Table 31.

Table 30: Adopted Ground Profile

Material	Top of Layer (m)
Fill	RL25.0 (initial ground surface)
Very low strength argillite	RL19.1
Extremely low strength argillite	RL16.0
Very low strength argillite	RL14.8
Medium strength argillite	RL11.9
High strength argillite	RL9.0

Table 31: Adopted Material Properties

Materials	E' (MPa)	c' (kPa)	ø' (°)	γ(kN/m^3)
Fill	45	0	38	21
Extremely low strength argillite	90	5	27	21
Very low strength argillite	120	15	28	22
Medium strength argillite	500	30	30	25
High strength argillite	3,000	100	42	25
Rock joint	-	10	30	-

Furthermore, a major rock joint was assumed to exist within the medium strength rock behind the excavation face, daylighting at RL10.5m (bulk excavation level) on the excavation face and rising upwards at 45^0 behind the face. The strength parameters of rock joints are also given in Table 31.

5.6.4.2 Retention System

The retention system adopted in the analysis model consisted of a soldier pile wall with shotcrete lagging stabilized by multiple rows of prestressed ground anchors. The wall properties adopted for the analysis are given in Table 32.

Table 32: Adopted Wall Properties

Parameter	Value
Wall Type	Soldier Pile Wall
Pile Diameter (m)	0.6
Assumed Pile c/c Spacing (m)	2.5
Toe Reduced Level (m)	RL9.0
Young's Modulus – E (GPa)	28

Four rows of prestressed ground anchors were adopted to maintain wall stability and to minimise wall deflection and bending moment. The anchor properties adopted are given in Table 33.

Table 33: Adopted Anchor Properties

Parameter	Anchor			
	A1	A2	A3	A4
Anchor Reduced Level	RL23.5m	RL20.5m	RL17.0m	RL13.8m
Area (m^2)	0.000556	0.000556	0.000695	0.000973
Young's Modulus – E (GPa)	200	200	200	200
Horizontal Spacing (m)	2.5	2.5	2.5	2.5
Anchor Declination Angle (°)	15	15	15	15
Assumed Free Length (m)	12.8	11.4	11.4	8.1
Preload (kN/anchor)	450	450	600	900

5.6.4.3 Groundwater Condition

The groundwater table was assumed to be at RL23.0m at a distance behind the wall, 2m below the ground surface level. The steady state pore water pressure was determined based on steady state seepage analysis with the water level maintained at the excavation level in front of the wall during each excavation stage. Drained effective stress analysis was conducted.

5.6.4.4 Surcharge

5.6.4.4.1 Assumed Building C28 Loads

Based on historical structural drawings of Building C28 provided to Butler Partners, the main structure is understood to be supported by twenty concrete piles of approximately 910mm in diameter, with the piles approximately 8m (centre to centre) apart between two adjacent piles and founded at approximately RL12.2m.

Based on the position and orientation of the existing structure (and foundation pile layout) with respect to the proposed NCP bulk excavations (and soldier pile wall), the piles closest to the proposed excavation were modelled in PLAXIS2D as three rows of piles; approximately 2.2m, 6.2m and 9.9m from the soldier pile wall. Each pile was applied with an assumed vertical load of 2100kN; **this load will need to be confirmed by a suitably qualified structural engineer**.

Figure 10 shows an overview of the PLAXIS model considering the three rows of piles.

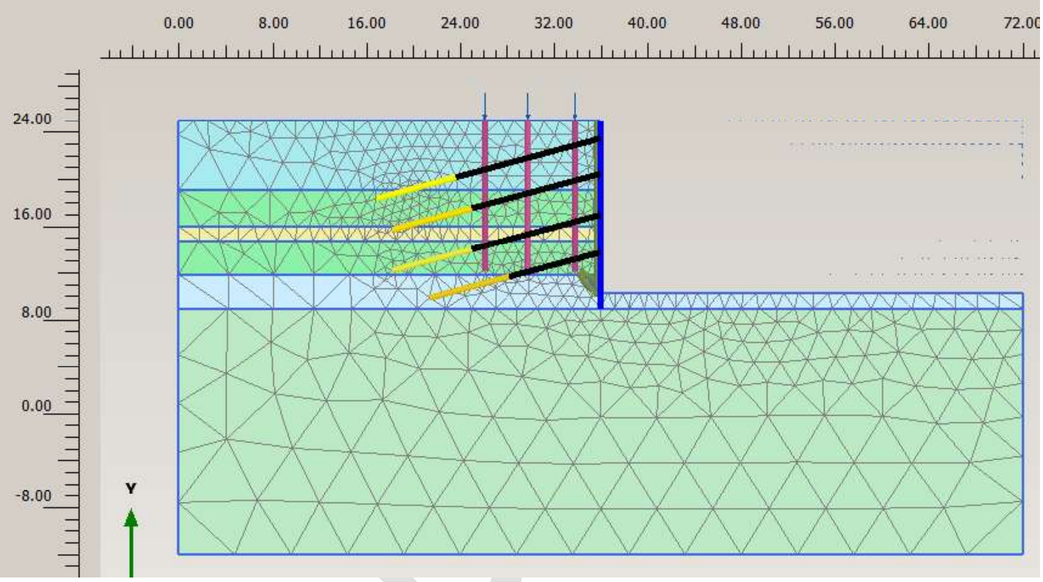


Figure 10: A snapshot of model at completion of excavation (surcharge not shown for clarity)

5.6.4.5 Construction Stages

The construction stages in the field are represented by the calculation phases in the analysis. The analysis of basement excavation comprised a repetition of two calculation phases for each excavation stage: excavation to 0.5m below the anchor level and installation of shotcrete panel and anchor, which was stressed to the specified preload. The detailed calculation phases are given in Table 34.

Table 34: Detailed Calculation Phases

Phase	Description
Initial phase	Activate rock joint
1	Activate Building C28 piles and loads
2	Zero displacement, activate surcharge and soldier pile wall
3	Excavate to RL23.0m
4	Install shotcrete panel, install and stress anchor A1
5	Excavate to RL20.0m
6	Install shotcrete panel, install and stress anchor A2
7	Excavate to RL16.5m
8	Install shotcrete panel, install and stress anchor A3
9	Excavate to RL13.3m
10	Install shotcrete panel, install and stress anchor A4
11	Excavate to RL10.3m
12	Install shotcrete to RL10.3m

5.6.4.6 *Analysis Results*

The analysis results for the wall movement and structural loads are summarised in Table 35 and Table 36.

Table 35: Calculated Maximum Wall Deflection and Structural Forces

Parameter		Maximum Calculated Value
Soldier Pile Wall	Bending Moment (kNm/pile)	277
	Shear Force (kN/pile)	513
Deflection (Horizontal Movement) (mm)		11

Table 36: Calculated Maximum Anchor Load

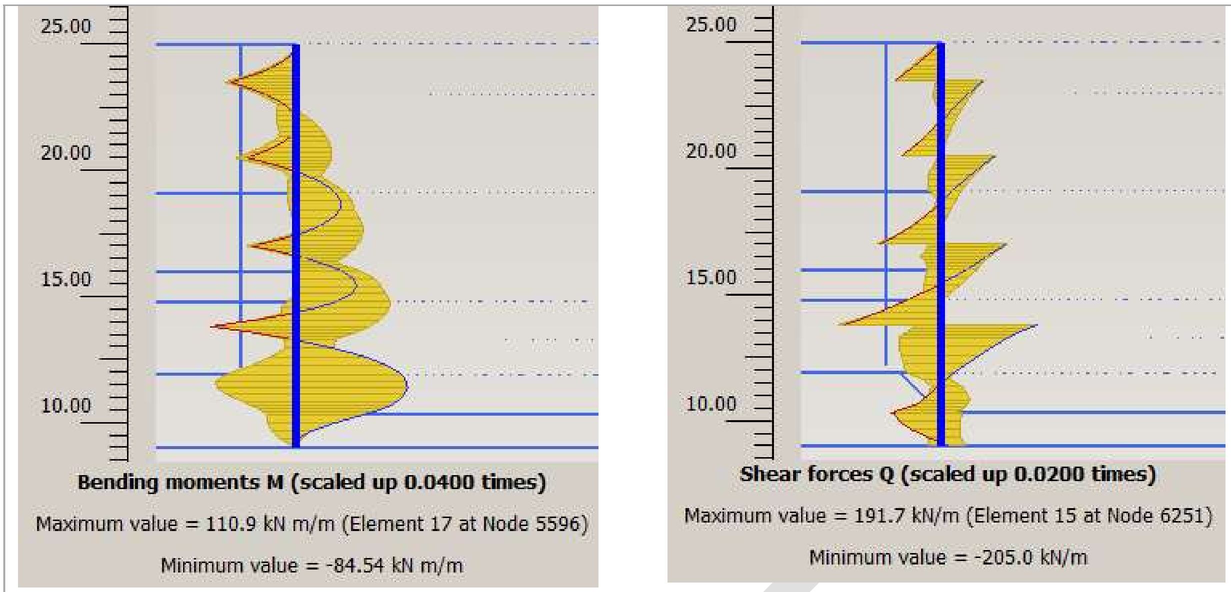
Anchor	Maximum Calculated Load (kN/anchor)
A1	454
A2	479
A3	646
A4	990

The pile head movement resulting from the Building C28 foundations is given in Table 37.

Table 37: Calculated Pile Head Movement from Building C28 Foundations

Pile Row	Maximum Calculated Movement	
	Horizontal Towards Excavation (mm)	Vertical Downwards (mm)
Row 1 (close to wall)	4	7
Row 2	7	4
Row 3	8	3

The distribution of wall bending moment and shear force are shown in Figure 11.



Bending moments M (scaled up 0.0400 times)
Maximum value = 110.9 kN m/m (Element 17 at Node 5596)
Minimum value = -84.54 kN m/m

Shear forces Q (scaled up 0.0200 times)
Maximum value = 191.7 kN/m (Element 15 at Node 6251)
Minimum value = -205.0 kN/m

(a) Bending Moment (b) Shear Force

Figure 11: Calculated envelopes of wall bending moment and shear force (vertical coordinate is RLm)

5.6.5 Stressed Anchor Design

5.6.5.1 Contractors and Construction Monitoring

Only fully experienced, pre-qualified contractors should be used to install rock anchors, due to the potential to cause damage to off-site properties if incorrect anchoring installation methods are employed, and to ensure that anchors achieve their full design capacity with all necessary load factors.

Detailed anchor construction and stressing/test records must be kept by the anchoring contractor, which must be continuously available for review and checking. Under no circumstances should excavation proceed below any row of anchors, until the installation and stressing records are reviewed and anchor capacity is accepted (as being in accordance with the design) by the Superintendent.

5.6.5.2 Preliminary Design Parameters

For the purpose of estimating anchor bond lengths, the maximum working stresses given in Table 38 may be used for preliminary sizing of anchors drilled (and maintained) dry. Insitu proof testing of anchors must be undertaken to confirm design capacity.

Table 38: Temporary Anchor Working Bond Stress

Rock Strength	Working Bond Stress (kPa)
extremely low	20 – 40(1)
very low	100
low	200
medium	500
high	1,000(2)

(1) Highly variable and possibly unreliable

(2) Subject to grout strength

It is considered essential that anchor lift-off tests be conducted on selected anchors, during the full time the anchors are required to provide excavation support, to confirm that creep is not occurring. Not less than 1.2m of strand should be left protruding from anchor heads in order to allow for lift off testing and restressing (if required). Protruding strands must be protected from damage.

Care should be taken with anchor stressing in fill, soils and extremely low to low strength rock to prevent 'outward' movement of temporary walls.

5.6.5.3 Approvals and Services Checks

It is strongly recommended that approvals for anchoring across site boundaries and beneath adjoining structures, be obtained as far in advance of construction as possible, to enable finalisation of excavation support design. Also, underground services that could be potentially affected by anchor/dowel installation and/or slope movement should be clearly identified well in advance of construction and well before finalisation of excavation support design.

5.7 Groundwater Control

Groundwater control will be required to enable economic sizing of support works, and to provide permanent long term drainage. It will be necessary to ensure that all existing drains/services that are (or will become) disconnected/disused as part of the site redevelopment are properly plugged (including backfill) to prevent any 'back-flow' to the site. In addition, the backfill to any drains should also be properly plugged.

5.7.1 Wall Drainage

As part of the retention analysis, assumptions as to groundwater level are required to be made, in order to ensure that the actual groundwater level behind the excavation face is controlled (and does not exceed the design assumptions).

For a soldier pile and shotcrete wall, temporary drainage is required behind shotcrete facing (above the average, long-term groundwater level), comprising not less than, full height strips of 'Core Drain' (or equivalent) of 20mm minimum thickness, 150mm wide, spaced at not more than approximately 1.8m centres. It is essential that the drains and drain joints be fully protected against ingress of shotcrete etc.

5.7.2 Under Slab Drainage

Groundwater seepage may occur through the floor of the car park excavation that could adversely affect floor slab performance and building amenity, unless either a fully tanked slab is adopted or an adequate groundwater control system is installed.

If a 'non-tanked' floor slab is adopted, it will be necessary to cast the slab over a free draining layer (incorporating agricultural drains) graded to sumps so that groundwater is removed and pressure does not build-up under the slab; 'normal' slab on ground joint spacing could be adopted with this option.

Groundwater inflow quantities (and 'design' groundwater level) should be determined by detailed groundwater modelling (incorporating both normal seasonal and extreme weather condition inputs) using computer packages such as MODFLOW, SEEPW etc., so that the drainage system can be properly designed.

It will be necessary to place the drainage layer over a geotextile to prevent clogging due to 'fine' particle ingress. Flushing/maintenance access points must be designed into the system to enable cleaning of the drainage layer as required.

5.8 Foundations

Due to the variable ground conditions encountered in the bores, including strength inversions, it is considered that bored pile foundations would provide the 'lowest risk' of foundation size/depth changes causing delays or significant construction cost increases during installation, unless either conservative working bearing pressures are adopted for pad footing design, or footing locations are pre-drilled (refer Section 5.10.1). Consideration could possibly be given to the use of pad footings to support any more lightly loaded columns/walls if/where present.

5.8.1 Maximum Bearing Pressure

Maximum allowable working pressure values are given in Table 39 provided that foundation pre-drilling has been undertaken (refer Section 5.10.1) for medium strength and stronger rock. Ultimate (failure) bearing pressures can be estimated by multiplying the working stress values by 2.5. It should be carefully noted that the potential presence of 'strength inversions' in the rock will require careful consideration in foundation design and the selection of maximum bearing pressures/founding depths.

Table 39: Maximum Working Bearing Pressures

Rock Strength	Maximum Allowable Working Bearing Pressure (kPa)(1)		
	Pad Footings	Bored Piles	
		Shaft	Base
extremely low	350	20	350
very low	500	75	750
low	1,000	150	1,500
medium	2,500	350	3,500(2)
high	5,000(2)	600	6,000(2)

(1) Not underlain by lower strength rock; foundation pre-drilling is not undertaken (refer Section 5.10.1)

(2) Provided core drilling to confirm rock strength and uniformity is carried out, otherwise limit to 2,500kPa (refer Section 5.10.1)

It is considered that local variations in (soil and) rock strength could be expected to occur over the site and it is suggested that a 'flexible' approach be adopted to the foundation design, construction methodology and costing, so that footing sizes/founding depths can be readily adjusted as required during construction, without cost/time penalties being incurred. Use of mass concrete may be required to transfer foundation stresses to suitable founding strata.

To provide a preliminary guide to the potential length of bored piles that may be required to carry anticipated column working loads over the northern area of the proposed NCP structure (i.e. in the vicinity of Bores 3A and 9), estimates of pile lengths below lowest bulk excavation level (of approximately RL9.5m at the location of Bore 3A and approximately RL6m at the location of Bore 9) have been made at selected bore locations, on the basis that medium strength rock exists below bore termination depth and the estimates are given in Table 40. The pile length estimates given in Table 40 are based on a maximum working end bearing capacity of 2,500kPa and the estimated pile lengths must be confirmed (and adjusted as required, based on the actual ground conditions encountered), by inspection during bored pile excavation and a pile diameter of 1.8m.

Table 40: Preliminary Estimates of Bored Pile Lengths (To Be Confirmed By Pile Inspection)

Bore	Maximum Pile Working Load (MN)	Pile Diameter (m)	Preliminary Estimate of 1.8m Diameter Bored Pile Length Below Approximate Bulk Excavation Level/Pile Toe Reduced Level	
			Pile Length Below Approximate Bulk Excavation Level (m)	Pile Toe Reduced Level (mAHD)
3A	15	1.8	10.7	RL-1.2
9	20	1.8	31.2	RL-25.2

5.8.2 Estimated Settlements

Foundation settlement analysis should be undertaken on a foundation by foundation basis, as part of detailed foundation design and the settlement modulus values given in Table 41 could be used as part of the analysis. Group effects must be considered where 'interaction' of foundations can occur.

Table 41: Estimated Settlement Modulus and Poisson's Ratio Values

Material	Strength	Estimated Settlement Modulus E' (MPa)	Poisson's Ratio
Rock	extremely low very low low medium high	30 – 50 50 – 70 70 – 200 200 – 800 800 – 4,000	0.25

5.8.2.1 Pad Footings

As a broad guide, preliminary estimates of pad footing settlement have been made for a nominal 10,000kN structural working load founding in very low strength rock, using the settlement modulus values given in Table 41 and the estimates are given in Table 42 for a single footing. Group effects must be considered where 'interaction' of footings can occur.

Table 42: Estimated Isolated Pad Footing Settlements

Footing (m)	Rock Strength	Maximum Allowable Working Pressure (kPa)	Estimated Settlement* (mm)
4.5 x 4.5	very low	500	29 – 41

* No allowance for underlying 'softer' zones

Given the 'significant' magnitude and range of estimated settlements given in Table 42, it is expected that piled footings would be considered as the preferred foundation option to support the NCP structure.

5.8.2.2 Bored Piles

As a guide to the potential load settlement response of a single bored pile socket, preliminary load – settlement analysis has been undertaken for:

- relatively 'heavily loaded' bored piles with the column load carried on a 8.5m rock socket constructed in medium strength rock for 1.0m diameter pile; and
- more 'lightly loaded' bored piles with the pile load carried on a 3m rock socket constructed in medium strength rock for 0.6m and 0.75m diameter piles.

In each case the estimates have been done for the lower and upper range of settlement modulus values given in Table 41 and the results are plotted in Figure 12 to Figure 14. The estimated load/settlement curves provided do not include any load/strength factors and represent the anticipated range of actual settlements under unfactored working loads for medium strength rock.

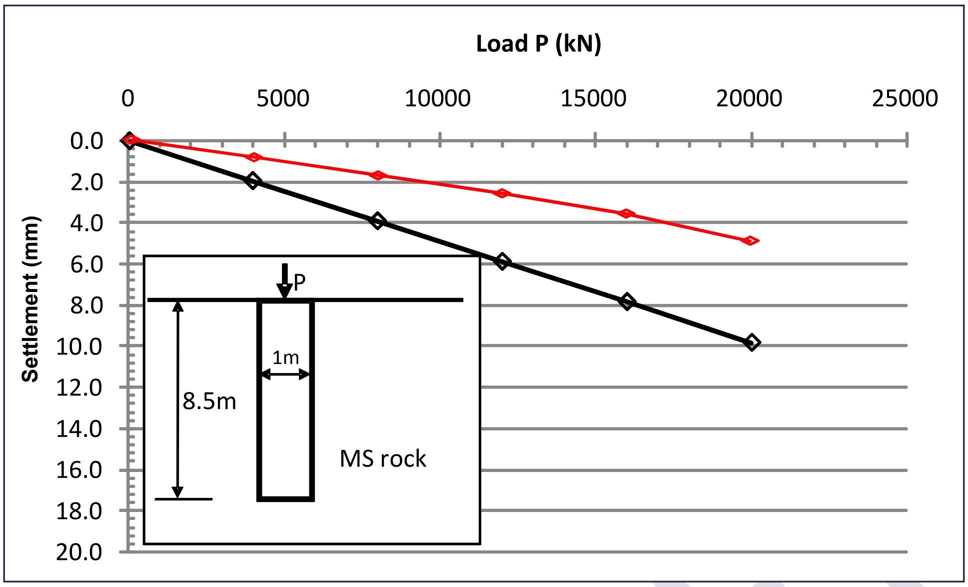


Figure 12: Estimated Range of Load Settlement Response – 1.0m diameter pile (8.5m socket)

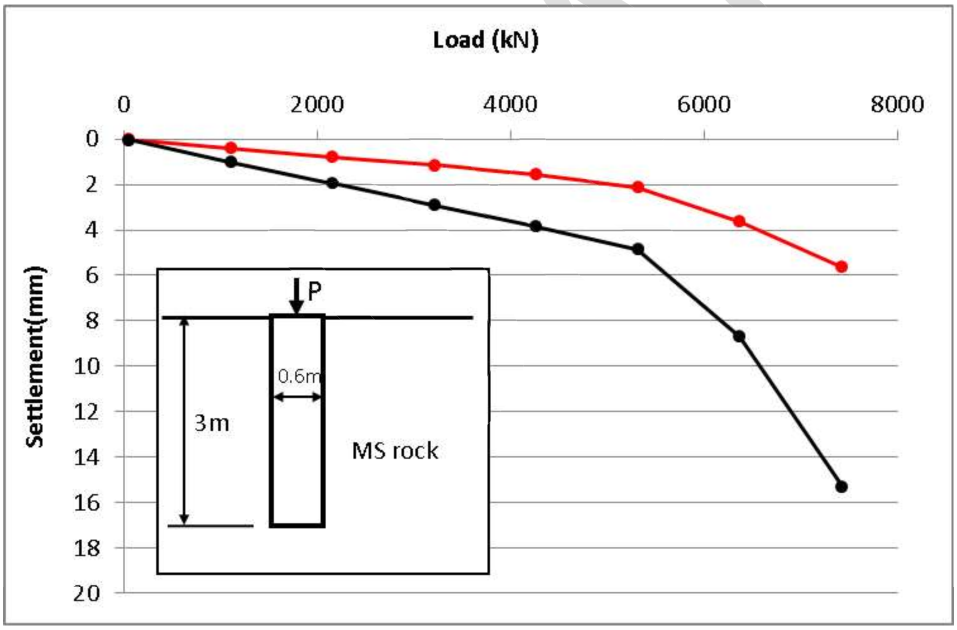


Figure 13: Estimated Range of Load Settlement Response – 0.6m diameter pile (3m socket)

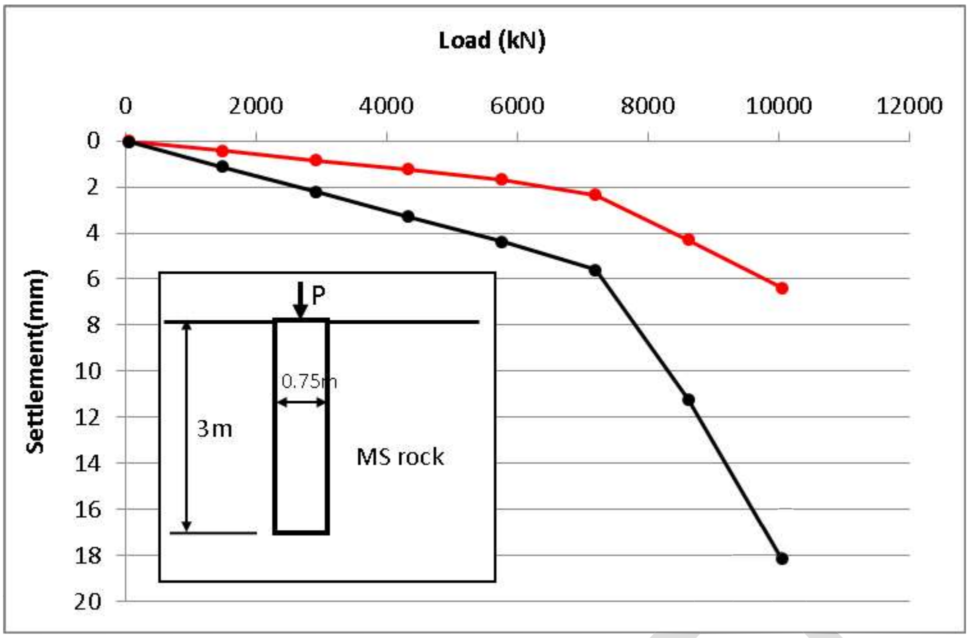


Figure 14: Estimated Range of Load Settlement Response – 0.75m diameter pile (3m socket)

5.8.3 Floor Slab Subgrade Properties

Subgrade properties will vary significantly over the site following excavation and testing will be required at the time of construction in order to confirm design values. For the purposes of initial costing and preliminary design, the values given in Table 43 may be adopted.

Table 43: Floor Slab Subgrade Properties

Subgrade Type	Strength	CBR (%)	Modulus of Subgrade Reaction (kPa/mm)
Shaley Clay(1)	Stiff	2 – 3	20 – 30
Rock	extremely low to very low low strength medium strength (or stronger)	2 – 3(2) 20 – 40(3) 40 – 60(3)	2 – 3(2) 80 – 120(3) 120 – 160(3)

(1) Not less than stiff; (2) Breakdown under trafficking/compaction likely; (3) No breakdown under compaction

The weathered argillite/siltstone at the site would be expected to breakdown significantly under the action of excavation equipment and tracked plant. As a result, where very low strength (or weaker) rock is exposed at subgrade level, it is possible that if the rock is disturbed, recompaction will result in a 'clayey gravel' (or possibly gravelly clay) type subgrade with properties significantly degraded below those of undisturbed rock.

5.9 Earthquake Site Factor

With reference to Australian Standard AS1170.4 – 2007 (R2018/Amdt2-2018) *Structural design actions, Part 4 - Earthquake actions in Australia*, it is considered that the following may be adopted for the site:

• Hazard Design Factor (Z):	0.08
• Class Definitions:	The Class Definition at the currently proposed excavation level of approximately RL5.5m to RL10.5m would be expected to be Class C_e- Shallow Soil Site.

5.10 Construction Monitoring

5.10.1 Foundation Pre-Drilling

Because of the 'high' column loads and foundation bearing pressures proposed for tower support, an appropriate number of 'cored' bores should be located beneath all 'major' foundations to a depth below founding levels of not less than two to three times minimum footing width, to ensure that clay seams, 'fragmented zones' or other rock defects do not exist within the zone of foundation stress influence and to assess in detail the location specific properties of the rock mass for confirmation/modification of working bearing pressures on a foundation by foundation basis as required. Additional insitu testing carried out during foundation predrilling (e.g. high capacity pressure meter testing) should be considered for the optimisation of location specific foundation founding depth and bearing capacity.

If cored bores are not undertaken, working design bearing pressures would require very substantial reductions from the values given in Table 39 to account for the uncertainty associated with encountering defects in the rock mass within the zone of foundation stress influence that could result in higher than anticipated foundation deflections under load.

5.10.2 Groundwater Level

It is suggested that groundwater levels around the site (outside the site boundary) be regularly monitored during the excavation and basement construction period to confirm that no offsite groundwater table lowering occurs as a result of the dewatering required for construction of the basement structure.

5.10.3 Wall Movement

Wall movements will occur as a consequence of the excavation process and these movements are not possible to prevent and are difficult to accurately predict. As a consequence, it is strongly recommended that the performance of the excavation support should be carefully monitored. It is considered essential than an accurate survey monitoring program of the excavation boundary, adjacent buildings, and retention system be put in place for not less than the duration of the excavation works, so that if untoward movement does occur (e.g. due to latent, adverse ground conditions) excavation face support loads can be upgraded/modified, if required.

The monitoring must be implemented prior to excavation commencing and without delay as construction proceeds, and all survey monitoring should be to an accuracy of not less than 1mm (horizontal and vertical), so that any movement trends can be readily identified.

5.10.4 Vibration

Vibration will be caused by excavation work at the site and will require monitoring and assessment to avoid nuisance and to avoid damage to adjoining structures.

British Standard BS 7385: Part 2 – 1993 *Evaluation and Measurement for Vibration in Buildings* provides vibration damage limits against which the likelihood of cosmetic building damage from ground vibration can be assessed and this Standard is referenced in Appendix J in Australian Standard AS 2187: Part 2 – 2006 *Explosives – Storage and use – Use of explosives* for assessment of transitory vibrations. Sources of vibration which are considered in the Standard include demolition, blasting (carried out during mineral extraction or construction excavation), piling, ground treatments (e.g. compaction), construction equipment, tunnelling, road and rail traffic and industrial machinery.

BS 7385 – 2 sets guide values which are given in Table 44 and are for building vibration based on vibration levels below which damage has been credibly demonstrated to be cosmetic only. These levels are judged to give a **minimal risk of vibration induced cosmetic damage**, where ‘minimal risk’ for a named effect is usually taken as a 95% probability of no effect.

Table 44: Transient Vibration Guide Values for Cosmetic Damage (BS 7385 – 2)

Line	Type of Building	Peak Component Particle Velocity in Frequency Range of Predominant Pulse	
		4Hz to 15Hz	15Hz and above
1	Reinforced or framed structures, industrial and heavy commercial buildings	50mm/s at 4Hz and above	
2	Unreinforced or light framed structure. Residential or light commercial type buildings	15mm/s at 4Hz increasing to 20mm/s at 15Hz	20mm/s at 15Hz increasing to 50mm/s at 40Hz and above

Alternatively, German Standard DIN 4150: Part 3 – 1986 also provides commonly referenced guidelines for evaluating the effects of vibration on structures. The DIN Standard give ‘safe levels’ up to which **no cosmetic damage** due to vibration effects has been observed and these levels are reproduced in Table 45.

Table 45: Vibration Guideline for Evaluating the Effects of Short-Term Vibration on Structures (DIN4150 – 3)

Line	Type of Structure	Guideline Values for Velocity, v_i, in mm/s			
		Vibration at the Foundation at a Frequency of			Vibration at Horizontal Plane of Highest Floor at All Frequencies
		1Hz to 10Hz	10Hz to 50Hz	50Hz to 100Hz*	
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or occupancy	5	5 to 15	15 to 20	15
3	Structures that, because of their particular sensitivity to vibration, cannot be classified under lines 1 and 2 and are of great intrinsic value (e.g. listed buildings under preservation order)	3	3 to 8	8 to 10	8

* At frequencies above 100Hz, the values given in this column may be used as minimum values

The DIN 4150 – 3 Levels are more conservative than BS 7385 – 2 (generally twice as stringent), to avoid a small risk of cosmetic cracking.

Unless very heavy and sustained hydraulic rock breaker excavation is undertaken in close proximity to site boundaries, it is considered unlikely that excavation induced vibration will pose any significant threat to adjacent buildings. However, a dilapidation survey of adjacent buildings (and services) is strongly recommended prior to commencement of site work.

BUTLER PARTNERS PTY LTD

CAMERON MURRAY
Senior Associate

BRUCE BUTLER
Senior Principal

Important Information about Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you —* should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

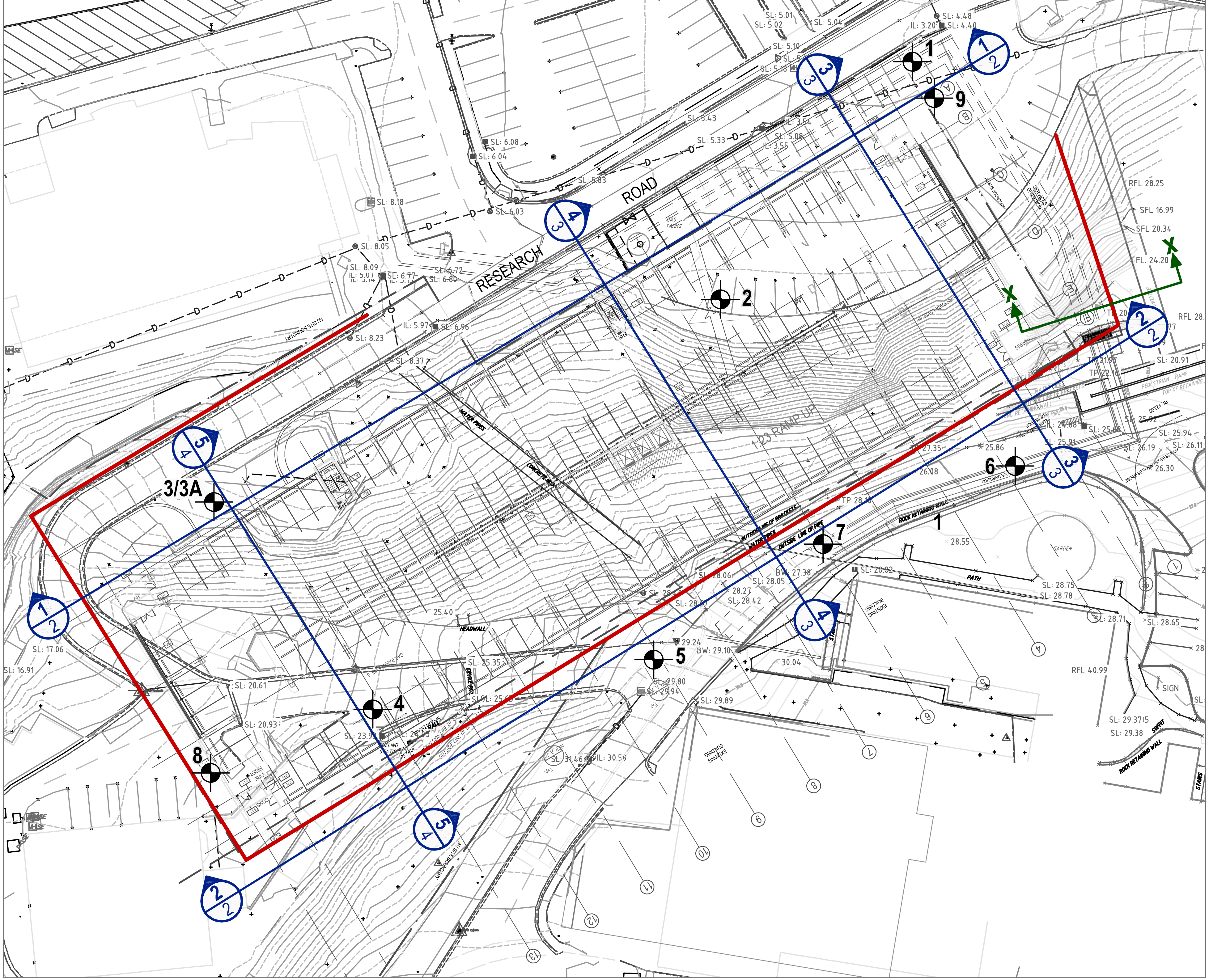
Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

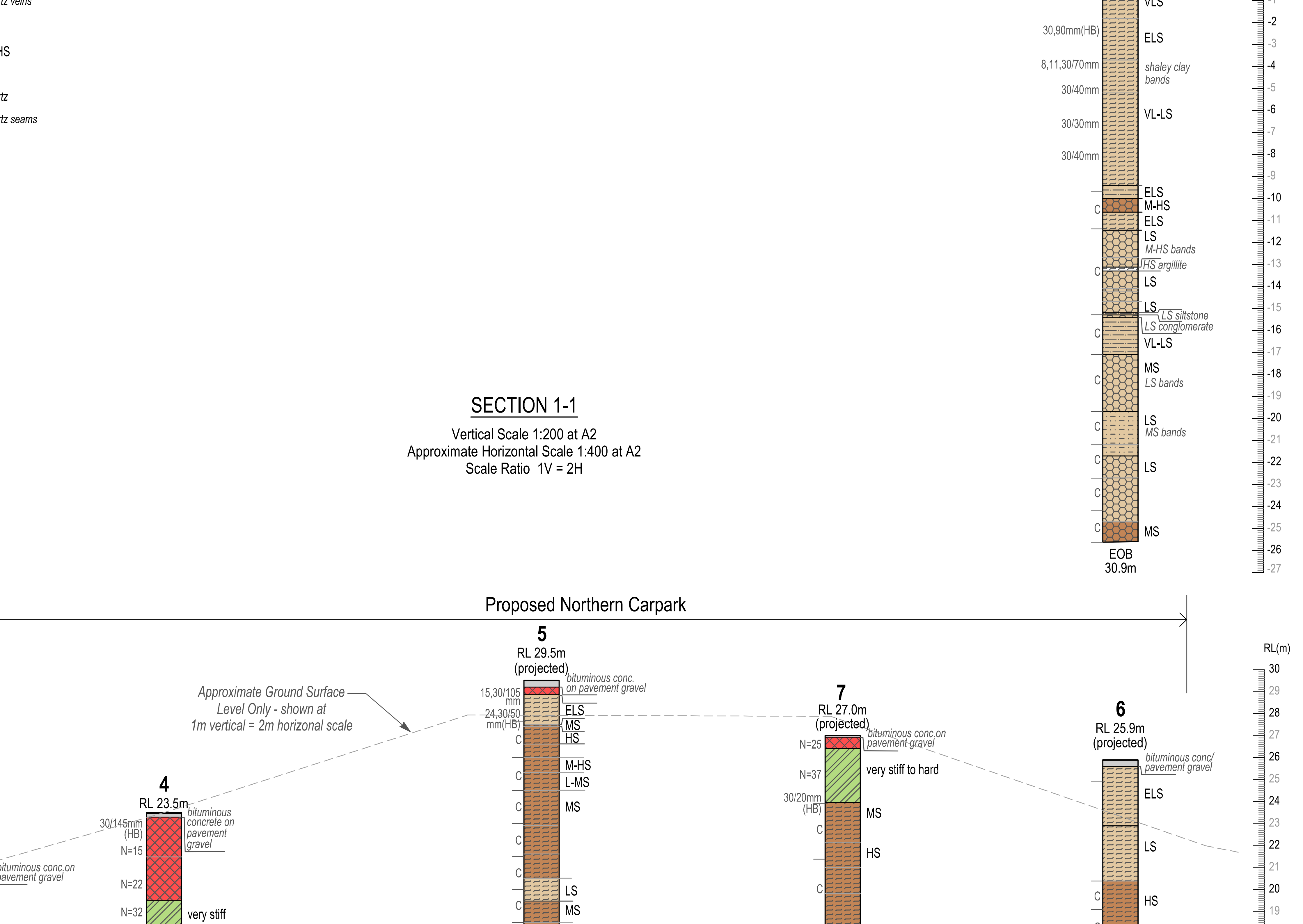
Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/THE BEST PEOPLE ON EARTH exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

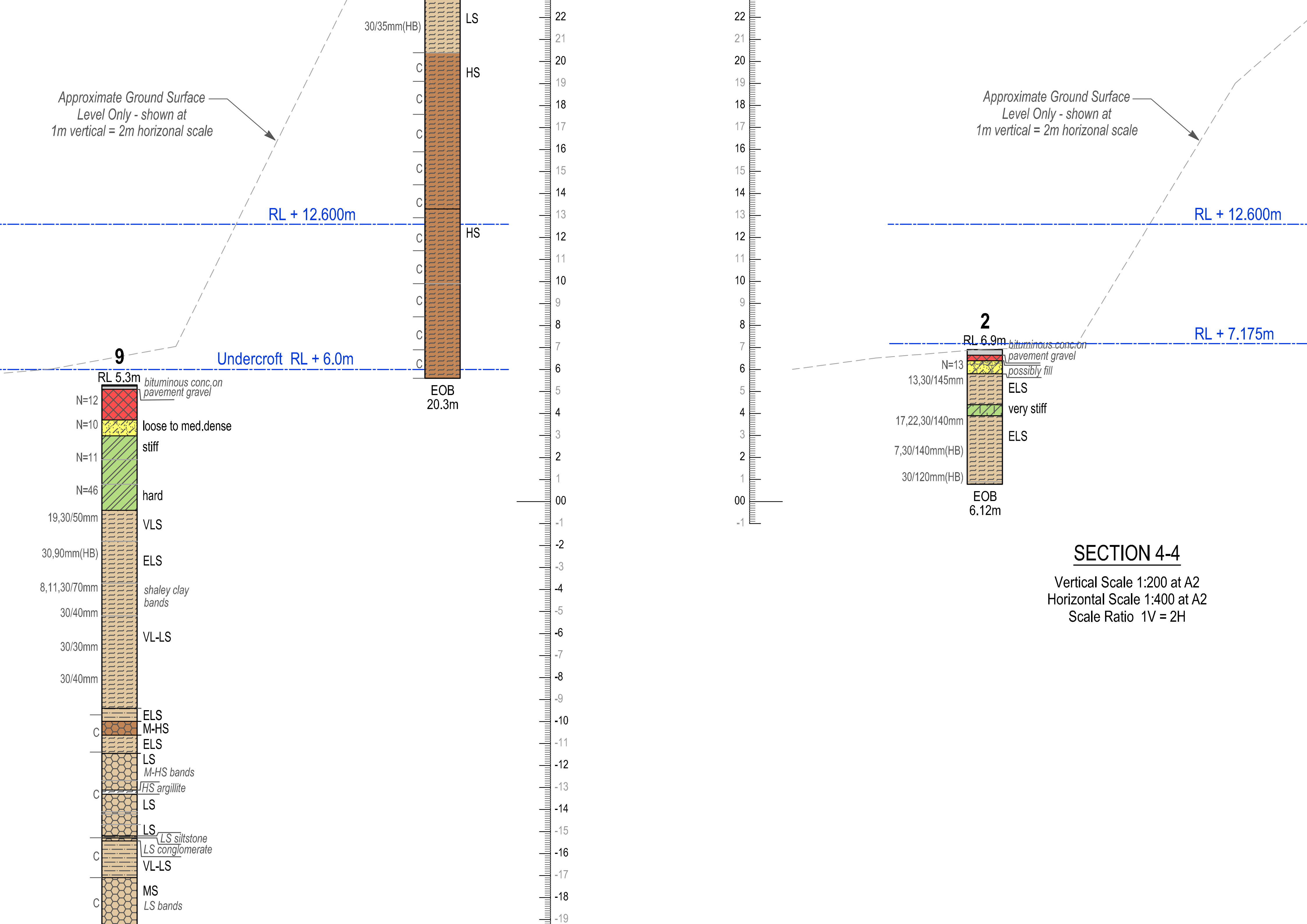
8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org


RESEARCH ROAD
1:23 RAMP UP
1
2
3/3A
4
5
6
7
8
9
RFL 28.25
SFL 16.99
SFL 20.34
FL. 24.20
SERVICES DIVERSION
AU SITE BOUNDARY
GAS TANKS
WATER PIPES
CONCRETE PIPE
HEADWALL
SERVICE PIT
OUTSIDE LINE OF BRACKETS
OUTSIDE LINE OF PIPE
ROCK RETAINING WALL
GARDEN
PATH
EXISTING BUILDING
SIGN
SOFFIT
STAIRS
PEDESTRIAN RAMP
TOP OF RETAINING
RFL 40.99
SL: 4.48
SL: 4.40
IL: 3.20
SL: 5.01
SL: 5.02
SL: 5.04
SL: 5.10
SL: 5.18
SL: 5.43
SL: 5.33
SL: 5.08
IL: 3.54
IL: 3.55
SL: 6.08
SL: 6.04
SL: 5.83
SL: 8.18
SL: 6.03
SL: 8.05
SL: 8.09
SL: 6.77
SL: 6.72
SL: 6.80
IL: 5.97
SL: 6.96
SL: 8.23
SL: 8.37
TP 21.97
TP 22.16
SL: 20.91
SL: 25.68
SL: 25.92
SL: 25.94
SL: 26.11
SL: 26.19
26.30
SL: 25.91
25.86
27.35
26.08
TP 28.10
28.55
SL: 28.75
SL: 28.78
SL: 28.71
SL: 28.65
SL: 28.82
BW: 27.38
SL: 28.05
28.27
SL: 28.42
SL: 28.06
25.40
29.24
BW: 29.10
30.04
SL: 29.80
SL: 29.94
SL: 29.89
SL: 25.35
SL: 20.61
SL: 20.93
SL: 23.93
SL: 31.46
IL: 30.56
SL: 17.06
SL: 16.91
SL: 29.3715
SL: 29.38
FIRE RISER
COMS
LV
HV



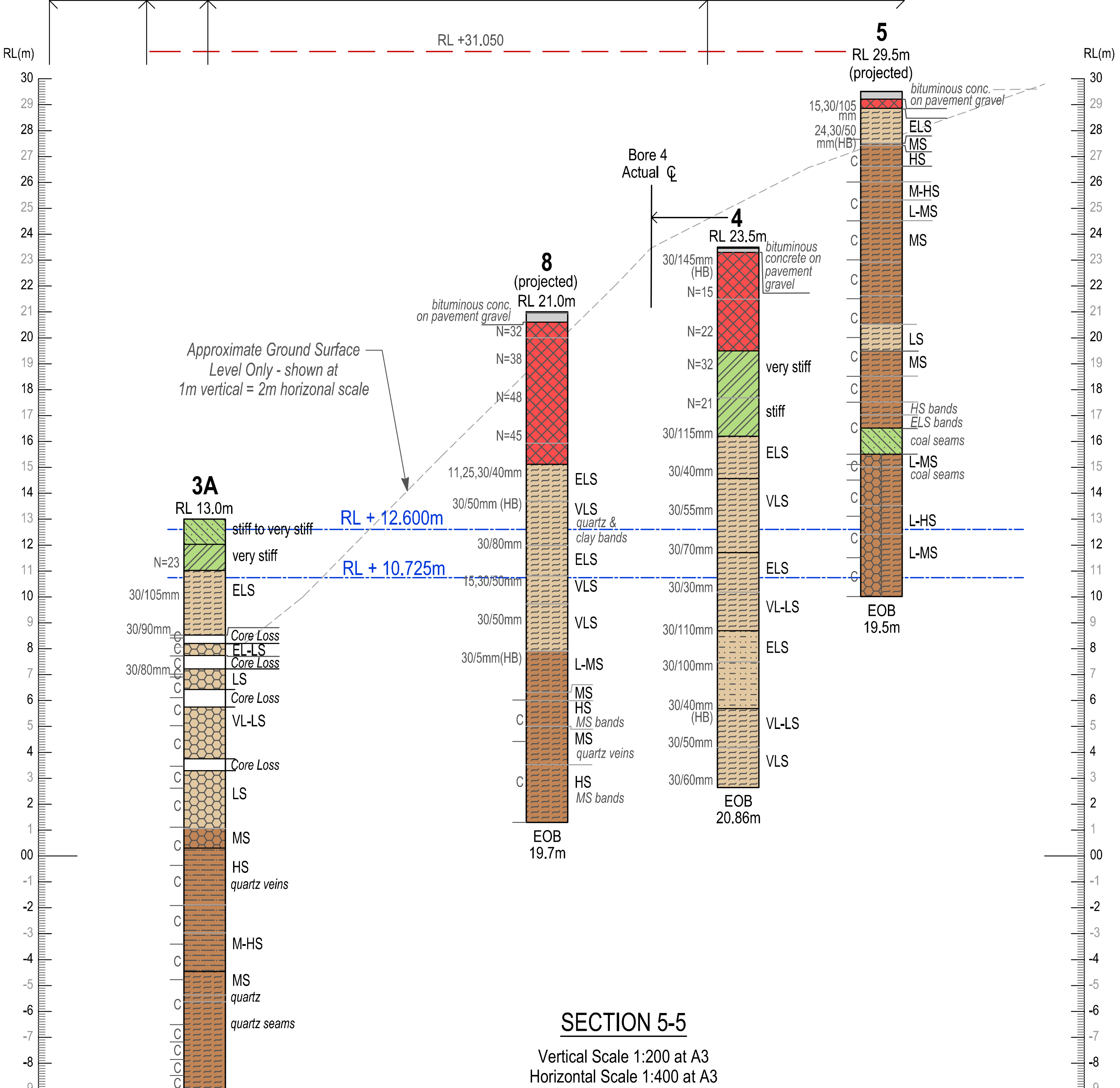
SECTION 1-1

Vertical Scale 1:200 at A2
Approximate Horizontal Scale 1:400 at A2
Scale Ratio 1V = 2H



SECTION 4-4

Vertical Scale 1:200 at A2
Horizontal Scale 1:400 at A2
Scale Ratio 1V = 2H



SECTION 5-5

Vertical Scale 1:200 at A3
Horizontal Scale 1:400 at A3

APPENDIX A

CURRENT INVESTIGATION BORE REPORT SHEETS WITH EXPLANATORY NOTES

BORE REPORT

Butler Partners
geotechnical • geo-environmental • groundwater

BORE 3A

Client: Herston Development Co Pty Ltd
Project: Herston Quarter Redevelopment - Northern Car Park
Location: Research Road, Herston
Project No: 017-141C

DRAFT

Page No: 1 of 3
Date: 3 May 2018
Ground Surface Level: RL13.0m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
0		13.0										
	SANDY CLAY (CI) - stiff to very stiff, brown, fine to coarse grained sand											
1	***SHALEY CLAY (CI)*** - very stiff, orange-brown	12.0										
						S	1.5 1.95					6,10,13 N=23
2		11.0										
	ARGILLITE (XW/HW) - extremely low strength, grey-brown mottled orange											
3		10.0				S	3.0 3.11					30/105mm
4		9.0										
	CORE LOSS			4.85m to 5.30m,C	>15	S/C C	4.5 4.59 5.3		50	0		30/90mm
5	***CONGLOMERATE (HW)*** - extremely low to low strength, grey mottled with dark grey and white, fragmented to highly fractured	8.0		5.85m to 6.0m,C 6.0m to 6.20m,C	>15	C			15	0		
6	***CORE LOSS***	7.0										30/80mm
	CONGLOMERATE (HW) - low strength, grey-brown			6.50m,J,75,iv,clean,1mm	>15	S/C C	6.0 6.08 6.4 6.9	0.1(d) 0.2(a)	65	25		
7	***CORE LOSS***	6.0		7.35m to 8.0m,C	>15	C	7.5	0.1(d) 0.2(a)	60	0		
	CONGLOMERATE (HW) - very low to low strength, grey-brown											
8		5.0		8.0m to 8.20m,C			8.0					
				8.70m,J,20,vii,clean,3mm 8.80m to 9.17m,S	6 >15	C	8.2 8.3 8.8	0.08(d) 0.06(a)	80	40		
9		4.0										
	CORE LOSS						9.5					
10		3.0			7	C	9.7 9.8	0.05(d) 0.1(a)	70	60		

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Multi-Purpose CE180

Logged by: CM

Drilling Method: Auger to 4.5m, casing to 4.5m, then NMLC

Groundwater: No free groundwater encountered during drilling

Remarks: *Approximate ground surface level interpolated from Land Solutions Australia Pty Ltd,'RBH_Survey_Northern Carpark3_LSA_17108(1).dwg',received 21/11/17

BORE REPORT

Butler Partners
geotechnical • geo-environmental • groundwater

Client: Herston Development Co Pty Ltd

Project: Herston Quarter Redevelopment - Northern Car Park

Location: Research Road, Herston

Project No: 017-141C

DRAFT

BORE 3A

Page No: 2 of 3

Date: 3 May 2018

Ground Surface Level: RL13.0m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
	CONGLOMERATE (MW) - low strength, brown mottled with grey and white, fractured			10.20m,J,15,iv,clean,1mm 10.30m to 10.60m,4J,15,vii,clean,3mm to 5mm			10.3 10.4	0.3(d) 0.3(a)				
11		2.0		10.80m to 11.40m,6J,15 to 30,clean,3mm to 5mm 11.60m to 11.80m,S	7	C	10.7	0.1(d) 0.2(a)	100	26		
12	- medium strength	1.0		12.0m,J,15,vii,clean,1mm 12.20m,J,15,iv,clean,1mm 12.66m to 12.69m,C	3	C	11.9 12.1	0.3(d) 0.6(a)	100	95		
13	***SILTSTONE (SW)*** - high strength, grey mottled with white, slightly fractured, occasional thin quartz veins, bedding planes dipping at approximately 45 degrees	0.0		12.80m,J,20,vii,clean,1mm 13.20m,J,30,vii,clean,1mm			13.1 13.4	0.9(d) 1.3(a)				
14		-1.0		13.90m,J,45,iv,clean,1mm 14.68m,J,30,vii,clean,1mm	2	C	14.1	1.1(d) 2.2(a)	100	100		
15	- fractured	-2.0		15.12m,J,15,vii,clean,3mm 15.23m,J,15,vii,cleam,3mm 15.37m to 15.40m,C 15.48m to 15.53m,C 15.68m,J,30,vii,clean,1mm 15.81m,J,30,vii,clean,1mm	8	C	14.9 15.1	1.6(d) 1.6(a)	100	45		
16	- medium to high strength	-3.0		16.0m to 16.40m,5J,15 to 30,clean,1mm to 3mm 16.46m,quartz,12mm			16.1 16.4	0.9(d) 2.2(a)				
17		-4.0		16.80m to 17.30m,9J,15 to 30,vii to iv,clean,1mm to 3mm 17.25m,J,60,vii,clean,1mm 17.45m to 17.53m,C 17.60m,17.72m,C	10	C	17.1	1.0(d) 1.7(a)	100	45		
	ARGILLITE (MW) - medium strength, dark grey mottled with white, foliations dipping at approximately 30 to 40 degrees, fractured			17.80m to 18.20m,5J,30 to 45,vii,clean,1mm to 3mm			17.8					
18	- quartz	-5.0		18.26m,quartz,30mm 18.35m,quartz,45mm 18.35m to 18.70m,6J,15 to 45,vii,clean,1mm to 2mm 18.41m,quartz,220mm	10	C	18.1 18.8	0.5(d) 1.1(d) 0.4(a)	100	30		
19	- abundant quartz seams	-6.0		19.26m,J,15,vii,clean,3mm 19.28m,J,60,vii,clean,1mm 19.60m to 19.68m,C			19.4 19.5	0.5(d) 0.4(a)				
20		-7.0										

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Multi-Purpose CE180

Logged by: CM

Drilling Method: Auger to 4.5m, casing to 4.5m, then NMLC

Groundwater: No free groundwater encountered during drilling

Remarks: *Approximate ground surface level interpolated from Land Solutions Australia Pty Ltd,'RBH_Survey_Northern Carpark3_LSA_17108(1).dwg',received 21/11/17

BORE REPORT

Butler Partners
geotechnical • geo-environmental • groundwater

DRAFT

BORE 3A

Client: Herston Development Co Pty Ltd
Project: Herston Quarter Redevelopment - Northern Car Park
Location: Research Road, Herston
Project No: 017-141C

Page No: 3 of 3
Date: 3 May 2018
Ground Surface Level: RL13.0m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
	ARGILLITE (MW) - medium strength, dark grey mottled with white, abundant quartz seams			20.16m to 20.20m,S 20.51m,quartz,20mm 20.51m,J,15,vii,clean,1mm 20.74m,J,15,iv,clean,1mm 20.77m,J,45,vii,clean,1mm	4	C	20.2	0.5(d) 1.1(a)	100	75		
21		-8.0		21.10m,quartz,20mm 21.10m,J,60,vii,clean,1mm 21.30m to 21.35m,C 21.42m,J,30,vii,clean,1mm	7	C	20.9		100	15		
				21.50m to 22.10m,8J,15 to 30,vii,clean,1mm to 5mm	12	C	21.5	0.2(d) 0.3(a)	100	0		
22		-9.0										
	End of Bore at 22.1 m						22.1	0.6(d) 0.6(a)				
23		-10.0										
24		-11.0										
25		-12.0										
26		-13.0										
27		-14.0										
28		-15.0										
29		-16.0										
30		-17.0										

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Multi-Purpose CE180

Logged by: CM

Drilling Method: Auger to 4.5m, casing to 4.5m, then NMLC

Groundwater: No free groundwater encountered during drilling

Remarks: *Approximate ground surface level interpolated from Land Solutions Australia Pty Ltd,'RBH_Survey_Northern Carpark3_LSA_17108(1).dwg',received 21/11/17

BORE REPORT

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BORE 7

Client: Herston Development Co Pty Ltd
Project: Herston Quarter Redevelopment - Northern Car Park
Location: Research Road, Herston
Project No: 017-141C

DRAFT

Page No: 1 of 3
Date: 12 May 2018
Ground Surface Level: RL27.0m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
0		27.0										
	BITUMINOUS CONCRETE - 30mm thick					S	0.5 0.95					12,15,10 N=25
1	***PAVEMENT GRAVEL*** - 50mm thick	26.0										
	FILL - brown, silty sand, fine to medium grained sand					S	1.5 1.95					4,14,23 N=37
2	***SHALEY CLAY (CI)*** - very stiff to hard, brown mottled pink-grey and light blue	25.0										
3	***TUFF (SW)*** - medium strength, light blue mottled with grey-brown, fractured to slightly fractured	24.0		3.0m to 3.37m,C 3.38m,J,10,vii,Fe,2mm 3.40m,J,75,vii,Fe,5mm	>20	(S)	3.0 3.02		100	0		30/20mm (HB)
				3.77m,J,45,vii,Fe,1mm 3.87m,J,15,iv,Fe,1mm 3.96m,J,30,vii,Fe,1mm	6	C	3.5 3.7	0.8(d) 0.5(d) 0.9(a)		65		
4		23.0		4.20m,J,30,vii,Fe,1mm 4.32m,J,30,vii,Fe,1mm 4.36m,J,30,vii,Fe,1mm 4.40m,J,45,vii,Fe,3mm 4.58m,J,45,vii,Fe,1mm 4.62m,J,30,vii,Fe,5mm			4.8	1.2(d) 1.1(d) 1.5(a)				
5	- high strength	22.0		5.15m,J,30,vii,Fe,1mm 5.19m,J,0,vii,Fe,2mm 5.52m,J,30,vii,Fe,1mm								
				5.90m,J,45,vii,Fe,3mm 5.93m,J,15,vii,Fe,1mm		C	5.6 5.9 6.0	1.4(d) 1.8(a)	100	70		
6	- slightly fractured	21.0		6.15m,J,15,vii,Fe,3mm 6.37m,J,15,vii,Fe,3mm	2							
7		20.0		7.23m,J,15,vii,Fe,5mm 7.28m,J,10,vii,Fe,5mm			6.9 7.2	1.1(d) 1.9(a)				
	- fractured to slightly fractured			7.34m,J,15,vii,Fe,3mm 7.40m,J,60,vii,clean,1mm 7.78m,J,60,vii,Fe,3mm	5		7.5	1.0(d) 1.0(d) 1.1(a)				
8		19.0		8.0m,J,75,vii,Fe,1mm 8.10m,J,75,vii,Fe,1mm 8.37m,J,30,iv,clean,1mm 8.47m,J,15,vii,Fe,2mm 8.55m,J,30,iv,Fe,2mm								
					6	C	8.6		100	55		
9		18.0		9.15m,J,15,vii,Fe,1mm 9.24m,J,15,iv,clean,1mm 9.30m,J,60,vii,Fe,1mm 9.68m to 9.70m,C 9.72m,J,15,vii,Fe,1mm 9.85m,J,30,vii,Fe,2mm			9.0	1.1(d) 2.1(a)				
10		17.0		10.10m,J,45,vii,Fe,1mm 10.23m,J,60,vii,Fe,10mm 10.35m,J,60,vii,Fe,15mm 10.58m,J,45,vii,Fe,1mm 10.42m,J,30,vii,Fe,1mm 10.72m,J,45,vii,Fe,1mm 10.82m,J,80,vii,Fe,10mm			10.0 10.6 10.9	1.6(d) 2.5(a) 1.7(d) 1.8(a)				
11	***TUFF (SW)*** - high strength, fractured to slightly fractured	16.0		11.13m,J,45,vii,Fe,1mm 11.58m,J,45,vii,Fe,1mm 11.61m,J,30,vii,Fe,1mm								

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Hydrapower Scout

Logged by: PZ

Drilling Method: Auger to 3.0m, casing to 3.0m, then washbore

Groundwater: No free groundwater encountered during drilling

Remarks: *Approximate ground surface level interpolated from Land Solutions Australia Pty Ltd,'RBH_Survey_Northern Carpark3_LSA_17108(1).dwg',received 21/11/17

BORE REPORT

Butler Partners
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DRAFT

BORE 7

Client: Herston Development Co Pty Ltd
Project: Herston Quarter Redevelopment - Northern Car Park
Location: Research Road, Herston
Project No: 017-141C

Page No: 2 of 3
Date: 12 May 2018
Ground Surface Level: RL27.0m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
12	***TUFF (SW)*** - high strength, fractured to slightly fractured	15.0		11.80,J,45,vii,Fe,2mm 12.15m to 12.19m,C 12.52m,J,30,vii,Fe,2mm 12.57m,J,30,vii,Fe,3mm 12.71m,J,30,vii,Fe,1mm 12.84m,J,45,vii,Fe,1mm	5	C	12.0	2.8(d) 3.3(a)	100	80		
13	- slightly fractured	14.0		13.20m,J,75,vii,Fe,3mm 13.45m,J,0,vii,clean,1mm 13.85m,J,75,vii,Fe,2mm	3		13.0 13.6	1.0(d) 2.4(a) 3.2(d) 2.4(a)				
14	- fractured to slightly fractured	13.0		14.12m,J,0,vii,clean,1mm 14.25m,J,0,vii,Fe,1mm 14.57m,J,0,vii,Fe,1mm 14.69m,J,0,vii,clean,1mm 14.94m to 15.07m,clay 15.0m to 15.15m,3J,vii,0,clean,3mm	7	C	14.0 14.8	1.8(d) 1.5(a)	100	70		
15	- approximately 130mm thick clay band	12.0		15.32m,J,30,vii,Fe,1mm	4	C	15.15 15.2	1.2(d) 0.6(a)	100	45		
16	- with very high strength bands	11.0		16.0m,J,60,vii,Fe,1mm 16.35m,J,60,vii,Fe,1mm 16.50m,J,0,vii,clean,1mm 16.95m,J,60,vii,Fe,2mm 17.0m to 17.80m,C			16.2	3.1(d) 3.5(d) 3.2(a)				
17 18	- highly fractured to fragmented	10.0 9.0		17.80m to 18.10m,C 18.24m,J,45,vii,Fe,3mm	>20		17.0 17.8			0		
19	- fractured to slightly fractured	8.0		18.30m,J,30,vii,Fe,3mm 18.63m,J,45,vii,clean,1mm 18.77m,J,0,vii,clean,1mm 18.84m,J,45,vii,clean,1mm 18.89m,J,45,vii,clean,1mm 19.15m,J,30,vii,clean,1mm 19.20m to 19.30m,3J,15,vii,clean,1mm	6	C	18.2 18.5	0.9(d) 1.4(a)	100	25		
	- approximately 200mm thick clay band			19.60m to 19.80m,clay 19.90m,J,75,vii,clean,3mm			19.7	1.3(d) 1.9(a)				
20	***CARBONACEOUS ARGILLITE (XW/HW)*** - very low to low strength, dark grey-black, with coal seams, fragmented	7.0		20.0m to 20.60m,C 20.80m to 21.0m,S	>20 >20	C	20.0 20.6 20.7 21.0	0.2(d) 0.2(a)	100	0 0		
21 22 23	***CONGLOMERATE (MW)*** - low strength, grey mottled brown-white, fractured to slightly fractured	6.0 5.0 4.0		21.20m,J,90,iv,clean,1mm 21.45m,J,45,vii,clean,1mm 21.56m,J,10,vii,clean,1mm 21.85m,J,30,vii,clean,1mm 22.08m,J,15,vii,clean,1mm 22.24m,J,30,vii,clean,1mm 22.78m,J,30,vii,clean,3mm	4	C	21.7 22.7	0.3(d) 0.2(a) 0.2(d) 0.3(a)	100	60		

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Hydrapower Scout

Logged by: PZ

Drilling Method: Auger to 3.0m, casing to 3.0m, then washbore

Groundwater: No free groundwater encountered during drilling

Remarks: *Approximate ground surface level interpolated from Land Solutions Australia Pty Ltd,'RBH_Survey_Northern Carpark3_LSA_17108(1).dwg',received 21/11/17

BORE REPORT

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Client: Herston Development Co Pty Ltd
Project: Herston Quarter Redevelopment - Northern Car Park
Location: Research Road, Herston
Project No: 017-141C

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BORE 7

Page No: 3 of 3
Date: 12 May 2018
Ground Surface Level: RL27.0m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
	CONGLOMERATE (MW) - low strength, grey mottled brown-white, fractured to slightly fractured			23.25m,J,30,vii,clean,1mm 23.38m,J,15,vii,clean,1mm			23.4 23.7	0.09(d) 0.1(a)				
24	End of Bore at 23.7 m	3.0										
25		2.0										
26		1.0										
27		0.0										
28		-1.0										
29		-2.0										
30		-3.0										
31		-4.0										
32		-5.0										
33		-6.0										
34		-7.0										

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Hydrapower Scout

Logged by: PZ

Drilling Method: Auger to 3.0m, casing to 3.0m, then washbore

Groundwater: No free groundwater encountered during drilling

Remarks: *Approximate ground surface level interpolated from Land Solutions Australia Pty Ltd,'RBH_Survey_Northern Carpark3_LSA_17108(1).dwg',received 21/11/17

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BORE 8

DRAFT

Client: Herston Development Co Pty Ltd
Project: Herston Quarter Redevelopment - Northern Car Park
Location: Research Road, Herston
Project No: 017-141C

Page No: 1 of 2
Date: 26 April 2018
Ground Surface Level: RL21.0m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
0		21.0										
	BITUMINOUS CONCRETE - 40mm thick											
	PAVEMENT GRAVEL - grey and brown, fine to coarse subangular, with fine to medium grained sand					S	0.5 0.95					16,15,17 N=32
1		20.0										
	FILL - brown, sandy gravel, fine to coarse angular, fine to coarse grained sand, with clay					S	1.5 1.95					11,15,23 N=38
2		19.0										
	- brown mottled grey and pale grey, clayey gravelly sand, fine to medium grained, fine to medium subangular											
3		18.0				S	3.0 3.45					14,20,28 N=48
4		17.0										
						S	4.5 4.95					21,20,25 N=45
5		16.0										
	- with quartz, fine to medium subangular to angular											
6	***ARGILLITE (XW)*** - extremely low strength, mottled orange-brown with grey	15.0				S	6.0 6.34					11,25,30/40mm
7		14.0										
	- very low strength, mottled grey-brown, with quartz and clay bands					S	7.5 7.55					30/50mm (HB)
8		13.0										
9	- extremely low strength	12.0				S	9.0 9.08					30/80mm
10		11.0										
	- very low strength, pale grey with mottled orange					S	10.5 10.7					15,30/50mm
11		10.0										
	ARGILLITE (XW/HW)											

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Hydrapower Scout

Logged by: RZ/BW

Drilling Method: Auger to 3.0m, casing to 3.0m, washbore to 15.0m, then NMLC

Groundwater: No free groundwater encountered during drilling

Remarks: *Approximate ground surface level interpolated from Land Solutions Australia Pty Ltd,'RBH_Survey_Northern Carpark3_LSA_17108(1).dwg',received 21/11/17

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BORE 8

DRAFT

Client: Herston Development Co Pty Ltd
Project: Herston Quarter Redevelopment - Northern Car Park
Location: Research Road, Herston
Project No: 017-141C

Page No: 2 of 2
Date: 26 April 2018
Ground Surface Level: RL21.0m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
	ARGILLITE (XW/HW) - very low strength, pale grey											
12		9.0				S	12.0 12.05					30/50mm
13	***ARGILLITE (HW)*** - low to medium strength, pale grey	8.0				(S)	13.5 13.505					30/5mm (HB)
14		7.0										
	ARGILLITE (MW) - medium strength, pale grey											
15		6.0		15.0m to 15.27m,5J,0 to 30,vii,clean and gravel,1mm to 15mm 15.37m to 15.57m,5J,0 to 35,vii,clean,1mm	15	C	15.0 15.1	0.3(d) 1.3(a)	100	70		
	ARGILLITE (MW/SW) - high strength, grey-brown, foliations dipping at approximately 30 to 40 degrees, with medium strength bands, fractured			15.70m,J,30,vii,clean,1mm 15.95m,J,5,vii,clean,1mm 16.09m,J,35,vii,clean,1mm	3		15.6 15.92	2.4(d) 1.9(a)				
16		5.0					16.6	0.7(d) 0.8(a)				
	ARGILLITE (SW) - medium strength, grey, with quartz veins up to 20mm thick, slightly fractured					C			100	85		
17		4.0		17.20m,J,30,vii,clean,1mm 17.46m,J,0,vii,clean,1mm 17.6m,J,30,vii,clean,1mm			17.55 17.6	1.4(d) 3.8(a)				
	- high strength, fractured to slightly fractured, with medium strength bands			17.95m,J,35,i,clean,1mm 18.0m to 18.09m,2J,20 to 30,vii and iv,clean,1mm 18.2m to 18.52m,3J,10 to 30,vii and iv,clean,1mm	6							
18		3.0					18.45 18.6	0.5(d) 0.7(a)				
19		2.0		18.82m to 19.15m,3J,40 to 60,vii,clean,Fe,1mm 19.31m,J,10,vii,clean,1mm	4		19.35 19.7	1.8(d) 1.4(a)				
	End of Bore at 19.7 m											
20		1.0										
21		0.0										
22		-1.0										
23		-2.0										

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Hydrapower Scout

Logged by: RZ/BW

Drilling Method: Auger to 3.0m, casing to 3.0m, washbore to 15.0m, then NMLC

Groundwater: No free groundwater encountered during drilling

Remarks: *Approximate ground surface level interpolated from Land Solutions Australia Pty Ltd,'RBH_Survey_Northern Carpark3_LSA_17108(1).dwg',received 21/11/17

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BORE 9

Client: Herston Development Co Pty Ltd
Project: Herston Quarter Redevelopment - Northern Car Park
Location: Research Road, Herston
Project No: 017-141C

Page No: 1 of 3
Date: 28 April and 19 May 2018
Ground Surface Level: RL5.3m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results	Groundwater Monitoring Welll Details
0	***BITUMINOUS CONCRETE*** - 50mm thick	5.3											Cement
1	***PAVEMENT GRAVEL*** - 150mm thick					S	0.5 0.95					13,23,9 N=32	
2	***FILL*** - brown, clayey gravelly sand, fine to coarse grained, fine to coarse subangular to subrounded gravel, with steel wire	4.0 3.0				S	1.5 1.95					4,4,6 N=10	
3	***CLAYEY SAND (SC)*** - loose to medium dense, green-grey mottled orange-brown (possibly fill)	2.0				S	3.0 3.45					2,5,6 N=11	
4	***SHALEY CLAY (CI)*** - stiff, white-yellow												Casing
	- orange-brown	1.0											Bentonite
5	- hard, brown	0.0				S	4.5 4.95					15,16,30 N=46	Sand
6 7	***ARGILLITE (XW)*** - very low strength, pale grey	-1.0				S	6.0 6.2					19,30/50mm	Screen
8	- extremely low strength, with fine to medium angular to subangular quartz gravel	-2.0 -3.0				S	7.5 7.59					30/90mm (HB)	
9 10	- with bands of pale grey mottled orange shaley clay	-4.0 -5.0				S	9.0 9.37					8,11,30/ 70mm	
11	***ARGILLITE (HW)*** - very low to low strength, grey-brown	-6.0				S	10.5 10.54					30/40mm	

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Hydrapower Scout

Logged by: PZ

Drilling Method: Auger to 3.0m, casing to 3.0m, washbore to 15.0m, then NMLC core

Groundwater: Free groundwater encountered at approximately 3.0m depth

Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark 3_LS_171018(I).dwg', received 21/11/17

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BORE 9

Client: Herston Development Co Pty Ltd
Project: Herston Quarter Redevelopment - Northern Car Park
Location: Research Road, Herston
Project No: 017-141C

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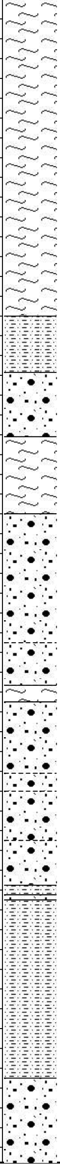
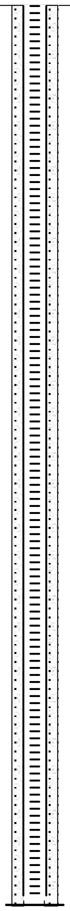
Page No: 2 of 3
Date: 28 April and 19 May 2018
Ground Surface Level: RL5.3m*





Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results	Groundwater Monitoring Weill Details
	ARGILLITE (HW)												
12	- very low to low strength, grey-brown						12.0						
						S						30/30mm	
		-7.0					12.03						
13													
		-8.0											
							13.5						
						S						30/40mm	
							13.54						
14													
		-9.0											
	SILTSTONE (XW)												
15	- extremely low strength, brown-white			15.0m to 15.15m,coreloss			15.0						
		-10.0		15.36m to 15.60m,SJ,0 to 10,vii,clean,1mm	8								
	CONGLOMERATE (MW)												
	- medium to high strength, grey, fractured			15.71m to 15.81m,fragmented (gravel and clay)		C	15.65	0.7(d)	90	15			
							15.92	1.1(a)					
16	to slightly fractured			15.92m to 16.24m,XW									
		-11.0		16.24m to 16.60m,fragmented (gravel and clay)	>20								
	CARBONACEOUS ARGILLITE (XW)												
	- extremely low strength, dark grey-black,			16.60m to 16.70m,coreloss			16.6 16.7						
17	with thin bands of SW argillite			16.97m to 17.20m,4J,0 to 35,iv and vii,clean and clay,1mm to 3mm			16.92	0.4(a)					
	CONGLOMERATE (HW/MW)	-12.0			9								
	- low strength, grey-brown, fractured to			17.45m to 17.56m,2J,5,vii,clean,1mm									
	slightly fractured, with medium to high			17.76m to 17.82m,2J,0 to 20,vii,clean,1mm			17.6	0.2(d) 1.8(a)					
18	strength bands			17.97m,J,5,iv,clean,1mm		C	18.0	0.1(d)	95	85			
		-13.0		18.43m,J,50,iv,clean,1mm			18.35	0.3(a)					
	- slightly fractured			18.60m,J,30,iv,clean,1mm	2		18.5	2.0(d) 1.2(a)					
	ARGILLITE (SW)												
19	- high strength, grey			19.18m,J,20,i,clean,1mm									
				19.32m,J,30,vii,clay,30mm			19.13	0.2(d)					
		-14.0		19.40m to 19.50m,XW seam	>20		19.32	0.2(a)					
	CONGLOMERATE (HW/MW)			19.65m,J,25,iv,clean,1mm			19.5	0.4(d)					
	- low strength, grey-brown			19.90m to 20.0m,fragmented			19.8	0.2(a)					
20	- dark grey-black			20.08m,C,40mm	6	C		0.2(d)	100	75			
							20.15	0.1(a)					
		-15.0		20.29m,J,60,vii,clean,1mm			20.38	0.04(d)					
				20.43m,J,30,vii,gravel,20mm			20.53						
	- grey-brown, fractured to slightly fractured			20.55m,J,0,vii,clean,1mm			20.6	0.2(d)					
	CONGLOMERATE (SW)							0.2(a)					
21	- low strength, grey						20.7	0.07(d) 0.05(a)					
	SILTSTONE (MW)	-16.0			>20	C			100	0			
	- low strength, pale brown						21.8	0.2(d)					
22	***CONGLOMERATE (SW)***							0.3(a)					
	- low strength, grey	-17.0					22.4						
				22.51m to 22.6m,C									
	SILTSTONE (XW/HW)			22.83m,J,0,vii,clean,1mm				0.2(d)					
23	- very low to low strength			23.12m,J,15,vii,clean,1mm			23.0	0.4(a)					

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Hydrapower Scout

Logged by: PZ

Drilling Method: Auger to 3.0m, casing to 3.0m, washbore to 15.0m, then NMLC core

Groundwater: Free groundwater encountered at approximately 3.0m depth

Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark 3_LS_171018(I).dwg', received 21/11/17

BORE REPORT

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BORE 9

Client: Herston Development Co Pty Ltd

Project: Herston Quarter Redevelopment - Northern Car Park

Location: Research Road, Herston

Project No: 017-141C

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Page No: 3 of 3

Date: 28 April and 19 May 2018

Ground Surface Level: RL5.3m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results	Groundwater Monitoring Welll Details
24 25	***CONGLOMERATE (HW)*** - medium strength, grey mottled with brown and white, fractured to slightly fractured with low strength bands	-18.0 -19.0		23.24m,J,30,vii,clean,1mm 23.62m,J,45,vii,clean,1mm 23.73m,J,45,iv,clean,1mm 24.15m,2J,45,vii,clean,1mm 24.54m,J,15,vii,clean,1mm 24.64m,J,15,vii,clean,1mm	5	C	24.0 25.0	0.3(d) 0.4(a) 0.08(d) 0.1(a)	100	50			
26	***MUDSTONE (MW)*** - low strength, dark grey mottled with white, bedding plane dipping at approximately 15 degrees, with medium strength bands	-20.0 -21.0		25.08m,J,10,vii,clean,1mm 25.25m,J,15,vii,clean,1mm 25.54m,J,10,vii,clean,1mm 25.60m to 25.66m,S 26.0m,J,0,vii,clean,1mm 26.32m,J,15,vii,clean,1mm 26.50m,J,0,vii,clean,1mm	4	C	25.5 26.5	0.3(d) 0.3(a) 0.3(d) 0.4(a)	100	65			
27 28	- grey mottled with white, fractured ***CONGLOMERATE (MW)*** - low strength, brown mottled with grey and white, with medium strength bands, fractured	-22.0		26.53m,J,10,vii,clean,1mm 26.71m to 26.83m,clay 27.0m to 27.90m,9J,10 to 30,vii,clean,1mm to 3mm 27.0m,J,15,vii,CL,3mm 27.56m,J,45,vii,clean,1mm 27.70m,J,45,vii,clean,4mm	11	C	27.0 28.0	0.1(d) 0.1(a) 0.5(d) 0.2(a)	100	30			
29	- fractured to slightly fractured	-23.0 -24.0		28.0m to 29.40m,9J,15 to 45,vii,clean,1mm to 4mm	6	C	29.0 29.46	0.1(d) 0.6(a)	100	65			
30	- medium strength	-25.0		29.61m,J,0,vii,clean,1mm 29.74m,J,0,vii,clean,1mm 29.90m,J,0,vii,clean,1mm 30.15m,J,45,iv,clean,1mm 30.34m,J,0,vii,clean,1mm 30.64m to 30.68m,C 30.73m,J,45,iv,clean,1mm 30.81m,quartz,25mm	6	C	30.0 30.9	0.6(d) 0.6(a)	100	70			
31 32 33 34	End of Bore at 30.9 m	-26.0 -27.0 -28.0 -29.0											

U	Undisturbed Tube Sample (50mm dia)	S	Standard Penetration Test (SPT)	E	Environmental Sample	Is(50)	Point Load Test Result (MPa)
D	Disturbed Sample	HB	SPT Hammer Bouncing	Up	Pushtube Sample	(d)	Diametral Test
B	Bulk Sample	( )	No Sample Recovery	C	NMLC Coring	(a)	Axial Test
pp	Pocket Penetrometer Test (kPa)	V	Vane Shear Strength, Uncorrected (kPa)			(i)	Lump Test

Rig: Hydrapower Scout

Logged by: PZ

Drilling Method: Auger to 3.0m, casing to 3.0m, washbore to 15.0m, then NMLC core

Groundwater: Free groundwater encountered at approximately 3.0m depth

Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark 3_LS_171018(I).dwg', received 21/11/17

Notes on Description and Classification of Soil

The methods of description and classification of soils used in this report are generally based on Australian Standard AS1726-1993 *Geotechnical Site Investigations*.

Soil description is based on an assessment of disturbed samples, as recovered from bores and excavations, or from undisturbed materials as seen in excavations and exposures or in undisturbed samples. Descriptions given on report sheets are an interpretation of the conditions encountered at the time of investigation.

In the case of cone or piezocone penetrometer tests, actual soil samples are not recovered and soil description is inferred based on published correlations, past experience and comparison with bore and/or test pit data (if available).

Soil classification is based on the particle size distribution of the soil and the plasticity of the portion of the material finer than 0.425mm. The description of particle size distribution and plasticity is based on the results of visual field estimation, laboratory testing or both. When assessed in the field, the properties of the soil are estimated; precise description will always require laboratory testing to define soil properties.

Where soil can be clearly identified as FILL this will be noted as the main soil type followed by a description of the composition of the fill (e.g. FILL – yellow-brown, fine to coarse grained gravelly clay fill with concrete rubble). If the soil is assessed as possibly being fill this will be noted as an additional observation.

Soils are generally described using the following sequence of terms. In certain instances, not all of the terms will be included in the soil description.

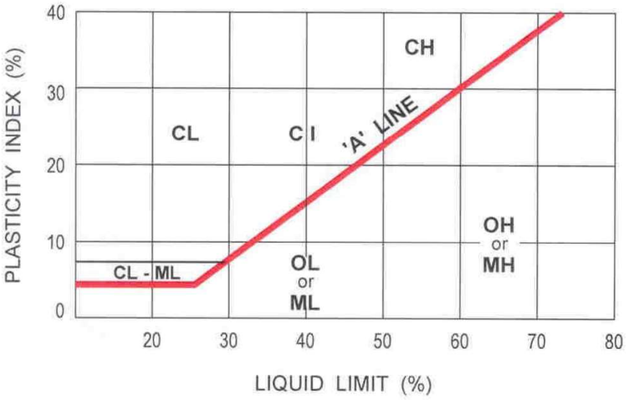
MAIN SOIL TYPE (CLASSIFICATION GROUP SYMBOL)
- strength/density, colour, structure/grain size, secondary and minor components, additional observations

Information on the definition of descriptive and classification terms follows.

SOIL TYPE and CLASSIFICATION GROUP SYMBOLS

	Major Divisions	Particle Size	Classification Group Symbol	Typical Names
COARSE GRAINED SOILS (more than half of material is larger than 0.075 mm)	BOULDERS	> 200mm		
	COBBLES	63 – 200mm		
	GRAVELS (more than half of coarse fraction is larger than 2.36mm)	Coarse: 20 – 63mm Medium: 6 – 20mm Fine: 2.36 – 6mm	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines, uniform gravels.
			GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than 2.36mm)	Coarse: 0.6 – 2.36mm Medium: 0.2 – 0.6mm Fine: 0.075 – 0.2mm	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands and gravelly sands; little or no fines, uniform sands.
			SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (more than half of material is smaller than 0.075 mm)	SILTS & CLAYS (liquid limit <50%)		ML	Inorganic silts and very fine sands, silty/clayey fine sands or clayey silts with low plasticity.
			CL and CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS & CLAYS (liquid limit >50%)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils.
			CH	Inorganic clays of high plasticity.
			OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

PLASTICITY CHART FOR CLASSIFICATION OF FINE GRAINED SOILS



(Reference: Australian Standard AS1726-1993 *Geotechnical site investigations*)

DESCRIPTIVE TERMS FOR MATERIAL PROPORTIONS

Coarse Grained Soils		Fine Grained Soils	
% Fines	**Modifier**	**% Coarse**	**Modifier**
< 5	Omit, or use 'trace'	< 15	Omit, or use trace.
5 – 12	Describe as 'with clay/silt' as applicable.	15 – 30	Describe as 'with sand/gravel' as applicable.
> 12	Prefix soil as 'silty/clayey' as applicable	> 30	Prefix soil as 'sandy/gravelly' as applicable.

STRENGTH TERMS – COHESIVE SOILS

Strength Term	Undrained Shear Strength	Field Guide to Strength
Very soft	< 12kPa	Exudes between the fingers when squeezed in hand.
Soft	12 – 25kPa	Can be moulded by light finger pressure.
Firm	25 – 50kPa	Can be moulded by strong finger pressure.
Stiff	50 – 100kPa	Cannot be moulded by fingers, can be indented by thumb.
Very stiff	100 – 200kPa	Can be indented by thumb nail.
Hard	> 200kPa	Can be indented with difficulty by thumb nail.

DENSITY TERMS – NON COHESIVE SOILS

Density Term	Density Index	SPT "N"	CPT Cone Resistance
Very loose	< 15%	0 – 5	0 – 2MPa
Loose	15 – 35%	5 – 10	2 – 5MPa
Medium dense	35 – 65%	10 – 30	5 – 15MPa
Dense	65 – 85%	30 – 50	15 – 25MPa
Very dense	> 85%	> 50	> 25MPa

COLOUR

The colour of a soil will generally be described in a 'moist' condition using simple colour terms (eg. black, grey, red, brown etc.) modified as necessary by "pale", "dark", "light" or "mottled". Borderline colours will be described as a combination of colours (eg. grey-brown).

EXAMPLE

e.g. CLAYEY SAND (SC) – medium dense, grey-brown, fine to medium grained with silt.

Indicates a medium dense, grey-brown, fine to medium grained clayey sand with silt.

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Notes on Description and Classification of Rock

The methods of description and classification of rock used in this report are generally based on Australian Standard AS1726-1993 *Geotechnical site investigations*.

Rock description is based on an assessment of disturbed samples, as recovered from bores and excavations, or from undisturbed materials as seen in excavations and exposures, or in core samples. Descriptions given on report sheets are an interpretation of the conditions encountered at the time of investigation.

Notes outlining the method and terminology adopted for the description of rock defects are given below, however, detailed information on defects can generally only be determined where rock core is taken, or excavations or exposures allow detailed observation and measurement.

Rocks are generally described using the following sequence of terms. In certain instances not all of the terms will be included in the rock description.

ROCK TYPE (WEATHERING SYMBOL), strength, colour, grain size, defect frequency

Information on the definition of descriptive and classification terms follows.

ROCK TYPE

In general, simple rock names are used rather than precise geological classifications.

ROCK MATERIALS WEATHERING CLASSIFICATION

Term	Weathering Symbol	Definition
Residual soil	RS	Soil developed from extremely weathered rock; the mass structure and substance fabrics are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered	XW	Rock is weathered to such an extent that it has ‘soil’ properties, i.e. it either disintegrates or can be remoulded in water.
Distinctly weathered *	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
- 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 extends throughout the whole of the rock substance and the original colour of the fresh rock may be no longer recognisable.
Slightly weathered	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh	FR	Rock shows no sign of decomposition or staining.

** Subdivision of this weathering grade into highly and moderately may be used where applicable.*

STRENGTH OF ROCK MATERIAL

Term	Symbol	Point Load Index I_s (50)	Field Guide To Strength
Extremely low	EL	<0.03MPa	Easily remoulded by hand to a material with soil properties.
Very low	VL	0.03 – 0.1MPa	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low	L	0.1 – 0.3MPa	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium	M	0.3 – 1.0MPa	Readily scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High	H	1.0 – 3.0MPa	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very high	VH	3.0 – 10.0MPa	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely high	EH	>10MPa	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Notes:

1. *These terms refer to the strength of the rock material and not to the strength of the rock mass which may be considerably weaker due to the effect of rock defects.*
2. *The field guide visual assessment for rock strength may be used for preliminary assessment or when point load testing is not available.*
3. *Anisotropy of rock may affect the field assessment of strength.*

COLOUR

The colour of a rock will generally be described in a ‘moist’ condition using simple colour terms (e.g. black, grey, red, brown, etc) modified as necessary by ‘pale’, ‘dark’, ‘light’ or ‘mottled’. Borderline colours will be described as a combination of colours (e.g. grey-brown).

GRAIN SIZE

Descriptive Term	Particle Size Range
Coarse grained	0.6 – 2.0mm
Medium grained	0.2 – 0.6mm
Fine grained	0.06 – 0.2mm

DEFECT FREQUENCY

Where appropriate, a defect frequency may be recorded as part of the rock description and will be expressed as the number of natural (or interpreted natural) defects present in an equivalent one metre length of core; by use of the following defect frequency descriptive terms; or both. The descriptive terms refer to the spacing of all types of natural defects along which the rock is discontinuous and include, bedding plane partings, joints and other rock defects, but excludes known artificial fractures such as drilling breaks.

Defect Frequency	Description
Fragmented	Rock 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.
Fractured to Slightly Fractured	Core lengths are mainly 100mm to 300mm with occasional shorter to longer sections.
Slightly Fractured	Core lengths are generally 300mm to 1,000mm with occasional longer sections and occasional sections of 100mm to 300mm.
Unbroken	The core does not contain any fractures.

EXAMPLE

e.g. SANDSTONE (XW) – low strength, pale brown, fine to coarse grained, slightly fractured.

ROCK DEFECT LOGGING

Defects are discontinuities in the rock mass and include joints, sheared zones, cleavages and bedding partings. The ability to observe and log defects will depend on the investigation methodology. Defects logged in core are described using the abbreviations noted in the following tables.

The *depth* noted in the description is measured in metres from the ground surface, the *defect angle* is measured in degrees from horizontal, and the defect *thickness* is measured normal to the plane of the defect and is in millimetres (unless otherwise noted).

Defects are generally described using the following sequence of terms:

Depth, Defect Type, Defect Angle (dip), Surface Roughness, Infill, Thickness

DEFECT TYPE

B	– Bedding
J	– Joint
S	– Shear Zone
C	– Crushed Zone

SURFACE ROUGHNESS

i	- rough or irregular, stepped
ii	- smooth, stepped
iii	- slickensided, stepped
iv	- rough or irregular, undulating
v	- smooth, undulating
vi	- slickensided, undulating
vii	- rough or irregular, planar
viii	- smooth planar
ix	- slickensided, planar

INFILL

Infill refers to secondary minerals or other materials formed on the surface of the defect and some common descriptions are given in the following table together with their abbreviations.

Ls	- limonite staining
Fe	- iron staining
Cl	- clay
Mn	- manganese staining
Qtz	- quartz
Ca	- calcite
Clean	- no visible infill

EXAMPLE

3.59m, J, 90, vii, Ls, 1mm

indicates a joint at 3.59m depth that is at 90° to horizontal (i.e. vertical), is rough or irregular and planar, limonite stained and 1mm thick.

APPENDIX B

PREVIOUS INVESTIGATION BORE REPORT SHEETS

BORE REPORT

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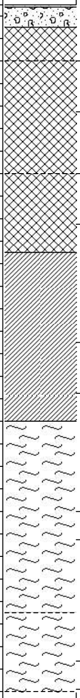
BORE 1

Client: Watpac Construction Pty Ltd
Project: Herston Quarter Redevelopment - Northern Carpark
Location: Research Road, Herston
Project No: 017-141B

Page No: 1 of 1
Date: 4 November 2017
Ground Surface Level: RL4.9m*



Depth (m)	Description	RL (m)	Lithology	Sample Type	Sample Depth (m)	Sample ID	Test Results
0	***BITUMINOUS CONCRETE*** - 20mm thick	4.9					
	PAVEMENT GRAVEL - 200mm thick			S	0.5 0.95		16,14,12 N=26
1		4.0					
	FILL - grey-brown, clayey gravelly sand			S	1.5 1.95		4,3,4 N=7
2	- grey-brown mottled orange-brown, clayey sandy gravel, with bands of clayey sand	3.0					
	- dark grey-brown, clayey sand, with bands of sandy clayey gravel, possible trace of ash						
3	***SHALEY CLAY (CH)*** - stiff, grey-brown mottled orange	2.0		S	3.0 3.45		4,7,6 N=13
4	***ARGILLITE (XW/HW)*** - extremely low strength, grey-brown	1.0		S	4.5 4.61		30/110mm (HB)
5		0.0					
	ARGILLITE (HW) - very low strength, grey-brown						
6		-1.0		S	6.0 6.08		30/80mm (HB)
	End of Bore at 6.1 m						
7		-2.0					
8		-3.0					
9		-4.0					
10		-5.0					

D	**Disturbed Sample**	**V**	**Vane Shear Strength, Uncorrected (kPa)**	**C**	**NMLC Coring**
B	**Bulk Sample**	**S**	**Standard Penetrometer Test (SPT)**	**Is(50)**	**Point Load Test Result (MPa)**
U	**Undisturbed Tube (50mm diameter)**	**SPT**	**Hammer Bouncing**	**(d)**	**Diametral Point Load Strength Test**
pp	**Pocket Penetrometer Test (kPa)**	**( )**	**No Sample Recovery**	**(a)**	**Axial Point Load Strength Test**
E	**Environmental Sample**	**A**	**Asbestos Sample**		

Rig: Jacro 350
Logged By: CM
Drilling Method: Auger wet at 1.7m depth in SPT
Groundwater: Free groundwater encountered at approximately 3.0m depth
Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark3_LSA_171018(1).dwg' received 21/11/17

BORE REPORT

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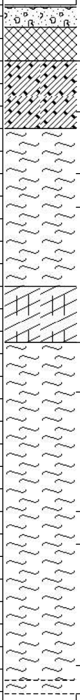
BORE 2

Client: Watpac Construction Pty Ltd
Project: Herston Quarter Redevelopment - Northern Carpark
Location: Research Road, Herston
Project No: 017-141B

Page No: 1 of 1
Date: 4 November 2017
Ground Surface Level: RL6.9m*

Depth (m)	Description	RL (m)	Lithology	Sample Type	Sample Depth (m)	Sample ID	Test Results
0	***BITUMINOUS CONCRETE*** - 20mm thick	6.9					
	PAVEMENT GRAVEL - 200mm thick			S	0.5 0.95		7,6,7 N=13
1	***FILL*** - brown, gravelly clayey sand	6.0					
				S	1.5 1.8		13,30/145mm
2	***CLAYEY GRAVELLY SAND (SC)*** - brown, fine to coarse grained, fine to medium angular to subangular gravel (possibly fill)	5.0					
3	***ARGILLITE (XW)*** - extremely low strength, orange-brown	4.0					
	SILTY CLAY (CI) - very stiff, dark brown, with fine to coarse grained sand, with fine to medium angular to subangular gravel			S	3.0 3.44		17,22,30/140mm
4	***ARGILLITE (XW)*** - extremely low strength, dark brown-grey	3.0					
				S	4.5 4.79		7,30/140mm (HB)
5		2.0					
6	- dark grey	1.0		S	6.0 6.12		30/120mm (HB)
	End of Bore at 6.12 m						
7		0.0					
8		-1.0					
9		-2.0					
10		-3.0					

D	**Disturbed Sample**	**V**	**Vane Shear Strength, Uncorrected (kPa)**	**C**	**NMLC Coring**
B	**Bulk Sample**	**S**	**Standard Penetrometer Test (SPT)**	**Is(50)**	**Point Load Test Result (MPa)**
U	**Undisturbed Tube (50mm diameter)**	**SPT**	**Hammer Bouncing**	**(d)**	**Diametral Point Load Strength Test**
pp	**Pocket Penetrometer Test (kPa)**	**( )**	**No Sample Recovery**	**(a)**	**Axial Point Load Strength Test**
E	**Environmental Sample**	**A**	**Asbestos Sample**		

Rig: Jacro 350
Logged By: CM
Drilling Method: Auger
Groundwater: No free groundwater encountered during drilling
Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark3_LSA_171018(1).dwg' received 21/11/17

BORE REPORT

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BORE 3

Client: Watpac Construction Pty Ltd
Project: Herston Quarter Redevelopment - Northern Carpark
Location: Research Road, Herston
Project No: 017-141B

Page No: 1 of 1
Date: 4 November 2017
Ground Surface Level: RL13.0m*

Depth (m)	Description	RL (m)	Lithology	Sample Type	Sample Depth (m)	Sample ID	Test Results
		13.0					
0	***SANDY CLAY (CI)*** - stiff, brown, fine to coarse grained sand			S	0.5 0.95		3,5,6 N=11
1	***SHALEY CLAY (CI)*** - very stiff to hard, orange-brown mottled grey	12.0		S	1.5 1.92		12,14,30/120mm (HB)
2	***ARGILLITE (XW/HW)*** - extremely low strength, grey-brown mottled orange	11.0					
3	- very low strength, grey-brown	10.0		S	3.0 3.09		30/90mm (HB)
4		9.0					
	- grey			S	4.5 4.59		30/90mm (HB)
5		8.0					
6	- dark grey-brown	7.0		S	6.0 6.04		30/40mm
	End of Bore at 6.04 m						
7		6.0					
8		5.0					
9		4.0					
10		3.0					

D	**Disturbed Sample**	**V**	**Vane Shear Strength, Uncorrected (kPa)**	**C**	**NMLC Coring**
B	**Bulk Sample**	**S**	**Standard Penetrometer Test (SPT)**	**Is(50)**	**Point Load Test Result (MPa)**
U	**Undisturbed Tube (50mm diameter)**	**SPT**	**Hammer Bouncing**	**(d)**	**Diametral Point Load Strength Test**
pp	**Pocket Penetrometer Test (kPa)**	**( )**	**No Sample Recovery**	**(a)**	**Axial Point Load Strength Test**
E	**Environmental Sample**	**A**	**Asbestos Sample**		

Rig: Jacro 350
Logged By: CM
Drilling Method: Auger
Groundwater: No free groundwater was encountered during drilling
Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark3_LSA_171018(1).dwg' received 21/11/17

BORE REPORT

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Client: Watpac Construction Pty Ltd
Project: Herston Quarter Redevelopment - Northern Carpark
Location: Research Road, Herston
Project No: 017-141B

BORE 4

Page No: 1 of 2
Date: 6 November 2017
Ground Surface Level: RL23.5m*

Depth (m)	Description	RL (m)	Lithology	Sample Type	Sample Depth (m)	Sample ID	Test Results
0	***BITUMINOUS CONCRETE*** - 10mm thick	23.5					
	PAVEMENT GRAVEL - 200mm thick			S	0.5 0.65		30/145mm (HB)
1	***FILL*** - brown, silty sandy gravel, fine to coarse grained angular to subrounded, with possible cobbles	22.0		S	1.5 1.95		3,7,8 N=15
2	- red-brown, with minor bands of clayey sand/sandy clay, with possible cobbles	21.0					
3		20.0		S	3.0 3.45		6,10,12 N=22
4	***SHALEY CLAY (CI)*** - very stiff, orange-brown mottled pale grey	19.0		S	4.3 4.75		9,14,18 N=32
5		18.0					
6	- stiff, dark grey mottled red-brown	17.0		S	5.8 6.25		4,9,12 N=21
7	***ARGILLITE (XW)*** - extremely low strength, grey mottled red-brown and dark grey, with carbonaceous bands	16.0		S	7.3 7.42		30/115mm
8		15.0					
9	***ARGILLITE (XW/HW)*** - very low strength, grey-brown	14.0		S	8.8 8.84		30/40mm
10		13.0		S	10.3 10.36		30/55mm

D	**Disturbed Sample**	**V**	**Vane Shear Strength, Uncorrected (kPa)**	**C**	**NMLC Coring**
B	**Bulk Sample**	**S**	**Standard Penetrometer Test (SPT)**	**Is(50)**	**Point Load Test Result (MPa)**
U	**Undisturbed Tube (50mm diameter)**	**SPT**	**Hammer Bouncing**	**(d)**	**Diametral Point Load Strength Test**
pp	**Pocket Penetrometer Test (kPa)**	**( )**	**No Sample Recovery**	**(a)**	**Axial Point Load Strength Test**
E	**Environmental Sample**	**A**	**Asbestos Sample**		

Rig: Jacro 350
Logged By: CM/RZ
Drilling Method: Auger to 3.0m, casing to 4.1m, the washbore
Groundwater: No free groundwater was encountered during drilling
Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark3_LSA_171018(1).dwg' received 21/11/17

BORE REPORT

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BORE 4

Client: Watpac Construction Pty Ltd
Project: Herston Quarter Redevelopment - Northern Carpark
Location: Research Road, Herston
Project No: 017-141B

Page No: 2 of 2
Date: 6 November 2017
Ground Surface Level: RL23.5m*

Depth (m)	Description	RL (m)	Lithology	Sample Type	Sample Depth (m)	Sample ID	Test Results
	ARGILLITE (XW/HW) - very low strength, grey-brown						
11							
		12.0					
	- extremely low strength			S	11.8 11.87		30/70mm
12							
		11.0					
13							
	- very low to low strength	10.0		S	13.3 13.33		30/30mm
14							
		9.0					
				S	14.8 14.91		30/110mm
15	***MUDSTONE (XW)*** - extremely low strength, dark grey						
		8.0					
16	- dark grey mottled pale grey						
		7.0		S	16.3 16.4		30/100mm
17							
		6.0					
18	***ARGILLITE (DW/MW)*** - very low to low strength, grey and pale grey			(S)	17.8 17.84		30/40mm (HB)
		5.0					
19							
	- very low strength	4.0		S	19.3 19.35		30/50mm
20							
		3.0		S	20.8 20.86		30/60mm
21	End of Bore at 20.86 m						

D	**Disturbed Sample**	**V**	**Vane Shear Strength, Uncorrected (kPa)**	**C**	**NMLC Coring**
B	**Bulk Sample**	**S**	**Standard Penetrometer Test (SPT)**	**Is(50)**	**Point Load Test Result (MPa)**
U	**Undisturbed Tube (50mm diameter)**	**SPT**	**Hammer Bouncing**	**(d)**	**Diametral Point Load Strength Test**
pp	**Pocket Penetrometer Test (kPa)**	**( )**	**No Sample Recovery**	**(a)**	**Axial Point Load Strength Test**
E	**Environmental Sample**	**A**	**Asbestos Sample**		

Rig: Jacro 350

Logged By: CM/RZ

Drilling Method: Auger to 3.0m, casing to 4.1m, the washbore

Groundwater: No free groundwater was encountered during drilling

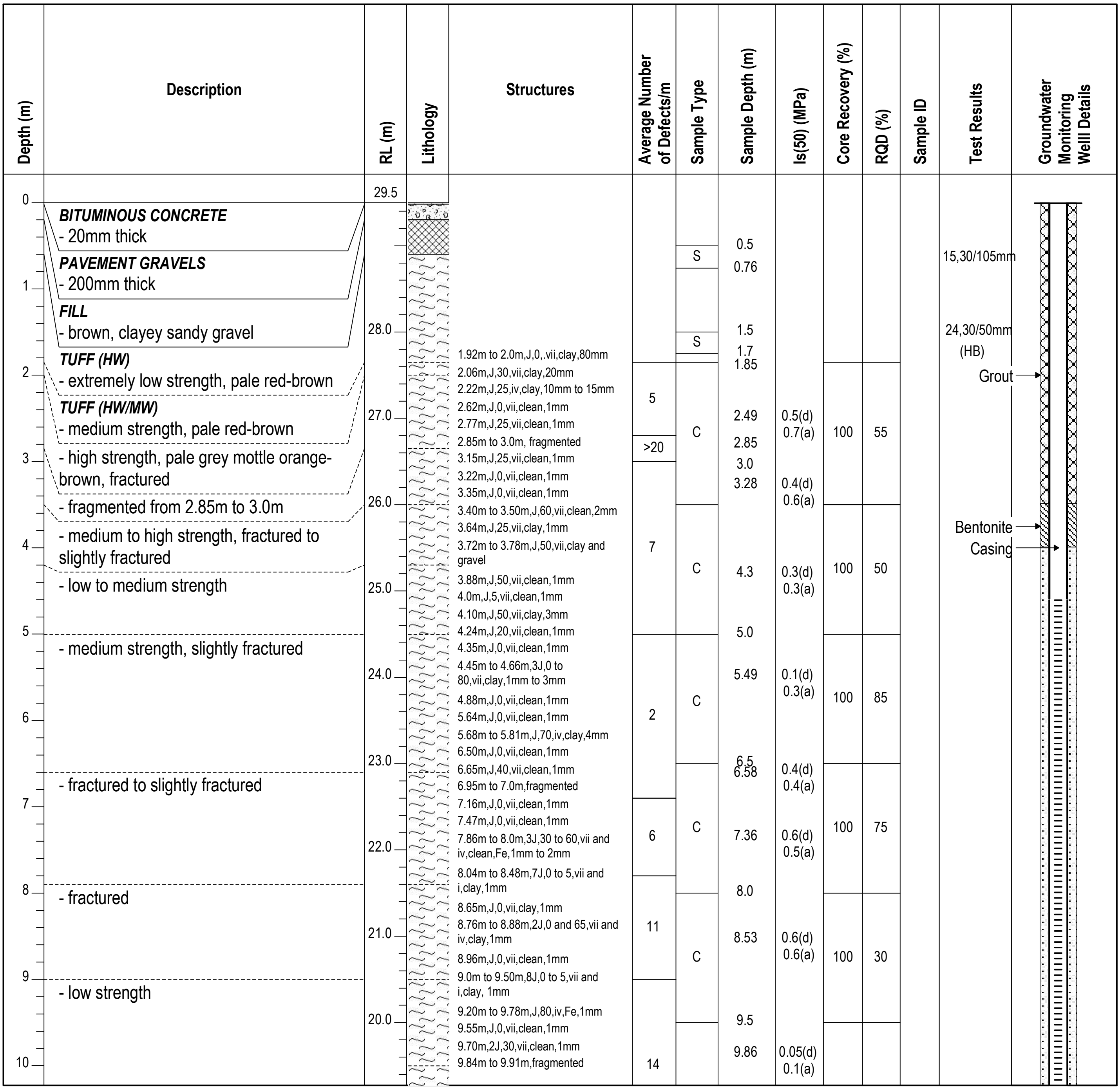
Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark3_LSA_171018(1).dwg' received 21/11/17

BORE REPORT

BORE 5

Client: Watpac Construction Pty Ltd
Project: Herston Quarter Redevelopment - Northern Car Park
Location: Research Road, Herston
Project No: 017-141B

Page No: 1 of 2
Date: 9 November 2017
Ground Surface Level: RL29.5m*



Description

BITUMINOUS CONCRETE
- 20mm thick

PAVEMENT GRAVELS
- 200mm thick

FILL
- brown, clayey sandy gravel

TUFF (HW)
- extremely low strength, pale red-brown

TUFF (HW/MW)
- medium strength, pale red-brown
- high strength, pale grey mottle orange-brown, fractured
- fragmented from 2.85m to 3.0m
- medium to high strength, fractured to slightly fractured
- low to medium strength
- medium strength, slightly fractured
- fractured to slightly fractured
- fractured
- low strength

Structures

1.92m to 2.0m,J,0,.vii,clay,80mm
2.06m,J,30,vii,clay,20mm
2.22m,J,25,iv,clay,10mm to 15mm
2.62m,J,0,vii,clean,1mm
2.77m,J,25,vii,clean,1mm
2.85m to 3.0m, fragmented
3.15m,J,25,vii,clean,1mm
3.22m,J,0,vii,clean,1mm
3.35m,J,0,vii,clean,1mm
3.40m to 3.50m,J,60,vii,clean,2mm
3.64m,J,25,vii,clay,1mm
3.72m to 3.78m,J,50,vii,clay and gravel
3.88m,J,50,vii,clean,1mm
4.0m,J,5,vii,clean,1mm
4.10m,J,50,vii,clay,3mm
4.24m,J,20,vii,clean,1mm
4.35m,J,0,vii,clean,1mm
4.45m to 4.66m,3J,0 to 80,vii,clay,1mm to 3mm
4.88m,J,0,vii,clean,1mm
5.64m,J,0,vii,clean,1mm
5.68m to 5.81m,J,70,iv,clay,4mm
6.50m,J,0,vii,clean,1mm
6.65m,J,40,vii,clean,1mm
6.95m to 7.0m,fragmented
7.16m,J,0,vii,clean,1mm
7.47m,J,0,vii,clean,1mm
7.86m to 8.0m,3J,30 to 60,vii and iv,clean,Fe,1mm to 2mm
8.04m to 8.48m,7J,0 to 5,vii and i,clay,1mm
8.65m,J,0,vii,clay,1mm
8.76m to 8.88m,2J,0 and 65,vii and iv,clay,1mm
8.96m,J,0,vii,clean,1mm
9.0m to 9.50m,8J,0 to 5,vii and i,clay, 1mm
9.20m to 9.78m,J,80,iv,Fe,1mm
9.55m,J,0,vii,clean,1mm
9.70m,2J,30,vii,clean,1mm
9.84m to 9.91m,fragmented

Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
	S	0.5 0.76					15,30/105mm
	S	1.5 1.7					24,30/50mm (HB)
5	C	1.85 2.49	0.5(d) 0.7(a)	100	55		
>20		2.85 3.0					
7		3.28	0.4(d) 0.6(a)				
	C	4.3	0.3(d) 0.3(a)	100	50		
2	C	5.0 5.49	0.1(d) 0.3(a)	100	85		
	C	6.5 6.58	0.4(d) 0.4(a)	100	75		
6		7.36	0.6(d) 0.5(a)				
11	C	8.0 8.53	0.6(d) 0.6(a)	100	30		
14		9.5 9.86	0.05(d) 0.1(a)				

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Jacro 350
Logged by: CM
Drilling Method: Auger to 1.2m, casing to 1.5m, then NMLC
Groundwater: No free groundwater encountered during auger drilling
Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark3_LSA_171018(1).dwg' received 21/11/17

BORE REPORT

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BORE 5

Client: Watpac Construction Pty Ltd

Project: Herston Quarter Redevelopment - Northern Car Park

Location: Research Road, Herston

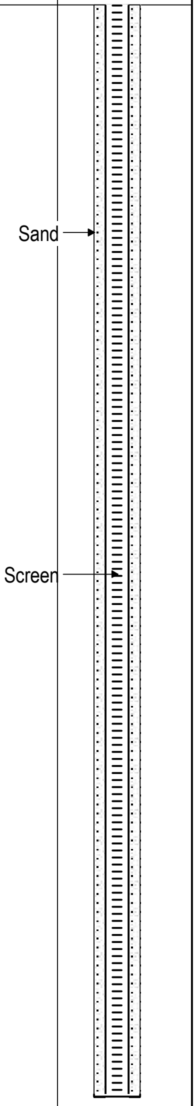
Project No: 017-141B

Page No: 2 of 2

Date: 9 November 2017

Ground Surface Level: RL29.5m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results	Groundwater Monitoring Welll Details
	TUFF (HW/MW) - medium strength, pale grey mottle orange-brown	19.0		10.0m to 10.13m,4J,20,vii,Fe,1mm 10.26m,J,50,vii,Fe,1mm 10.32m to 10.48m,4J,15 to 45,vii and iv,clean,Fe,1mm 10.62m to 10.93m,7J,10 to 30,vii,Fe,clean,1mm		C	10.57	0.3(d) 0.3(a)	100	5			
11	***TUFF (MW)*** - medium strength, pale grey, fractured to slightly fractured	18.0		11.06m to 11.23m,3J,20,vii,Fe,clean,1mm 11.34m to 11.42m,J,45,i,clean,1mm 11.46m,J,0,vii,clean,1mm	8	C	11.0 11.72	0.3(d) 0.3(a)	100	60			
12	- with high strength bands			11.66m to 11.9m,3J,0 to 20,clean,1mm	4							Sand	
	- with extremely low strength, XW bands	17.0		12.32m,J,45,clean,1mm 12.47m,J,0,vii,clean,1mm 12.65m to 12.71m,XW band		C	12.5 12.55						
13	***SANDY CLAY (CI)*** - brown mottled dark brown-black, with XW argillite bands and coal seams	16.0		12.90m to 13.0m,XW bamd 13.13m,J,0,vii,clay and coal,1mm 13.39m to 13.50,3J,0 to 30,vii,clay,1mm 13.75m to 14.0m,core loss			13.3	0.01(d) 0.01(a)	85	35			
14	***CONGLOMERATE/ARGILLITE (HW)*** - low to medium strength, grey-brown, with coal seams, fragmented	15.0		14.0m to 14.38m,fragmented 14.50m,J,0,vii,gravel,40mm 14.63m,J,0,vii,gravel,40mm 14.71m to 15.0m,3J,0,vii,clean,1mm	10	C	14.0 14.36	0.2(d) 0.3(a)	100	20			
15	- fractured	14.0		14.84m to 14.93m,clay seam 15.0m to 15.10m,XW sean 15.18m,J,5,vii,clean,1mm 15.48m to 16.0m,7J,0 to 25,vii and i,clean and clay,1mm to 3mm 15.67m to 15.73m,XW seam		C	15.1 15.28	0.2(d) 0.6(a)	100	25		Screen	
16 17	- low to high strength, fractured to fragmented	13.0		16.06m to 16.45m,highly fractured to fragmented 16.52m to 17.14m,highly fractured to fragmented 17.22m to 17.29m ,fragmented	>20	C	16.4 16.85	0.0(d) 0.1(a)	100	20			
18 19	- low to medium strength, fractured	12.0 11.0 10.0		17.51m to 17.61m,3J,0,vii and vi,clean,1mm 17.70m,J,30,vii,cl;ean,1mm 17.81m to 17.91m,fragmented 18.0m,J,0,vii,clean,1mm 18.05m and 18.14m,2J,5,vii,clean,1mm 18.26m to 18.36m,fragmented 18.52m to 18.66m,fragmented 18.73m to 18.79m,3J,25,vii,clean,12mm 18.90,J,25,vii,clay and coal,15mm	15	C C	17.64 17.9 18.43 19.12 19.5	0.2(d) 0.3(a) 0.08(d) 0.08(a) 0.1(d) 0.2(a)	100	30			
20	End of Bore at 19.5 m			19.07m to 19.23m,4J,0 to 35,vii,clean,1mm 19.36m to 19.5m,3J,5,vii,clean,1mm									



U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Jacro 350

Logged by: CM

Drilling Method: Auger to 1.2m, casing to 1.5m, then NMLC

Groundwater: No free groundwater encountered during auger drilling

Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark3_LSA_171018(1).dwg' received 21/11/17

BORE REPORT

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BORE 6

Client: Watpac Construction Pty Ltd
Project: Herston Quarter Redevelopment - Northern Carpark
Location: Research Road, Herston
Project No: 017-141B

Page No: 1 of 2
Date: 7 November 2017
Ground Surface Level: RL25.9m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
0	***BITUMINOUS CONCRETE*** - 20mm thick	25.9										
	PAVEMENT GRAVEL - orange-brown, fine to coarse subangular					S	0.5 0.59					30/90mm
1	***TUFF (XW/DW)*** - extremely low strength, pale pink and pale grey	25.0				S	1.5 1.92					14,11,30/120mm
2		24.0										
3	***TUFF (HW)*** - low strength, pale pink mottled pale grey	23.0				(S)	3.0 3.03					30/30mm (HB)
4		22.0				S	4.5 4.54					30/35mm (HB)
5		21.0					5.41	1.1(d) 0.9(a)				
	- high strength, pale grey mottled orange, slightly fractured						5.5					
6		20.0		6.21m,J,10,vii,clean,1mm 6.62m,J,50,vii,Fe,1mm	2	C	6.42	0.6(d) 0.4(a)	100	100		
7		19.0		7.46m,J,20,vii,Fe,1mm 7.82,crush zone,40mm	2	C	6.8 7.4	0.4(d) 0.5(a)	97	100		
8		18.0										
	- fractured to slightly fractured			8.33m,crush zone,20mm 8.41m,J,10,vii,Fe,1mm 8.52m,J,20,ix,clean,2mm 8.63m,J,45,vii,Fe,1mm 8.80m,J,15,vii,Fe,1mm	4	C	8.3 8.45	0.5(d) 1.1(a)	87	100		
9		17.0		9.05m,J,20,vii,Fe,1mm 9.05m to 9.34m,J,80 to 90,vii,Fe,1mm 9.51m,J,20,vii,Fe,1mm			9.54	0.8(d) 0.5(a)				
	- slightly fractured						9.8					
10		16.0										

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Jacro 350

Logged by: RZ

Drilling Method: Auger to 3.0m, casing to 3.0m, washbore to 5.5m, then NMLC

Groundwater: No free groundwater was encountered during drilling

Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark3_LSA_171018(1).dwg' received 21/11/17

BORE REPORT

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BORE 6

Client: Watpac Construction Pty Ltd

Project: Herston Quarter Redevelopment - Northern Carpark

Location: Research Road, Herston

Project No: 017-141B

Page No: 2 of 2

Date: 7 November 2017

Ground Surface Level: RL25.9m*

Depth (m)	Description	RL (m)	Lithology	Structures	Average Number of Defects/m	Sample Type	Sample Depth (m)	Is(50) (MPa)	Core Recovery (%)	RQD (%)	Sample ID	Test Results
11	***TUFF (HW)*** - high strength, pale grey mottled orange,slightly fractured	15.0		10.39m,J,10,vii,Fe,1mm 10.60m,J,10,vii,Fe,1mm	2	C	10.43	1.4(d) 0.7(a)	100	100		
12		14.0		11.63m,J,0,vii,clay,5mm 11.65m,J,0,vii,clay,3mm 12.28m,J,10,vii,Fe,1mm 12.56m,J,20,vii,Fe,1mm	2	C	11.4 11.44 12.62	1.1(d) 1.1(a) 0.9(d) 1.1(a)	98	100		
13 14	***TUFF (SW)*** - high strength, grey	13.0 12.0		13.54m,J,10,vii,Fe,2mm 13.64m,J,0,vii,Fe,1mm 14.30m,J,20,vii,Fe,2mm	2	C	13.0 13.59	1.8(d) 1.2(a)	100	100		
15	- fractured to slightly fractured	11.0		14.95m,J,60,vii,Fe,3mm 15.74m,J,5,vii,Fe,1mm	7	C	14.4 14.62 15.41	2.5(d) 2.3(a) 2.8(d) 2.3(a)	97	100		
16 17	- green-grey, slightly fractured	10.0 9.0		16.30m,J,0,vii,clean,1mm 16.58m,J,30,vii,Fe,1mm 17.22m to 17.50m,J,70,vii,Fe,4mm	2	C	15.85 16.43	2.1(d) 2.9(a)		100		
18		8.0		17.90m to 18.78m,J,80 to 90,vii,clean,1mm	2	C	17.3 17.49	3.0(d) 2.8(a)	100			
19 20	- fractured to slightly fractured	7.0 6.0		19.15m,J,70,vii,clean,1mm 19.34m,J,50,vii,clean,1mm 19.49m,J,30,vii,Fe,1mm 19.61m,J,10,vii,clean,1mm 19.90m to 20.10m,J,80 to 90,vii,clean,1mm 20.22m,J,0,vii,clean,1mm	5	C	18.8 18.84 19.46 20.3	4.5(d) 4.4(a) 2.6(d) 1.7(a)	91	100		
	End of Bore at 20.3 m											

U	**Undisturbed Tube Sample (50mm dia)**	**S**	**Standard Penetration Test (SPT)**	**E**	**Environmental Sample**	**Is(50)**	**Point Load Test Result (MPa)**
D	**Disturbed Sample**	**HB**	**SPT Hammer Bouncing**	**Up**	**Pushtube Sample**	**(d)**	**Diametral Test**
B	**Bulk Sample**	**( )**	**No Sample Recovery**	**C**	**NMLC Coring**	**(a)**	**Axial Test**
pp	**Pocket Penetrometer Test (kPa)**	**V**	**Vane Shear Strength, Uncorrected (kPa)**			**(i)**	**Lump Test**

Rig: Jacro 350

Logged by: RZ

Drilling Method: Auger to 3.0m, casing to 3.0m, washbore to 5.5m, then NMLC

Groundwater: No free groundwater was encountered during drilling

Remarks: *Approximate ground surface level interpolated from Land Solution Australia Pty Ltd's 'RBH_Survey_Northern Carpark3_LSA_171018(1).dwg' received 21/11/17

APPENDIX C

LABORATORY TEST RESULTS

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EMERSON CLASS NUMBER TEST REPORT

Test Procedure: AS1289.3.8.1

pH TEST REPORT

Test Procedure: AS1289.4.3.1

CONDUCTIVITY REPORT

Soil Chemical Methods, Rayment & Lyons

Client:	Watpac Construction Pty Ltd	**Report No.:**	017-141B_ECN_B1711-347
Project:	Herston Quarter Redevelopment - Northern Car Park	Tested by:	CS
Location:	Research Road, Herston	Date:	22/11/2017
		Checked by:	CS
Project No:	017-141B	Date:	24/11/2017

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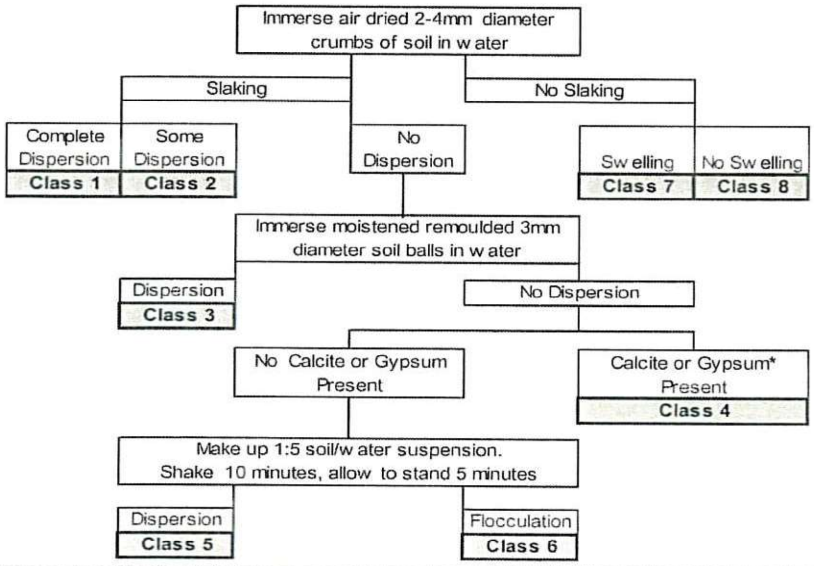
Determination of Emerson Class Number



Sample Number:	B1711-347	B1711-349	B1711-351	B1711-353	B1711-356
Sampling Method:					
AS1289.1.2.1	Clause 6.5.3	Clause 6.5.3	Clause 6.5.3	Clause 6.5.3	Clause 6.5.3
Bore:	1	2	3	4	6
Depth (m):	0.5-0.95	0.5-0.95	0.5-0.95	1.5-1.95	0.5-0.59
Date Sampled:	4/11/2017	4/11/2017	4/11/2017	6/11/2017	7/11/2017
Sample Description:	Clayey Gravelly Sand	Clayey Gravelly Sand	Shaley Clay	Silty Sandy Gravel	Tuff
Water Type:	Distilled	Distilled	Distilled	Distilled	Distilled
Water Temperature (^{o}C):	20.8	20.8	20.9	20.9	20.9
Emerson Class Number	4	4	6	4	4
pH	7.5	6.9	6.5	7.5	7.7
Conductivity (mS/cm)	0.56	0.62	0.56	0.54	0.60

Comments:

Disclaimer:- Conductivity method is not NATA accredited

Authorised Signatory

Bruce Butler

Date 24/11/2017

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Accreditation No. 19529

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EMERSON CLASS NUMBER TEST REPORT

Test Procedure: AS1289.3.8.1

pH TEST REPORT

Test Procedure: AS1289.4.3.1

CONDUCTIVITY REPORT

Soil Chemical Methods, Rayment & Lyons

Client:	Herston Development Co Pty Ltd	**Report No.:**	017-141C_ECN_T1806-198
Project:	Northern Car Park	Tested by:	DN
Location:	Herston Quarte, Herston	Date:	22/06/2018
		Checked by:	DN
Project No:	017-141C	Date:	22/06/2018

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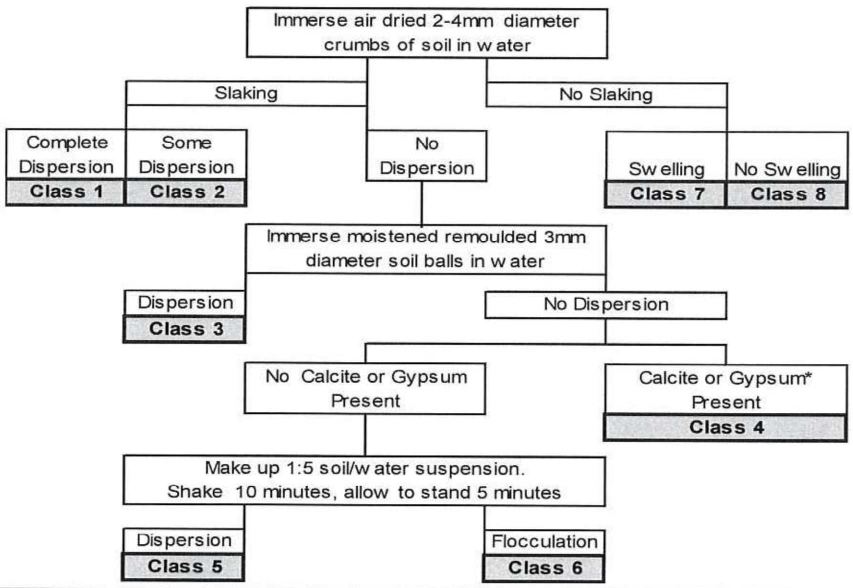
Determination of Emerson Class Number



Sample Number:	T1806-198	T1805-146			
Sampling Method:					
AS1289.1.2.1	Clause 6.5.3	Clause 6.5.3			
Bore:	7	8			
Depth (m):	0.5-0.95	1.5-1.95			
Date Sampled:	12/05/2018	26/04/2018			
Sample Description:	Shaley Clay	Sandy Clayey Gravel			
Water Type:	Distilled	Distilled			
Water Temperature (^{o}C):	20.1	19.4			
Emerson Class Number	**5**	**2**			
pH	**7.4**	**5.8**			
Conductivity (mS/cm)	**0.07**	**0.06**			

Comments:

Disclaimer:- Conductivity method is not NATA accredited

Authorised Signatory

DNash

Dennis Nash

22/06/2018

Date

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PARTICLE SIZE DISTRIBUTION TEST REPORT

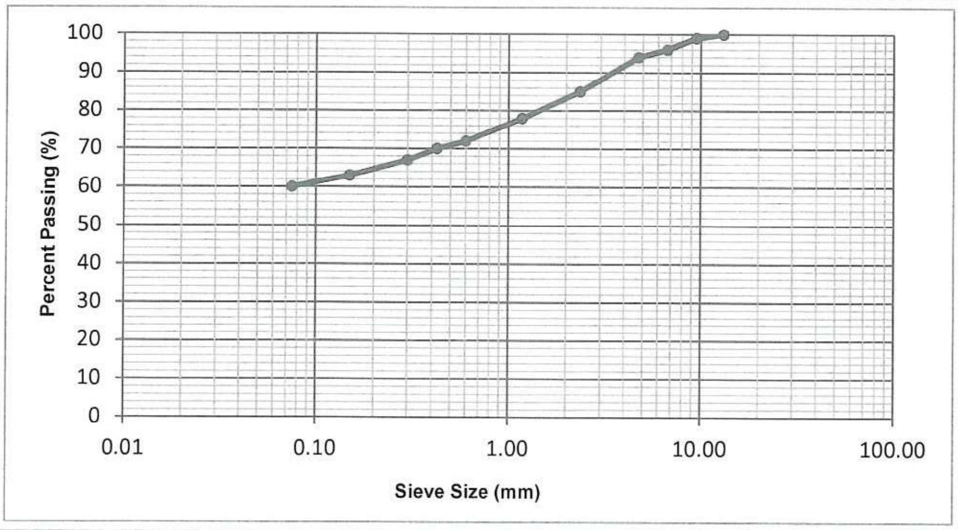
Test Procedure: AS1289.3.6.1 ☑ *Test Procedure: Q103A* ☐

Test Procedure: AS1289.2.1.1 ☑ *Test Procedure: Q103B* ☐

Client:	Herston Development Co Pty Ltd	Tested by:	DN	Date:	14/05/2018
Project:	Northern Car Park	Checked by:	DN	Date:	14/05/2018
Location:	Herston Quarter, Herston	**Report No.:**	**017-141C_PSD_T1805-144**		
Project No:	017-141C	THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL			

Sample No.:	T1805-144
Sampling Method:	AS1289.1.2.1 Cl.6.5.3
Sample Moisture Content (%):	10.3
Bore:	3A
Depth (m):	1.5-1.95

AS SIEVE SIZE (mm)	PERCENT PASSING
13.2	100
9.5	99
6.7	96
4.75	94
2.36	85
1.18	78
0.600	72
0.425	70
0.300	67
0.150	63
0.075	60



Comments:

Authorised Signatory

Dennis Nash

Date 14/05/2018

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ISO 9001

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PARTICLE SIZE DISTRIBUTION TEST REPORT

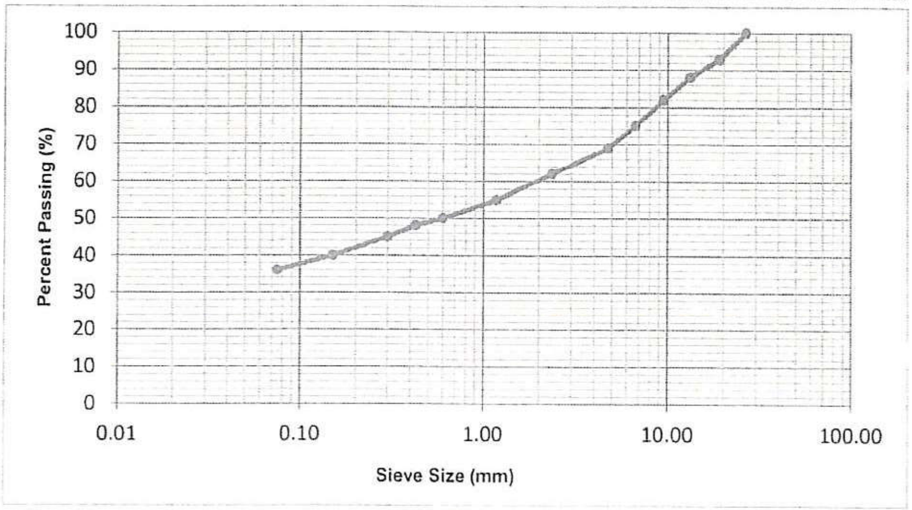
Test Procedure: AS1289.3.6.1 ☑ *Test Procedure: Q103A* ☐

Test Procedure: AS1289.2.1.1 ☑ *Test Procedure: Q103B* ☐

Client:	Watpac Construction Pty Ltd	Tested by:	DN	Date:	21/11/2017
Project:	Herston Quarter Redevelopment - Northern Car Par	Checked by:	DN	Date:	22/11/2017
Location:	Research Road, Herston	**Report No.:**	**017-141A_PSD_B1711-354**		
Project No:	017-141B	THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL			

Sample No.:	B1711-354
Sampling Method:	AS1289.1.2.1 Cl.6.5.3
Sample Moisture Content (%):	8.4
Bore:	4
Depth (m):	3.0-3.45

AS SIEVE SIZE (mm)	PERCENT PASSING
26.5	100
19.0	93
13.2	88
9.5	82
6.7	75
4.75	69
2.36	62
1.18	55
0.600	50
0.425	48
0.300	45
0.150	40
0.075	36



Comments:

Authorised Signatory

Bruce Butler

Date 22 | 11 | 2017

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PARTICLE SIZE DISTRIBUTION TEST REPORT

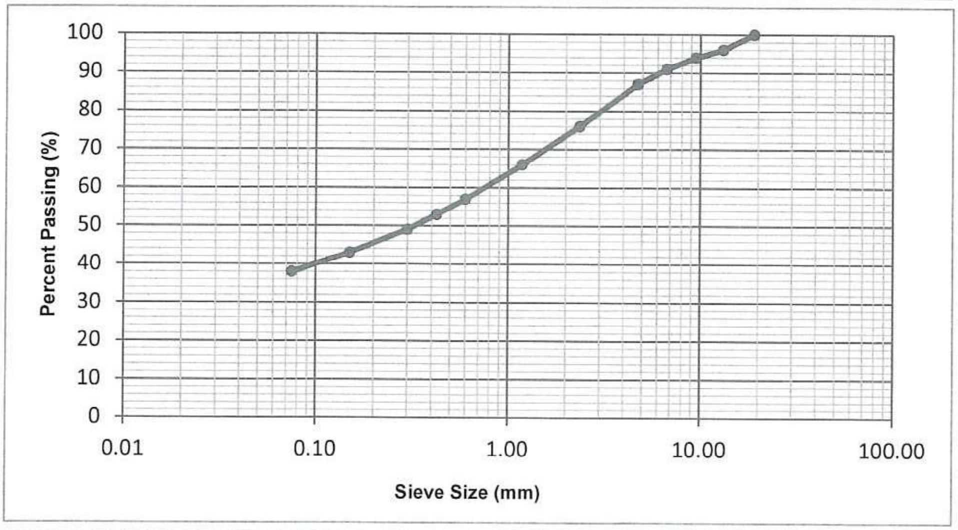
Test Procedure: AS1289.3.6.1 ☑ *Test Procedure: Q103A* ☐

Test Procedure: AS1289.2.1.1 ☑ *Test Procedure: Q103B* ☐

Client:	Herston Development Co Pty Ltd	Tested by:	DN	Date:	22/06/2018
Project:	Northern Car Park	Checked by:	DN	Date:	22/06/2018
Location:	Herstone Quarter, Herston	**Report No.:**	**017-141C_PSD_T11806-200**		
Project No:	017-141C	THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL			

Sample No.:	T1806-200
Sampling Method:	AS1289.1.2.1 Cl.6.5.3
Sample Moisture Content (%):	8.3
Bore:	8
Depth (m):	3.0-3.45

AS SIEVE SIZE (mm)	PERCENT PASSING
19.0	100
13.2	96
9.5	94
6.7	91
4.75	87
2.36	76
1.18	66
0.600	57
0.425	53
0.300	49
0.150	43
0.075	38



Comments:

Authorised Signatory

DNash

Dennis Nash

22/06/2018

Date

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Atterberg Limits Test Report

Test Procedure: AS1289.2.1.1
Test Procedure: AS1289.3.1.2
Test Procedure: AS1289.3.2.1
Test Procedure: AS1289.3.3.1
Test Procedure: AS1289.3.4.1

Client:	Watpac Construction Pty Ltd	**Report No.:**	**017-141B_ATL_B1711-348**		
Project:	Herston Quarter Redevelopment - Northern Car Park	Tested by:	DN	Date:	23/11/2017
Location:	Research Road, Herston	Checked by:	DN	Date:	24/11/2017
Project No:	017-141B	THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL			

Sample Number:	B1711-348	B1711-350	B1711-352	B1711-355	B1711-357
Sampling Method: AS1289.1.2.1	Cl.6.5.3	Cl.6.5.3	Cl.6.5.3	Cl.6.5.3	Cl.6.5.3
Bore:	1	2	3	4	6
Depth (m):	3.0-3.45	3.0-3.44	1.5-1.92	5.8-6.25	1.5-1.92
Liquid Limit (%)	81	44	42	38	31
Plastic Limit (%)	29	19	17	17	19
Plasticity Index (%)	52	25	25	21	12
Linear Shrinkage (%)	25.5	12.0	13.0	10.5	5.5
Sample Moisture Content (%)	38.6	11.1	9.2	12.9	11.1
Shrinkage Mould Length (mm)	*124.86*	*124.91*	*124.89*	*124.80*	*124.78*
Sample History	*Oven Dried*	*Oven Dried*	*Oven Dried*	*Oven Dried*	*Oven Dried*
Sample Preparation	*Dry Sieved*	*Dry Sieved*	*Dry Sieved*	*Dry Sieved*	*Dry Sieved*
Cracking of Linear Shrinkage Sample	*None*	*None*	*None*	*None*	*None*
Crumbling of Linear Shrinkage Sample	*None*	*None*	*None*	*None*	*None*
Curling of Linear Shrinkage Sample	*None*	*None*	*None*	*None*	*None*

Comments

Authorised Signatory

Bruce Butler

Date 24/11/2017

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Atterberg Limits Test Report

Test Procedure: AS1289.2.1.1
Test Procedure: AS1289.3.1.1
Test Procedure: AS1289.3.2.1
Test Procedure: AS1289.3.3.1
Test Procedure: AS1289.3.4.1

Client:	Herston Development Co Pty Ltd	**Report No.:**	**017-141C_ATL_T1806-145**		
Project:	Herston Quarter Redevelopment – Northern Car Park	Tested by:	DN	Date:	22/06/2018
Location:	Research Road, Herston	Checked by:	DN	Date:	25/06/2018
Project No:	017-141C	THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL			

Sample Number:	T1805-145
Sampling Method:	AS1289.1.2.1 Cl.6.5.3
Bore:	3A
Depth (m):	3.0-3.11

Liquid Limit (%)	36
Plastic Limit (%)	21
Plasticity Index (%)	15
Linear Shrinkage (%)	7.5
Sample Moisture Content (%)	6.5

Shrinkage Mould Length (mm)	*124.83*
Sample History	*Oven Dried*
Sample Preparation	*Dry Sieved*
Cracking of Linear Shrinkage Sample	*None*
Crumbling of Linear Shrinkage Sample	*None*
Curling of Linear Shrinkage Sample	*None*

Comments	***Authorised Signatory*** ***Dennis Nash*** 25/06/2018 ***Date***

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Atterberg Limits Test Report

Test Procedure: AS1289.2.1.1
Test Procedure: AS1289.3.1.2
Test Procedure: AS1289.3.2.1
Test Procedure: AS1289.3.3.1
Test Procedure: AS1289.3.4.1

Client:	Herston Development Co Pty Ltd	**Report No.:**	**017-141C_ATL_T1806-199**		
Project:	Herston Quarter Redevelopment - Northern Car Park	Tested by:	DN	Date:	22/06/2018
Location:	Research Road, Herston	Checked by:	DN	Date:	25/06/2018
Project No:	017-141C	THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL			

Sample Number:	T1806-199	T1805-147	T1806-201	T1805-148	T1805-149
Sampling Method: AS1289.1.2.1	Cl.6.5.3	Cl.6.5.3	Cl.6.5.3	Cl.6.5.3	Cl.6.5.3
Bore:	7	8	8	9	9
Depth (m):	1.5-1.95	6.0-6.34	10.5-10.7	4.5-4.95	9.0-9.37
Liquid Limit (%)	48	43	36	39	68
Plastic Limit (%)	19	19	21	20	16
Plasticity Index (%)	29	24	15	19	52
Linear Shrinkage (%)	13.5	10.0	7.5	9.5	18.5
Sample Moisture Content (%)	29.1	14.4	14.2	15.5	20.0
Shrinkage Mould Length (mm)	*127.02*	*124.71*	*126.94*	*124.94*	*124.95*
Sample History	*Oven Dried*	*Oven Dried*	*Oven Dried*	*Oven Dried*	*Oven Dried*
Sample Preparation	*Dry Sieved*	*Dry Sieved*	*Dry Sieved*	*Dry Sieved*	*Dry Sieved*
Cracking of Linear Shrinkage Sample	*None*	*None*	*None*	*None*	*None*
Crumbling of Linear Shrinkage Sample	*None*	*None*	*None*	*None*	*None*
Curling of Linear Shrinkage Sample	*None*	*None*	*None*	*None*	*None*

Comments

Authorised Signatory

Dennis Nash

25/06/2018

Date

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POINT LOAD STRENGTH INDEX TEST REPORT

Test Method: AS4133.4.1 AS1726

Client:	Herston Development Co Pty Ltd	**Report No.:**	**017-141C_PLS_3A**
Project:	Herston Quarter Redevelopment - Northern Car Park	**Tested by:**	PZ
Location:	Research Road, Herston	**Date:**	10/05/2018
		Checked by:	RZ
Project No:	017-141C	**Date:**	1/06/2018

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Bore:	**Depth (m)**	**Test Type**	**Sample Description**	**Point Load Strength [$I_s(50)$] (MPa)**	**Rock Strength Category***
3A	6.40	Diametral	Conglomerate	0.1	Low
	6.40	Axial	Conglomerate	0.2	Low
	7.50	Diametral	Conglomerate	0.1	Low
	7.50	Axial	Conglomerate	0.2	Low
	8.30	Diametral	Conglomerate	0.08	Very low
	8.30	Axial	Conglomerate	0.06	Very low
	9.80	Diametral	Conglomerate	0.05	Very low
	9.80	Axial	Conglomerate	0.1	Low
	10.30	Diametral	Conglomerate	0.3	Low
	10.30	Axial	Conglomerate	0.2	Low
	10.70	Diametral	Conglomerate	0.1	Low
	10.70	Axial	Conglomerate	0.2	Low
	12.10	Diametral	Conglomerate	0.3	Medium
	12.10	Axial	Conglomerate	0.6	Medium
	13.10	Diametral	Siltstone	0.9	Medium
	13.10	Axial	Siltstone	1.3	High
	14.10	Diametral	Sitstone	1.1	High
	14.10	Axial	Siltstone	2.2	High
	15.10	Diametral	Siltstone	1.6	High
	15.10	Axial	Siltstone	1.6	High
	16.10	Diametral	Siltstone	0.9	Medium
	16.10	Axial	Siltstone	2.2	High
	17.10	Diametral	Siltstone	1.0	Medium
	17.10	Axial	Siltstone	1.7	High
	18.10	Diametral	Argillite	0.5	Medium
	18.80	Diametral	Argillite	1.1	High
	18.80	Axial	Argillite	0.4	Medium
	19.50	Diametral	Argillite	0.5	Medium
	19.50	Axial	Argillite	0.4	Medium
	20.20	Diametral	Argillite	0.4	Medium
	20.20	Axial	Argillite	1.1	High
	21.50	Diametral	Argillite	0.2	Low
	21.50	Axial	Argillite	0.3	Medium
	22.10	Diametral	Argillite	0.6	Medium
	22.10	Axial	Argillite	0.6	Medium

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POINT LOAD STRENGTH INDEX TEST REPORT

Test Method: AS4133.4.1 AS1726

Client:	Herston Development Co Pty Ltd	**Report No.:**	**017-141C_PLS_7**
Project:	Herston Quarter Redevelopment - Northern Car Park	**Tested by:**	PZ
Location:	Research Road,	**Date:**	15/05/2018
	Herston	**Checked by:**	RZ
Project No:	017-141C	**Date:**	1/06/2018

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Bore:	**Depth (m)**	**Test Type**	**Sample Description**	**Point Load Strength [$I_s(50)$] (MPa)**	**Rock Strength Category***
7	3.70	Diametral	Tuff	0.8	Medium
	3.70	Diametral	Tuff	0.5	Medium
	3.70	Axial	Tuff	0.9	Medium
	4.80	Diametral	Tuff	1.2	High
	4.80	Diametral	Tuff	1.1	High
	4.80	Axial	Tuff	1.5	High
	5.90	Diametral	Tuff	1.4	High
	5.90	Axial	Tuff	1.8	High
	6.90	Diametral	Tuff	1.1	High
	6.90	Axial	Tuff	1.9	High
	7.50	Diametral	Tuff	1.0	Medium
	7.50	Diametral	Tuff	1.0	High
	7.50	Axial	Tuff	1.1	High
	9.00	Diametral	Tuff	1.1	High
	9.00	Axial	Tuff	2.1	High
	10.00	Diametral	Tuff	1.6	High
	10.00	Axial	Tuff	2.5	High
	10.60	Diametral	Tuff	1.7	High
	10.60	Axial	Tuff	1.8	High
	12.00	Diametral	Tuff	2.8	High
	12.00	Axial	Tuff	3.3	Very high
	13.00	Diametral	Tuff	1.0	Medium
	13.00	Axial	Tuff	2.4	High
	13.60	Diametral	Tuff	3.2	Very high
	13.60	Axial	Tuff	2.4	High
	14.80	Diametral	Tuff	1.8	High
	14.80	Axial	Tuff	1.5	High
	15.20	Diametral	Tuff	1.2	High
	15.20	Axial	Tuff	0.6	Medium
	16.20	Diametral	Tuff	3.1	Very high
	16.20	Diametral	Tuff	3.5	Very high
	16.20	Axial	Tuff	3.2	Very high
	18.50	Diametral	Tuff	0.9	Medium
	18.50	Axial	Tuff	1.4	High
	19.70	Diametral	Tuff	1.3	High
	19.70	Axial	Tuff	1.9	High
	20.70	Diametral	Carbonaceous Argillite	0.2	Low
	20.70	Axial	Carbonaceous Argillite	0.2	Low

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POINT LOAD STRENGTH INDEX TEST REPORT

Test Method: AS4133.4.1 AS1726

Client:	Herston Development Co Pty Ltd	**Report No.:**	**017-141C_PLS_BH7**
Project:	Herston Quarter Redevelopment - Northern Car Park	**Tested by:**	PZ
Location:	Research Road, Herston	**Date:**	15/05/2018
		Checked by:	RZ
Project No:	017-141C	**Date:**	1/06/2018

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Bore:	Depth (m)	Test Type	Sample Description	Point Load Strength [$I_s(50)$] (MPa)	Rock Strength Category*
7	21.70	Diametral	Conglomerate	0.3	Low
	21.70	Axial	Conglomerate	0.2	Low
	22.70	Diametral	Conglomerate	0.2	Low
	22.70	Axial	Conglomerate	0.3	Low
	23.40	Diametral	Conglomerate	0.09	Very low
	23.40	Axial	Conglomerate	0.1	Low
8	15.10	Diametral	Argillite	0.3	Medium
	15.10	Axial	Argillite	1.3	High
	15.92	Diametral	Argillite	2.4	High
	15.92	Axial	Argillite	1.9	High
	16.60	Diametral	Argillite	0.7	Medium
	16.60	Axial	Argillite	0.8	Medium
	17.55	Diametral	Argillite	1.4	High
	17.55	Axial	Argillite	3.8	Very high
	18.45	Diametral	Argillite	0.5	Medium
	18.45	Axial	Argillite	0.6	Medium
	19.35	Diametral	Argillite	1.8	High
	19.35	Axial	Argillite	1.4	High
9	15.65	Diametral	Conglomerate	0.7	Medium
	15.65	Axial	Conglomerate	1.1	High
	16.92	Axial	Conglomerate	0.5	Medium
	17.60	Diametral	Conglomerate	0.2	Low
	17.60	Axial	Conglomerate	1.8	High
	18.35	Diametral	Conglomerate	0.1	Low
	18.35	Axial	Conglomerate	0.3	Low
	18.50	Diametral	Argillite	2.0	High
	18.50	Axial	Argillite	1.2	High
	19.13	Diametral	Conglomerate	0.1	Low
	19.13	Axial	Conglomerate	0.1	Low
	19.80	Diametral	Conglomerate	0.4	Medium
	19.80	Axial	Conglomerate	0.2	Low
	20.15	Diametral	Conglomerate	0.2	Low
	20.15	Axial	Conglomerate	0.1	Low
	20.38	Diametral	Conglomerate	0.04	Very low
	20.53	Diametral	Siltstone	0.2	Low
	20.53	Axial	Siltstone	0.2	Low
	20.70	Diametral	Siltstone	0.07	Very low
	20.70	Axial	Siltstone	0.05	Very low

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POINT LOAD STRENGTH INDEX TEST REPORT

Test Method: AS4133.4.1 AS1726

Client:	Herston Development Co Pty Ltd	Report No.:	017-141C_PLS_BH9
Project:	Herston Quarter Redevelopment - Northern Car Park	Tested by:	CM and PZ
Location:	Research Road,	Date:	28/04/2018 and 29/05/2018
	Herston	Checked by:	RZ
Project No:	017-141C	Date:	1/06/2018

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Bore:	Depth (m)	Test Type	Sample Description	Point Load Strength [$I_s(50)$] (MPa)	Rock Strength Category*
9	21.80	Diametral	Siltstone	0.2	Low
	21.80	Axial	Siltstone	0.3	Medium
	23.00	Diametral	Conglomerate	0.2	Low
	23.00	Axial	Conglomerate	0.4	Medium
	24.00	Diametral	Conglomerate	0.3	Medium
	24.00	Axial	Siltstone	0.4	Medium
	25.00	Diametral	Mudstone	0.08	Very low
	25.00	Axial	Mudstone	0.1	Low
	25.50	Diametral	Mudstone	0.3	Low
	25.50	Axial	Mudstone	0.3	Low
	26.50	Diametral	Mudstone	0.3	Low
	26.50	Axial	Mudstone	0.4	Medium
	27.00	Diametral	Conglomerate	0.1	Low
	27.00	Axial	Conglomerate	0.1	Low
	28.00	Diametral	Conglomerate	0.5	Medium
	28.00	Axial	Conglomerate	0.2	Low
	29.00	Diametral	Conglomerate	0.1	Very low
	29.00	Axial	Conglomerate	0.6	Medium
	30.00	Diametral	Conglomerate	0.6	Medium
	30.00	Axial	Conglomerate	0.6	Medium

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APPENDIX D

PRELIMINARY WALLAP OUTPUT

```
                                                       Units: KN,m
INPUT DATA

SOIL PROFILE
Stratum   Elevation of    ------------------ Soil types -------------------
  no.    top of stratum    Left side                Right side
   1          24.60      23  FILL                  23  FILL
   2          19.60       5  CLAY vstiff            5  CLAY vstiff
   3          16.30      18  ROCK xls              18  ROCK xls
   4          14.60      19  ROCK vls              19  ROCK vls
   5           8.80      18  ROCK xls              18  ROCK xls
   6           5.80      19  ROCK vls              19  ROCK vls


GROUND WATER CONDITIONS
 Density of water = 10.00 KN/m3
                                   Left side     Right side
 Initial water table elevation       22.50           22.50

 Automatic water pressure balancing at toe of wall :  No

 Water             Left side                      Right side
 press. -------------------------------  -------------------------------
profile Point   Elev.    Piezo   Water   Point   Elev.    Piezo   Water
  no.    no.            elev.   press.   no.            elev.   press.
                  m        m     KN/m2             m        m    KN/m2
   1      1     22.60    22.60      0.0     1     19.20    19.20      0.0

   2      1     22.60    22.60      0.0     1     16.20    16.20      0.0

   3      1     22.60    22.60      0.0     1     13.20    13.20      0.0

   4      1     22.60    22.60      0.0     1     11.40    11.40      0.0

   5      1     22.60    22.60      0.0     1     10.00    10.00      0.0

WALL PROPERTIES
                         Type of structure = Soldier Pile Wall
                        Soldier Pile width =  0.75 m
                      Soldier Pile spacing =  2.00 m
               Passive mobilisation factor =  2.50
                  Elevation of toe of wall =  1.70
             Maximum finite element length =  1.20 m
                  Youngs modulus of wall E = 2.8000E+07 KN/m2
               Moment of inertia of wall I = 7.7620E-03 m4/m run
                                           = 0.015524 m4 per pile
                                       E.I = 217336 KN.m2/m run
                      Yield Moment of wall = Not defined
```

STRUTS and ANCHORS

```
Strut/                 X-section                    Inclin    Pre-
anchor         Strut     area      Youngs    Free  -ation   stress  Tension
 no.   Elev.  spacing  of strut    modulus  length (degs)   /strut  allowed
                 m       sq.m       KN/m2     m               KN
  1    23.10    2.00   0.000556  2.000E+08  35.00   30.00    450.0    No
  2    20.00    2.00   0.000695  2.000E+08  29.00   30.00    600.0    No
  3    17.00    2.00   0.000695  2.000E+08  23.40   30.00    600.0    No
  4    14.00    2.00   0.000973  2.000E+08  17.60   30.00    900.0    No
  5    12.20    2.00   0.001112  2.000E+08  13.00   30.00     1050    No
```

SURCHARGE LOADS

```
Surch         Distance   Length    Width        Surcharge      Equiv. Partial
-arge           from    parallel  perpend. -----  KN/m2  -----  soil  factor/
 no.   Elev.    wall    to wall   to wall  Near edge  Far edge  type  Category
  1    24.60    0.00(L)  130.00     14.60     16.00    176.00   N/A     N/A
  2    24.60   14.60(L)  130.00     35.40    176.00     =       N/A     N/A

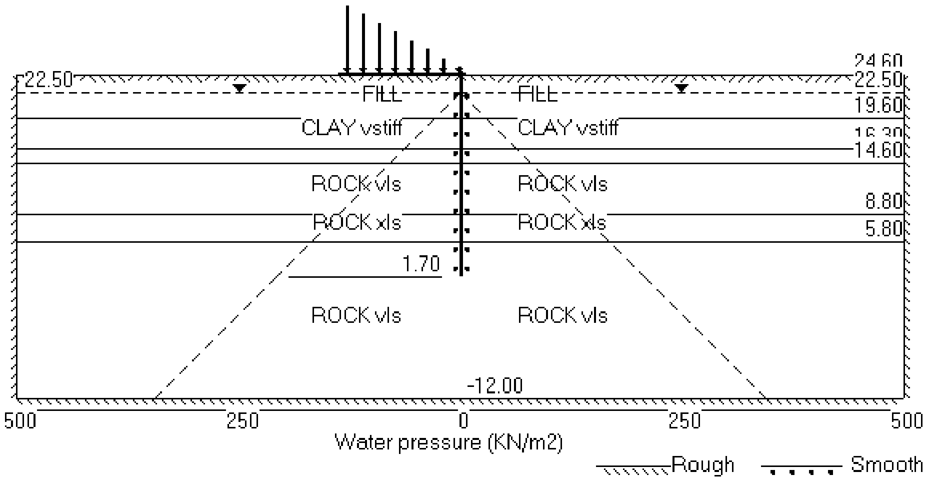
    Note: L = Left side,  R = Right side
          A trapezoidal surcharge is defined by two values:
          N = at edge near to wall,  F = at edge far from wall
```

CONSTRUCTION STAGES

```
Construction   Stage description
  stage no.    ---------------------------------------------------------
      1        Apply surcharge no.1 at elevation 24.60
      2        Apply surcharge no.2 at elevation 24.60
      3        Excavate to elevation 22.60 on RIGHT side
      4        Install strut or anchor no.1 at elevation 23.10
      5        Apply water pressure profile no.1
               No analysis at this stage
      6        Excavate to elevation 19.50 on RIGHT side
      7        Install strut or anchor no.2 at elevation 20.00
      8        Apply water pressure profile no.2
               No analysis at this stage
      9        Excavate to elevation 16.50 on RIGHT side
     10        Install strut or anchor no.3 at elevation 17.00
     11        Apply water pressure profile no.3
               No analysis at this stage
     12        Excavate to elevation 13.50 on RIGHT side
     13        Install strut or anchor no.4 at elevation 14.00
     14        Apply water pressure profile no.4
               No analysis at this stage
     15        Excavate to elevation 11.70 on RIGHT side
     16        Install strut or anchor no.5 at elevation 12.20
     17        Apply water pressure profile no.5
               No analysis at this stage
     18        Excavate to elevation 10.30 on RIGHT side
     19        Change properties of soil type 5 to soil type 11
               Ko pressures will be reset
```

```
                                                     Units: KN,m
```

Stage No.1   Apply surcharge no.1 at elev. 24.60



```
                                                       Units: KN,m
Stage No. 1   Apply surcharge no.1 at elevation 24.60
```

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method

```
 Factor of safety on gross pressure (excluding water pressure)

                                FoS for toe       Toe elev. for
                               elev. =    1.70     FoS = 2.000
                               ---------------    -------------
 Stage  --- G.L. ---    Strut  Factor  Moment      Toe    Wall    Direction
  No.    Act.   Pass.   Elev.    of    equilib.   elev.  Penetr       of
                               Safety  at elev.           -ation    failure
   1   24.60   24.60    Cant.   9.787     2.36     ***     ***      L to R

   Legend: *** Result not found
```

BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall

Analysis options

```
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

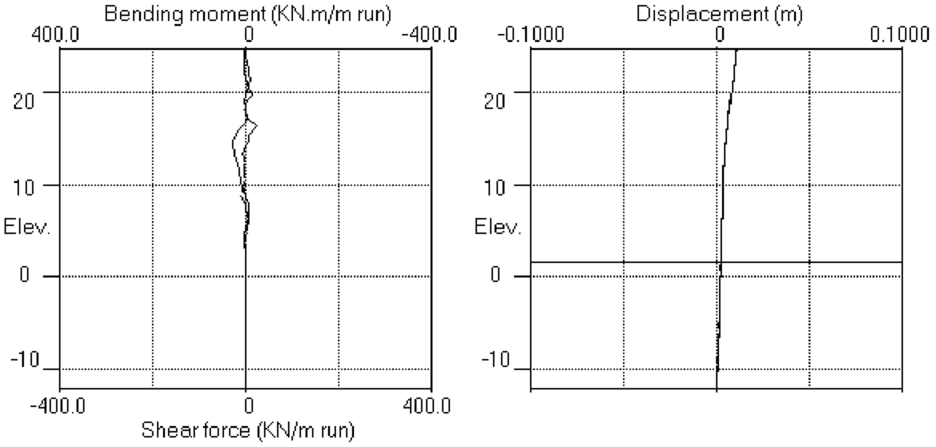
Node     Y       Nett       Wall      Wall      Shear   Bending   Strut
 no.   coord   pressure     disp.   rotation    force   moment    forces
                  KN/m2       m       rad.       KN/m    KN.m/m    KN/m
  1    24.60      -2.76     0.011   5.72E-04      0.0       0.0
  2    23.85      -3.40     0.010   5.73E-04     -2.3      -0.6
  3    23.10      -0.94     0.010   5.79E-04     -3.9      -3.0
  4    22.60       0.69     0.010   5.89E-04     -4.0      -5.1
  5    22.50       1.16     0.010   5.91E-04     -3.9      -5.5
  6    21.88       3.13     0.009   6.10E-04     -2.6      -7.6
  7    21.25       5.15     0.009   6.33E-04      0.0      -8.5
  8    20.63       7.14     0.008   6.56E-04      3.9      -7.3
  9    20.00       8.99     0.008   6.71E-04      8.9      -3.4
 10    19.60       8.12     0.008   6.74E-04     12.3       0.9
                 -14.94     0.008   6.74E-04     12.3       0.9
 11    19.50     -18.82     0.008   6.73E-04     10.6       2.0
 12    19.20     -19.30     0.008   6.68E-04      4.9       4.4
 13    18.10       2.63     0.007   6.60E-04     -4.2      -1.0
 14    17.00      19.64     0.006   6.71E-04      8.0      -3.4
 15    16.50      22.49     0.006   6.71E-04     18.5       3.1
 16    16.30      22.14     0.006   6.66E-04     23.0       7.3
                 -26.40     0.006   6.66E-04     23.0       7.3
 17    16.20     -16.93     0.006   6.62E-04     20.8       9.5
 18    15.40      -8.32     0.005   6.07E-04     10.7      20.8
 19    14.60      -0.69     0.005   5.19E-04      7.1      26.8
                 -18.90     0.005   5.19E-04      7.1      26.8
 20    14.00     -10.03     0.004   4.44E-04     -1.6      27.7
 21    13.50      -4.29     0.004   3.83E-04     -5.1      25.7
 22    13.20      -0.80     0.004   3.48E-04     -5.9      23.9
 23    12.20       1.25     0.004   2.53E-04     -5.7      17.7
 24    11.70       1.98     0.004   2.15E-04     -4.9      15.0
 25    11.40       1.18     0.003   1.95E-04     -4.4      13.6
 26    10.30       0.20     0.003   1.37E-04     -3.6       9.5
```

```
                                                                      (continued)
Stage No.1   Apply surcharge no.1 at elevation 24.60

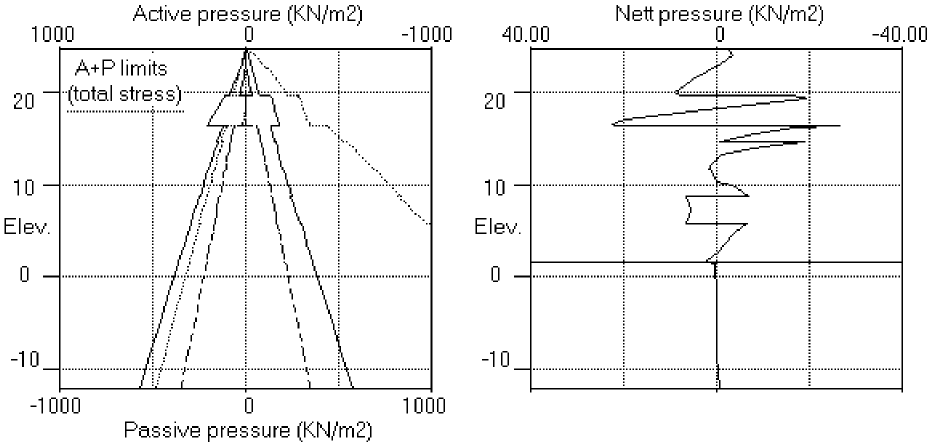
Node     Y       Nett       Wall      Wall       Shear   Bending   Strut
 no.  coord  pressure      disp.   rotation      force   moment    forces
               KN/m2         m       rad.        KN/m    KN.m/m    KN/m
 27   10.00     -3.42      0.003   1.24E-04      -4.1       8.4
 28    8.80     -6.73      0.003   9.90E-05     -10.2       1.0
                 6.82      0.003   9.90E-05     -10.2       1.0
 29    8.00      6.09      0.003   1.06E-04      -5.0      -5.0
 30    7.20      5.51      0.003   1.29E-04      -0.4      -7.2
 31    6.50      5.97      0.003   1.50E-04       3.6      -6.1
 32    5.80      6.58      0.003   1.63E-04       8.0      -2.2
                -6.40      0.003   1.63E-04       8.0      -2.2
 33    4.70     -3.86      0.003   1.62E-04       2.4       2.7
 34    3.60     -1.55      0.002   1.48E-04      -0.6       2.8
 35    2.65     -0.15      0.002   1.39E-04      -1.4       1.5
 36    1.70      2.40      0.002   1.35E-04      -0.4       0.0
 37    1.40      0.47      0.002          0       0.1       0.0
 38   -1.70     -0.01      0.002          0       0.8       0.0
 39   -4.80      0.06      0.001          0       0.9       0.0
 40   -8.40     -0.02      0.001          0       0.9       0.0
 41  -12.00     -0.50      0.000          0       0.0       0.0
```

Units: KN,m

Stage No.1 Apply surcharge no.1 at elev. 24.60

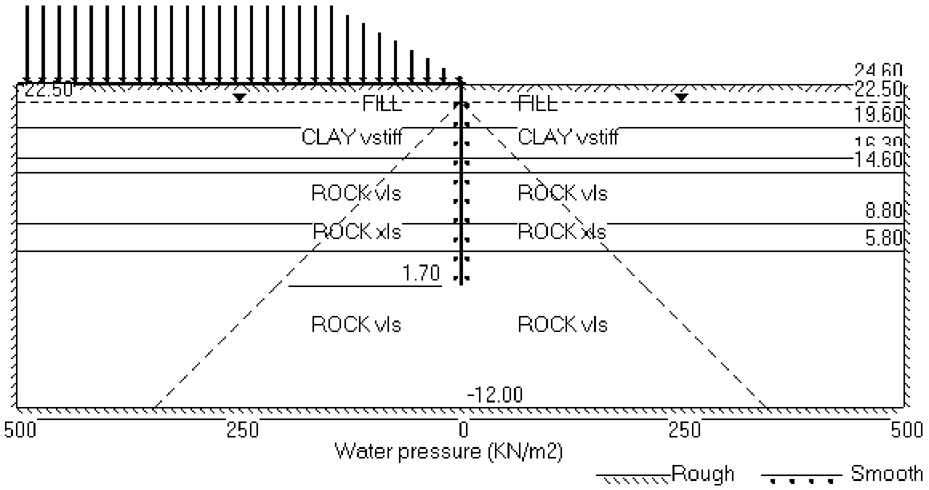


Stage No.1 Apply surcharge no.1 at elev. 24.60



Units: KN,m

Stage No.2 Apply surcharge no.2 at elev. 24.60



```
                                                       Units: KN,m
Stage No. 2   Apply surcharge no.2 at elevation 24.60

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                                 FoS for toe       Toe elev. for
                                elev. =    1.70     FoS = 2.000
                                ---------------    -------------
 Stage  --- G.L. ---    Strut   Factor  Moment       Toe     Wall    Direction
  No.    Act.    Pass.   Elev.     of    equilib.   elev.   Penetr      of
                                Safety  at elev.            -ation     failure
   2   24.60   24.60    ---    Conditions not suitable for FoS calc.


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

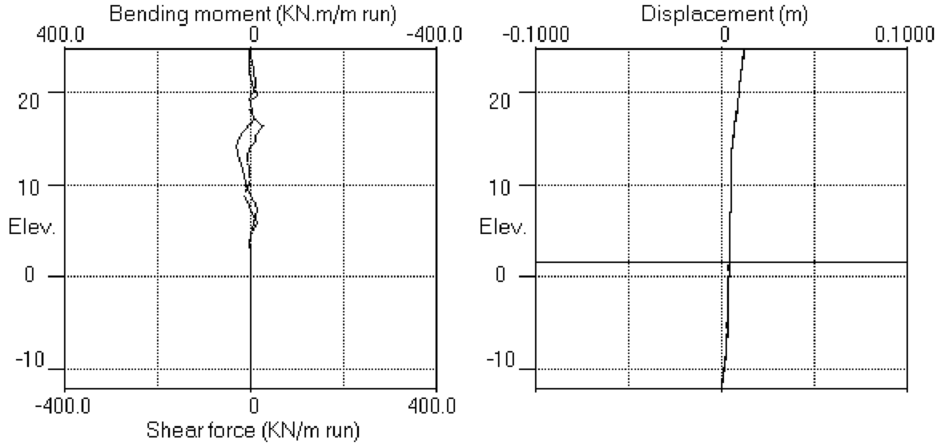
Node    Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
               KN/m2        m       rad.       KN/m    KN.m/m    KN/m
  1   24.60     -2.75    0.012   5.38E-04      0.0       0.0
  2   23.85     -3.49    0.012   5.39E-04     -2.3      -0.6
  3   23.10     -1.12    0.011   5.46E-04     -4.1      -3.1
  4   22.60      0.49    0.011   5.55E-04     -4.2      -5.2
  5   22.50      0.91    0.011   5.58E-04     -4.2      -5.6
  6   21.88      2.93    0.010   5.77E-04     -3.0      -7.9
  7   21.25      5.03    0.010   6.02E-04     -0.5      -9.1
  8   20.63      7.18    0.010   6.27E-04      3.3      -8.3
  9   20.00      9.28    0.009   6.45E-04      8.5      -4.7
 10   19.60      8.75    0.009   6.50E-04     12.1      -0.5
               -14.78    0.009   6.50E-04     12.1      -0.5
 11   19.50    -19.35    0.009   6.50E-04     10.4       0.6
 12   19.20    -19.65    0.009   6.48E-04      4.5       2.9
 13   18.10      2.60    0.008   6.47E-04     -4.8      -2.7
 14   17.00     21.65    0.007   6.67E-04      8.5      -5.3
 15   16.50     25.87    0.007   6.71E-04     20.4       1.8
 16   16.30     26.29    0.007   6.68E-04     25.6       6.4
               -29.68    0.007   6.68E-04     25.6       6.4
 17   16.20    -18.44    0.007   6.64E-04     23.2       8.8
 18   15.40     -8.62    0.006   6.08E-04     12.4      21.6
 19   14.60      0.25    0.006   5.15E-04      9.0      28.9
               -21.42    0.006   5.15E-04      9.0      28.9
 20   14.00    -11.40    0.006   4.33E-04     -0.8      30.5
 21   13.50     -4.96    0.005   3.65E-04     -4.9      28.7
 22   13.20     -1.43    0.005   3.27E-04     -5.9      27.0
 23   12.20      1.42    0.005   2.17E-04     -5.9      20.7
 24   11.70      2.07    0.005   1.72E-04     -5.0      18.0
 25   11.40      0.45    0.005   1.48E-04     -4.6      16.6
 26   10.30     -0.21    0.005   7.63E-05     -4.5      12.1
 27   10.00     -6.11    0.005   6.06E-05     -5.4      10.7
 28    8.80    -10.77    0.005   3.05E-05    -15.6       0.2
                10.23    0.005   3.05E-05    -15.6       0.2
```

```
                                                              (continued)
Stage No.2   Apply surcharge no.2 at elevation 24.60

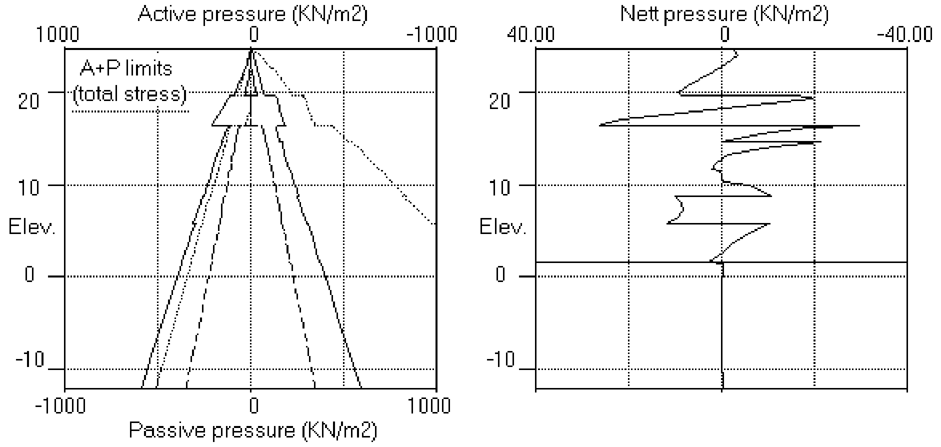
Node     Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
                KN/m2       m       rad.       KN/m    KN.m/m    KN/m
 29    8.00      8.60     0.005   4.66E-05     -8.0      -8.9
 30    7.20      8.35     0.005   8.61E-05     -1.3     -12.5
 31    6.50      9.43     0.004   1.24E-04      5.0     -11.3
 32    5.80     11.78     0.004   1.51E-04     12.4      -5.5
               -10.22     0.004   1.51E-04     12.4      -5.5
 33    4.70     -5.63     0.004   1.59E-04      3.7       2.2
 34    3.60     -2.10     0.004   1.46E-04     -0.6       3.0
 35    2.65     -0.15     0.004   1.36E-04     -1.6       1.6
 36    1.70      2.74     0.004   1.33E-04     -0.4       0.0
 37    1.40     -0.36     0.004          0     -0.1       0.0
 38   -1.70      0.10     0.003          0     -0.5       0.0
 39   -4.80      0.07     0.003          0     -0.2       0.0
 40   -8.40      0.08     0.002          0      0.1       0.0
 41  -12.00     -0.12     0.000          0      0.0       0.0
```

Units: KN,m

Stage No.2 Apply surcharge no.2 at elev. 24.60

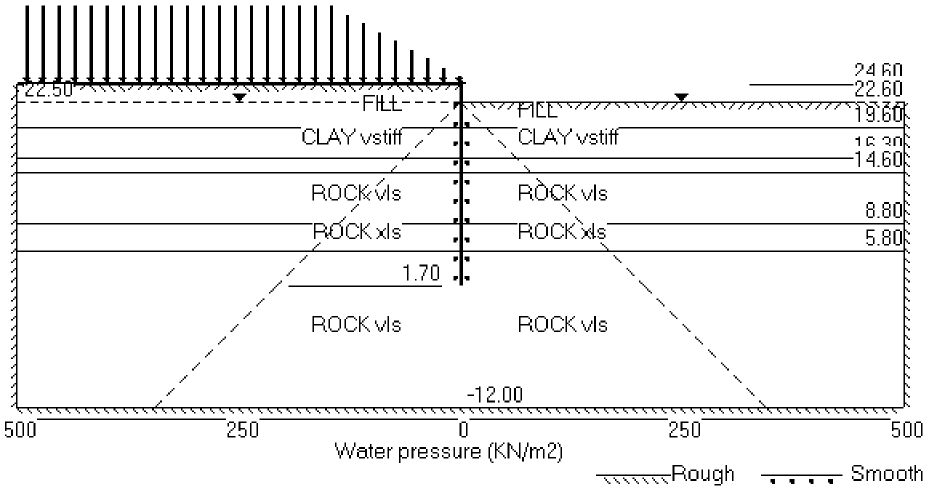


Stage No.2 Apply surcharge no.2 at elev. 24.60



Units: KN,m

Stage No.3   Excav. to elev. 22.60 on RIGHT side



```
                                                       Units: KN,m
Stage No. 3   Excavate to elevation 22.60 on RIGHT side

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                                FoS for toe       Toe elev. for
                               elev. =    1.70     FoS = 2.000
                               ---------------    -------------
 Stage  --- G.L. ---   Strut   Factor  Moment      Toe    Wall    Direction
  No.   Act.    Pass.    Elev.    of    equilib.   elev.  Penetr      of
                               Safety  at elev.          -ation     failure
   3   24.60   22.60    Cant.   6.365     2.39    16.65    5.95     L to R


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

Node     Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment     forces
                KN/m2       m       rad.      KN/m     KN.m/m    KN/m
  1   24.60      3.54     0.037   3.84E-03      0.0       0.0
  2   23.85     11.35     0.034   3.84E-03      5.6       2.0
  3   23.10     19.14     0.031   3.82E-03     17.0      10.4
  4   22.60     24.33     0.029   3.78E-03     27.9      21.6
                16.63     0.029   3.78E-03     27.9      21.6
  5   22.50     12.08     0.029   3.77E-03     29.3      24.5
  6   21.88     -1.26     0.026   3.67E-03     32.7      45.3
  7   21.25     -0.95     0.024   3.51E-03     32.0      65.6
  8   20.63      4.27     0.022   3.29E-03     33.1      85.5
  9   20.00      9.83     0.020   3.02E-03     37.5     107.1
 10   19.60     -4.03     0.019   2.80E-03     38.6     122.9
              -125.95     0.019   2.80E-03     38.6     122.9
 11   19.50   -104.57     0.019   2.75E-03     27.1     126.2
 12   19.20    -84.11     0.018   2.57E-03     -1.2     129.6
 13   18.10     11.94     0.015   2.04E-03    -40.9      78.6
 14   17.00     51.84     0.013   1.74E-03     -5.8      42.1
 15   16.50     55.82     0.012   1.64E-03     21.1      45.7
 16   16.30     53.00     0.012   1.59E-03     32.0      51.1
               -30.08     0.012   1.59E-03     32.0      51.1
 17   16.20    -34.41     0.012   1.57E-03     28.7      54.1
 18   15.40    -14.79     0.011   1.35E-03      9.1      66.3
 19   14.60      1.53     0.010   1.10E-03      3.8      69.0
               -30.70     0.010   1.10E-03      3.8      69.0
 20   14.00    -15.60     0.009   9.16E-04    -10.1      65.8
 21   13.50     -4.61     0.009   7.72E-04    -15.2      58.8
 22   13.20      0.92     0.008   6.95E-04    -15.7      54.0
 23   12.20      4.78     0.008   4.80E-04    -12.9      39.0
 24   11.70      5.34     0.008   3.97E-04    -10.4      33.2
 25   11.40      2.43     0.008   3.53E-04     -9.2      30.4
 26   10.30      0.88     0.007   2.21E-04     -7.4      22.0
 27   10.00     -8.90     0.007   1.92E-04     -8.6      19.9
```

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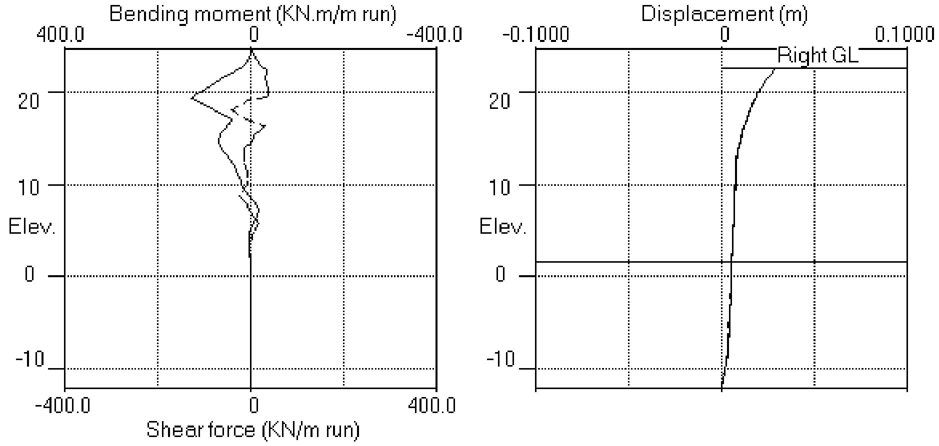
---------------------------------------------------------------------------
                                                                      (continued)
Stage No.3   Excavate to elevation 22.60 on RIGHT side

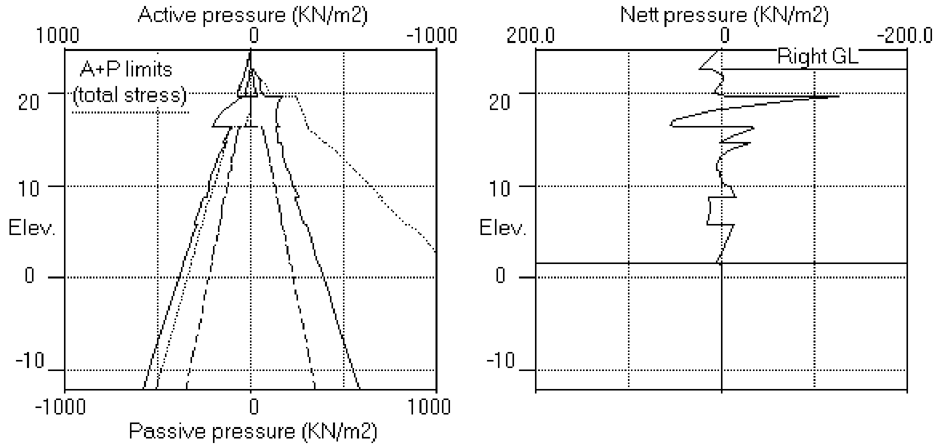
Node     Y       Nett       Wall      Wall      Shear   Bending    Strut
 no.   coord   pressure     disp.   rotation    force   moment     forces
                 KN/m2        m       rad.       KN/m    KN.m/m     KN/m
 28    8.80    -15.87     0.007   1.27E-04    -23.4       3.5
                13.98     0.007   1.27E-04    -23.4       3.5
 29    8.00     13.11     0.007   1.41E-04    -12.6     -10.7
 30    7.20     12.55     0.007   1.90E-04     -2.3     -16.5
 31    6.50     14.21     0.007   2.41E-04      7.0     -15.0
 32    5.80     16.19     0.006   2.76E-04     17.7      -6.5
               -14.14     0.006   2.76E-04     17.7      -6.5
 33    4.70     -8.45     0.006   2.80E-04      5.3       4.6
 34    3.60     -3.29     0.006   2.55E-04     -1.2       5.4
 35    2.65     -0.29     0.006   2.37E-04     -2.9       2.8
 36    1.70      4.87     0.005   2.31E-04     -0.7       0.0
 37    1.40     -0.30     0.005          0     -0.1       0.0
 38   -1.70      0.10     0.005          0     -0.4       0.0
 39   -4.80      0.09     0.004          0     -0.1       0.0
 40   -8.40      0.09     0.003          0      0.3       0.0
 41  -12.00     -0.23     0.000          0      0.0       0.0
```

Units: KN,m

Stage No.3 Excav. to elev. 22.60 on RIGHT side



Stage No.3 Excav. to elev. 22.60 on RIGHT side



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                                                     Units: KN,m
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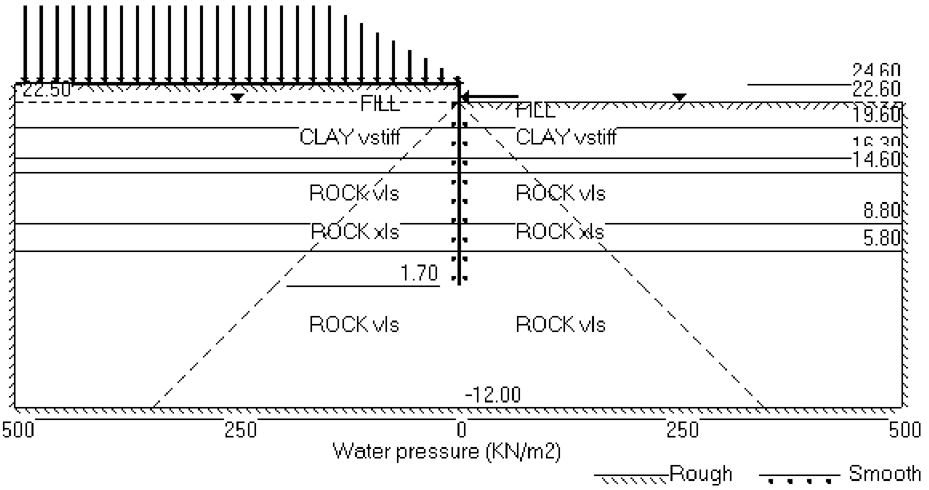
Stage No.4   Install strut no.1 at elev. 23.10



```
                                                       Units: KN,m
Stage No. 4   Install strut or anchor no.1 at elevation 23.10

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                               FoS for toe       Toe elev. for
                              elev. =    1.70     FoS = 2.000
                              ---------------    -------------
 Stage  --- G.L. ---   Strut  Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.    Elev.    of   equilib.  elev.  Penetr      of
                              Safety  at elev.          -ation    failure
   4   24.60   22.60    23.10  7.245     n/a     19.53    3.07     L to R


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

Node     Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure      disp.   rotation    force   moment    forces
                KN/m2        m       rad.      KN/m    KN.m/m    KN/m
  1   24.60     45.28      0.004  -1.48E-03      0.0      -0.0
  2   23.85     46.91      0.005  -1.50E-03     34.6      13.8
  3   23.10     48.31      0.006  -1.62E-03     70.3      53.9    194.9
                48.31      0.006  -1.62E-03   -124.6      53.9
  4   22.60     49.30      0.007  -1.68E-03   -100.2      -2.1
  5   22.50     46.63      0.007  -1.68E-03    -95.4     -11.8
  6   21.88     43.83      0.008  -1.57E-03    -67.1     -62.2
  7   21.25     31.60      0.009  -1.34E-03    -43.5     -95.5
  8   20.63     27.71      0.010  -1.04E-03    -25.0    -116.4
  9   20.00     27.08      0.010  -6.92E-04     -7.9    -126.5
 10   19.60     37.37      0.011  -4.59E-04      5.0    -127.5
                81.03      0.011  -4.59E-04      5.0    -127.5
 11   19.50     43.08      0.011  -4.00E-04     11.2    -126.6
 12   19.20     18.64      0.011  -2.29E-04     20.5    -121.3
 13   18.10     -2.96      0.011   2.95E-04     29.1     -86.2
 14   17.00     18.45      0.010   6.52E-04     37.6     -54.8
 15   16.50     25.42      0.010   7.53E-04     48.6     -33.6
 16   16.30     27.18      0.010   7.80E-04     53.8     -23.4
               -68.81      0.010   7.80E-04     53.8     -23.4
 17   16.20    -28.59      0.010   7.89E-04     49.0     -18.3
 18   15.40    -16.07      0.009   8.01E-04     31.1      11.9
 19   14.60     -3.69      0.008   7.21E-04     23.2      31.8
               -39.41      0.008   7.21E-04     23.2      31.8
 20   14.00    -21.48      0.008   6.23E-04      4.9      38.7
 21   13.50    -11.15      0.008   5.35E-04     -3.2      38.5
 22   13.20     -4.44      0.007   4.83E-04     -5.6      37.0
 23   12.20      0.18      0.007   3.29E-04     -7.7      29.5
 24   11.70      1.99      0.007   2.66E-04     -7.1      25.8
 25   11.40      0.50      0.007   2.32E-04     -6.8      23.7
 26   10.30     -0.55      0.007   1.29E-04     -6.8      16.9
 27   10.00     -7.75      0.007   1.07E-04     -8.0      14.8
```

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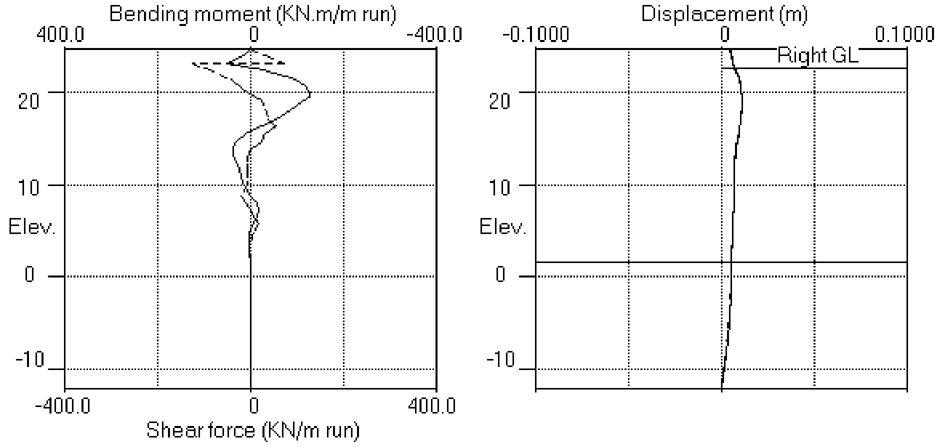
                                                              (continued)
Stage No.4   Install strut or anchor no.1 at elevation 23.10

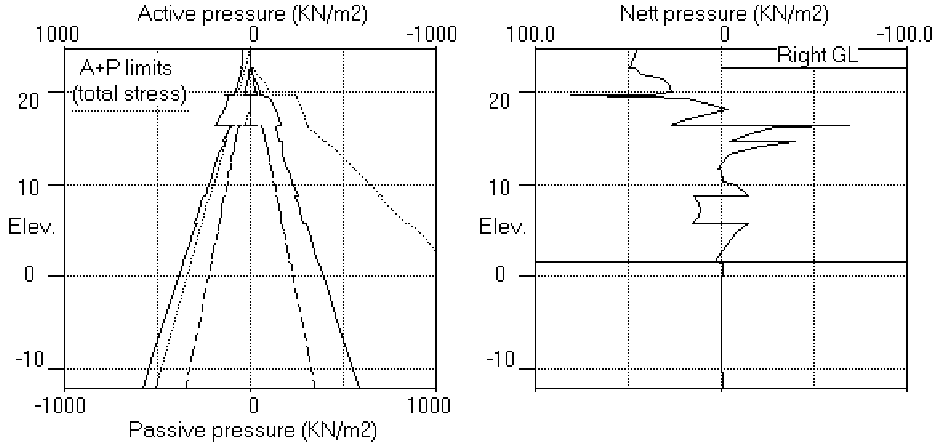
Node     Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation     force   moment    forces
               KN/m2        m        rad.       KN/m   KN.m/m     KN/m
 28    8.80    -14.40     0.006   6.68E-05     -21.3      -0.1
                14.86     0.006   6.68E-05     -21.3      -0.1
 29    8.00     11.95     0.006   8.96E-05     -10.6     -12.3
 30    7.20     11.38     0.006   1.43E-04      -1.3     -16.9
 31    6.50     12.65     0.006   1.94E-04       7.1     -14.9
 32    5.80     16.03     0.006   2.29E-04      17.2      -6.8
               -14.40     0.006   2.29E-04      17.2      -6.8
 33    4.70     -7.79     0.006   2.37E-04       5.0       3.5
 34    3.60     -2.92     0.006   2.17E-04      -0.9       4.4
 35    2.65     -0.19     0.005   2.03E-04      -2.4       2.3
 36    1.70      3.99     0.005   1.97E-04      -0.6       0.0
 37    1.40     -0.30     0.005          0      -0.1       0.0
 38   -1.70      0.10     0.005          0      -0.4       0.0
 39   -4.80      0.09     0.004          0      -0.1       0.0
 40   -8.40      0.09     0.003          0       0.3       0.0
 41  -12.00     -0.23     0.000          0       0.0       0.0
```

Units: KN,m

Stage No.4 Install strut no.1 at elev. 23.10

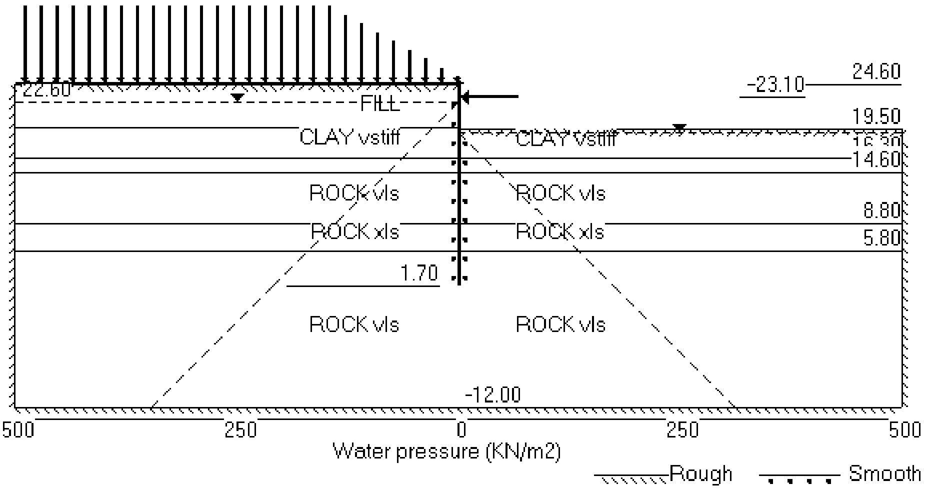


Stage No.4 Install strut no.1 at elev. 23.10



Units: KN,m

Stage No.6   Excav. to elev. 19.50 on RIGHT side



```
                                                      Units: KN,m
Stage No. 6   Excavate to elevation 19.50 on RIGHT side

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                               FoS for toe       Toe elev. for
                              elev. =    1.70     FoS = 2.000
                              ---------------    -------------
 Stage  --- G.L. ---   Strut  Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.   Elev.    of    equilib.   elev.  Penetr      of
                              Safety  at elev.          -ation    failure
   6   24.60   19.50   23.10  4.341     n/a     18.71    0.79     L to R


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

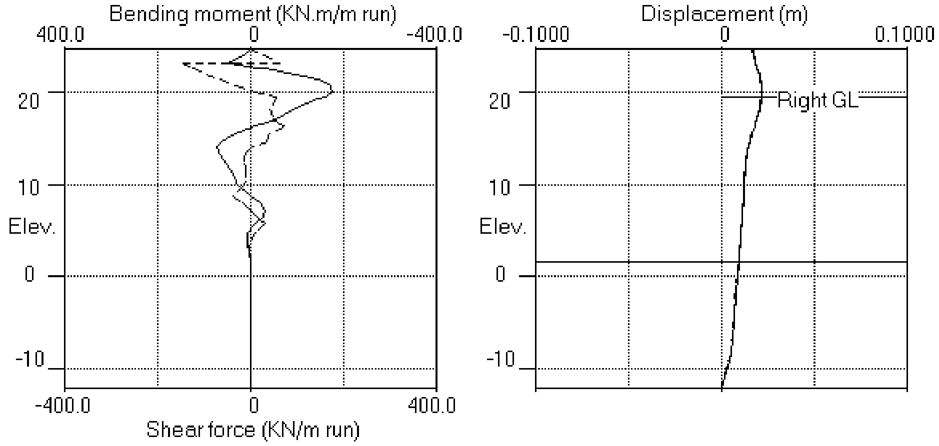
Node    Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
               KN/m2        m       rad.       KN/m    KN.m/m    KN/m
  1   24.60     41.46     0.016  -1.39E-03      0.0      -0.0
  2   23.85     42.14     0.017  -1.41E-03     31.4      12.7
  3   23.10     42.71     0.018  -1.52E-03     63.2      49.1    209.3
                42.71     0.018  -1.52E-03   -146.2      49.1
  4   22.60     43.20     0.019  -1.55E-03   -124.7     -18.4
  5   22.50     40.68     0.019  -1.54E-03   -120.5     -30.6
  6   21.88     46.84     0.020  -1.36E-03    -93.1     -97.3
  7   21.25     52.21     0.021  -1.01E-03    -62.2    -145.0
  8   20.63     60.41     0.021  -5.58E-04    -27.0    -172.8
  9   20.00     71.08     0.022  -5.57E-05     14.1    -177.0
 10   19.60     83.01     0.022   2.59E-04     44.9    -165.6
               103.86     0.022   2.59E-04     44.9    -165.6
 11   19.50     83.04     0.021   3.34E-04     54.3    -160.6
                17.40     0.021   3.34E-04     54.3    -160.6
 12   19.20    -14.66     0.021   5.44E-04     54.7    -143.5
 13   18.10    -12.41     0.020   1.13E-03     39.8     -90.9
 14   17.00     29.46     0.019   1.50E-03     49.2     -53.5
 15   16.50     37.71     0.018   1.59E-03     66.0     -25.1
 16   16.30     39.04     0.018   1.61E-03     73.6     -11.2
               -82.11     0.018   1.61E-03     73.6     -11.2
 17   16.20    -42.62     0.018   1.61E-03     67.4      -4.2
 18   15.40    -20.74     0.016   1.55E-03     42.1      36.2
 19   14.60      0.33     0.015   1.37E-03     33.9      63.4
               -63.37     0.015   1.37E-03     33.9      63.4
 20   14.00    -34.42     0.014   1.18E-03      4.6      72.4
 21   13.50    -17.00     0.014   1.01E-03     -8.3      70.4
 22   13.20     -5.80     0.014   9.24E-04    -11.7      67.2
 23   12.20      1.31     0.013   6.48E-04    -14.0      52.8
 24   11.70      4.07     0.013   5.35E-04    -12.6      46.0
 25   11.40      1.35     0.012   4.73E-04    -11.8      42.5
 26   10.30     -1.18     0.012   2.89E-04    -11.7      30.6
```

```
                                                              (continued)
Stage No.6   Excavate to elevation 19.50 on RIGHT side

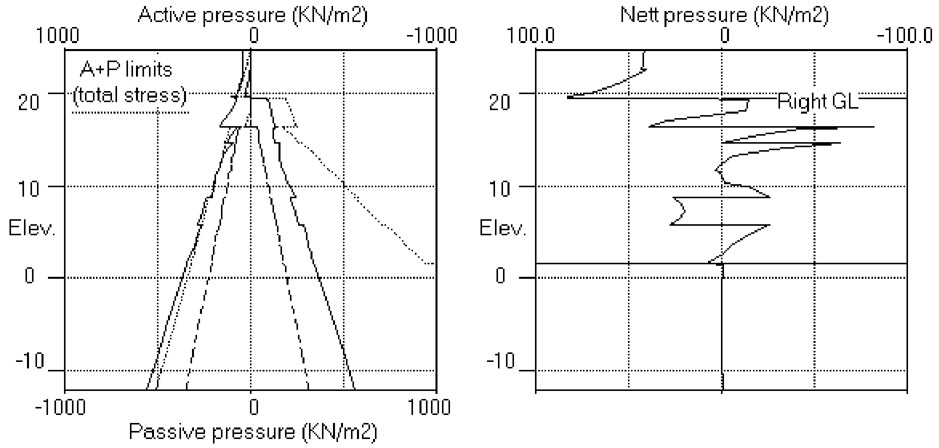
Node     Y       Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure      disp.   rotation     force   moment    forces
              KN/m2         m        rad.        KN/m   KN.m/m    KN/m
 27   10.00    -13.92     0.012   2.49E-04    -14.0      27.0
 28    8.80    -25.99     0.012   1.72E-04    -37.9       0.6
                26.29     0.012   1.72E-04    -37.9       0.6
 29    8.00     21.58     0.012   2.10E-04    -18.8     -21.2
 30    7.20     20.30     0.011   3.03E-04     -2.0     -29.2
 31    6.50     22.46     0.011   3.92E-04     13.0     -25.6
 32    5.80     27.76     0.011   4.51E-04     30.5     -11.0
               -25.45     0.011   4.51E-04     30.5     -11.0
 33    4.70    -14.07     0.010   4.60E-04      8.8       7.4
 34    3.60     -5.37     0.010   4.19E-04     -1.9       8.7
 35    2.65     -0.40     0.009   3.90E-04     -4.6       4.5
 36    1.70      7.71     0.009   3.81E-04     -1.2       0.0
 37    1.40     -0.24     0.009          0     -0.0       0.0
 38   -1.70      0.11     0.008          0     -0.2       0.0
 39   -4.80      0.12     0.007          0      0.1       0.0
 40   -8.40      0.10     0.005          0      0.5       0.0
 41  -12.00     -0.38     0.000          0      0.0       0.0
 At elev.  23.10 Strut force =    418.6 KN/strut =    209.3 KN/m run (horiz.)
                                                 =    241.7 KN/m run (inclined)
```

Units: KN,m

Stage No.6 Excav. to elev. 19.50 on RIGHT side

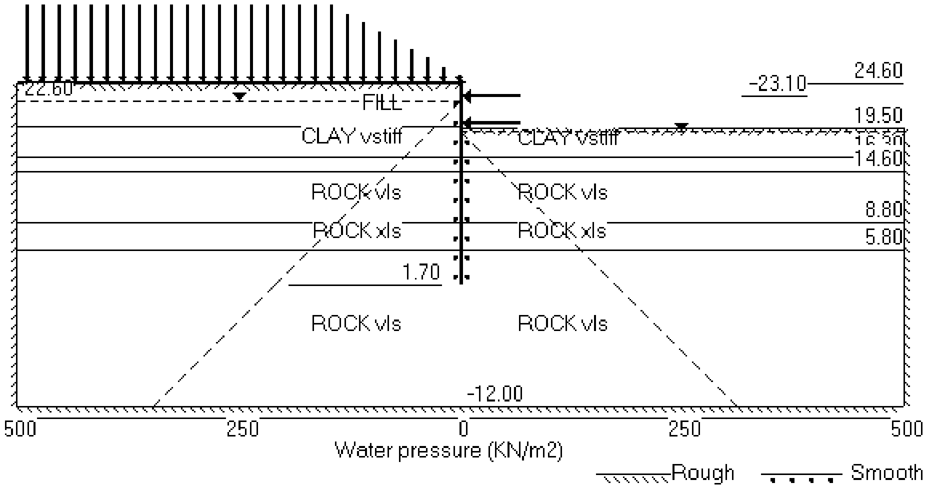


Stage No.6 Excav. to elev. 19.50 on RIGHT side



Units: KN,m

Stage No.7   Install strut no.2 at elev. 20.00



```
                                                       Units: KN,m
Stage No. 7   Install strut or anchor no.2 at elevation 20.00

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                                FoS for toe       Toe elev. for
                               elev. =    1.70     FoS = 2.000
                               ---------------    -------------
 Stage  --- G.L. ---   Strut   Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.    Elev.    of    equilib.   elev.  Penetr      of
                               Safety  at elev.           -ation     failure
   7   24.60   19.50           More than one strut. No FoS calc.


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

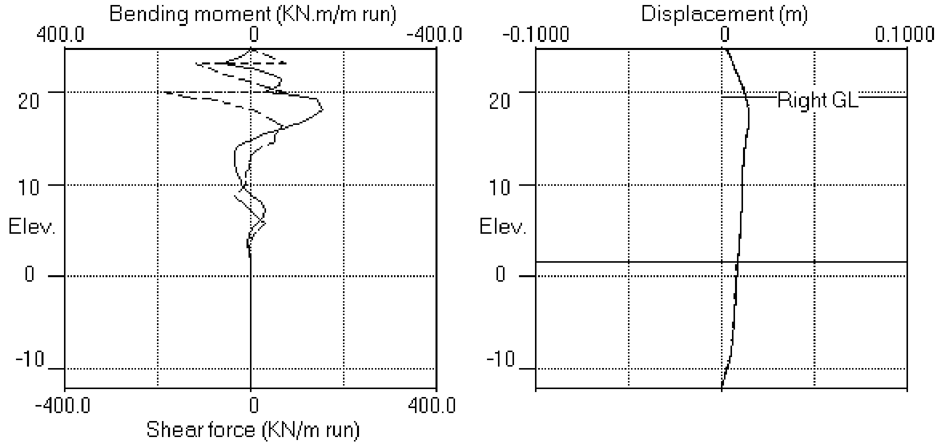
Node     Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
                KN/m2       m       rad.      KN/m    KN.m/m    KN/m
   1  24.60     49.69     0.003  -1.97E-03      0.0      -0.0
   2  23.85     51.17     0.004  -2.00E-03     37.8      15.0
   3  23.10     52.37     0.006  -2.13E-03     76.6      58.8    194.2
                52.37     0.006  -2.13E-03   -117.5      58.8
   4  22.60     53.23     0.007  -2.20E-03    -91.1       6.9
   5  22.50     51.05     0.007  -2.20E-03    -85.9      -2.0
   6  21.88     57.54     0.008  -2.14E-03    -52.0     -45.0
   7  21.25     63.30     0.010  -1.98E-03    -14.2     -64.9
   8  20.63     71.70     0.011  -1.80E-03     28.0     -60.6
   9  20.00     82.21     0.012  -1.67E-03     76.1     -28.2    259.8
                82.21     0.012  -1.67E-03   -183.7     -28.2
  10  19.60     97.72     0.012  -1.56E-03   -147.7     -95.1
               177.44     0.012  -1.56E-03   -147.7     -95.1
  11  19.50    154.74     0.013  -1.51E-03   -131.1    -108.9
  12  19.20    134.83     0.013  -1.34E-03    -87.7    -141.3
  13  18.10     36.51     0.014  -5.92E-04      6.5    -154.9
  14  17.00     31.00     0.014   1.14E-04     43.7    -124.4
  15  16.50     40.14     0.014   3.71E-04     61.5     -98.6
  16  16.30     47.68     0.014   4.55E-04     70.2     -85.5
               -69.15     0.014   4.55E-04     70.2     -85.5
  17  16.20    -19.28     0.014   4.93E-04     65.8     -78.9
  18  15.40     -9.97     0.014   6.97E-04     54.1     -32.2
  19  14.60      2.71     0.013   7.42E-04     51.2       8.1
               -59.41     0.013   7.42E-04     51.2       8.1
  20  14.00    -35.29     0.013   6.92E-04     22.8      28.2
  21  13.50    -21.78     0.012   6.19E-04      8.5      35.2
  22  13.20    -11.53     0.012   5.69E-04      3.5      36.8
  23  12.20     -4.37     0.012   4.04E-04     -4.4      34.8
  24  11.70     -0.80     0.011   3.27E-04     -5.7      32.1
  25  11.40     -2.07     0.011   2.84E-04     -6.1      30.4
  26  10.30     -3.85     0.011   1.50E-04     -9.4      22.7
```

```
                                                              (continued)
Stage No.7   Install strut or anchor no.2 at elevation 20.00

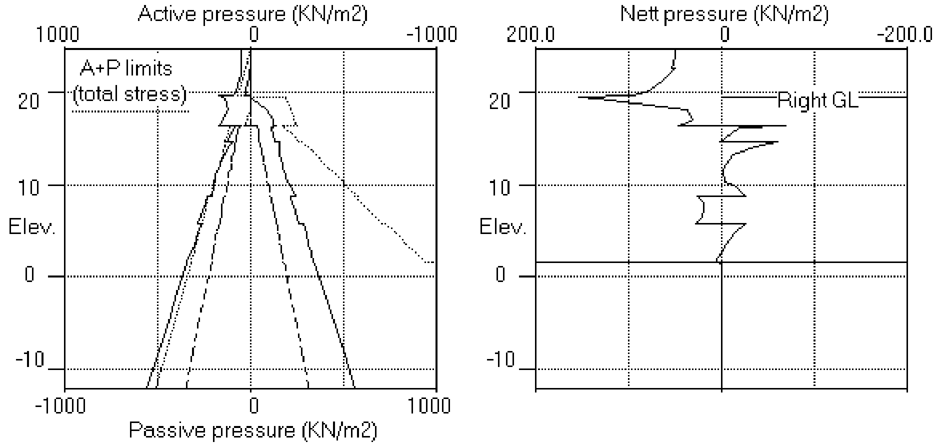
Node     Y       Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
                KN/m2        m       rad.      KN/m   KN.m/m    KN/m
 27   10.00    -13.70     0.011   1.21E-04    -12.0      19.7
 28    8.80    -25.12     0.011   7.89E-05    -35.3      -4.3
                26.81     0.011   7.89E-05    -35.3      -4.3
 29    8.00     20.21     0.011   1.30E-04    -16.5     -23.9
 30    7.20     19.04     0.011   2.30E-04     -0.8     -30.5
 31    6.50     20.85     0.011   3.22E-04     13.1     -26.4
 32    5.80     27.65     0.010   3.84E-04     30.1     -12.0
               -25.64     0.010   3.84E-04     30.1     -12.0
 33    4.70    -13.36     0.010   4.00E-04      8.7       5.7
 34    3.60     -4.96     0.009   3.67E-04     -1.4       7.3
 35    2.65     -0.31     0.009   3.43E-04     -3.9       3.8
 36    1.70      6.50     0.009   3.34E-04     -1.0       0.0
 37    1.40     -0.24     0.009          0     -0.0       0.0
 38   -1.70      0.11     0.008          0     -0.2       0.0
 39   -4.80      0.12     0.007          0      0.1       0.0
 40   -8.40      0.10     0.005          0      0.5       0.0
 41  -12.00     -0.38     0.000          0      0.0       0.0
 At elev.  23.10 Strut force =    388.3 KN/strut =    194.2 KN/m run (horiz.)
                                                =    224.2 KN/m run (inclined)
```

Units: KN,m

Stage No.7 Install strut no.2 at elev. 20.00

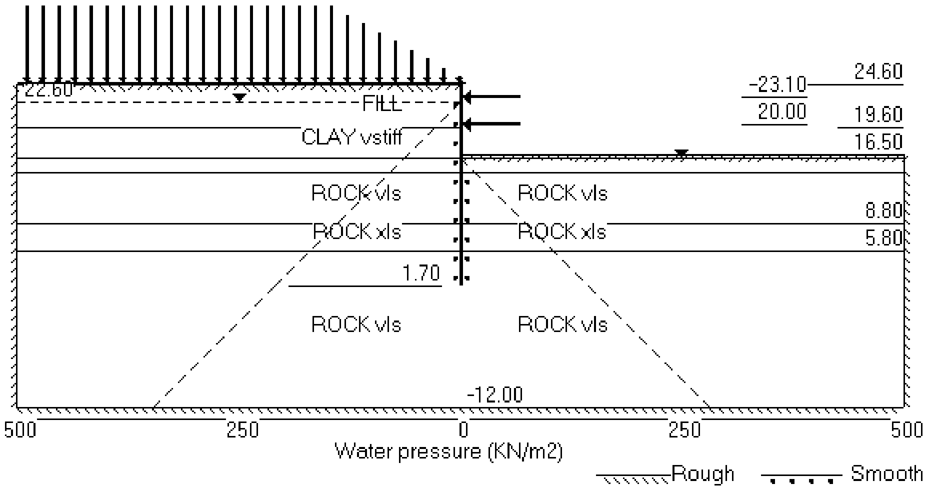


Stage No.7 Install strut no.2 at elev. 20.00



Units: KN,m

Stage No.9   Excav. to elev. 16.50 on RIGHT side



```
                                                     Units: KN,m
Stage No. 9   Excavate to elevation 16.50 on RIGHT side
```

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method

```
 Factor of safety on gross pressure (excluding water pressure)

                              FoS for toe       Toe elev. for
                             elev. =    1.70     FoS = 2.000
                             ---------------    -------------
 Stage  --- G.L. ---   Strut  Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.   Elev.    of    equilib.  elev.  Penetr       of
                             Safety  at elev.          -ation     failure
   9   24.60   16.50          More than one strut. No FoS calc.
```

BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall

Analysis options

```
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

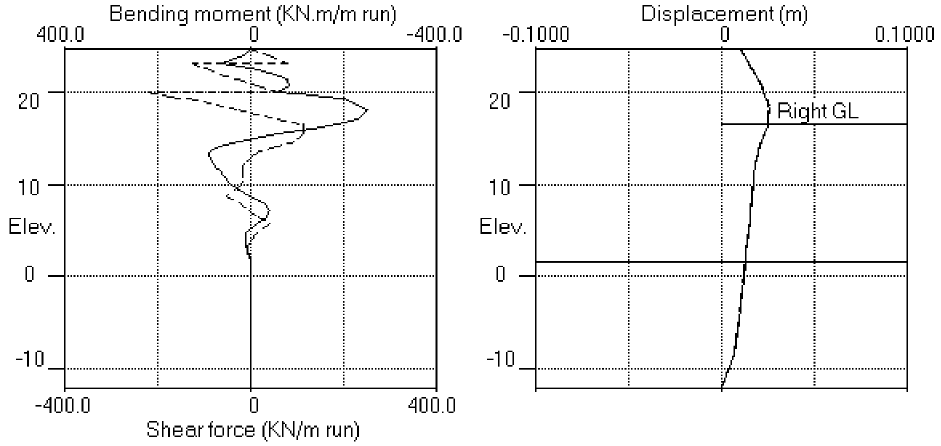
Node     Y       Nett       Wall      Wall      Shear   Bending   Strut
 no.   coord   pressure    disp.    rotation    force   moment    forces
                 KN/m2       m        rad.       KN/m    KN.m/m    KN/m
  1   24.60      52.42     0.010  -2.67E-03      0.0      -0.0
  2   23.85      52.84     0.012  -2.70E-03     39.5      15.8
  3   23.10      52.55     0.014  -2.83E-03     79.0      61.2    204.4
                 52.55     0.014  -2.83E-03   -125.4      61.2
  4   22.60      52.55     0.016  -2.90E-03    -99.2       5.4
  5   22.50      49.65     0.016  -2.90E-03    -94.1      -4.3
  6   21.88      55.46     0.018  -2.82E-03    -61.2     -52.6
  7   21.25      60.36     0.019  -2.63E-03    -25.0     -78.8
  8   20.63      68.03     0.021  -2.40E-03     15.1     -81.7
  9   20.00      77.93     0.022  -2.20E-03     60.7     -58.1    278.9
                 77.93     0.022  -2.20E-03   -218.2     -58.1
 10   19.60      97.95     0.023  -2.02E-03   -183.0    -139.1
                178.57     0.023  -2.02E-03   -183.0    -139.1
 11   19.50     140.15     0.023  -1.95E-03   -167.1    -156.5
 12   19.20     117.91     0.024  -1.71E-03   -128.4    -200.1
 13   18.10      69.92     0.025  -5.65E-04    -25.1    -253.0
 14   17.00      89.94     0.025   6.60E-04     62.8    -231.3
 15   16.50     107.18     0.025   1.14E-03    112.1    -188.3
                 -1.39     0.025   1.14E-03    112.1    -188.3
 16   16.30       7.70     0.025   1.30E-03    112.7    -165.9
                 21.70     0.025   1.30E-03    112.7    -165.9
 17   16.20      14.93     0.024   1.37E-03    114.6    -154.6
 18   15.40     -17.67     0.023   1.77E-03    113.5     -59.6
 19   14.60     -31.65     0.022   1.83E-03     93.8      25.7
                -66.80     0.022   1.83E-03     93.8      25.7
 20   14.00     -76.11     0.021   1.70E-03     50.9      70.0
 21   13.50     -55.04     0.020   1.52E-03     18.1      86.0
 22   13.20     -28.67     0.019   1.40E-03      5.5      88.9
 23   12.20     -10.90     0.018   1.01E-03    -14.2      80.4
 24   11.70      -0.81     0.018   8.39E-04    -17.2      71.9
 25   11.40      -0.72     0.017   7.43E-04    -17.4      66.8
```

```
                                                             (continued)
Stage No.9   Excavate to elevation 16.50 on RIGHT side

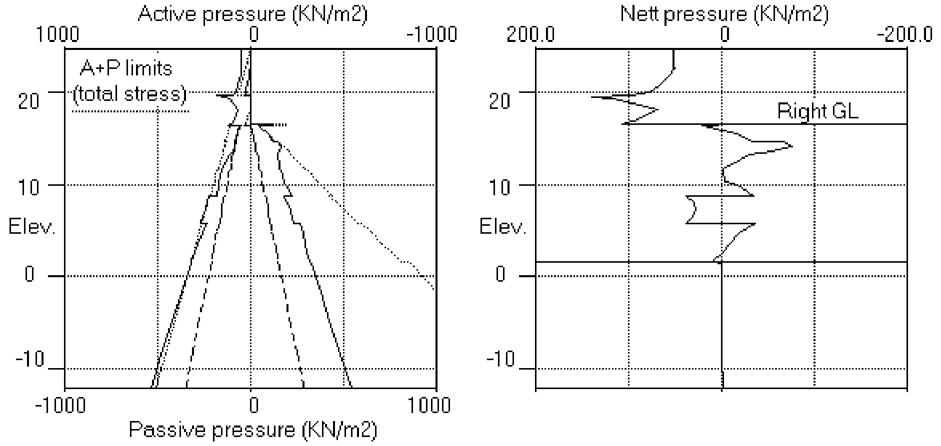
Node    Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
               KN/m2        m       rad.       KN/m   KN.m/m    KN/m
 26   10.30     -2.73     0.017   4.54E-04    -19.3      47.5
 27   10.00    -17.63     0.017   3.92E-04    -22.4      41.6
 28    8.80    -34.59     0.016   2.71E-04    -53.7       2.4
                38.30     0.016   2.71E-04    -53.7       2.4
 29    8.00     30.37     0.016   3.18E-04    -26.2     -28.2
 30    7.20     28.20     0.016   4.43E-04     -2.8     -39.3
 31    6.50     30.82     0.015   5.61E-04     17.9     -34.3
 32    5.80     38.76     0.015   6.39E-04     42.2     -14.2
               -35.73     0.015   6.39E-04     42.2     -14.2
 33    4.70    -19.38     0.014   6.48E-04     11.9      10.8
 34    3.60     -7.36     0.014   5.90E-04     -2.8      12.3
 35    2.65     -0.49     0.013   5.49E-04     -6.5       6.4
 36    1.70     10.81     0.013   5.35E-04     -1.6      -0.0
 37    1.40     -0.21     0.012          0     -0.0       0.0
 38   -1.70      0.12     0.011          0     -0.2       0.0
 39   -4.80      0.14     0.009          0      0.2       0.0
 40   -8.40      0.11     0.007          0      0.7       0.0
 41  -12.00     -0.49     0.000          0      0.0       0.0
 At elev.  23.10 Strut force =    408.9 KN/strut =    204.4 KN/m run (horiz.)
                                                 =    236.1 KN/m run (inclined)
 At elev.  20.00 Strut force =    557.9 KN/strut =    278.9 KN/m run (horiz.)
                                                 =    322.1 KN/m run (inclined)
```

```
                                                       Units: KN,m
```

Stage No.9 Excav. to elev. 16.50 on RIGHT side

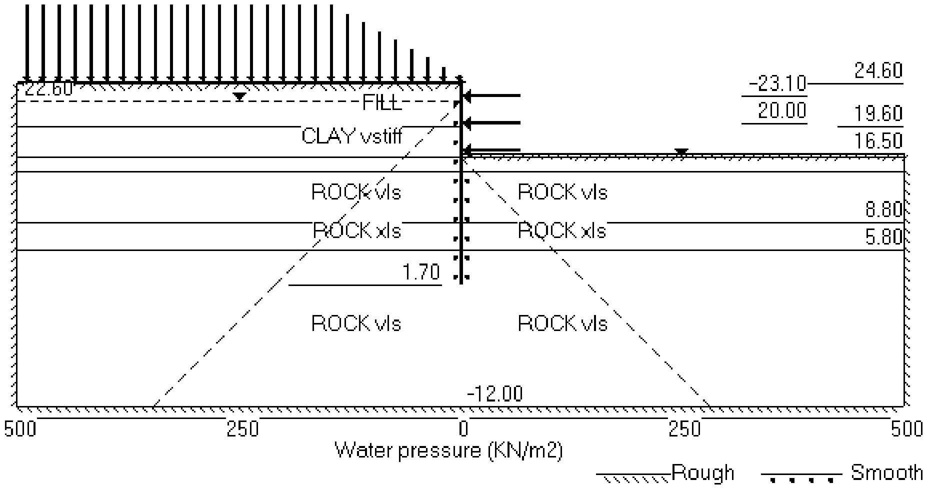


Stage No.9 Excav. to elev. 16.50 on RIGHT side



Units: KN,m

Stage No.10   Install strut no.3 at elev. 17.00



```
                                                       Units: KN,m
Stage No. 10   Install strut or anchor no.3 at elevation 17.00

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                                FoS for toe       Toe elev. for
                               elev. =    1.70     FoS = 2.000
                               ---------------    -------------
 Stage  --- G.L. ---   Strut   Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.   Elev.     of    equilib.  elev.  Penetr       of
                               Safety  at elev.          -ation     failure
  10   24.60   16.50           More than one strut. No FoS calc.


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

Node     Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
              KN/m2        m        rad.      KN/m    KN.m/m    KN/m
  1   24.60     50.58     0.006  -2.29E-03      0.0      -0.0
  2   23.85     52.02     0.008  -2.31E-03     38.5      15.2
  3   23.10     52.68     0.010  -2.44E-03     77.7      59.7    199.0
                52.68     0.010  -2.44E-03   -121.3      59.7
  4   22.60     53.22     0.011  -2.52E-03    -94.8       6.0
  5   22.50     50.77     0.011  -2.52E-03    -89.6      -3.2
  6   21.88     57.00     0.013  -2.44E-03    -55.9     -48.6
  7   21.25     62.43     0.014  -2.27E-03    -18.6     -71.1
  8   20.63     70.54     0.016  -2.07E-03     23.0     -69.6
  9   20.00     80.84     0.017  -1.91E-03     70.3     -40.7    268.8
                80.84     0.017  -1.91E-03   -198.5     -40.7
 10   19.60     97.29     0.017  -1.77E-03   -162.9    -113.5
               175.25     0.017  -1.77E-03   -162.9    -113.5
 11   19.50    149.18     0.018  -1.71E-03   -146.6    -128.9
 12   19.20    135.37     0.018  -1.51E-03   -104.0    -166.0
 13   18.10    109.59     0.019  -6.35E-04     30.8    -181.4
 14   17.00    134.23     0.020   1.13E-05    164.9     -74.4    259.8
               134.23     0.020   1.13E-05    -94.9     -74.4
 15   16.50    145.40     0.020   2.17E-04    -25.0    -104.7
                87.08     0.020   2.17E-04    -25.0    -104.7
 16   16.30     84.61     0.020   3.15E-04     -7.9    -107.9
               120.17     0.020   3.15E-04     -7.9    -107.9
 17   16.20     90.14     0.020   3.64E-04      2.6    -108.2
 18   15.40     36.19     0.019   7.08E-04     53.2     -78.7
 19   14.60      1.18     0.018   8.98E-04     68.1     -24.4
               -12.08     0.018   8.98E-04     68.1     -24.4
 20   14.00    -47.65     0.018   9.12E-04     50.2      14.3
 21   13.50    -41.17     0.017   8.57E-04     28.0      33.5
 22   13.20    -24.59     0.017   8.06E-04     18.1      40.1
 23   12.20    -12.27     0.016   6.08E-04     -0.3      46.2
 24   11.70     -5.15     0.016   5.03E-04     -4.6      44.6
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                                                                 (continued)
Stage No.10  Install strut or anchor no.3 at elevation 17.00

Node     Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation     force   moment    forces
                KN/m2       m       rad.        KN/m    KN.m/m    KN/m
 25   11.40     -5.27     0.016  4.43E-04      -6.2      42.9
 26   10.30     -7.03     0.016  2.50E-04     -13.0      33.2
 27   10.00    -19.52     0.016  2.07E-04     -17.0      29.0
 28    8.80    -35.34     0.015  1.41E-04     -49.9      -5.0
                37.85     0.015  1.41E-04     -49.9      -5.0
 29    8.00     28.59     0.015  2.11E-04     -23.3     -32.7
 30    7.20     26.79     0.015  3.48E-04      -1.1     -42.1
 31    6.50     29.23     0.015  4.75E-04      18.5     -36.3
 32    5.80     38.60     0.014  5.59E-04      42.2     -16.2
               -36.01     0.014  5.59E-04      42.2     -16.2
 33    4.70    -18.81     0.014  5.78E-04      12.1       8.6
 34    3.60     -7.01     0.013  5.29E-04      -2.1      10.7
 35    2.65     -0.44     0.013  4.94E-04      -5.7       5.5
 36    1.70      9.41     0.012  4.82E-04      -1.4      -0.0
 37    1.40     -0.21     0.012         0      -0.0       0.0
 38   -1.70      0.12     0.011         0      -0.2       0.0
 39   -4.80      0.14     0.009         0       0.2       0.0
 40   -8.40      0.11     0.006         0       0.7       0.0
 41  -12.00     -0.49     0.000         0       0.0       0.0
 At elev.  23.10 Strut force =    398.0 KN/strut =    199.0 KN/m run (horiz.)
                                                 =    229.8 KN/m run (inclined)
 At elev.  20.00 Strut force =    537.5 KN/strut =    268.8 KN/m run (horiz.)
                                                 =    310.3 KN/m run (inclined)
```

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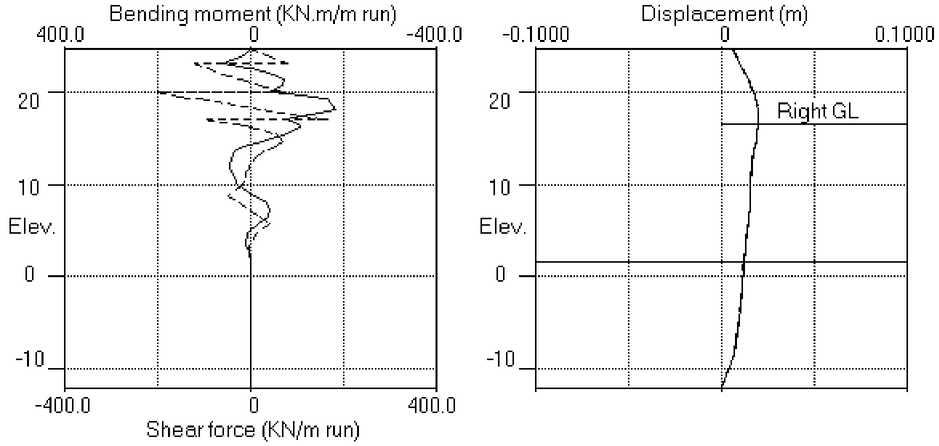
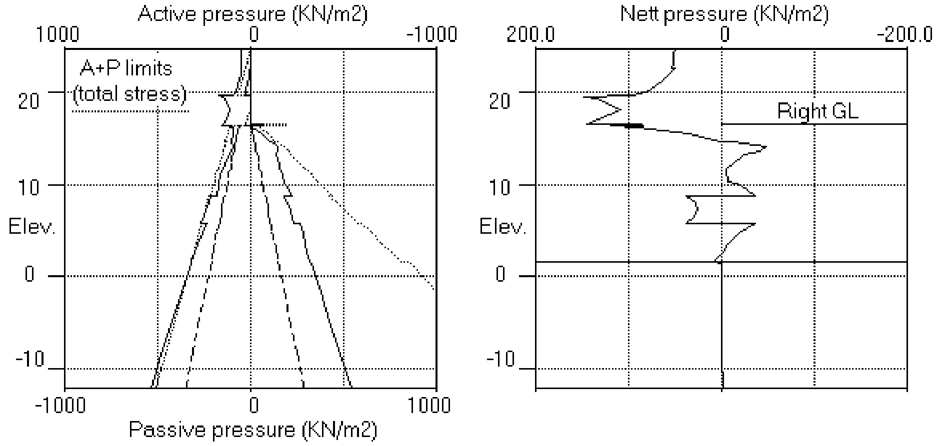
BUTLER PARTNERS PTY LTD                                    | Sheet No.
Program: WALLAP  Version 6.06  Revision A51.B69.R54        | Job No. 17-141C
                        Licensed from GEOSOLVE             | Made by :    PZ
Data filename/Run ID: 017-141A_Herston_Bore4_PZ            |
Herston Quarter Redevelopment - Northern Carpark           | Date:26-06-2018
Retaining Wall Stability                                   | Checked :
------------------------------------------------------------------------------
                                                     Units: KN,m
```

Stage No.10 Install strut no.3 at elev. 17.00

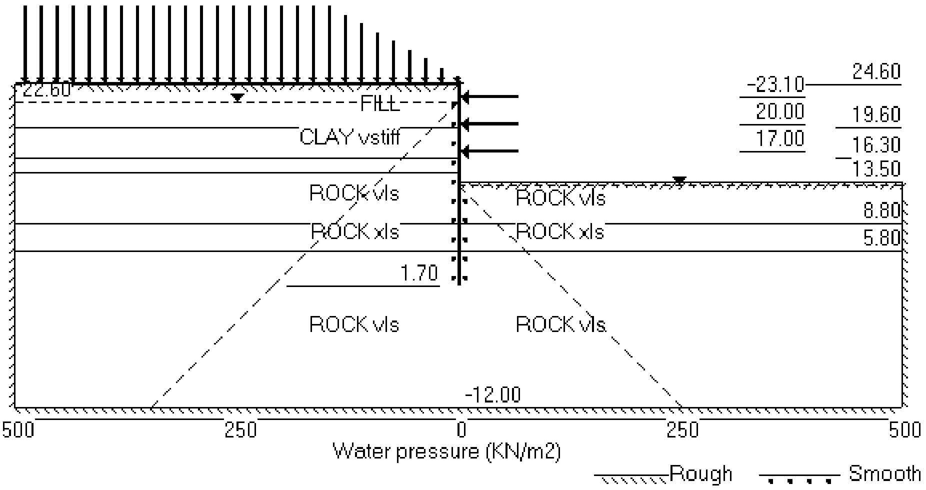


Stage No.10 Install strut no.3 at elev. 17.00



Units: KN,m

Stage No.12 Excav. to elev. 13.50 on RIGHT side



```
                                                      Units: KN,m
Stage No. 12   Excavate to elevation 13.50 on RIGHT side

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                                FoS for toe       Toe elev. for
                               elev. =    1.70     FoS = 2.000
                               ---------------    -------------
 Stage  --- G.L. ---   Strut   Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.   Elev.    of    equilib.   elev.  Penetr       of
                               Safety  at elev.          -ation     failure
  12   24.60   13.50           More than one strut. No FoS calc.


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

Node     Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
               KN/m2        m        rad.      KN/m    KN.m/m    KN/m
  1   24.60     52.42     0.013  -3.75E-03      0.0      -0.0
  2   23.85     60.09     0.015  -3.78E-03     42.2      15.8
  3   23.10     59.14     0.018  -3.92E-03     86.9      65.3    209.4
                59.14     0.018  -3.92E-03   -122.5      65.3
  4   22.60     58.72     0.020  -4.01E-03    -93.0      11.8
  5   22.50     55.50     0.021  -4.01E-03    -87.3       2.8
  6   21.88     60.97     0.023  -3.96E-03    -50.9     -40.2
  7   21.25     65.48     0.026  -3.81E-03    -11.4     -58.8
  8   20.63     72.83     0.028  -3.65E-03     31.9     -52.2
  9   20.00     82.59     0.030  -3.55E-03     80.4     -17.2    292.9
                82.59     0.030  -3.55E-03   -212.5     -17.2
 10   19.60    101.37     0.032  -3.45E-03   -175.7     -95.5
               195.68     0.032  -3.45E-03   -175.7     -95.5
 11   19.50    174.08     0.032  -3.40E-03   -157.2    -112.1
 12   19.20    162.30     0.033  -3.22E-03   -106.8    -151.3
 13   18.10     87.93     0.036  -2.45E-03     30.9    -153.4
 14   17.00     73.16     0.038  -1.91E-03    119.5     -59.3    301.7
                73.16     0.038  -1.91E-03   -182.2     -59.3
 15   16.50     76.81     0.039  -1.68E-03   -144.7    -140.9
 16   16.30    106.01     0.040  -1.54E-03   -126.4    -168.2
                63.00     0.040  -1.54E-03   -126.4    -168.2
 17   16.20     64.00     0.040  -1.46E-03   -120.1    -180.6
 18   15.40     72.00     0.041  -6.67E-04    -65.7    -252.1
 19   14.60     81.85     0.041   3.05E-04     -4.1    -276.6
                97.86     0.041   3.05E-04     -4.1    -276.6
 20   14.00     86.00     0.040   1.04E-03     51.0    -259.6
 21   13.50     91.00     0.040   1.60E-03     95.3    -223.3
                46.11     0.040   1.60E-03     95.3    -223.3
 22   13.20     26.07     0.039   1.88E-03    106.1    -193.1
 23   12.20     -8.17     0.037   2.51E-03    115.0     -78.1
 24   11.70    -20.45     0.036   2.62E-03    107.9     -21.6
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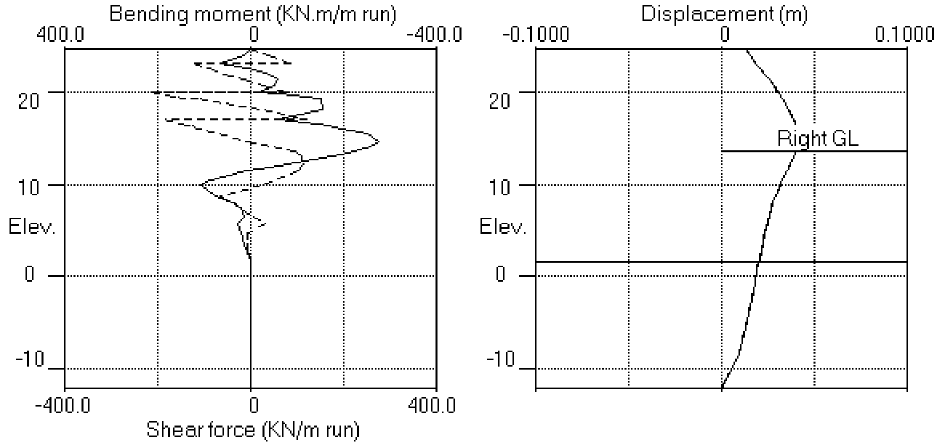
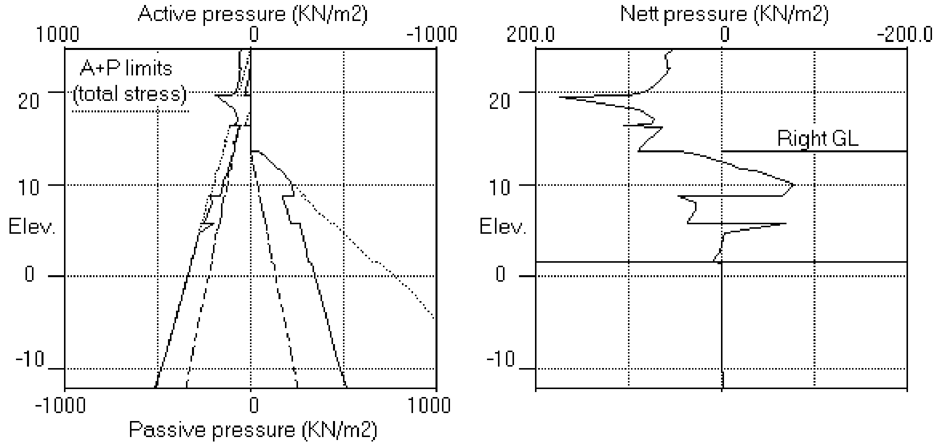
-----------------------------------------------------------------------------
                                                              (continued)
Stage No.12  Excavate to elevation 13.50 on RIGHT side

Node    Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
                KN/m2       m       rad.       KN/m    KN.m/m    KN/m
 25   11.40    -36.22     0.035   2.63E-03     99.4       9.9
 26   10.30    -69.67     0.032   2.36E-03     41.2      97.6
 27   10.00    -76.76     0.031   2.22E-03     19.2     106.8
 28    8.80    -65.00     0.029   1.72E-03    -65.9      74.9
               46.76     0.029   1.72E-03    -65.9      74.9
 29    8.00     27.29     0.028   1.51E-03    -36.3      37.3
 30    7.20     28.46     0.027   1.41E-03    -13.9      17.1
 31    6.50     33.02     0.026   1.36E-03      7.6      14.3
 32    5.80     37.41     0.025   1.29E-03     32.2      27.8
              -68.25     0.025   1.29E-03     32.2      27.8
 33    4.70     -2.31     0.023   1.16E-03     -6.6      22.1
 34    3.60     -0.23     0.022   1.07E-03     -8.0      13.5
 35    2.65      1.91     0.021   1.03E-03     -7.2       5.9
 36    1.70     10.06     0.020   1.02E-03     -1.5       0.0
 37    1.40     -0.21     0.020          0     -0.0       0.0
 38   -1.70      0.13     0.017          0     -0.1       0.0
 39   -4.80      0.16     0.014          0      0.3       0.0
 40   -8.40      0.13     0.009          0      0.8       0.0
 41  -12.00     -0.57     0.000          0      0.0       0.0
 At elev.  23.10 Strut force =    418.7 KN/strut =    209.4 KN/m run (horiz.)
                                                 =    241.7 KN/m run (inclined)
 At elev.  20.00 Strut force =    585.8 KN/strut =    292.9 KN/m run (horiz.)
                                                 =    338.2 KN/m run (inclined)
 At elev.  17.00 Strut force =    603.3 KN/strut =    301.7 KN/m run (horiz.)
                                                 =    348.3 KN/m run (inclined)
```

Stage No.12 Excav. to elev. 13.50 on RIGHT side

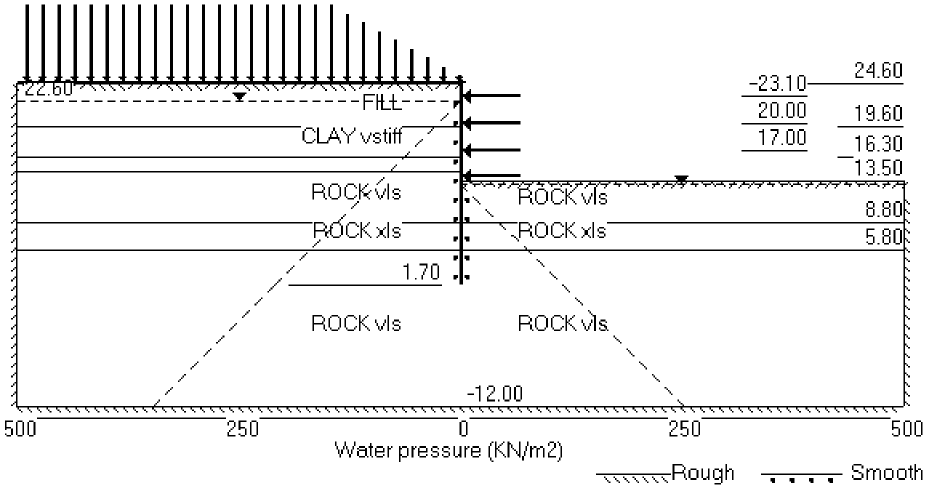


Stage No.12 Excav. to elev. 13.50 on RIGHT side



Units: KN,m

Stage No.13 Install strut no.4 at elev. 14.00



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                                                       Units: KN,m
Stage No. 13   Install strut or anchor no.4 at elevation 14.00

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                                FoS for toe       Toe elev. for
                               elev. =    1.70     FoS = 2.000
                               ---------------    -------------
 Stage  --- G.L. ---   Strut   Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.    Elev.    of    equilib.   elev.  Penetr       of
                               Safety  at elev.          -ation     failure
  13   24.60   13.50           More than one strut. No FoS calc.


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

Node    Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
                KN/m2       m       rad.       KN/m    KN.m/m    KN/m
  1   24.60     49.89     0.011  -3.39E-03      0.0      -0.0
  2   23.85     57.99     0.013  -3.41E-03     40.5      15.1
  3   23.10     57.50     0.016  -3.55E-03     83.8      62.7    206.6
                57.50     0.016  -3.55E-03   -122.8      62.7
  4   22.60     57.34     0.018  -3.63E-03    -94.1       8.8
  5   22.50     54.34     0.018  -3.63E-03    -88.5      -0.3
  6   21.88     60.01     0.020  -3.57E-03    -52.8     -44.3
  7   21.25     64.77     0.023  -3.41E-03    -13.8     -64.3
  8   20.63     72.33     0.025  -3.23E-03     29.0     -59.4
  9   20.00     82.22     0.027  -3.11E-03     77.3     -26.2    286.5
                82.22     0.027  -3.11E-03   -209.2     -26.2
 10   19.60    100.23     0.028  -2.99E-03   -172.7    -103.3
               189.95     0.028  -2.99E-03   -172.7    -103.3
 11   19.50    166.68     0.028  -2.94E-03   -154.9    -119.5
 12   19.20    154.60     0.029  -2.75E-03   -106.7    -158.3
 13   18.10     95.75     0.032  -1.93E-03     31.0    -165.1
 14   17.00     94.67     0.033  -1.34E-03    135.8     -66.1    290.4
                94.67     0.033  -1.34E-03   -154.6     -66.1
 15   16.50    101.38     0.034  -1.12E-03   -105.6    -131.2
 16   16.30    123.58     0.034  -9.93E-04    -83.1    -150.2
                89.36     0.034  -9.93E-04    -83.1    -150.2
 17   16.20     91.74     0.034  -9.22E-04    -74.0    -158.1
 18   15.40    110.22     0.035  -2.93E-04      6.7    -183.9
 19   14.60    129.93     0.035   2.99E-04    102.8    -138.2
               178.00     0.035   2.99E-04    102.8    -138.2
 20   14.00    172.69     0.035   5.49E-04    208.0     -42.6    389.7
               172.69     0.035   5.49E-04   -181.7     -42.6
 21   13.50    168.34     0.034   7.26E-04    -96.4    -111.9
 22   13.20    149.94     0.034   8.96E-04    -48.7    -133.6
 23   12.20     74.01     0.033   1.46E-03     63.3    -111.6
 24   11.70     26.92     0.032   1.66E-03     88.5     -70.6
```

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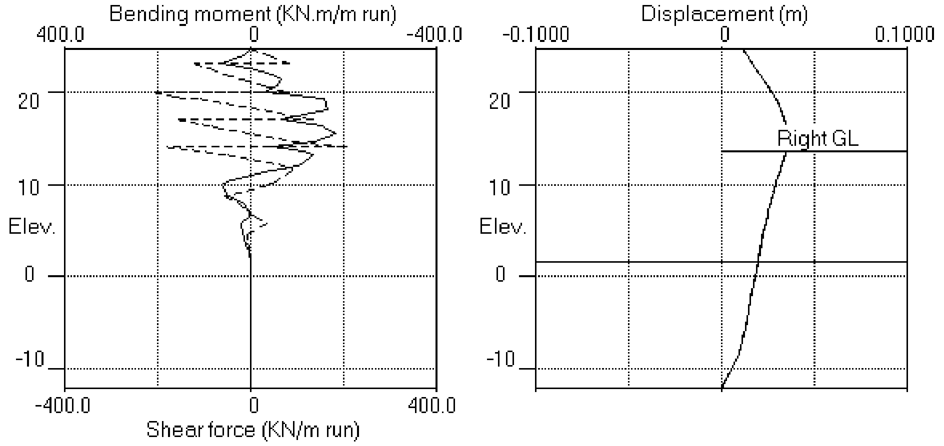
----------------------------------------------------------------------------
                                                              (continued)
Stage No.13  Install strut or anchor no.4 at elevation 14.00

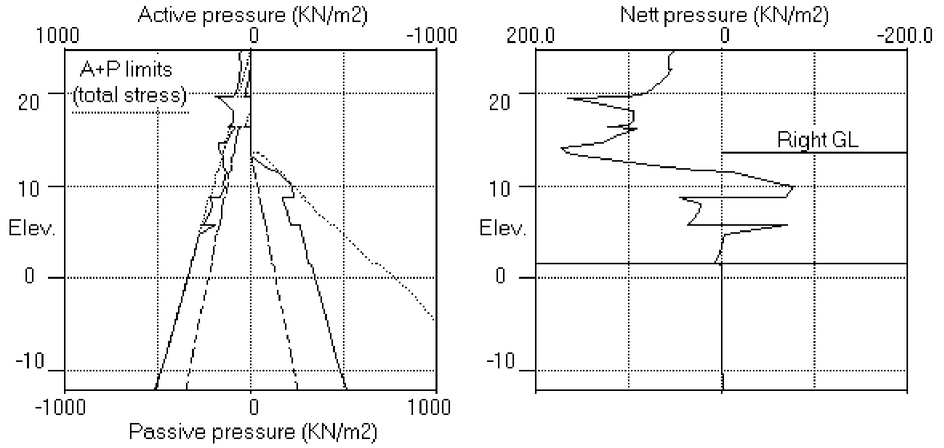
Node     Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation     force   moment    forces
                KN/m2       m       rad.       KN/m    KN.m/m    KN/m
 25   11.40    -11.03     0.031   1.74E-03     90.9     -42.9
 26   10.30    -58.63     0.030   1.72E-03     52.6      50.7
 27   10.00    -76.75     0.029   1.64E-03     32.3      63.9
 28    8.80    -67.98     0.027   1.34E-03    -54.6      47.7
                44.98     0.027   1.34E-03    -54.6      47.7
 29    8.00     23.17     0.026   1.21E-03    -27.3      18.5
 30    7.20     25.27     0.025   1.17E-03     -7.9       4.2
 31    6.50     29.98     0.024   1.16E-03     11.4       4.9
 32    5.80     36.61     0.024   1.12E-03     34.7      20.2
               -69.57     0.024   1.12E-03     34.7      20.2
 33    4.70     -2.25     0.022   1.02E-03     -4.8      16.5
 34    3.60     -0.12     0.021   9.62E-04     -6.1      10.0
 35    2.65      1.74     0.020   9.31E-04     -5.3       4.2
 36    1.70      7.18     0.020   9.22E-04     -1.1       0.0
 37    1.40     -0.21     0.019          0     -0.0       0.0
 38   -1.70      0.13     0.017          0     -0.1       0.0
 39   -4.80      0.16     0.014          0      0.3       0.0
 40   -8.40      0.13     0.009          0      0.8       0.0
 41  -12.00     -0.57     0.000          0      0.0       0.0
 At elev.  23.10 Strut force =    413.2 KN/strut =    206.6 KN/m run (horiz.)
                                                 =    238.5 KN/m run (inclined)
 At elev.  20.00 Strut force =    573.0 KN/strut =    286.5 KN/m run (horiz.)
                                                 =    330.9 KN/m run (inclined)
 At elev.  17.00 Strut force =    580.7 KN/strut =    290.4 KN/m run (horiz.)
                                                 =    335.3 KN/m run (inclined)
```

Units: KN,m

Stage No.13 Install strut no.4 at elev. 14.00



Stage No.13 Install strut no.4 at elev. 14.00



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                                                               Units: KN,m
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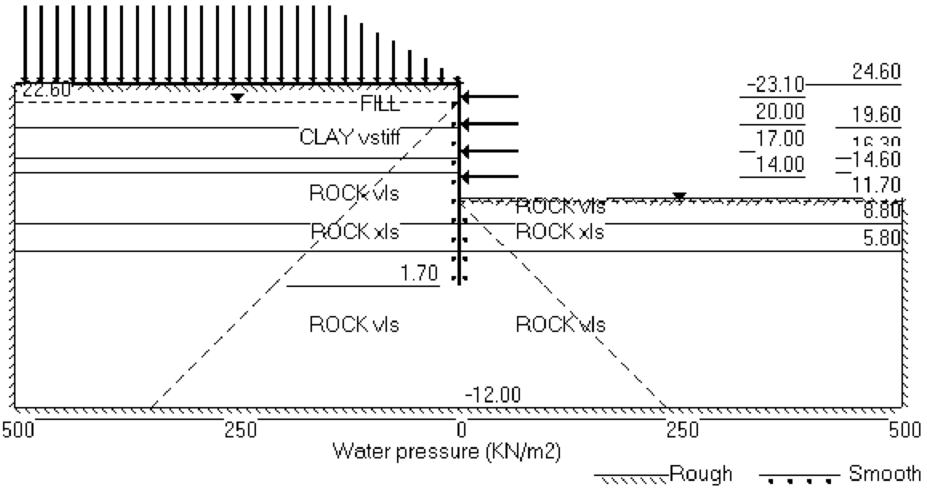
Stage No.15   Excav. to elev. 11.70 on RIGHT side



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                                                       Units: KN,m
Stage No. 15   Excavate to elevation 11.70 on RIGHT side

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                                FoS for toe       Toe elev. for
                               elev. =    1.70     FoS = 2.000
                               ---------------    -------------
 Stage  --- G.L. ---   Strut   Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.   Elev.    of    equilib.   elev.  Penetr       of
                               Safety  at elev.          -ation     failure
  15   24.60   11.70           More than one strut. No FoS calc.


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

Node     Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
               KN/m2        m       rad.       KN/m    KN.m/m    KN/m
  1   24.60     51.88     0.013  -3.74E-03      0.0      -0.0
  2   23.85     59.96     0.016  -3.77E-03     41.9      15.6
  3   23.10     59.36     0.019  -3.91E-03     86.7      64.9    209.8
                59.36     0.019  -3.91E-03   -123.1      64.9
  4   22.60     59.11     0.021  -4.00E-03    -93.5      11.1
  5   22.50     56.04     0.021  -4.00E-03    -87.7       2.0
  6   21.88     61.62     0.024  -3.94E-03    -51.0     -41.1
  7   21.25     66.26     0.026  -3.80E-03    -11.0     -59.7
  8   20.63     73.70     0.028  -3.64E-03     32.7     -52.7
  9   20.00     83.52     0.031  -3.54E-03     81.8     -17.0    293.5
                83.52     0.031  -3.54E-03   -211.7     -17.0
 10   19.60    100.87     0.032  -3.43E-03   -174.8     -94.9
               193.14     0.032  -3.43E-03   -174.8     -94.9
 11   19.50    173.39     0.032  -3.39E-03   -156.5    -111.4
 12   19.20    164.89     0.033  -3.20E-03   -105.7    -150.4
 13   18.10    101.53     0.036  -2.45E-03     40.8    -149.9
 14   17.00     97.47     0.039  -1.97E-03    150.2     -36.7    302.4
                97.47     0.039  -1.97E-03   -152.2     -36.7
 15   16.50    105.26     0.040  -1.82E-03   -101.5    -100.2
 16   16.30    128.17     0.040  -1.71E-03    -78.1    -118.3
                96.23     0.040  -1.71E-03    -78.1    -118.3
 17   16.20     93.70     0.040  -1.66E-03    -68.6    -125.7
 18   15.40    108.69     0.041  -1.16E-03     12.3    -146.5
 19   14.60    123.48     0.042  -7.15E-04    105.2     -96.9
               167.24     0.042  -7.15E-04    105.2     -96.9
 20   14.00    147.45     0.042  -5.78E-04    199.6      -1.8    422.8
               147.45     0.042  -5.78E-04   -223.3      -1.8
 21   13.50    127.41     0.043  -4.67E-04   -154.5     -95.0
 22   13.20     94.00     0.043  -3.08E-04   -121.3    -136.0
 23   12.20    104.00     0.043   4.71E-04    -22.3    -202.7
 24   11.70    109.00     0.042   9.33E-04     30.9    -199.0
                75.89     0.042   9.33E-04     30.9    -199.0
```

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                                                                (continued)
Stage No.15  Excavate to elevation 11.70 on RIGHT side

Node     Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation     force   moment    forces
               KN/m2        m       rad.        KN/m    KN.m/m    KN/m
 25   11.40     43.51     0.042  1.19E-03      48.8    -186.2
 26   10.30      1.04     0.040  1.93E-03      73.3    -104.3
 27   10.00     -9.75     0.040  2.06E-03      72.0     -82.3
 28    8.80    -53.03     0.037  2.32E-03      34.4     -14.3
                26.02     0.037  2.32E-03      34.4     -14.3
 29    8.00    -27.65     0.035  2.31E-03      33.7      21.6
 30    7.20     -6.19     0.033  2.20E-03      20.2      39.8
 31    6.50     11.15     0.032  2.05E-03      21.9      52.5
 32    5.80     25.67     0.031  1.85E-03      34.8      70.6
               -99.63     0.031  1.85E-03      34.8      70.6
 33    4.70     -0.00     0.029  1.55E-03     -20.0      48.7
 34    3.60      3.09     0.027  1.36E-03     -18.3      26.9
 35    2.65      5.54     0.026  1.27E-03     -14.2      10.9
 36    1.70     18.54     0.025  1.25E-03      -2.8       0.0
 37    1.40     -0.22     0.024         0      -0.0       0.0
 38   -1.70      0.13     0.021         0      -0.2       0.0
 39   -4.80      0.16     0.017         0       0.3       0.0
 40   -8.40      0.14     0.011         0       0.8       0.0
 41  -12.00     -0.59     0.000         0       0.0       0.0
 At elev.  23.10 Strut force =    419.6 KN/strut =    209.8 KN/m run (horiz.)
                                                 =    242.3 KN/m run (inclined)
 At elev.  20.00 Strut force =    587.1 KN/strut =    293.5 KN/m run (horiz.)
                                                 =    338.9 KN/m run (inclined)
 At elev.  17.00 Strut force =    604.8 KN/strut =    302.4 KN/m run (horiz.)
                                                 =    349.2 KN/m run (inclined)
 At elev.  14.00 Strut force =    845.7 KN/strut =    422.8 KN/m run (horiz.)
                                                 =    488.2 KN/m run (inclined)
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                                                    Units: KN,m
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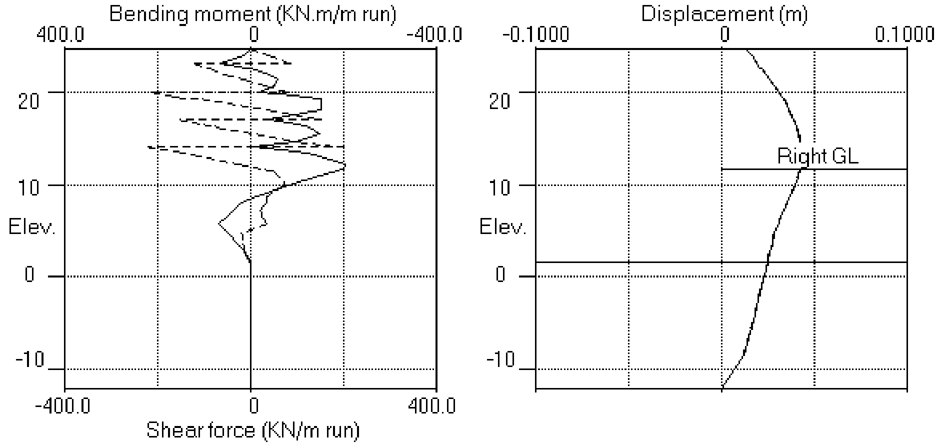
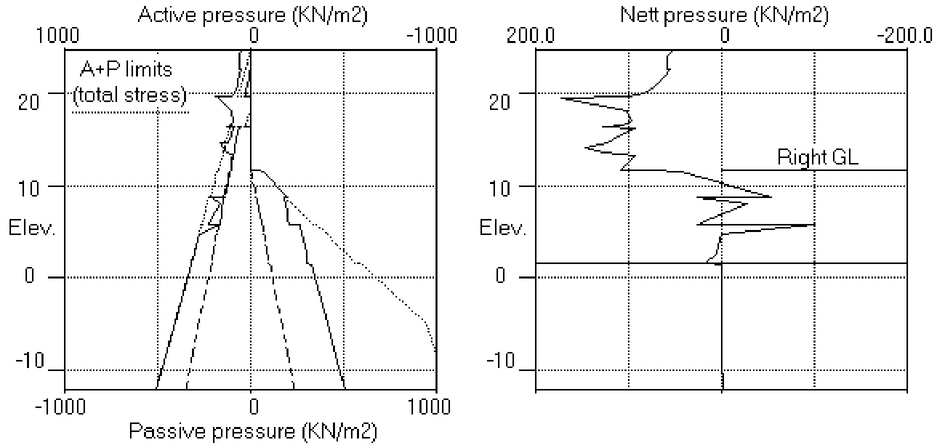
Stage No.15 Excav. to elev. 11.70 on RIGHT side



Stage No.15 Excav. to elev. 11.70 on RIGHT side



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                                                       Units: KN,m
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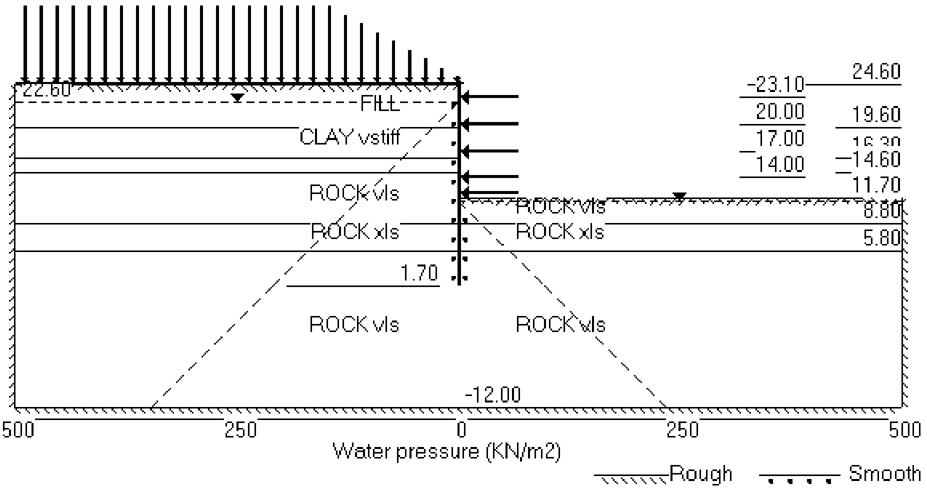
Stage No.16   Install strut no.5 at elev. 12.20



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                                                       Units: KN,m
Stage No. 16   Install strut or anchor no.5 at elevation 12.20

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                                FoS for toe       Toe elev. for
                               elev. =    1.70     FoS = 2.000
                               ---------------    -------------
 Stage  --- G.L. ---   Strut   Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.   Elev.    of    equilib.   elev.  Penetr       of
                               Safety  at elev.          -ation     failure
  16   24.60   11.70           More than one strut. No FoS calc.


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

Node    Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
               KN/m2        m       rad.       KN/m    KN.m/m    KN/m
  1   24.60     50.24     0.012  -3.49E-03      0.0      -0.0
  2   23.85     58.45     0.014  -3.52E-03     40.8      15.2
  3   23.10     58.02     0.017  -3.65E-03     84.4      63.2     207.9
                58.02     0.017  -3.65E-03   -123.4      63.2
  4   22.60     57.89     0.019  -3.74E-03    -94.5       9.0
  5   22.50     54.90     0.019  -3.74E-03    -88.8      -0.2
  6   21.88     60.57     0.022  -3.67E-03    -52.7     -44.2
  7   21.25     65.32     0.024  -3.52E-03    -13.4     -64.1
  8   20.63     72.86     0.026  -3.34E-03     29.8     -58.8
  9   20.00     82.73     0.028  -3.22E-03     78.4     -25.1     289.1
                82.73     0.028  -3.22E-03   -210.6     -25.1
 10   19.60    100.30     0.029  -3.10E-03   -174.0    -102.6
               190.30     0.029  -3.10E-03   -174.0    -102.6
 11   19.50    167.54     0.030  -3.05E-03   -156.1    -119.0
 12   19.20    156.93     0.031  -2.86E-03   -107.5    -158.2
 13   18.10    100.08     0.033  -2.05E-03     33.9    -164.4
 14   17.00    101.30     0.035  -1.48E-03    144.6     -59.6     294.3
               101.30     0.035  -1.48E-03   -149.6     -59.6
 15   16.50    108.92     0.036  -1.27E-03    -97.1    -121.4
 16   16.30    129.13     0.036  -1.15E-03    -73.3    -138.6
                97.68     0.036  -1.15E-03    -73.3    -138.6
 17   16.20     99.46     0.036  -1.08E-03    -63.4    -145.5
 18   15.40    119.85     0.037  -5.27E-04     24.3    -160.3
 19   14.60    141.10     0.037  -5.26E-05    128.7     -97.5
               196.61     0.037  -5.26E-05    128.7     -97.5
 20   14.00    193.58     0.037   5.85E-05    245.7      16.9     400.1
               193.58     0.037   5.85E-05   -154.4      16.9
 21   13.50    189.65     0.037   8.05E-05    -58.6     -36.1
 22   13.20    174.43     0.037   1.36E-04     -4.0     -45.5
 23   12.20    196.64     0.037   1.37E-04    181.5      45.4     454.7
               196.64     0.037   1.37E-04   -273.1      45.4
```

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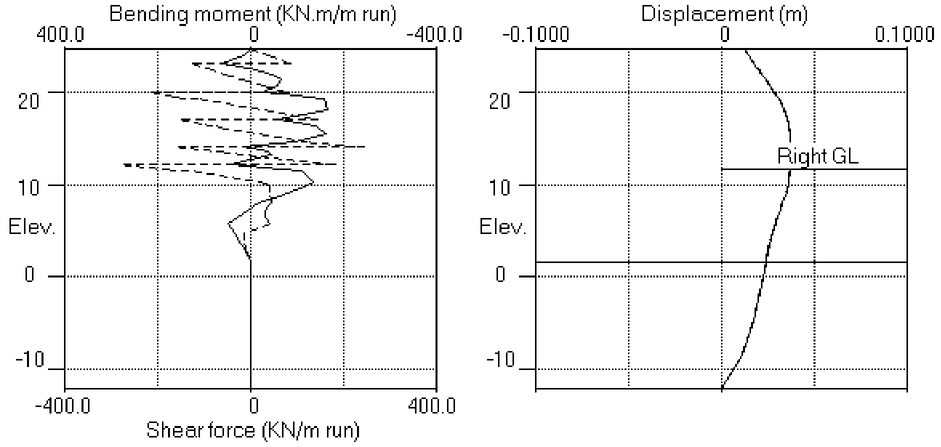
                                                              (continued)
Stage No.16  Install strut or anchor no.5 at elevation 12.20

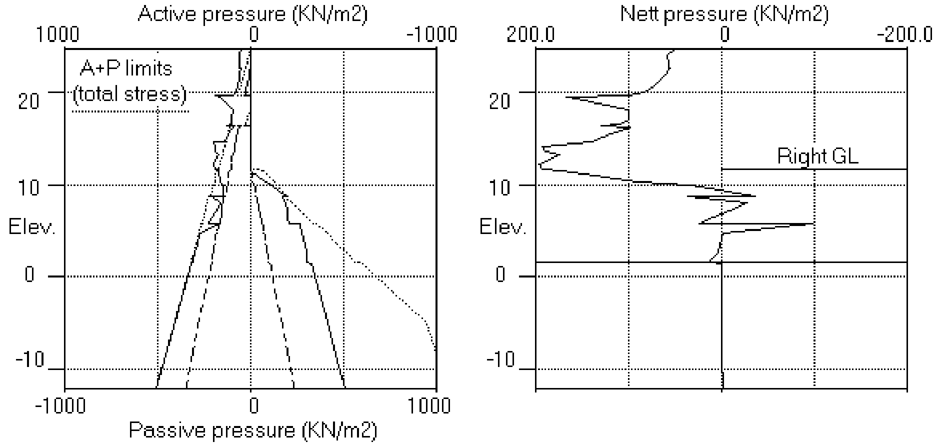
Node     Y       Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure      disp.   rotation     force   moment    forces
                KN/m2         m       rad.       KN/m    KN.m/m    KN/m
 24   11.70    193.78      0.037  1.59E-04   -175.5     -64.7
 25   11.40    172.76      0.037  2.78E-04   -120.5    -108.6
 26   10.30     89.84      0.036  8.94E-04     23.9    -134.8
 27   10.00     32.46      0.036  1.07E-03     42.2    -123.6
 28    8.80    -36.13      0.034  1.58E-03     40.0     -61.0
                36.16      0.034  1.58E-03     40.0     -61.0
 29    8.00    -27.37      0.033  1.72E-03     43.5     -17.3
 30    7.20     -7.79      0.031  1.74E-03     29.5       8.9
 31    6.50      8.32      0.030  1.68E-03     29.7      27.6
 32    5.80     24.45      0.029  1.55E-03     41.1      50.5
               -98.67      0.029  1.55E-03     41.1      50.5
 33    4.70     -1.50      0.027  1.33E-03    -14.0      36.2
 34    3.60      1.99      0.026  1.19E-03    -13.7      20.0
 35    2.65      4.51      0.025  1.13E-03    -10.6       8.0
 36    1.70     13.55      0.024  1.11E-03     -2.0       0.0
 37    1.40     -0.22      0.024         0     -0.0       0.0
 38   -1.70      0.13      0.020         0     -0.2       0.0
 39   -4.80      0.16      0.017         0      0.3       0.0
 40   -8.40      0.14      0.011         0      0.8       0.0
 41  -12.00     -0.59      0.000         0      0.0       0.0
 At elev.  23.10 Strut force =    415.7 KN/strut =    207.9 KN/m run (horiz.)
                                                 =    240.0 KN/m run (inclined)
 At elev.  20.00 Strut force =    578.1 KN/strut =    289.1 KN/m run (horiz.)
                                                 =    333.8 KN/m run (inclined)
 At elev.  17.00 Strut force =    588.6 KN/strut =    294.3 KN/m run (horiz.)
                                                 =    339.8 KN/m run (inclined)
 At elev.  14.00 Strut force =    800.3 KN/strut =    400.1 KN/m run (horiz.)
                                                 =    462.0 KN/m run (inclined)
```

Units: KN,m

Stage No.16 Install strut no.5 at elev. 12.20

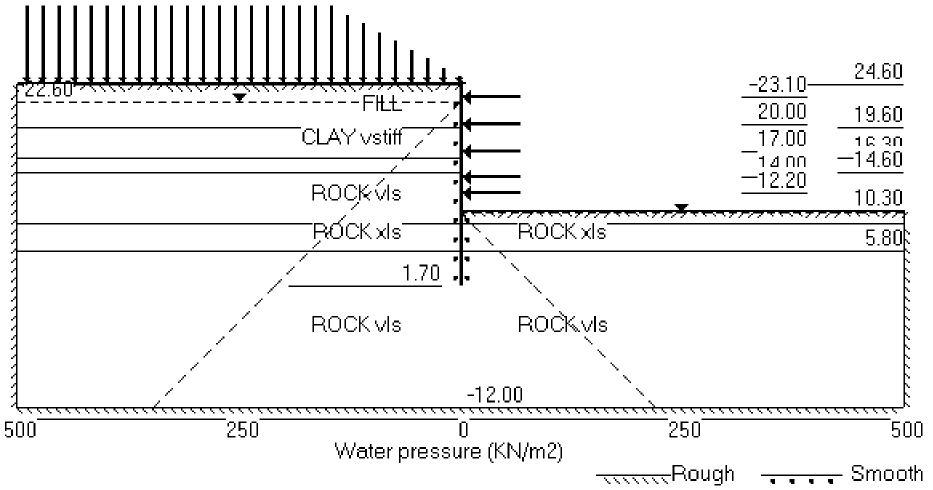


Stage No.16 Install strut no.5 at elev. 12.20



Units: KN,m

Stage No.18   Excav. to elev. 10.30 on RIGHT side



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                                                       Units: KN,m
Stage No. 18   Excavate to elevation 10.30 on RIGHT side

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                                FoS for toe       Toe elev. for
                               elev. =    1.70     FoS = 2.000
                               ---------------    -------------
 Stage  --- G.L. ---   Strut   Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.   Elev.    of    equilib.   elev.  Penetr       of
                               Safety  at elev.          -ation     failure
  18   24.60   10.30           More than one strut. No FoS calc.


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

Node     Y       Nett       Wall      Wall      Shear   Bending   Strut
 no.   coord   pressure     disp.   rotation    force   moment    forces
                 KN/m2        m       rad.       KN/m    KN.m/m    KN/m
  1    24.60      50.68     0.012  -3.58E-03      0.0      -0.0
  2    23.85      58.92     0.015  -3.61E-03     41.1      15.3
  3    23.10      58.49     0.018  -3.75E-03     85.1      63.7    208.8
                  58.49     0.018  -3.75E-03   -123.7      63.7
  4    22.60      58.35     0.020  -3.83E-03    -94.5       9.4
  5    22.50      55.36     0.020  -3.83E-03    -88.8       0.3
  6    21.88      61.01     0.023  -3.77E-03    -52.4     -43.7
  7    21.25      65.73     0.025  -3.62E-03    -12.8     -63.2
  8    20.63      73.25     0.027  -3.44E-03     30.6     -57.5
  9    20.00      83.10     0.029  -3.33E-03     79.5     -23.2    291.0
                  83.10     0.029  -3.33E-03   -211.6     -23.2
 10    19.60     100.43     0.030  -3.21E-03   -174.9    -101.1
                 190.96     0.030  -3.21E-03   -174.9    -101.1
 11    19.50     168.88     0.031  -3.16E-03   -156.9    -117.6
 12    19.20     159.28     0.032  -2.97E-03   -107.7    -156.9
 13    18.10     102.29     0.035  -2.17E-03     36.2    -162.0
 14    17.00     103.69     0.037  -1.62E-03    149.5     -53.3    297.5
                 103.69     0.037  -1.62E-03   -148.1     -53.3
 15    16.50     111.72     0.037  -1.43E-03    -94.2    -114.0
 16    16.30     131.42     0.038  -1.32E-03    -69.9    -130.5
                 101.11     0.038  -1.32E-03    -69.9    -130.5
 17    16.20     102.00     0.038  -1.25E-03    -59.7    -137.1
 18    15.40     122.40     0.039  -7.33E-04     30.0    -148.1
 19    14.60     143.34     0.039  -3.14E-04    136.3     -79.9
                 200.34     0.039  -3.14E-04    136.3     -79.9
 20    14.00     195.91     0.039  -2.58E-04    255.2      39.8    408.8
                 195.91     0.039  -2.58E-04   -153.6      39.8
 21    13.50     189.84     0.039  -2.90E-04    -57.2     -12.5
 22    13.20     170.06     0.039  -2.66E-04     -3.2     -21.5
 23    12.20     186.27     0.040  -3.73E-04    175.0      68.0    472.9
                 186.27     0.040  -3.73E-04   -297.9      68.0
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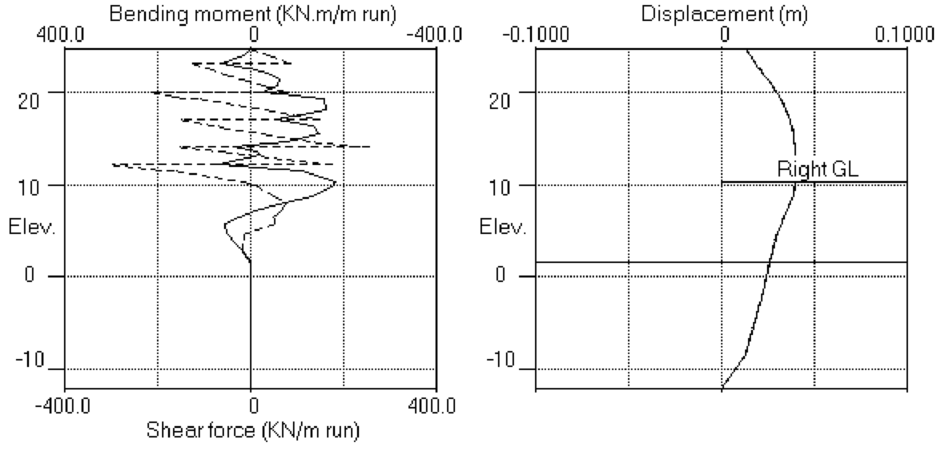
                                                                (continued)
Stage No.18  Excavate to elevation 10.30 on RIGHT side

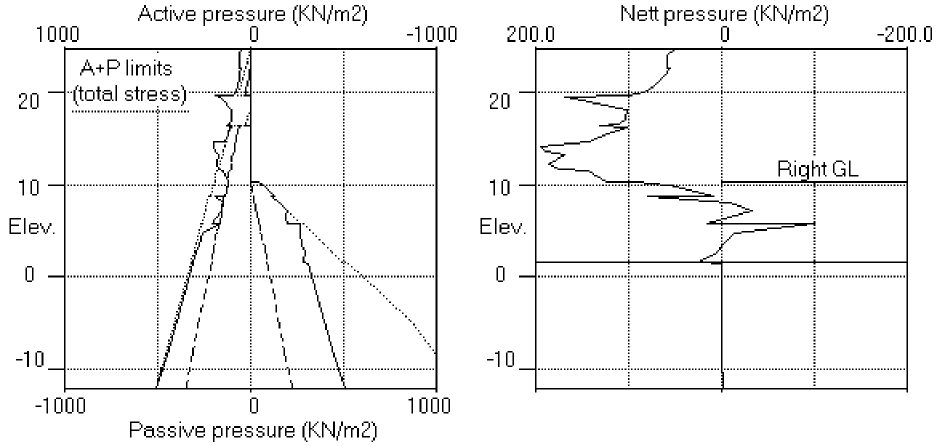
Node     Y       Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.  rotation     force   moment    forces
                KN/m2        m      rad.       KN/m   KN.m/m    KN/m
 24   11.70    175.56     0.040 -3.87E-04    -207.5     -55.9
 25   11.40    143.98     0.040 -2.73E-04    -159.5    -110.1
 26   10.30    125.36     0.040  4.65E-04     -11.4    -181.6
                92.92     0.040  4.65E-04     -11.4    -181.6
 27   10.00     57.41     0.040  7.15E-04      11.1    -180.7
 28    8.80      8.24     0.038  1.58E-03      50.5    -134.3
                80.71     0.038  1.58E-03      50.5    -134.3
 29    8.00     -7.81     0.037  1.96E-03      79.7     -70.8
 30    7.20    -32.94     0.035  2.11E-03      63.4     -11.0
 31    6.50     -8.67     0.034  2.08E-03      48.8      25.4
 32    5.80     16.36     0.032  1.95E-03      51.5      57.5
              -100.17     0.032  1.95E-03      51.5      57.5
 33    4.70    -13.70     0.030  1.67E-03     -11.1      53.7
 34    3.60     -1.50     0.029  1.45E-03     -19.5      33.4
 35    2.65      5.38     0.027  1.34E-03     -17.6      14.3
 36    1.70     24.12     0.026  1.31E-03      -3.6       0.0
 37    1.40     -0.24     0.026          0      -0.0       0.0
 38   -1.70      0.14     0.022          0      -0.2       0.0
 39   -4.80      0.16     0.018          0       0.3       0.0
 40   -8.40      0.14     0.012          0       0.8       0.0
 41  -12.00     -0.60     0.000          0       0.0       0.0
 At elev.  23.10 Strut force =    417.7 KN/strut =    208.8 KN/m run (horiz.)
                                                 =    241.1 KN/m run (inclined)
 At elev.  20.00 Strut force =    582.1 KN/strut =    291.0 KN/m run (horiz.)
                                                 =    336.1 KN/m run (inclined)
 At elev.  17.00 Strut force =    595.1 KN/strut =    297.5 KN/m run (horiz.)
                                                 =    343.6 KN/m run (inclined)
 At elev.  14.00 Strut force =    817.6 KN/strut =    408.8 KN/m run (horiz.)
                                                 =    472.1 KN/m run (inclined)
 At elev.  12.20 Strut force =    945.8 KN/strut =    472.9 KN/m run (horiz.)
                                                 =    546.0 KN/m run (inclined)
```

Units: KN,m

Stage No.18 Excav. to elev. 10.30 on RIGHT side

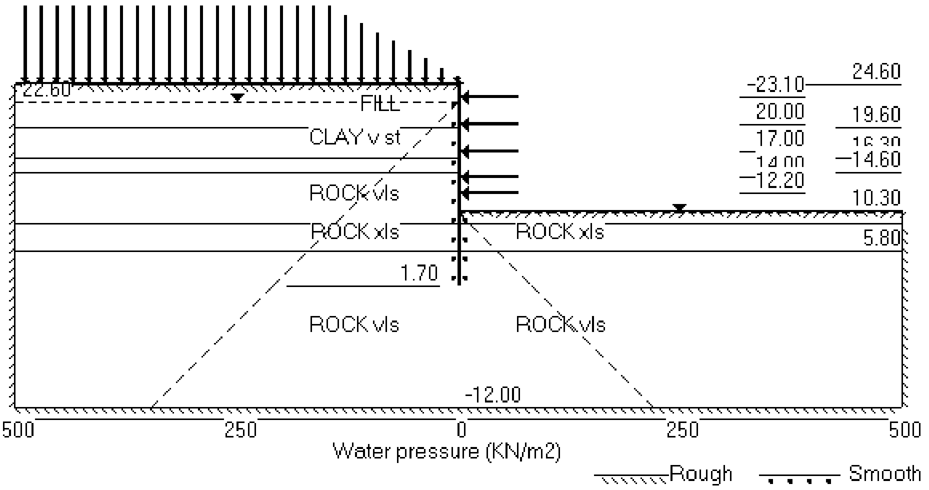


Stage No.18 Excav. to elev. 10.30 on RIGHT side



```
                                                                Units: KN,m
```

Stage No.19   Change soil type 5 to soil type 11



```
                                                       Units: KN,m
Stage No. 19   Change properties of soil type 5 to soil type 11
              Ko pressures will be reset

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                                FoS for toe       Toe elev. for
                               elev. =    1.70     FoS = 2.000
                               ---------------    -------------
 Stage  --- G.L. ---   Strut   Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.    Elev.    of    equilib.  elev.  Penetr      of
                               Safety  at elev.          -ation     failure
  19   24.60   10.30           More than one strut. No FoS calc.


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary

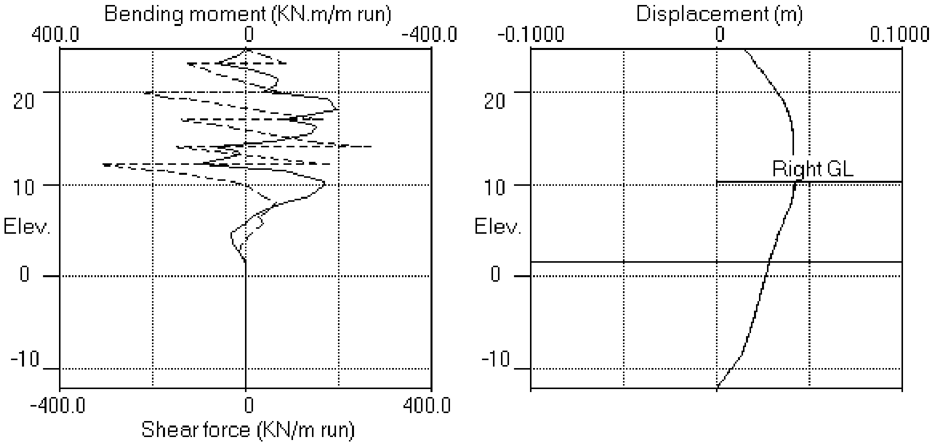
Node    Y      Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure     disp.   rotation    force   moment    forces
                KN/m2       m        rad.      KN/m    KN.m/m    KN/m
  1   24.60     51.44     0.015  -3.79E-03      0.0      -0.0
  2   23.85     59.27     0.018  -3.82E-03     41.5      15.5
  3   23.10     58.46     0.020  -3.96E-03     85.7      64.3    211.9
                58.46     0.020  -3.96E-03   -126.2      64.3
  4   22.60     58.09     0.022  -4.04E-03    -97.1       8.8
  5   22.50     54.90     0.023  -4.04E-03    -91.5      -0.6
  6   21.88     60.35     0.025  -3.97E-03    -55.4     -46.3
  7   21.25     64.83     0.028  -3.81E-03    -16.3     -67.9
  8   20.63     72.11     0.030  -3.62E-03     26.5     -64.6
  9   20.00     81.78     0.032  -3.48E-03     74.6     -33.1    296.8
                81.78     0.032  -3.48E-03   -222.2     -33.1
 10   19.60     99.68     0.034  -3.34E-03   -185.9    -115.4
               187.23     0.034  -3.34E-03   -185.9    -115.4
 11   19.50    162.58     0.034  -3.29E-03   -168.4    -133.0
 12   19.20    150.67     0.035  -3.07E-03   -121.5    -176.0
 13   18.10    116.96     0.038  -2.13E-03     25.7    -196.3
 14   17.00    137.49     0.040  -1.42E-03    165.7     -86.4    304.7
               137.49     0.040  -1.42E-03   -139.0     -86.4
 15   16.50    147.64     0.040  -1.16E-03    -67.7    -138.3
 16   16.30    151.45     0.041  -1.02E-03    -37.8    -148.9
                85.57     0.041  -1.02E-03    -37.8    -148.9
 17   16.20     89.66     0.041  -9.60E-04    -29.1    -152.3
 18   15.40    113.63     0.041  -4.17E-04     52.3    -142.7
 19   14.60    138.22     0.041  -4.54E-05    153.0     -59.5
               191.82     0.041  -4.54E-05    153.0     -59.5
 20   14.00    192.81     0.041  -5.80E-05    268.4      68.6    418.7
               192.81     0.041  -5.80E-05   -150.3      68.6
 21   13.50    189.74     0.042  -1.57E-04    -54.6      17.6
 22   13.20    171.59     0.042  -1.75E-04     -0.4       9.4
 23   12.20    188.05     0.042  -4.33E-04    179.4     102.4    487.2
               188.05     0.042  -4.33E-04   -307.8     102.4
```

```
                                                               (continued)
Stage No.19  Change properties of soil type 5 to soil type 11
              Ko pressures will be reset

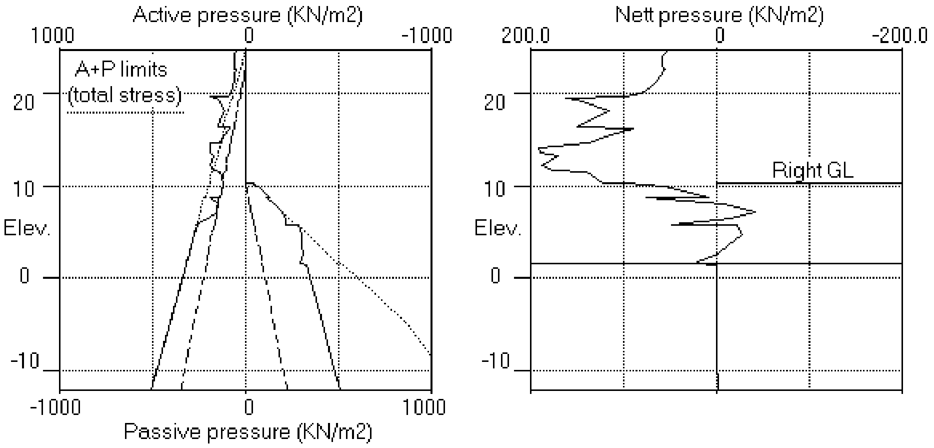
Node     Y       Nett       Wall      Wall      Shear   Bending   Strut
 no.  coord  pressure      disp.   rotation     force   moment    forces
              KN/m2          m       rad.       KN/m    KN.m/m    KN/m
 24   11.70    176.04     0.042  -5.20E-04   -216.8     -26.1
 25   11.40    140.67     0.042  -4.45E-04   -169.3     -83.2
 26   10.30    123.00     0.042   1.88E-04    -24.3    -167.4
                98.83     0.042   1.88E-04    -24.3    -167.4
 27   10.00     57.41     0.042   4.21E-04     -0.9    -170.1
 28    8.80      8.24     0.041   1.27E-03     38.5    -138.1
                76.56     0.041   1.27E-03     38.5    -138.1
 29    8.00     -7.81     0.040   1.68E-03     66.0     -85.6
 30    7.20    -40.42     0.039   1.90E-03     46.7     -36.7
 31    6.50    -15.92     0.037   1.99E-03     27.0     -13.8
 32    5.80     48.96     0.036   2.01E-03     38.6       1.2
               -21.37     0.036   2.01E-03     38.6       1.2
 33    4.70    -26.58     0.034   1.92E-03     12.2      30.9
 34    3.60    -12.05     0.032   1.77E-03     -9.0      28.4
 35    2.65     -0.55     0.030   1.68E-03    -15.0      14.5
 36    1.70     24.46     0.028   1.65E-03     -3.7      -0.0
 37    1.40     -0.24     0.028          0     -0.0       0.0
 38   -1.70      0.14     0.024          0     -0.2       0.0
 39   -4.80      0.16     0.020          0      0.3       0.0
 40   -8.40      0.14     0.013          0      0.8       0.0
 41  -12.00     -0.60     0.000          0      0.0       0.0
 At elev.  23.10 Strut force =    423.8 KN/strut =    211.9 KN/m run (horiz.)
                                                 =    244.7 KN/m run (inclined)
 At elev.  20.00 Strut force =    593.6 KN/strut =    296.8 KN/m run (horiz.)
                                                 =    342.7 KN/m run (inclined)
 At elev.  17.00 Strut force =    609.4 KN/strut =    304.7 KN/m run (horiz.)
                                                 =    351.8 KN/m run (inclined)
 At elev.  14.00 Strut force =    837.3 KN/strut =    418.7 KN/m run (horiz.)
                                                 =    483.4 KN/m run (inclined)
 At elev.  12.20 Strut force =    974.5 KN/strut =    487.2 KN/m run (horiz.)
                                                 =    562.6 KN/m run (inclined)
```

Units: KN,m

Stage No.19 Change soil type 5 to soil type 11



Stage No.19 Change soil type 5 to soil type 11



```
                                                       Units: KN,m
Summary of results

STABILITY ANALYSIS of Soldier Pile Wall according to CP2 method
 Factor of safety on gross pressure (excluding water pressure)

                                FoS for toe       Toe elev. for
                               elev. =    1.70     FoS = 2.000
                               ---------------    -------------
 Stage  --- G.L. ---   Strut   Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.    Elev.    of    equilib.  elev.  Penetr       of
                               Safety  at elev.          -ation     failure
   1   24.60   24.60    Cant.   9.787     2.36     ***     ***      L to R
   2   24.60   24.60   ---    Conditions not suitable for FoS calc.
   3   24.60   22.60    Cant.   6.365     2.39    16.65    5.95     L to R
   4   24.60   22.60    23.10   7.245     n/a     19.53    3.07     L to R
   5   24.60   22.60           No analysis at this stage
   6   24.60   19.50    23.10   4.341     n/a     18.71    0.79     L to R
   7   24.60   19.50           More than one strut. No FoS calc.
   All remaining stages have more than one strut - FoS calculation n/a

   Legend: *** Result not found
```

```
                                                       Units: KN,m
```

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall

Analysis options

```
  Soldier Pile width = 0.75m;  spacing = 2.00m
  Passive mobilisation factor = 2.500
  Length of wall perpendicular to section = 130.00m
  2-D finite element model.    Soil arching modelled.
  Soil deformations are elastic until the active or passive limit is reached

  Rigid boundaries:     Left side 50.00 from wall        Rough boundary
                       Right side 50.00 from wall        Rough boundary
  Lower rigid boundary at elevation -12.00               Rough boundary
```

Bending moment, shear force and displacement envelopes

```
Node     Y       Displacement          Bending moment       Shear force
 no.   coord   maximum   minimum     maximum   minimum    maximum   minimum
                  m         m         KN.m/m    KN.m/m      KN/m      KN/m
  1    24.60    0.037     0.000         0.0      -0.0        0.0       0.0
  2    23.85    0.034     0.000        15.8      -0.6       42.2      -2.3
  3    23.10    0.031     0.000        65.3      -3.1       86.9    -146.2
  4    22.60    0.029     0.000        21.6     -18.4       27.9    -124.7
  5    22.50    0.029     0.000        24.5     -30.6       29.3    -120.5
  6    21.88    0.026     0.000        45.3     -97.3       32.7     -93.1
  7    21.25    0.028     0.000        65.6    -145.0       32.0     -62.2
  8    20.63    0.030     0.000        85.5    -172.8       33.1     -27.0
  9    20.00    0.032     0.000       107.1    -177.0       81.8    -222.2
 10    19.60    0.034     0.000       122.9    -165.6       44.9    -185.9
 11    19.50    0.034     0.000       126.2    -160.6       54.3    -168.4
 12    19.20    0.035     0.000       129.6    -200.1       54.7    -128.4
 13    18.10    0.038     0.000        78.6    -253.0       40.8     -40.9
 14    17.00    0.040     0.000        42.1    -231.3      165.7    -182.2
 15    16.50    0.040     0.000        45.7    -188.3      112.1    -144.7
 16    16.30    0.041     0.000        51.1    -168.2      112.7    -126.4
 17    16.20    0.041     0.000        54.1    -180.6      114.6    -120.1
 18    15.40    0.041     0.000        66.3    -252.1      113.5     -65.7
 19    14.60    0.042     0.000        69.0    -276.6      153.0      -4.1
 20    14.00    0.042     0.000        72.4    -259.6      268.4    -223.3
 21    13.50    0.043     0.000        86.0    -223.3       95.3    -154.5
 22    13.20    0.043     0.000        88.9    -193.1      106.1    -121.3
 23    12.20    0.043     0.000       102.4    -202.7      181.5    -307.8
 24    11.70    0.042     0.000        71.9    -199.0      107.9    -216.8
 25    11.40    0.042     0.000        66.8    -186.2       99.4    -169.3
 26    10.30    0.042     0.000        97.6    -181.6       73.3     -24.3
 27    10.00    0.042     0.000       106.8    -180.7       72.0     -22.4
 28     8.80    0.041     0.000        74.9    -138.1       50.5     -65.9
 29     8.00    0.040     0.000        37.3     -85.6       79.7     -36.3
 30     7.20    0.039     0.000        39.8     -42.1       63.4     -13.9
 31     6.50    0.037     0.000        52.5     -36.3       48.8       0.0
 32     5.80    0.036     0.000        70.6     -16.2       51.5       0.0
 33     4.70    0.034     0.000        53.7       0.0       12.2     -20.0
 34     3.60    0.032     0.000        33.4       0.0        0.0     -19.5
 35     2.65    0.030     0.000        14.5       0.0        0.0     -17.6
 36     1.70    0.028     0.000         0.0      -0.0        0.0      -3.7
 37     1.40    0.028     0.000         0.0       0.0        0.1      -0.1
 38    -1.70    0.024     0.000         0.0       0.0        0.8      -0.5
 39    -4.80    0.020     0.000         0.0       0.0        0.9      -0.2
 40    -8.40    0.013     0.000         0.0       0.0        0.9       0.0
 41   -12.00    0.000     0.000         0.0       0.0        0.0       0.0
```

```
Summary of results   (continued)

Maximum and minimum bending moment and shear force at each stage
Stage  --------- Bending moment --------   ---------- Shear force ----------
 no.   maximum  elev.   minimum   elev.   maximum  elev.   minimum   elev.
        KN.m/m            KN.m/m            KN/m             KN/m
  1       27.7  14.00      -8.5   21.25      23.0  16.30     -10.2    8.80
  2       30.5  14.00     -12.5    7.20      25.6  16.30     -15.6    8.80
  3      129.6  19.20     -16.5    7.20      38.6  19.60     -40.9   18.10
  4       53.9  23.10    -127.5   19.60      70.3  23.10    -124.6   23.10
  5    No calculation at this stage
  6       72.4  14.00    -177.0   20.00      73.6  16.30    -146.2   23.10
  7       58.8  23.10    -154.9   18.10      76.6  23.10    -183.7   20.00
  8    No calculation at this stage
  9       88.9  13.20    -253.0   18.10     114.6  16.20    -218.2   20.00
 10       59.7  23.10    -181.4   18.10     164.9  17.00    -198.5   20.00
 11    No calculation at this stage
 12      106.8  10.00    -276.6   14.60     119.5  17.00    -212.5   20.00
 13       63.9  10.00    -183.9   15.40     208.0  14.00    -209.2   20.00
 14    No calculation at this stage
 15       70.6   5.80    -202.7   12.20     199.6  14.00    -223.3   14.00
 16       63.2  23.10    -164.4   18.10     245.7  14.00    -273.1   12.20
 17    No calculation at this stage
 18       68.0  12.20    -181.6   10.30     255.2  14.00    -297.9   12.20
 19      102.4  12.20    -196.3   18.10     268.4  14.00    -307.8   12.20

Maximum and minimum displacement at each stage
Stage -------- Displacement ---------   Stage description
 no.  maximum  elev.   minimum  elev.   -----------------
         m               m
  1    0.011   24.60    0.000   24.60   Apply surcharge no.1 at elev. 24.60
  2    0.012   24.60    0.000   24.60   Apply surcharge no.2 at elev. 24.60
  3    0.037   24.60    0.000   24.60   Excav. to elev. 22.60 on RIGHT side
  4    0.011   19.20    0.000   24.60   Install strut no.1 at elev. 23.10
  5    No calculation at this stage     Apply water pressure profile no.1
  6    0.022   20.00    0.000   24.60   Excav. to elev. 19.50 on RIGHT side
  7    0.014   17.00    0.000   24.60   Install strut no.2 at elev. 20.00
  8    No calculation at this stage     Apply water pressure profile no.2
  9    0.025   18.10    0.000   24.60   Excav. to elev. 16.50 on RIGHT side
 10    0.020   17.00    0.000   24.60   Install strut no.3 at elev. 17.00
 11    No calculation at this stage     Apply water pressure profile no.3
 12    0.041   14.60    0.000   24.60   Excav. to elev. 13.50 on RIGHT side
 13    0.035   15.40    0.000   24.60   Install strut no.4 at elev. 14.00
 14    No calculation at this stage     Apply water pressure profile no.4
 15    0.043   13.20    0.000   24.60   Excav. to elev. 11.70 on RIGHT side
 16    0.037   14.60    0.000   24.60   Install strut no.5 at elev. 12.20
 17    No calculation at this stage     Apply water pressure profile no.5
 18    0.040   11.40    0.000   24.60   Excav. to elev. 10.30 on RIGHT side
 19    0.042   10.30    0.000   24.60   Change soil type 5 to soil type 11
```

Summary of results   (continued)

Strut forces at each stage (horizontal components)

```
Stage   --- Strut no. 1 ---   --- Strut no. 2 ---   --- Strut no. 3 ---
 no.       at elev. 23.10        at elev. 20.00        at elev. 17.00
         KN/m run  KN/strut    KN/m run  KN/strut    KN/m run  KN/strut
  4      194.86    389.71        ---       ---         ---       ---
  6      209.32    418.65        ---       ---         ---       ---
  7      194.17    388.33      259.81    519.62        ---       ---
  9      204.43    408.86      278.94    557.88        ---       ---
 10      199.01    398.02      268.75    537.51      259.81    519.62
 12      209.35    418.71      292.92    585.85      301.67    603.34
 13      206.59    413.17      286.52    573.05      290.37    580.75
 15      209.81    419.62      293.53    587.06      302.41    604.82
 16      207.87    415.74      289.06    578.12      294.29    588.58
 18      208.83    417.65      291.05    582.09      297.55    595.09
 19      211.92    423.83      296.79    593.57      304.69    609.38

Stage   --- Strut no. 4 ---   --- Strut no. 5 ---
 no.       at elev. 14.00        at elev. 12.20
         KN/m run  KN/strut    KN/m run  KN/strut
 13      389.71    779.42        ---       ---
 15      422.84    845.67        ---       ---
 16      400.14    800.28      454.66    909.33
 18      408.82    817.65      472.89    945.77
 19      418.66    837.31      487.24    974.48
```

```
BUTLER PARTNERS PTY LTD                                      | Sheet No.
Program: WALLAP  Version 6.06  Revision A51.B69.R54          | Job No. 17-141C
                             Licensed from GEOSOLVE          | Made by :    PZ
Data filename/Run ID: 017-141A_Herston_Bore4_PZ              |
Herston Quarter Redevelopment - Northern Carpark             | Date:26-06-2018
Retaining Wall Stability                                     | Checked :
-----------------------------------------------------------------------------
                                                     Units: KN,m
```

Bending moment, shear force, displacement envelopes