REPORT

TO

THE OWNERS OF 29, 31 AND 33 PACIFIC STREET
AND 23A, 23B AND 25C OCEAN VIEW DRIVE,
WAMBERAL, NSW

ON

GEOTECHNICAL ASSESSMENT

OF

PROPOSED COASTAL PROTECTION WORKS

AT

29, 31 AND 33 PACIFIC STREET
AND 23A, 23B AND 25C OCEAN VIEW DRIVE,
WAMBERAL, NSW

3 March 2017
Ref: 30243ZRrpt

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1 INTRODUCTION

This report presents our geotechnical assessment of the proposed coastal protection works at 29, 31 and 33 Pacific Street and 23a, 23b and 25c Ocean View Drive Wamberal, NSW (the subject properties). The assessment was commissioned on behalf of The Owners of 29, 31 and 33 Pacific Street and 23a, 23b and 25c Ocean View Drive, Wamberal, by Eugene Marchese (the owner of 29 Pacific Street) in an email dated 22 February 2017. The commission was on the basis of our fee proposal (Ref. P44420ZR rev1) dated 22 February 2017.

In June 2016, an East Coast Low Storm caused erosion over the seaward portions of the subject properties. The affected property owners engaged Peter Horton (Horton Coastal Engineering Pty Ltd [HCEPL]) to provide advice on coastal protection works. HCEPL prepared a coastal engineering report which included a statement of environmental effects for construction of rock revetment works at the properties (Ref. lrJ0021-29-33 Pacific St & 23a,b & 25c Ocean View Dr Wamberal-v2.docx) dated 19 September 2016. This was part of a Development Application (DA) submitted to the NSW Coastal Panel.

Following deemed refusal of the DA, a Class 1 Application was filed in the Land and Environment Court (Horton Coastal Engineering ats NSW Coastal Panel). A Section 34 Conciliation Conference on this matter was attended by Peter Horton (HCEPL) and representatives of the NSW Coastal Panel on 1 February 2017. Following the conciliation conference, a list of required information was provided by the Respondent to enable an assessment of the proposed coastal protection works.

A meeting was held on 21 February 2017 at the offices of James Taylor & Associates (JTA) and was attended by Richard Yates (JTA), Peter Horton (HCEPL) and Paul Roberts (JK Geotechnics [JK]). At the meeting, a concept design for revised coastal protection works was discussed which included the following:

- A secant or sheet piled wall (possibly anchored) installed on the seaward boundary of the subject properties. The top of the pile wall capping beam to be at RL 2.5m AHD.
- A rock revetment placed landward of the crest of the piled wall formed with a seaward face sloping at 1 Vertical (V) in 1.5 Horizontal (H) and a crest at RL6m AHD.
- Where appropriate, re-profiling the seaward portions of the yard areas landward of the rock revetment at RL6m AHD, with localised removal of existing landscape walls over the footprint of the rock revetment, particularly at No. 29 and No. 31 Pacific Street.

Subsequent discussions were also held with Greg Britton (Haskoning Australia [HK]).

We note that we have completed a previous geotechnical investigation report for a proposed redevelopment of the residence at 23b Ocean View Drive, Wamberal (Ref. 25111SPrpt) dated 10 January 2012. The results of the investigation are presented in Appendix A.

The purpose of the assessment herein was to:

1. Complete a numerical analysis with regard to the stability of the alternative secant or sheet piled wall options, and the need for ground anchors;
2. Complete a stability analysis with regard to the 'global' stability of the proposed coastal protection works;
3. Provide the results of the analyses to JTA and HCEPL in order to assist in their selection of the most suitable piled wall; and
4. Based on the results of our analyses provide our comments and recommendations on the geotechnical aspects of the preferred proposed coastal protection works.

With regard to item 3, following our initial analysis results and a review of potential corrosion issues and costs, the following was agreed:

- Steel sheet piles were not considered appropriate due to long term corrosion concerns;
- A secant pile wall was appropriate from a corrosion perspective, but the costs associated with installing a secant pile wall were deemed prohibitive; and
- An alternative contiguous piled wall option was selected, using 0.6m diameter piles installed 'hard up' against each other to a similar depth as the secant piles. On the landward side of the contiguous piles a second row of smaller diameter piles would also be installed down to RL -4m AHD in order to plug any gaps between the contiguous piles.

The assessment therefore addresses the geotechnical aspects of the Section 34 Conciliation Conference List of Information Required.

We understand that HCEPL and JTA will provide comments on coastal and structural engineering issues, respectively, and that HK will provide an overall peer review.

2 ASSESSMENT PROCEDURE

2.1 Provided Information and Assumptions

To assist with our assessment we were provided with the following information:

- A site survey plan (Ref. 20265, dated 24 June 2016) prepared by Clarke Dowdle & Associates.
- A preliminary sketch of the proposed coastal protection works (Drawing No. S.01) prepared by JTA, with annotations by HCEPL indicating the final cross sectional profile of the seaward portion of the rear yard following construction of the coastal protection works.
- A drone image of the foreshore area following the June 2016 storms.

Between 21 and 27 February 2017, based on information presented in a number of emails prepared by HCEPL and JTA we note the following pertinent issues in relation to this assessment:

- The Highest Astronomical Tide (HAT) is about RL 1.0m AHD and the Lowest Astronomical Tide (LAT) is RL -1m AHD.
- However, HCEPL advised that it was more appropriate to design for the 100 year ARI still water level which is RL 1.5m AHD, or RL 2m AHD when adding 0.5m to account for sea level rise (SLR), and representing a groundwater level. HCEPL also assumed that wave setup effects were not significant in defining groundwater levels.
- The RL 2m AHD groundwater level outlined above was further increased to RL 3m AHD on the landward side of the capping beam (to conservatively account for sea level rise and a build-up of groundwater landward of the capping beam due to rainfall and infiltration of wave overtopping waters) and RL -0.5m on the seaward side to account for a 0.5m SLR above LAT. This RL 3m AHD level is considered unlikely to be reached. This was agreed between HCEPL, JTA, JK and HK to be the worst case groundwater condition which attempts to model 'rapid drawdown' at low tide with a build-up behind the capping beam in excess of HAT. Whilst we consider that the modelled groundwater levels present a reasonable 'worst case' scenario, the adopted tidal lag range is probably pessimistic. The high tidal groundwater level landward of the seawall will probably lower almost immediately as tidal levels recede and the landward tidal water drains laterally to the north and south beyond the coastal protection works. Even so, for the purposes of our analyses, use of these pessimistic worst case groundwater levels is considered valid.
- Precedence from similar coastal settings within, and to the north and south of Sydney, indicate that a landward groundwater level of RL3m is not typical and would only locally apply. A higher landward groundwater level would be influenced by the topographic setting and hydrogeological regime of the coastal area. In this regard, we note that the groundwater level recorded in the

borehole drilled at 23b Ocean View Drive was RL 1.15m. An increase of 1.85m to achieve the design landward elevated groundwater level is therefore considered reasonable.

- It is commonly assumed that wave overtopping waters infiltrate sandy soils landward of seawalls, dune scarps etc. However, in this instance the near surface soils landward of the proposed works have been assessed to comprise a mix of silty sand and clay fill and natural silty sands. On this basis, it was considered that the overtopping waters would preferentially drain along the sloping rear yard surfaces, through the higher permeability boulder revetment and over the proposed capping beam, rather than infiltrate the near surface soils. Any infiltrating overtopping waters would take some time to reach the groundwater level and would also drain laterally as well as vertically. Consequently, it was considered that such infiltrating overtopping water would have little impact on groundwater levels compared to the natural tidal fluctuations. However, an allowance for groundwater damming landward of the capping beam to RL 3m AHD (as described above) was allowed for. Any potentially detrimental impact of the overtopping water (irrespective of tidal level, indeed for analysis purposes the 3m AHD groundwater level was assumed to apply with an LAT tailwater) has been included as a surface surcharge equivalent to a 0.5m depth of overtopped water, which is conservative.
- A design beach scour level at RL -3m AHD has been adopted seaward of the proposed piled wall.
- For the comparison of piled wall options, JTA nominated a 0.6m diameter secant pile and two alternative steel sheet pile sections; AZ 17-700 and AZ 48.
- A design life of 60 years has been adopted.

3 NUMERICAL AND STABILITY ANALYSES

3.1 Geotechnical Model

We have not visited the site area since completion of our geotechnical report 10 January 2012. However, we have reviewed the provided survey plan and drone image and the contents of our geotechnical report.

The location of the cross section (Figure 2) selected for the analyses is indicated on the attached Figure 1. When compared to No. 33 Pacific Street and 23a, 23b and 25c Ocean View Drive, the houses and rear yard deck/pergola areas at No 29 and No. 31 Pacific Street were situated closer to the foreshore. With the exception of the rear yard landscape retaining walls, the set-back distances of the buildings and structures from the foreshore at No 29 and No. 31 Pacific Street were similar. However, the existing rear yard surface levels at No. 29 were slightly more elevated overall, and therefore the cross section shown on Figure 1 represents the worst case.

The subsurface profile adopted for the analyses was based on the results of our geotechnical investigation at 23b Ocean View Drive. The borehole logs and location plan from our previous investigation are presented in the attached Appendix A. The subsurface profile adopted for our geotechnical model and presented on the attached Figure 2 is as follows:

- Loose natural silty sand and sands/variable fill from the rear yard surface level to RL -2.15m AHD
- Medium dense natural sands from RL -2.15m AHD to RL- 5.65m AHD.
- Dense natural sands below RL -5.65m AHD.

3.2 Model Parameters

The analysis parameters adopted for the subsurface profile are tabulated below. The soil strength parameters were assessed from our previous geotechnical investigation, our past experience of similar material types and empirical correlations well established in geotechnical engineering.

TABLE OF MATERIAL PARAMETERS

Parameters	Natural Loose Sands and Fill	Medium Dense Sand	Dense Sand	Boulder Revetment
Unit Weight γ (kN/m^3)	16	18	20	18
Elastic Modulus E (MPa)	20	40	80	40
Poisson's Ratio v	0.3	0.3	0.3	0.3
Cohesion c (kPa)	0	0	0	0
Internal Angle of Friction ϕ (º)	30	33	36	40

An interface friction angle of 50% of the internal angle of friction was adopted for the soil to wall interface on the active side of the piled wall. On the passive side of the piled wall, soil wall friction was ignored.

The parameters adopted for the secant and contiguous pile wall and the two alternative steel sheet pile walls are tabulated below:

TABLE OF PILE WALL PARAMETERS

Piled Wall Type	Pile Diameter (m)	Moment of Inertia $(m^4)/m$	Elastic Modulus (MPa)
Sheet Pile wall (AZ 17-700)	N/A	3.623×10^{-4}	200,000
Sheet Pile wall (AZ 48)	N/A	1.156×10^{-3}	200,000
Secant and Contiguous Pile Wall	0.6	6.36×10^{-3}	20,000

The Moment of Inertia for the steel sheet piles was sourced from the product data sheets for the two alternative steel sheet pile sections prepared by J Steel Australasia Pty Ltd.

The surcharge loads adopted for the analyses are outlined below:

- Standing water over the yard surface due to wave overtopping; assumed to be a depth of 0.5m resulting in a 5kPa surcharge acting over the rear yard landward of the pile wall to the seaward margin of the house. This is not considered to be a realistic assumption as overtopping would be expected to immediately drain away. However, although conservative, the short term surcharge of the overtopped water was included.
- Above the capping beam level, in the WALLAP analysis, the rear yard profile was input as a series of surcharges based on the surface level and the bulk unit weight of the soil profile.
- A 100kPa surcharge for the house footing founded in the natural sands; this was not applied to the WALLAP analysis as the footing was located outside the zone of influence of the pile wall (defined by an angle projected up from the maximum scour level [RL -3m AHD] at 45^{o}).

3.3 Analysis Procedure

The computer program "WALLAP" was adopted for the analyses of the proposed pile wall and to assess the need for anchors. A pile toe embedment at RL -10m was initially adopted.

The stability of the proposed pile wall retention systems and the prediction of wall deflections was analysed by balancing disturbing forces and moments created by the 'active' earth pressures on the outside of the pile wall with restoring forces and moments from the sand profile on the seaward side of the pile wall below the nominated beach scour level. The analyses were initially run without ground anchors, but due to excessive deflections (in excess of 400mm) at the crest of the pile wall (secant and sheet pile), ground anchors were included in subsequent analyses.

Ground anchors were nominated to be installed at 3m lateral spacing, in 150mm diameter holes at RL2m, with a downward angle of 45^{o} below the horizontal.

The ‘global’ stability analysis was completed using the computer program “SLOPE/W” which applies circular slip surface analyses to the model. The analysis considered a worst case post beach erosion scenario (RL -3m AHD scour level) with elevated landward groundwater levels of 3m AHD and a 0.5m depth of water inundation landward of the revetment.

Slip circle analyses were run for the above scenario in order to determine the lowest Factor of Safety (FOS) for a theoretical global circular failure plane passing under the toe of the pile wall. Ground anchors were included, but had no impact on overall stability because the theoretical failure plane was landward of the embedded end of the anchor.

Based on past experience for similar analyses in coastal erosion settings, we adopted an ‘acceptable’ Factor of Safety (FOS) of 1.5 for ‘global’ stability.

Following completing the WALLAP analysis for an anchored pile wall embedded at RL -10m and running the SLOPE/W analysis, the results were presented to JTA and HCEPL for review. At this stage a sensitivity analysis was run in SLOPE/W to determine the minimum pile toe embedment for a FOS of 1.5. The WALLAP analyses were then re-run to optimise pile toe embedment depths.

3.4 Analysis Results

The results for WALLAP analyses for all three pile wall options are provided below and print outs from WALLAP are presented in the attached Appendix B.

Type of wall	Toe Level of Pile RL (m)	Maximum Bending Moment (kNm/m)/ acting at RL(m)	Maximum Shear Force (kN/m)/ acting at RL(m)	Maximum Deflections (mm)			Maximum Horizontal Load (kN)/anchor
				At Crest/ Crest RL (m)	Maximum /acting at RL (m)	At Pile Toe	First Row/ RL (m)
Sheet pile AZ 17-700	-10	228/ -1.48	114/ 2.0	39/ 2.5	51/ -0.8 & -1.48	3	360/ 2
Sheet pile AZ 17-700	-8	278/ -1.68	128/ 2.0	42/ 2.5	61/ -1.2 & -1.68	2	400/ 2
Sheet pile AZ 48	-10	225/ -2.15	116/ 2.0	39/ 2.5	39/ 2.5 to 1.0	3	363/ 2
Sheet pile AZ 48	-8	287/ -1.68	130/ 2.0	44/ 2.5	44/ 2.5 to 1.5	2	407/ 2
Secant or Contiguous pile 600mm	-10	229/ -1.48	114/ 2.0	39/ 2.5	42/ 0.5 to – 0.8	3	360/ 2
Secant or Contiguous pile 600mm	-8	283/ -1.68	129/ 2.0	43/ 2.5	49/ – 0.6	2	404/ 2

The results of the stability analyses are presented on the attached Figures 3 and 4, and have indicated the following:

- For a pile toe embedment at RL -10m the minimum FOS for a theoretical 'global' circular failure was 2; see Figure 3.
- For a pile toe embedment at RL -8m the minimum FOS for a theoretical 'global' circular failure was 1.7; see Figure 4.

3.5 Conclusions

Based on the results of our numerical analyses and stability analysis we note the following:

- One row of ground anchors will be required and installed at about RL 2m AHD, i.e. through the capping beam.
- A minimum pile toe embedment depth to RL -8m will be required to achieve a 'stable' pile wall retention system and achieve a minimum FOS of 1.5 for theoretical 'global' stability.

- A sheet pile wall, contiguous or secant pile wall retention system is suitable. However the selection of the appropriate sheet pile section will be dependent on an assessment of corrosion over the design life, bending moments and shear forces. We understand from JTA that steel sheet piles are not a suitable option when considering corrosion over the 60 year design life.

4 ADDITIONAL GEOTECHNICAL ADVICE

4.1 Proposed Construction Sequence and Methodology

Based on the proposed form of the coastal protection works, we recommend the following generalised construction sequence and methodology:

1. Preparation of a Construction Environmental Management Plan (CEMP) and Construction Methodology Plan (CMP).
2. Review and approval of the CEMP by the relevant statutory bodies and review and approval of the CMP by the project coastal, structural and geotechnical consultants.
3. Complete dilapidation surveys and detailed condition surveys of the neighbouring properties and, if required, dilapidation surveys of the subject properties.
4. Establishment of appropriate construction zone fencing/traffic control, etc to Council requirements.
5. Geotechnical consultant to complete a piling rig working platform design based on information supplied by the piling contractor.
6. Excavate along seaward property boundary to remove any obstructions (boulders etc).
7. Reinstate sand up to piling working platform level (approximately the underside of the capping beam) in accordance with geotechnical advice.
8. Form a sand bund seaward of works site.
9. Form the batter slope within the seaward portion of the subject properties to enable rock revetment construction. Excavated sand to be placed on the beach seaward of the works.
10. Install contiguous pile wall and landward ‘plug’ piles down to design toe levels.
11. Form and pour concrete capping beam.
12. Install and test permanent ground anchors.
13. Place second underlayer of revetment (0.4m thick) over the batter slope.
14. Place 2 layers of secondary armour rock (1m thick).
15. Place 2 layers of primary armour rock (2.3m thick).
16. Wash sand into the revetment to fill voids, fill interstitial voids at the primary armour crest with rip rap and place geotextile over the revetment crest as a foundation for the reinstated lawn areas.

17. Reinstate remaining seaward portion of rear yard areas within the subject properties, including establishing lawn areas landward of the rock revetment.
18. Replace sand mounded seaward of the pile wall back over the revetment as required. Where necessary, material would be screened to remove any inclusions.
19. Planting of the sand dune formed over the rock revetment with suitable dune vegetation.
20. Post construction dilapidation survey.

4.2 Site Preparation

4.2.1 General

We recommend that the contractor prepares a Construction Methodology Plan (CMP) prior to works commencing which should be completed with due regard to the geotechnical advice provided in this report and the contents of the Construction Environmental Management Plan (CEMP). The CMP must include, but not be limited to, proposed excavation and piling techniques, the proposed excavation and piling equipment, sequencing of the excavation, piling and anchoring works, placement of the rock revetment, required inspections by the geotechnical, structural and coastal engineers, hold points etc, if required. The geotechnical, structural and coastal engineers should review and approve the CMP.

4.2.2 Dilapidation Survey

Prior to works commencing, a detailed dilapidation report should be compiled on the neighbouring property to the south (No. 25 Pacific Street) including the rear yard structures (e.g. decks, landscape walls etc) and rock revetment. Detailed condition survey reports of the vegetated area to the north of the subject properties, and the sandy beach seaward of the subject properties (particularly recording the existence of any unsuitable material such as debris and strewn rock), should also be carried out. The neighbouring property owners should be asked to confirm that the reports present a fair record of existing conditions as the reports may assist in the resolution of any damage claims.

In addition, prior to works commencing, consideration should be given to preparing detailed dilapidation reports on the subject properties. The property owners should be asked to confirm that the reports present a fair record of existing conditions as the reports may assist the clients in pursuing any claims against the contractor for damage.

4.2.3 Excavation Conditions

Excavation recommendations provided below should be completed by reference to the Safe Work Australia Code of Practice ‘Excavation Work’, dated July 2015.

Bulk excavations to the underside of the capping beam level (RL2m) will extend to maximum depths between about 4m and 6m below the existing surface levels of the foreshore areas of the subject properties. The excavations are not expected to extend any lower than RL -1m AHD. The excavations will extend through the sandy soil profile and encounter gravel, cobble and boulder sized inclusions. The excavations are expected to be readily completed using tracked excavators but with over excavation to remove obstructions. Any topsoil or root affected soils should be stripped and separately stockpiled for re-use in landscape areas as such soils are not suitable for re-use as engineered fill.

Care will need to be exercised in order to maintain the stability of the adjacent section of neighbouring rock revetment to the south. This work will need to be completed using suitably experienced (and insured) contractors and supervised by a suitably qualified engineer.

4.2.4 Potential Ground Surface Movement Risks

Due to the presence of loose natural sands (including beach sands), which we expect will extend across the general area, we advise that sudden stop/start movements of tracked excavators and piling rigs should be avoided in order to reduce transmission of ground vibrations to the adjacent sections of buildings and structures within and neighbouring the site.

We also recommend that during the drilling of CFA piles the ground surface is monitored by the site supervisor to check for signs of sand drawdown, particularly when drilling through the dense sands to form the sockets. If drawdown is occurring then piling operations should immediately cease and further geotechnical advice sought.

4.2.5 Groundwater Seepage and Tidal Levels

Groundwater inflow is expected within the excavations within the sandy soil profile, due to tidal fluctuations. In general, we expect any inflows to be of small volume and managed by infiltration into the sandy subgrade. Inspection and monitoring of groundwater seepage during excavations is recommended, so that any unexpected conditions, which may be revealed, can be incorporated into the drainage design.

The Highest Astronomical Tide (HAT) is about RL 1.0m AHD and so excavations are not expected to extend below tidal water levels, although locally this may occur when removing boulders. However, there will be the potential for erosion of the piling rig working platform (see Section 4.4, below). The formation of a sand bund seaward of the works area will assist in controlling potential erosion of the working platform and consideration should be given to providing temporary erosion protection.

4.3 Temporary Batter Slopes and Retention

4.3.1 General

Temporary excavation batters no steeper than 1 Vertical (V) in 1.5 Horizontal (H) are considered feasible for the sandy soils above the groundwater levels. These temporary batter slopes are only expected to be accommodated over the landward and seaward sides of the proposed works and not over the northern and southern end of the proposed works.

The temporary batter slopes will extend over the northern and southern site boundaries. To maintain the stability of the northern end of the neighbouring rock revetment to the south (within No. 25 Pacific Street) particular care will be required. We note the following:

- The northern site boundary will be supported by the piled wall retention system that will be constructed as part of the coastal protection works and return landward along the northern boundary of 25C Ocean View Drive. This section of the piled wall will protect the property from potential future erosion of the neighbouring Department of Planning and Environment land which has no coastal protection works.
- Where the works abut the northern end of the neighbouring rock revetment to the south within No. 25 Pacific Street, excavations should be carefully completed in order to expose the basal profile of the rock revetment. The coastal engineer will need to inspect the exposed profile in order to assess the extent of any additional boulders to support the neighbouring revetment and provide a smooth transition between the two sections of coastal protection works. Such details will need to be confirmed by initially excavating test pits which should be inspected by the coastal engineer and possibly the geotechnical engineer.

We note that the bulk excavations over the landward side of the works will extend close to the encountered groundwater level. If groundwater is encountered this may affect the stability of the excavation sides. Some allowance should be made for use of sand bags to support temporary batters close to the groundwater levels.

4.3.2 Preferred Retention System

Based on the results of the analyses presented in Section 3 above and review by JTA, the preferred retention method for the coastal protection works (including the northern boundary of 25c Ocean View Drive) will comprise:

- An anchored contiguous piled wall embedded into dense sands at RL -8m AHD. Due to the collapsible nature of the sandy soil profile and the tidal groundwater levels, auger grout injected (CFA) piles would need to be used.
- To prevent soil loss through any gaps in the contiguous piles, a second row of smaller diameter CFA piles would be installed landward of the piled wall and installed to RL -4m AHD, i.e. a depth of 1m below the maximum adopted design scour level of RL -3m AHD.
- One row of ground anchors installed into at least medium dense sands at a lateral spacing of 3m.

With regard to the ground anchors along the northern boundary of 25c Ocean View Drive, consideration will need to be given to the level and inclination angle of the anchors in order that they do not intersect the ground anchors installed along the seaward boundary of the subject properties.

The capping beam will need to be provided with landward thickenings at 3m lateral spacing and provided with a PVC conduit (sloping at 45^o) in order to allow subsequent installation of the ground anchors. The capping beam thickenings will protect the ground anchors from damage due to placement or any movement of the rock revetment, and provide additional corrosion protection.

4.3.3 Retention Design Parameters

The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the static design of the anchored pile walls:

- For design of the permanent piled retention system, we recommend the use of a uniform rectangular earth pressure distribution of 4H kPa for the soil profile, where H is the retained height in metres from the design beach scour level (RL -3m AHD).
- Assuming ground surface movements can be tolerated, any new landscape retaining walls landward of the rock revetment (if required) should be designed using a triangular lateral earth pressure distribution and an ‘active’ earth pressure coefficient, K_a, of 0.35 for the soil profile, assuming a horizontal backfill surface.

- Any surcharge (including construction traffic, the rock revetment, compaction stresses, landscaping, inclined retained surfaces, footings etc) affecting the piled retention system and/or landscape walls should be allowed for in the design using the above earth pressure coefficient.
- The piled wall must be designed for hydrostatic pressures based on the adopted design groundwater level at RL 3m AHD) as discussed in Section 2, above.
- Toe restraint may be provided by the passive pressure of the soil below the design beach erosion scour level. A passive earth pressure coefficient, K_p, of 3 may be adopted, provided a Factor of Safety of 2 is used in order to reduce deflections.
- Bulk unit weights as outlined in Section 3.2, above should be adopted for the retained profile above the water level, and reduced by 9.8kN/m^3 below the groundwater level.
- Permanent ground anchors bonded into medium dense (or better) sands may be designed using an effective angle of internal friction, ϕ', of 33º. All anchors should be proof tested to 1.3 times the working load under the supervision of an experienced engineer independent of the anchor contractor. The anchors will need to be load tested in incremental stages with the test load held for at least 10 minutes and the anchors locked off at 85% of the working load. A lift off test should also be completed 24 hours later to confirm that the anchors are holding the working load. Any anchors that fail the load testing will need to be replaced. The permanent anchors must be designed with due regard for long term corrosion, i.e. double sheathed and fully grouted. We understand that the ground anchoring will be a ‘design and construct’ package and the design should be reviewed by the structural and geotechnical engineers.

4.4 Piling Rig Working Platform

The piling rig may need to be provided with a suitable working platform determined by a geotechnical engineer. The design of the working platform will need to be based on the specific loadings and track dimensions supplied by piling contractor for the proposed piling rig. Further, the assessment of the working platform thickness will need to be based on the methodology outlined in BR 470 *‘Working Platforms for Tracked Plant’ (2004, prepared by BRE)*. In addition, should any works be completed close to sloping surfaces then computer based stability analyses may also be required.

The working platform will need to be constructed using DGB20 (or a similar durable granular material approved by the geotechnical engineer) compacted to at least 95% Modified Maximum Dry Density (MMDD) using a large roller. The subgrade will need to be prepared as outlined in below.

Density tests should be regularly carried out on the working platform materials to confirm the above density has been achieved. The frequency of density testing should be at least one test per layer per 2500m^2 or three tests per visit, whichever requires the most tests. Level 2 testing of fill compaction is the minimum permissible in AS3798-2007. However, our preference would be for Level 1 control of fill placement and compaction, in accordance with AS3798-2007.

On completion of the piling works, the granular fill should be removed from the beach (with the assistance of screening, as required) and may be used as backfill to reinstate the rear yards.

4.5 Earthworks

The following earthworks recommendations should be complemented by reference to AS3798-2007 "Guidelines on Earthworks for Commercial and Residential Developments".

With regard to the proposed works, the following earthworks are envisaged:

- Formation of a piling rig working platform.
- Reinstating the rear yards of the subject properties landward of the rock revetment.

4.5.1 Subgrade Preparation

Prior to placement of fill to reinstate the rear yards and/or construction of the piling rig platform (if required), preparation of the soil subgrade should consist of the following:

- Following completion of bulk excavations the sandy subgrade over the areas of the piling rig working platform should be proof rolled with at least eight passes of a static (non-vibratory) smooth drum roller of at least 12 tonnes deadweight. Over landscape areas proof rolling may be completed using a vibrating plate compactor (attached to an excavator or hand held). The sandy subgrade should be thoroughly moistened prior to proof rolling.
- The final pass of proof rolling should be carried out under the direction of an experienced geotechnical engineer for the detection of unstable or soft areas which should be removed and replaced with engineered fill (as outlined in Section 4.5.2, below).
- Care should also be taken when using vibrating equipment not to cause damage to any adjacent structures. The vibrations should be qualitatively monitored by site personnel. If there is any cause for concern then proof rolling should cease and further advice sought. Alternatively, where appropriate, the static (non-vibration) mode may be used.

4.5.2 Engineered Fill

Fill required to reinstate rear yard areas and unstable areas of subgrade and to backfill any landscape walls should comprise engineered fill.

Engineered fill (including backfill to any landscape retaining walls) should be free from organic materials, other contaminants and deleterious substances and have a maximum particle size not exceeding 40mm. We expect the excavated sands may be used as engineered fill. Engineered fill should be placed in layers of maximum 100mm loose thickness and compacted with the above mentioned roller(s) to achieve a minimum density index (I_D) of 65% for the sandy soils. Care will be required to ensure excessive compaction stresses are not transferred to the retaining walls.

Density tests should be carried out at the frequencies outlined in AS3798. At least Level 2 testing of earthworks should be carried out in accordance with AS3798. Any areas of insufficient compaction will require reworking.

4.5.3 Wave Inundation Erosion Protection

Any potential inundation of the rear yard areas due to wave overtopping is expected to naturally infiltrate through the sandy subgrade materials and the rock revetment. There is the potential for some localised erosion of the landscape surface but this is expected to be satisfactorily controlled by establishing a rapid growing grass cover suitable for this marine environment, and reinstating divots, as required.

4.6 Further Geotechnical Input

The following summarises the scope of further geotechnical work recommended within this report. For specific details reference should be made to the relevant sections of this report.

- Review of contractors CMP.
- Dilapidation reports of buildings and structures within and neighbouring the subject properties and condition surveys of the neighbouring land to the north and the beach seaward of the works.
- Inspection of excavations exposing the neighbouring revetment to the south.
- Monitoring of groundwater seepage into bulk excavations.
- Proof rolling of exposed sub-grade.
- Qualitative vibration monitoring during use of vibratory compaction equipment.
- Density testing of engineered fill.
- Review of anchor design.

- Witnessing installation and testing of ground anchors.
- Piling rig working platform design.
- Witnessing the drilling of CFA piles to check for sand drawdown.

5 GENERAL COMMENTS

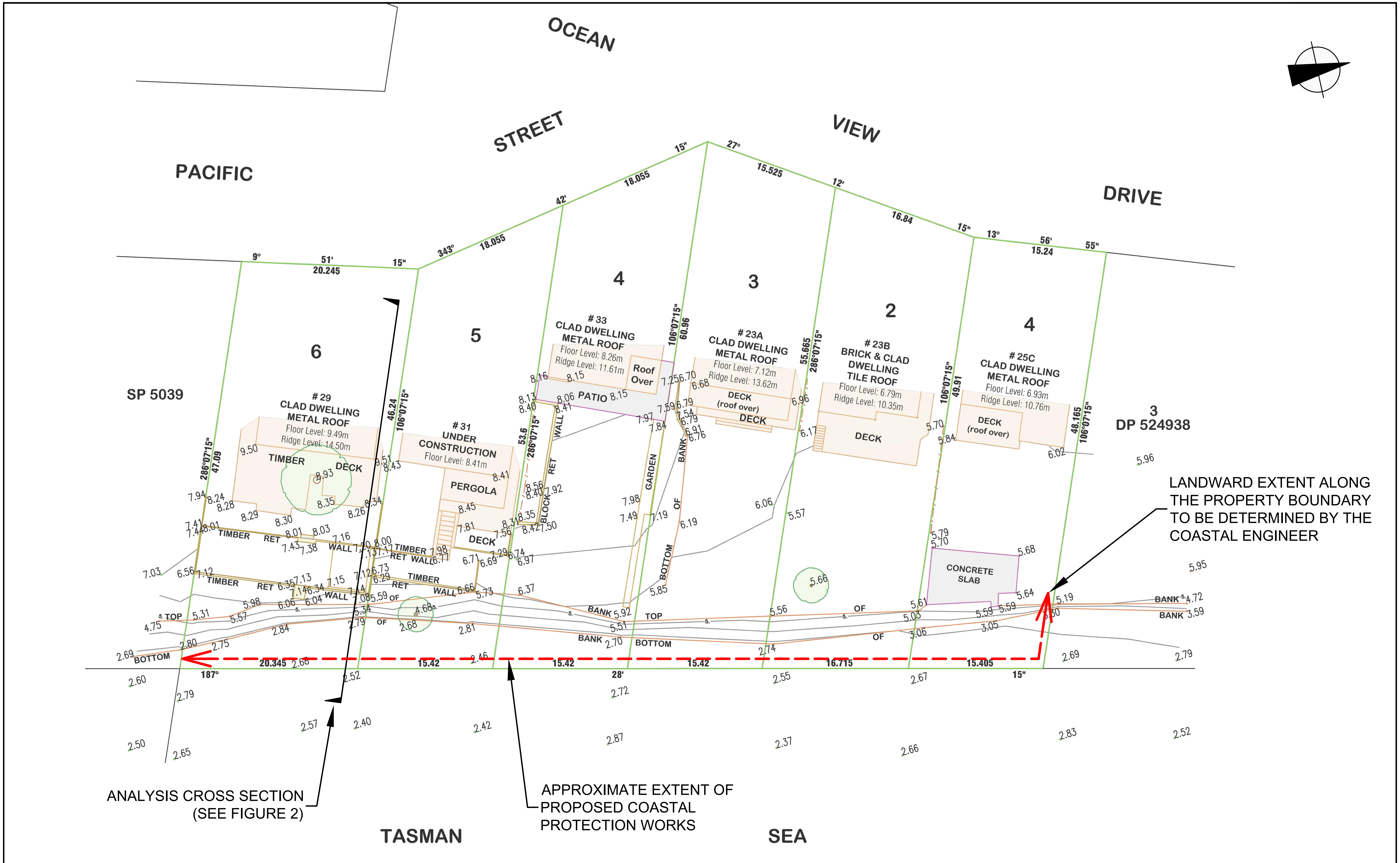
The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

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

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

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

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



0 4 8 12 16 20

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

Title: **SITE LOCATION PLAN**

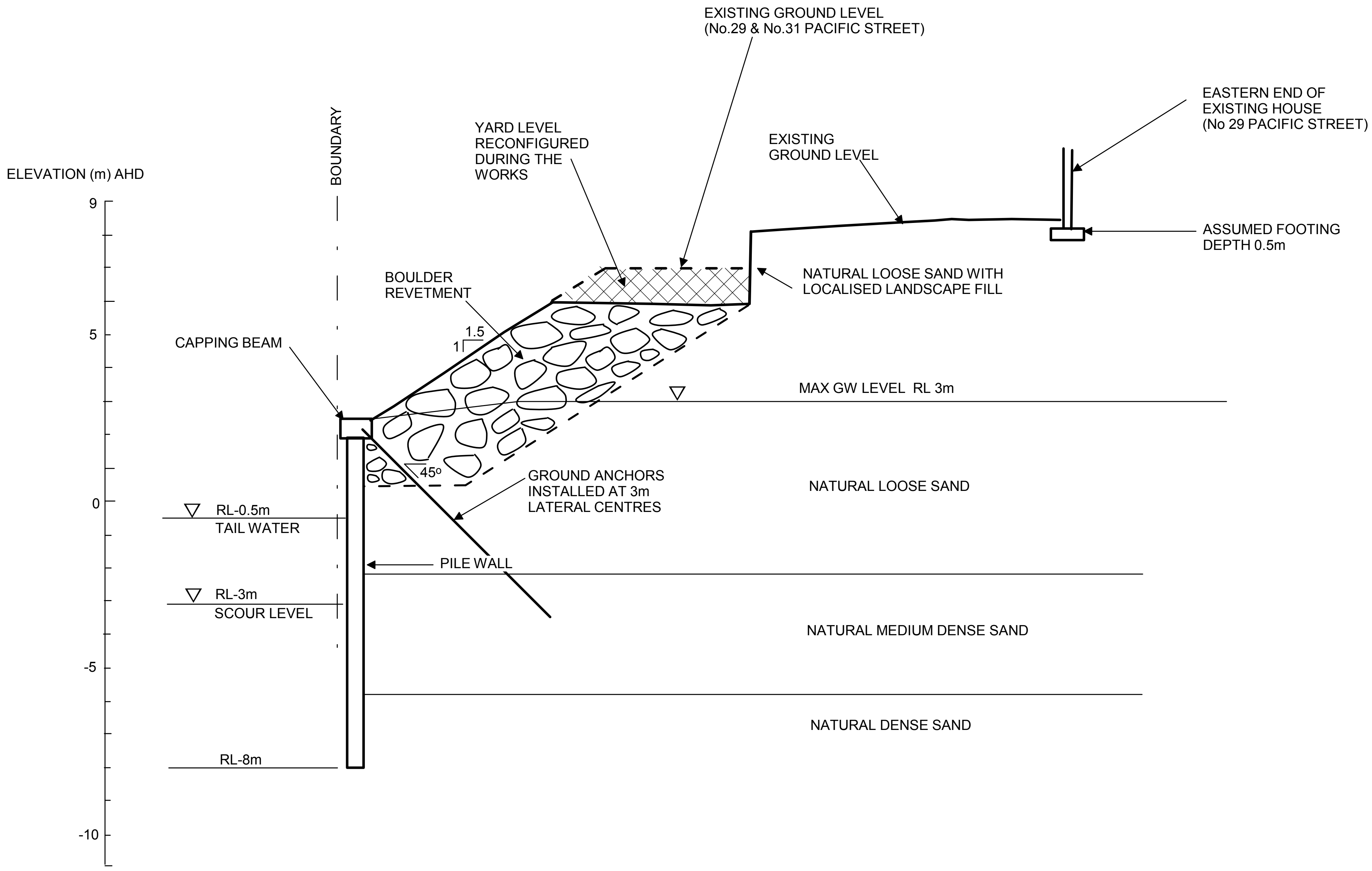
Location: LOT 29, 31 & 33, 23A, 23B, 25C
PACIFIC STREET & OCEAN VIEW DRIVE, WAMBERAL, NSW

Report No: 30243ZR

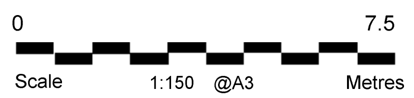
Figure No: 1

JK Geotechnics

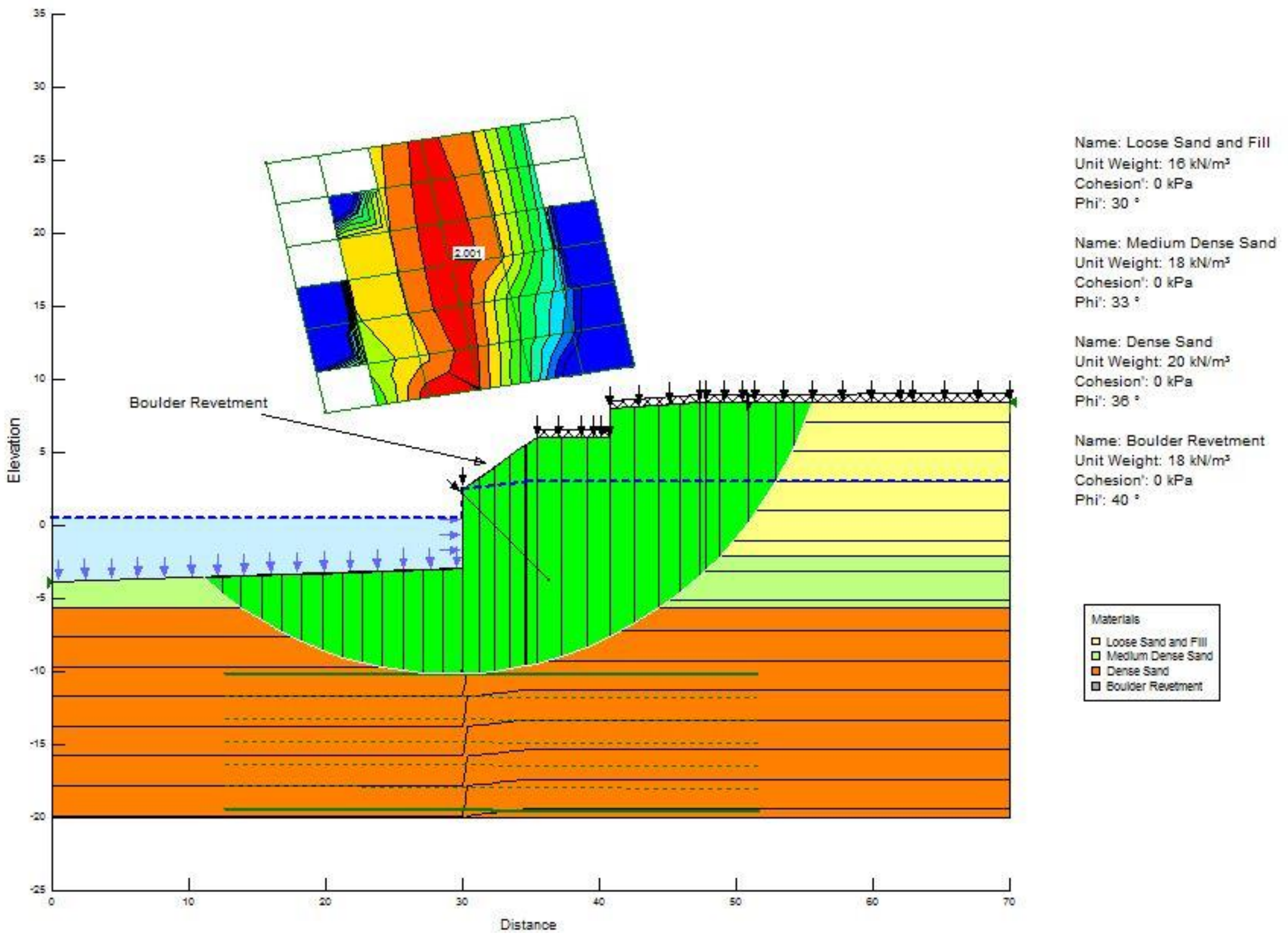
PLOT DATE: 2/03/2017 10:16:54 AM DWG FILE: S:\16 GEOTECHNICAL\6F GEOTECHNICAL JOBS\30000'S\30243ZR WAMBERAL\CAD\30243ZR.DWG



GEOTECHNICAL MODEL



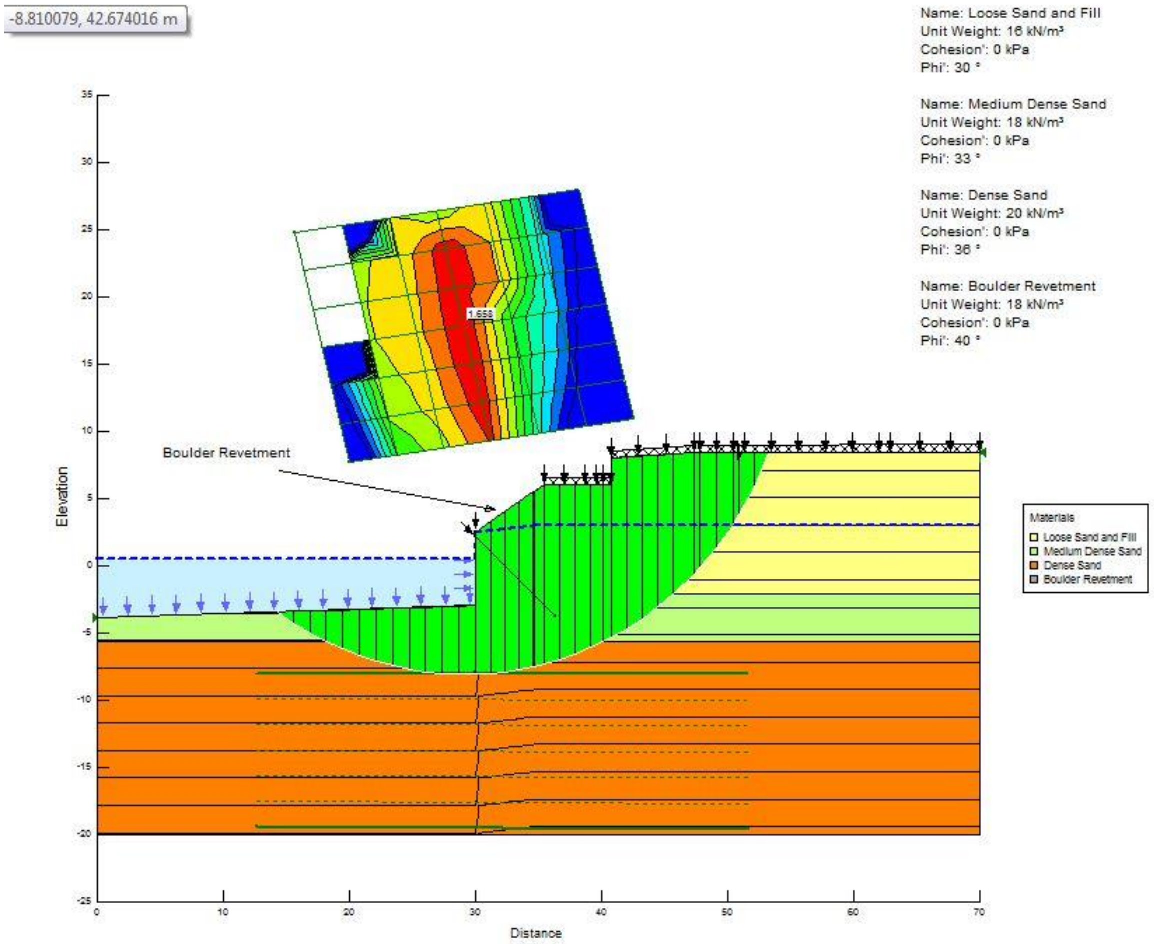
JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS
Report No. 30243ZR
Figure No. 2



THEORETICAL SLIP CIRCLE – GLOBAL FAILURE
PILE TOE AT RL – 10m

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 30243ZR Figure No. 3



THEORETICAL SLIP CIRCLE – GLOBAL FAILURE
PILE TOE AT RL – 8m

JK Geotechnics
GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 30243ZR   Figure No. 4

APPENDIX A

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. **1**
1/2

BOREHOLE LOG

Client: MR & MRS SHIELDS
Project: PROPOSED REDEVELOPMENT
Location: 23B OCEAN VIEW DRIVE, WAMBERAL, NSW

Job No. 25111P **Method:** SPIRAL AUGER JK250 **R.L. Surface:** ≈ 3.35m
Date: 16-8-11 **Datum:** AHD
Logged/Checked by: A.M./PW

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0		SM	SILTY SAND: medium grained, brown, with root fibres.	M	L		GRASS COVER
	DS	N = 5 1,3,2				SILTY SAND: medium grained, brown and light grey.				
			1		SP	SAND: medium grained, light grey.				
ON COMPLETION; DURING DRILLING	DS	N = 7 3,3,4	2							
						SAND: medium to coarse grained, grey.	W			
	DS	N = 8 4,4,4	3							
			4							
	DS	N = 9 3,5,4	5					MD		
	DS	N = 15 5,7,8	6							
			7							

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
1
2/2

Client: MR & MRS SHIELDS
Project: PROPOSED REDEVELOPMENT
Location: 23B OCEAN VIEW DRIVE, WAMBERAL, NSW

Job No. 25111P
Date: 16-8-11

Method: SPIRAL AUGER
JK250

Logged/Checked by: A.M./RW

R.L. Surface: ≈ 3.35m
Datum: AHD

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
					SP	SAND: medium to coarse grained, grey.	M	MD		
	DS	N = 18 5,9,9								
			8							
	DS	N = 33 4,11,22	9					D		
			10			END OF BOREHOLE AT 10.0m				
			11							
			12							
			13							
			14							

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

Borehole No. 2

1/1

BOREHOLE LOG

Client: MR & MRS SHIELDS

Project: PROPOSED REDEVELOPMENT

Location: 23B OCEAN VIEW DRIVE, WAMBERAL, NSW

Job No. 25111P **Method:** HAND AUGER **R.L. Surface:** ≈ 5.66m

Date: 16-8-11 **Datum:** AHD

Logged/Checked by: A.M./PW

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		REFER TO DCP TEST RESULTS	0			FILL: Silty sandy clay topsoil, dark brown, fine to coarse grained sand.	MC>PL			APPEARS POORLY COMPACTED
						FILL: Silty sand, medium to coarse grained, orange brown.	M			APPEARS MODERATELY COMPACTED
						FILL: Silty sandy clay, low plasticity, orange brown and light grey.	MC≈PL		150	
			1			FILL: Silty clay, medium plasticity, light grey mottled orange brown, trace of medium to coarse grained sub rounded shale and sandstone gravel.	MC>PL		160 150 100 90 100 150 120	
			2						130	APPEARS WELL COMPACTED
						as above, but with sandstone cobbles.				
						END OF BOREHOLE AT 2.2m				HAND AUGER REFUSAL ON OBSTRUCTION IN FILL
			3							
			4							
			5							
			6							
			7							

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

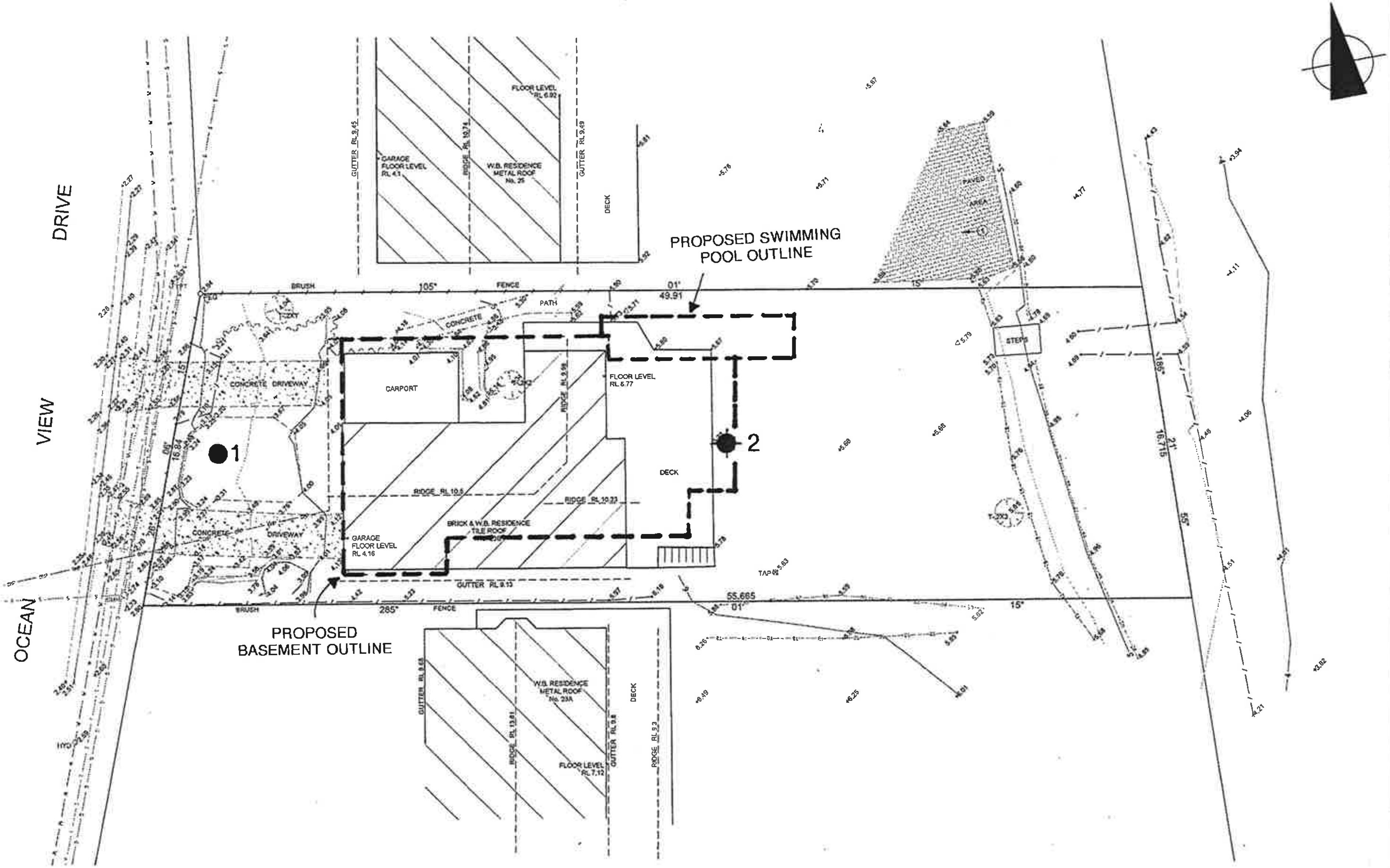
Client:	MR & MRS SHIELDS		
Project:	PROPOSED REDEVELOPMENT		
Location:	23B OCEAN VIEW DRIVE, WAMBERAL, NSW		
Job No.	25111P	Hammer Weight & Drop: 9kg/510mm	
Date:	16-8-11	Rod Diameter: 16mm	
Tested By:	A.M.	Point Diameter: 20mm	

Number of Blows per 100mm Penetration

Test Location	RL ~5.66m			Test Location			
Depth (mm)	2			Depth (mm)	2		
0 - 100	1			3000-3100	9		
100 - 200	2			3100-3200	11		
200 - 300	3			3200-3300	9		
300 - 400	5			3300-3400	9		
400 - 500	4			3400-3500	9		
500 - 600	7			3500-3600	9		
600 - 700	4			3600-3700	9		
700 - 800	4			3700-3800	13		
800 - 900	4			3800-3900	21		
900 - 1000	4			3900-4000	20		
1000 - 1100	3			4000-4100	22		
1100 - 1200	4			4100-4200	23		
1200 - 1300	7			4200-4300	23		
1300 - 1400	9			4300-4400	23		
1400 - 1500	12			4400-4500	24		
1500 - 1600	9			4500-4600	25		
1600 - 1700	7			4600-4700	26		
1700 - 1800	9			4700-4800	26		
1800 - 1900	17			4800-4900	28		
1900 - 2000	17			4900-5000	29		
2000 - 2100	7			5000-5100	20		
2100 - 2200	7			5100-5200	15		
2200 - 2300	7			5200-5300	17		
2300 - 2400	8			5300-5400	18		
2400 - 2500	9			5400-5500	19		
2500 - 2600	7			5500-5600	19		
2600 - 2700	8			5600-5700	15		
2700 - 2800	9			5700-5800	22		
2800 - 2900	11			5800-5900	24		
2900 - 3000	8			5900-6000	29 END		

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal

Ref: Scala6.xls April 99



LEGEND

BOREHOLE

BOREHOLE AND DCP TEST

SCALE (m)

0 10

INVESTIGATION LOCATION PLAN

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

Report No. 25111P Figure No. 1

APPENDIX B

Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types: Active side	Soil types: Passive side
1	2.50	1 Loose Sand	1 Loose Sand
2	-2.15	2 Medium Dense Sand	2 Medium Dense Sand
3	-5.65	3 Dense Sand	3 Dense Sand

SOIL PROPERTIES

-- Soil type -- No. Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1 Loose Sand	16.00	20000	0.500	OC	0.292	3.000	
				(0.300)	(0.000)	(0.000)	
2 Medium Dense Sand	18.00	40000	0.600	OC	0.245	3.537	
				(0.300)	(0.000)	(0.000)	
3 Dense Sand	20.00	80000	0.500	OC	0.224	3.852	
				(0.300)	(0.000)	(0.000)	

Additional soil parameters associated with Ka and Kp

------- Soil type ------- No. Description	Ka: Soil friction angle	Ka: Wall adhesion coeff.	Ka: Back-fill angle	Kp: Soil friction angle	Kp: Wall adhesion coeff.	Kp: Back-fill angle
1 Loose Sand	30.00	0.500	0.00	30.00	0.000	0.00
2 Medium Dense Sand	34.00	0.500	0.00	34.00	0.000	0.00
3 Dense Sand	36.00	0.500	0.00	36.00	0.000	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	3.00	0.50

Automatic water pressure balancing at toe of wall : No

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = -10.00
Maximum finite element length = 0.80 m
Youngs modulus of wall E = 2.0000E+08 kN/m2
Moment of inertia of wall I = 3.6230E-04 m4/m run
E.I = 72460 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	2.00	3.00	0.000363	2.000E+08	3.50	45.00	150.0	No

SURCHARGE LOADS

Surch-arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Surcharge kN/m2 Far edge	Equiv. soil type	Partial factor/ Category
1	2.50	5.80(A)	50.00	15.80	5.00	=	N/A	N/A
2	2.50	0.00(A)	50.00	5.80	0.00	100.00	N/A	N/A
3	2.50	5.80(A)	50.00	5.30	72.00	=	N/A	N/A
4	2.50	11.10(A)	50.00	6.70	128.00	134.40	N/A	N/A
5	2.50	17.80(A)	50.00	3.70	134.40	=	N/A	N/A

Note: A = Active side, P = Passive 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 no.	Stage description
1	Apply surcharge no.1 at elevation 2.50
2	Apply surcharge no.2 at elevation 2.50
3	Apply surcharge no.3 at elevation 2.50
4	Apply surcharge no.4 at elevation 2.50
5	Apply surcharge no.5 at elevation 2.50
6	Excavate to elevation 1.50 on PASSIVE side
7	Install strut or anchor no.1 at elevation 2.00
8	Excavate to elevation -3.00 on PASSIVE side

FACTORS OF SAFETY and ANALYSIS OPTIONS

Stability analysis:
Method of analysis - CP2
Factor on passive for calculating wall depth = 1.50

Parameters for undrained strata:
Minimum equivalent fluid density = 5.00 kN/m3
Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:
Method - Subgrade reaction model using Influence Coefficients
Open Tension Crack analysis? - No
Non-linear Modulus Parameter (L) = 0 m

Boundary conditions:
Length of wall (normal to plane of analysis) = 30.00 m

Width of excavation on active side of wall = 100.00 m
Width of excavation on passive side of wall = 100.00 m

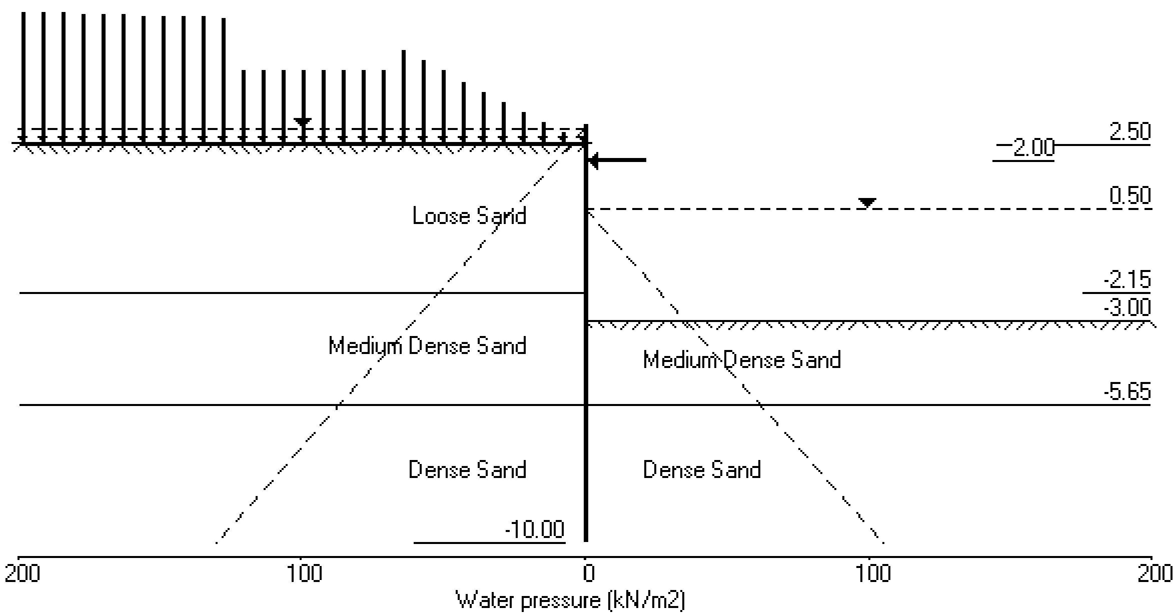
Distance to rigid boundary on active side = 100.00 m
Distance to rigid boundary on passive side = 100.00 m

OUTPUT OPTIONS

Stage no.	Stage description	Displacement Bending mom. Shear force	Active, Passive pressures	Graph. output
1	Apply surcharge no.1 at elev. 2.50	No	No	No
2	Apply surcharge no.2 at elev. 2.50	No	No	No
3	Apply surcharge no.3 at elev. 2.50	No	No	No
4	Apply surcharge no.4 at elev. 2.50	No	No	No
5	Apply surcharge no.5 at elev. 2.50	No	No	No
6	Excav. to elev. 1.50 on PASSIVE side	No	No	No
7	Install strut no.1 at elev. 2.00	No	No	No
8	Excav. to elev. -3.00 on PASSIVE side	No	No	No
*	Summary output	Yes	-	Yes

Units: kN,m

Stage No.8 Excav. to elev. -3.00 on PASSIVE side



Units: kN,m

Stage No. 8 Excavate to elevation -3.00 on PASSIVE side

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	FoS for toe elev. = -10.00 Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.500 Toe elev.	Wall Penetr -ation
8	2.50	-3.00	2.00	2.144	n/a	-8.32	5.32

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Node no.	Y coord	Nett pressure kN/m2	Wall disp. m	Wall rotation rad.	Shear force kN/m	Bending moment kN.m/m	Strut forces kN/m
1	3.00	0.00	0.036	-5.18E-03	0.0	-0.0	
2	2.50	5.00	0.039	-5.18E-03	1.3	0.2	
3	2.00	12.48	0.042	-5.19E-03	5.6	1.8	119.7
		12.48	0.042	-5.19E-03	-114.1	1.8	
4	1.50	19.92	0.044	-5.01E-03	-106.0	-53.2	
5	1.00	27.30	0.047	-4.47E-03	-94.2	-103.8	
6	0.50	34.61	0.049	-3.60E-03	-78.7	-146.9	
7	-0.15	37.46	0.050	-2.10E-03	-55.3	-188.4	
8	-0.80	40.13	0.051	-2.85E-04	-30.1	-216.4	
9	-1.48	42.73	0.051	1.78E-03	-2.1	-227.6	
10	-2.15	45.16	0.049	3.86E-03	27.6	-219.1	
		41.92	0.049	3.86E-03	27.6	-219.1	
11	-2.58	43.35	0.047	5.10E-03	45.7	-203.4	
12	-3.00	44.72	0.045	6.22E-03	64.4	-179.9	
13	-3.50	32.14	0.041	7.33E-03	83.6	-142.8	
14	-4.00	19.50	0.037	8.16E-03	96.5	-97.6	
15	-4.80	-0.82	0.030	8.80E-03	104.0	-17.1	
16	-5.23	-11.67	0.027	8.77E-03	101.3	26.6	
17	-5.65	-22.54	0.023	8.49E-03	94.1	68.2	
		-31.60	0.023	8.49E-03	94.1	68.2	
18	-6.43	-59.25	0.017	7.44E-03	58.9	128.5	
19	-7.20	-86.97	0.012	5.93E-03	2.2	153.1	
20	-8.00	-51.86	0.007	4.33E-03	-53.3	136.8	
21	-8.80	-10.13	0.004	3.14E-03	-78.1	78.8	
22	-9.40	44.06	0.003	2.69E-03	-67.9	30.2	
23	-10.00	182.40	0.001	2.57E-03	0.0	-0.0	

Strut force at elev. 2.00 = 119.70 kN/m run = 359.09 kN/strut (horiz.)
= 507.83 kN/strut (incl.)

(continued)

Stage No.8 Excavate to elevation -3.00 on PASSIVE side

Units: kN,m

Summary of results

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

				FoS for toe elev. = -10.00		Toe elev. for FoS = 1.500	
Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	Factor of Safety	Moment equilib. at elev.	Toe elev.	Wall Penetr -ation
1	2.50	2.50	Cant.	4.831	-8.69	1.16	1.34
2	2.50	2.50	Cant.	4.020	-8.87	1.11	1.39
3	2.50	2.50	Cant.	3.906	-9.04	1.11	1.39
4	2.50	2.50	Cant.	3.862	-9.16	1.11	1.39
5	2.50	2.50	Cant.	3.855	-9.18	1.11	1.39
6	2.50	1.50	Cant.	2.552	-9.14	-4.53	6.03
7	2.50	1.50	2.00	4.753	n/a	-0.90	2.40
8	2.50	-3.00	2.00	2.144	n/a	-8.32	5.32

Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement maximum m	Displacement minimum m	Bending moment maximum kN.m/m	Bending moment minimum kN.m/m	Shear force maximum kN/m	Shear force minimum kN/m
1	3.00	0.036	0.000	0.0	-0.0	0.0	0.0
2	2.50	0.039	0.000	0.2	0.0	1.3	0.0
3	2.00	0.042	0.000	2.5	0.0	9.8	-114.1
4	1.50	0.044	0.000	7.2	-53.2	13.7	-106.0
5	1.00	0.047	0.000	15.8	-103.8	19.5	-94.2
6	0.50	0.049	0.000	25.3	-146.9	17.0	-78.7
7	-0.15	0.050	0.000	35.1	-188.4	11.0	-55.3
8	-0.80	0.051	0.000	37.9	-216.4	9.0	-30.1
9	-1.48	0.051	0.000	38.6	-227.6	9.4	-2.1
10	-2.15	0.049	0.000	41.0	-219.1	27.6	0.0
11	-2.58	0.047	0.000	41.9	-203.4	45.7	-1.5
12	-3.00	0.045	0.000	40.3	-179.9	64.4	-6.6
13	-3.50	0.041	0.000	35.6	-142.8	83.6	-10.7
14	-4.00	0.037	0.000	29.6	-97.6	96.5	-12.3
15	-4.80	0.030	0.000	20.0	-17.1	104.0	-8.5
16	-5.23	0.027	0.000	26.6	0.0	101.3	-3.9
17	-5.65	0.023	0.000	68.2	0.0	94.1	0.0
18	-6.43	0.017	0.000	128.5	0.0	58.9	-3.6
19	-7.20	0.012	0.000	153.1	0.0	2.2	-6.2
20	-8.00	0.007	0.000	136.8	0.0	0.0	-53.3
21	-8.80	0.004	0.000	78.8	0.0	0.0	-78.1
22	-9.40	0.003	0.000	30.2	0.0	0.0	-67.9
23	-10.00	0.003	0.000	0.0	-0.0	0.0	0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment maximum kN.m/m	elev.	Bending moment minimum kN.m/m	elev.	Shear force maximum kN/m	elev.	Shear force minimum kN/m	elev.
1	10.2	-3.00	-0.5	-0.15	6.9	-2.15	-2.3	-7.20
2	11.8	-6.43	-6.8	-0.80	11.1	-2.15	-5.0	1.00
3	12.1	-6.43	-8.3	-0.80	11.1	-2.15	-5.5	1.00
4	12.5	-6.43	-8.6	-0.80	11.1	-2.15	-5.6	1.00
5	12.6	-6.43	-8.6	-0.80	11.1	-2.15	-5.6	1.00
6	41.9	-2.58	-0.0	-10.00	19.5	1.00	-12.3	-4.00
7	31.4	-3.00	-7.8	1.00	12.9	-2.15	-25.5	2.00
8	153.1	-7.20	-227.6	-1.48	104.0	-4.80	-114.1	2.00

Summary of results (continued)

Maximum and minimum displacement at each stage

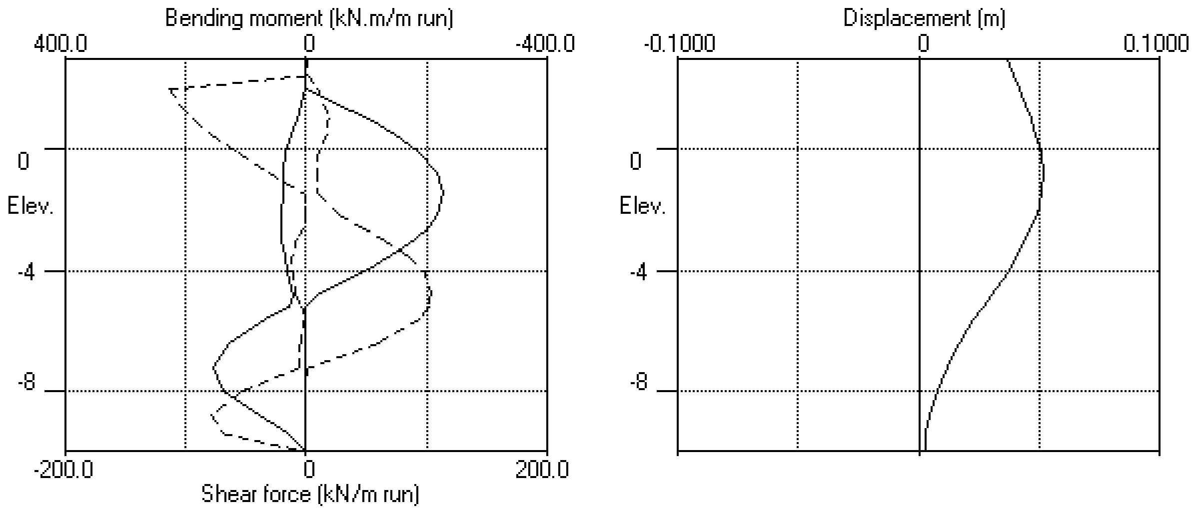
Stage no.	Displacement maximum m	elev.	minimum m	elev.	Stage description
1	0.006	3.00	0.000	3.00	Apply surcharge no.1 at elev. 2.50
2	0.009	3.00	0.000	3.00	Apply surcharge no.2 at elev. 2.50
3	0.009	3.00	0.000	3.00	Apply surcharge no.3 at elev. 2.50
4	0.009	3.00	0.000	3.00	Apply surcharge no.4 at elev. 2.50
5	0.009	3.00	0.000	3.00	Apply surcharge no.5 at elev. 2.50
6	0.024	3.00	0.000	3.00	Excav. to elev. 1.50 on PASSIVE side
7	0.019	3.00	0.000	3.00	Install strut no.1 at elev. 2.00
8	0.051	-0.80	0.000	3.00	Excav. to elev. -3.00 on PASSIVE side

Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 2.00 kN/m run	kN/strut
7	35.36	106.07
8	119.70	359.09

Units: kN,m

Bending moment, shear force, displacement envelopes



Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types: Active side	Soil types: Passive side
1	2.50	1 Loose Sand	1 Loose Sand
2	-2.15	2 Medium Dense Sand	2 Medium Dense Sand
3	-5.65	3 Dense Sand	3 Dense Sand

SOIL PROPERTIES

-- Soil type -- No. Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1 Loose Sand	16.00	20000	0.500	OC (0.300)	0.292 (0.000)	3.000 (0.000)	
2 Medium Dense Sand	18.00	40000	0.600	OC (0.300)	0.245 (0.000)	3.537 (0.000)	
3 Dense Sand	20.00	80000	0.500	OC (0.300)	0.224 (0.000)	3.852 (0.000)	

Additional soil parameters associated with Ka and Kp

------- Soil type ------- No. Description	Ka: Soil friction angle	Ka: Wall adhesion coeff.	Ka: Back-fill angle	Kp: Soil friction angle	Kp: Wall adhesion coeff.	Kp: Back-fill angle
1 Loose Sand	30.00	0.500	0.00	30.00	0.000	0.00
2 Medium Dense Sand	34.00	0.500	0.00	34.00	0.000	0.00
3 Dense Sand	36.00	0.500	0.00	36.00	0.000	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	3.00	0.50

Automatic water pressure balancing at toe of wall : No

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = -10.00
Maximum finite element length = 0.80 m
Youngs modulus of wall E = 2.0000E+08 kN/m2
Moment of inertia of wall I = 1.1567E-03 m4/m run
(Arcelor AZ48) E.I = 231340 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre- stress /strut kN	Tension allowed
1	2.00	3.00	0.000363	2.000E+08	3.50	45.00	150.0	No

SURCHARGE LOADS

Surch -arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Surcharge kN/m2 Far edge	Equiv. soil type	Partial factor/ Category
1	2.50	5.80(A)	50.00	15.80	5.00	=	N/A	N/A
2	2.50	0.00(A)	50.00	5.80	0.00	100.00	N/A	N/A
3	2.50	5.80(A)	50.00	5.30	72.00	=	N/A	N/A
4	2.50	11.10(A)	50.00	6.70	128.00	134.40	N/A	N/A
5	2.50	17.80(A)	50.00	3.70	134.40	=	N/A	N/A

Note: A = Active side, P = Passive 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 no.	Stage description
1	Apply surcharge no.1 at elevation 2.50
2	Apply surcharge no.2 at elevation 2.50
3	Apply surcharge no.3 at elevation 2.50
4	Apply surcharge no.4 at elevation 2.50
5	Apply surcharge no.5 at elevation 2.50
6	Excavate to elevation 1.50 on PASSIVE side
7	Install strut or anchor no.1 at elevation 2.00
8	Excavate to elevation -3.00 on PASSIVE side

FACTORS OF SAFETY and ANALYSIS OPTIONS

Stability analysis:
Method of analysis - CP2
Factor on passive for calculating wall depth = 1.50

Parameters for undrained strata:
Minimum equivalent fluid density = 5.00 kN/m3
Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:
Method - Subgrade reaction model using Influence Coefficients
Open Tension Crack analysis? - No
Non-linear Modulus Parameter (L) = 0 m

Boundary conditions:
Length of wall (normal to plane of analysis) = 30.00 m

Width of excavation on active side of wall = 100.00 m
Width of excavation on passive side of wall = 100.00 m

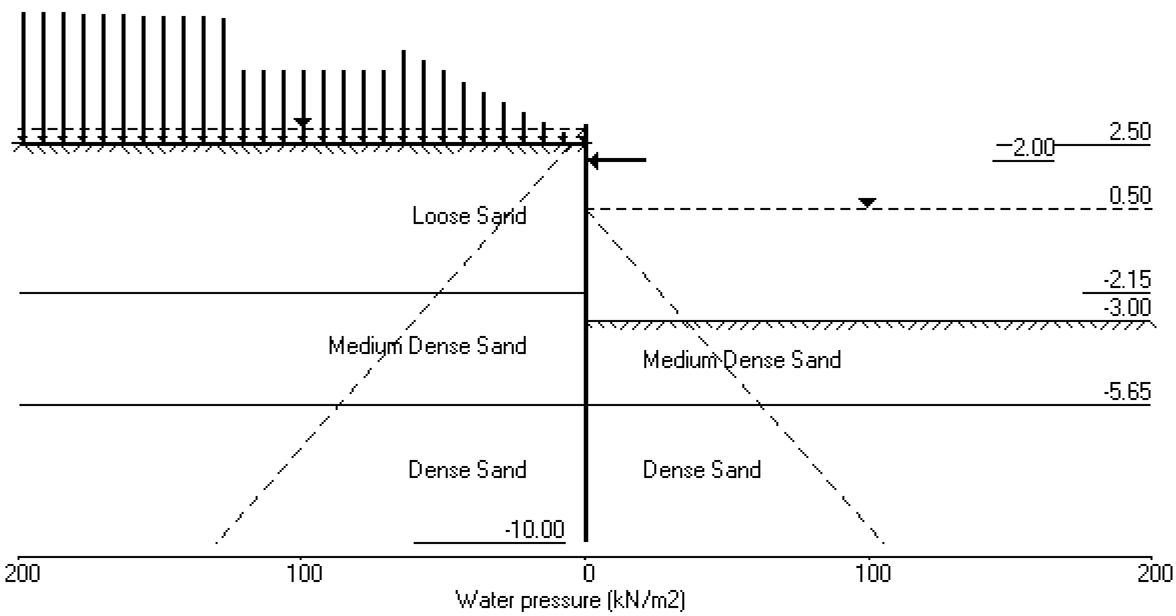
Distance to rigid boundary on active side = 100.00 m
Distance to rigid boundary on passive side = 100.00 m

OUTPUT OPTIONS

Stage no.	Stage description	Displacement Bending mom. Shear force	Active, Passive pressures	Graph. output
1	Apply surcharge no.1 at elev. 2.50	No	No	No
2	Apply surcharge no.2 at elev. 2.50	No	No	No
3	Apply surcharge no.3 at elev. 2.50	No	No	No
4	Apply surcharge no.4 at elev. 2.50	No	No	No
5	Apply surcharge no.5 at elev. 2.50	No	No	No
6	Excav. to elev. 1.50 on PASSIVE side	No	No	No
7	Install strut no.1 at elev. 2.00	No	No	No
8	Excav. to elev. -3.00 on PASSIVE side	No	No	No
*	Summary output	Yes	-	Yes

Units: kN,m

Stage No.8 Excav. to elev. -3.00 on PASSIVE side



Units: kN,m

Stage No. 8 Excavate to elevation -3.00 on PASSIVE side

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	FoS for toe elev. = -10.00 Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.500 Toe elev.	Wall Penetr -ation
8	2.50	-3.00	2.00	2.144	n/a	-8.32	5.32

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Node no.	Y coord	Nett pressure kN/m2	Wall disp. m	Wall rotation rad.	Shear force kN/m	Bending moment kN.m/m	Strut forces kN/m
1	3.00	0.00	0.039	3.53E-04	0.0	0.0	
2	2.50	5.00	0.039	3.53E-04	1.3	0.2	
3	2.00	12.48	0.039	3.50E-04	5.6	2.0	121.1
		12.48	0.039	3.50E-04	-115.5	2.0	
4	1.50	19.92	0.039	4.06E-04	-107.4	-53.8	
5	1.00	27.30	0.039	5.78E-04	-95.6	-105.1	
6	0.50	34.61	0.038	8.52E-04	-80.1	-148.2	
7	-0.15	37.46	0.038	1.33E-03	-56.7	-192.8	
8	-0.80	40.13	0.037	1.91E-03	-31.5	-221.7	
9	-1.48	42.73	0.035	2.57E-03	-3.5	-233.7	
10	-2.15	45.16	0.033	3.24E-03	26.1	-225.4	
		41.92	0.033	3.24E-03	26.1	-225.4	
11	-2.58	43.35	0.032	3.64E-03	44.2	-210.2	
12	-3.00	44.72	0.030	4.01E-03	63.0	-187.3	
13	-3.50	32.14	0.028	4.37E-03	82.2	-150.9	
14	-4.00	19.50	0.026	4.65E-03	95.1	-106.3	
15	-4.80	-0.82	0.022	4.88E-03	102.5	-27.0	
16	-5.23	-11.67	0.020	4.89E-03	99.9	16.1	
17	-5.65	-22.54	0.018	4.82E-03	92.6	57.1	
		-31.60	0.018	4.82E-03	92.6	57.1	
18	-6.43	-59.25	0.014	4.53E-03	57.4	117.0	
19	-7.20	-79.65	0.011	4.09E-03	3.6	149.5	
20	-8.00	-45.52	0.008	3.61E-03	-46.5	126.4	
21	-8.80	-11.01	0.005	3.26E-03	-69.1	75.9	
22	-9.40	23.74	0.003	3.12E-03	-65.3	32.5	
23	-10.00	193.80	0.001	3.08E-03	0.0	-0.0	

Strut force at elev. 2.00 = 121.15 kN/m run = 363.44 kN/strut (horiz.)
= 513.99 kN/strut (incl.)

(continued)

Stage No.8   Excavate to elevation -3.00 on PASSIVE side

Units: kN,m

Summary of results

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

				FoS for toe elev. = -10.00		Toe elev. for FoS = 1.500	
Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	Factor of Safety	Moment equilib. at elev.	Toe elev.	Wall Penetr -ation
1	2.50	2.50	Cant.	4.831	-8.69	1.16	1.34
2	2.50	2.50	Cant.	4.020	-8.87	1.11	1.39
3	2.50	2.50	Cant.	3.906	-9.04	1.11	1.39
4	2.50	2.50	Cant.	3.862	-9.16	1.11	1.39
5	2.50	2.50	Cant.	3.855	-9.18	1.11	1.39
6	2.50	1.50	Cant.	2.552	-9.14	-4.53	6.03
7	2.50	1.50	2.00	4.753	n/a	-0.90	2.40
8	2.50	-3.00	2.00	2.144	n/a	-8.32	5.32

Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement maximum m	Displacement minimum m	Bending moment maximum kN.m/m	Bending moment minimum kN.m/m	Shear force maximum kN/m	Shear force minimum kN/m
1	3.00	0.039	0.000	0.0	0.0	0.0	0.0
2	2.50	0.039	0.000	0.2	0.0	1.3	0.0
3	2.00	0.039	0.000	2.5	0.0	8.5	-115.5
4	1.50	0.039	0.000	7.2	-53.8	13.7	-107.4
5	1.00	0.039	0.000	15.8	-105.1	19.5	-95.6
6	0.50	0.038	0.000	26.2	-148.2	17.8	-80.1
7	-0.15	0.038	0.000	35.7	-192.8	12.8	-56.7
8	-0.80	0.037	0.000	43.0	-221.7	12.4	-31.5
9	-1.48	0.035	0.000	50.5	-233.7	17.4	-3.5
10	-2.15	0.033	0.000	60.8	-225.4	26.1	0.0
11	-2.58	0.032	0.000	66.5	-210.2	44.2	0.0
12	-3.00	0.030	0.000	69.2	-187.3	63.0	0.0
13	-3.50	0.028	0.000	68.9	-150.9	82.2	-2.8
14	-4.00	0.026	0.000	66.5	-106.3	95.1	-7.1
15	-4.80	0.022	0.000	58.6	-27.0	102.5	-8.9
16	-5.23	0.020	0.000	55.1	0.0	99.9	-6.8
17	-5.65	0.018	0.000	57.1	0.0	92.6	-2.9
18	-6.43	0.014	0.000	117.0	0.0	57.4	-12.2
19	-7.20	0.011	0.000	149.5	0.0	3.6	-16.9
20	-8.00	0.008	0.000	126.4	0.0	0.0	-46.5
21	-8.80	0.005	0.000	75.9	0.0	0.0	-69.1
22	-9.40	0.003	0.000	32.5	0.0	0.0	-65.3
23	-10.00	0.003	0.000	0.0	-0.0	0.0	0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment maximum kN.m/m	elev.	Bending moment minimum kN.m/m	elev.	Shear force maximum kN/m	elev.	Shear force minimum kN/m	elev.
1	20.5	-3.50	0.0	3.00	9.5	-2.15	-5.5	-7.20
2	23.7	-5.65	-4.4	-0.15	13.7	-2.15	-8.6	-8.00
3	22.9	-6.43	-6.0	-0.15	13.2	-2.15	-8.5	-8.00
4	23.1	-6.43	-6.3	-0.80	13.1	-2.15	-8.8	-8.00
5	23.3	-6.43	-6.3	-0.80	13.1	-2.15	-8.9	-8.00
6	69.2	-3.00	0.0	3.00	19.5	1.00	-16.9	-7.20
7	47.8	-4.00	-11.3	1.00	23.6	-2.15	-26.8	2.00
8	149.5	-7.20	-233.7	-1.48	102.5	-4.80	-115.5	2.00

Summary of results (continued)

Maximum and minimum displacement at each stage

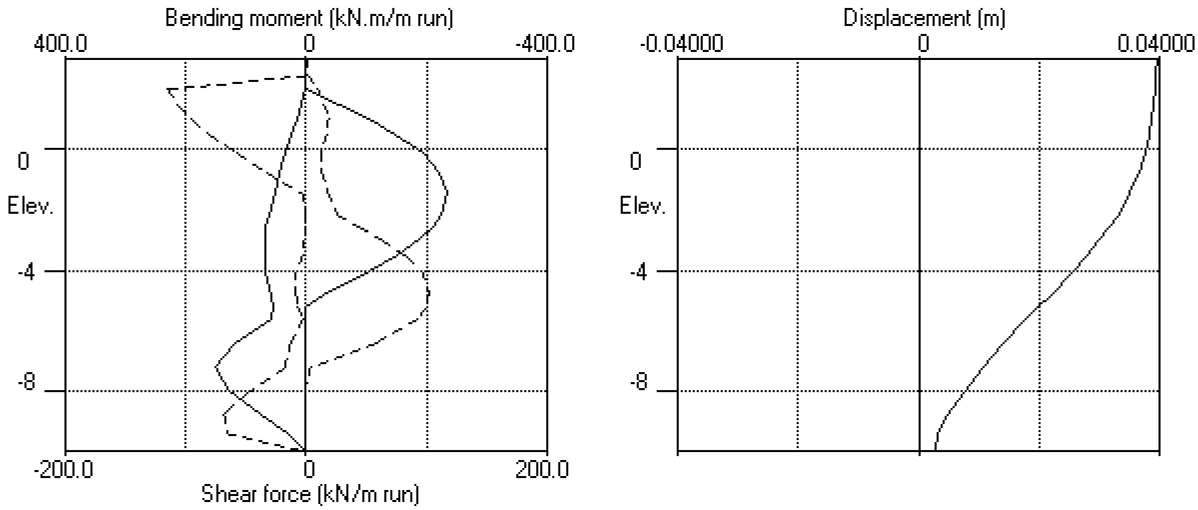
Stage no.	Displacement maximum m	elev.	minimum m	elev.	Stage description
1	0.006	3.00	0.000	3.00	Apply surcharge no.1 at elev. 2.50
2	0.009	3.00	0.000	3.00	Apply surcharge no.2 at elev. 2.50
3	0.010	3.00	0.000	3.00	Apply surcharge no.3 at elev. 2.50
4	0.010	3.00	0.000	3.00	Apply surcharge no.4 at elev. 2.50
5	0.010	3.00	0.000	3.00	Apply surcharge no.5 at elev. 2.50
6	0.019	3.00	0.000	3.00	Excav. to elev. 1.50 on PASSIVE side
7	0.016	3.00	0.000	3.00	Install strut no.1 at elev. 2.00
8	0.039	3.00	0.000	3.00	Excav. to elev. -3.00 on PASSIVE side

Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 2.00 kN/m run	kN/strut
7	35.36	106.07
8	121.15	363.44

Units: kN,m

Bending moment, shear force, displacement envelopes



Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types: Active side	Soil types: Passive side
1	2.50	1 Loose Sand	1 Loose Sand
2	-2.15	2 Medium Dense Sand	2 Medium Dense Sand
3	-5.65	3 Dense Sand	3 Dense Sand

SOIL PROPERTIES

-- Soil type -- No. Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1 Loose Sand	16.00	20000	0.500	OC	0.292	3.000	
				(0.300)	(0.000)	(0.000)	
2 Medium Dense Sand	18.00	40000	0.600	OC	0.245	3.537	
				(0.300)	(0.000)	(0.000)	
3 Dense Sand	20.00	80000	0.500	OC	0.224	3.852	
				(0.300)	(0.000)	(0.000)	

Additional soil parameters associated with Ka and Kp

------- Soil type ------- No. Description	--- parameters for Ka --- Soil friction angle	Wall adhesion coeff.	Back-fill angle	--- parameters for Kp --- Soil friction angle	Wall adhesion coeff.	Back-fill angle
1 Loose Sand	30.00	0.500	0.00	30.00	0.000	0.00
2 Medium Dense Sand	34.00	0.500	0.00	34.00	0.000	0.00
3 Dense Sand	36.00	0.500	0.00	36.00	0.000	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	3.00	0.50

Automatic water pressure balancing at toe of wall : No

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = -10.00
Maximum finite element length = 0.80 m
Youngs modulus of wall E = 2.0000E+07 kN/m2
Moment of inertia of wall I = 6.3600E-03 m4/m run
E.I = 127200 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	2.00	3.00	0.000363	2.000E+08	3.50	45.00	150.0	No

SURCHARGE LOADS

Surch -arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Surcharge kN/m2 Far edge	Equiv. soil type	Partial factor/ Category
1	2.50	5.80(A)	50.00	15.80	5.00	=	N/A	N/A
2	2.50	0.00(A)	50.00	5.80	0.00	100.00	N/A	N/A
3	2.50	5.80(A)	50.00	5.30	72.00	=	N/A	N/A
4	2.50	11.10(A)	50.00	6.70	128.00	134.40	N/A	N/A
5	2.50	17.80(A)	50.00	3.70	134.40	=	N/A	N/A

Note: A = Active side, P = Passive 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 no.	Stage description
1	Apply surcharge no.1 at elevation 2.50
2	Apply surcharge no.2 at elevation 2.50
3	Apply surcharge no.3 at elevation 2.50
4	Apply surcharge no.4 at elevation 2.50
5	Apply surcharge no.5 at elevation 2.50
6	Excavate to elevation 1.50 on PASSIVE side
7	Install strut or anchor no.1 at elevation 2.00
8	Excavate to elevation -3.00 on PASSIVE side

FACTORS OF SAFETY and ANALYSIS OPTIONS

Stability analysis:
Method of analysis - CP2
Factor on passive for calculating wall depth = 1.50

Parameters for undrained strata:
Minimum equivalent fluid density = 5.00 kN/m3
Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:
Method - Subgrade reaction model using Influence Coefficients
Open Tension Crack analysis? - No
Non-linear Modulus Parameter (L) = 0 m

Boundary conditions:
Length of wall (normal to plane of analysis) = 30.00 m

Width of excavation on active side of wall = 100.00 m
Width of excavation on passive side of wall = 100.00 m

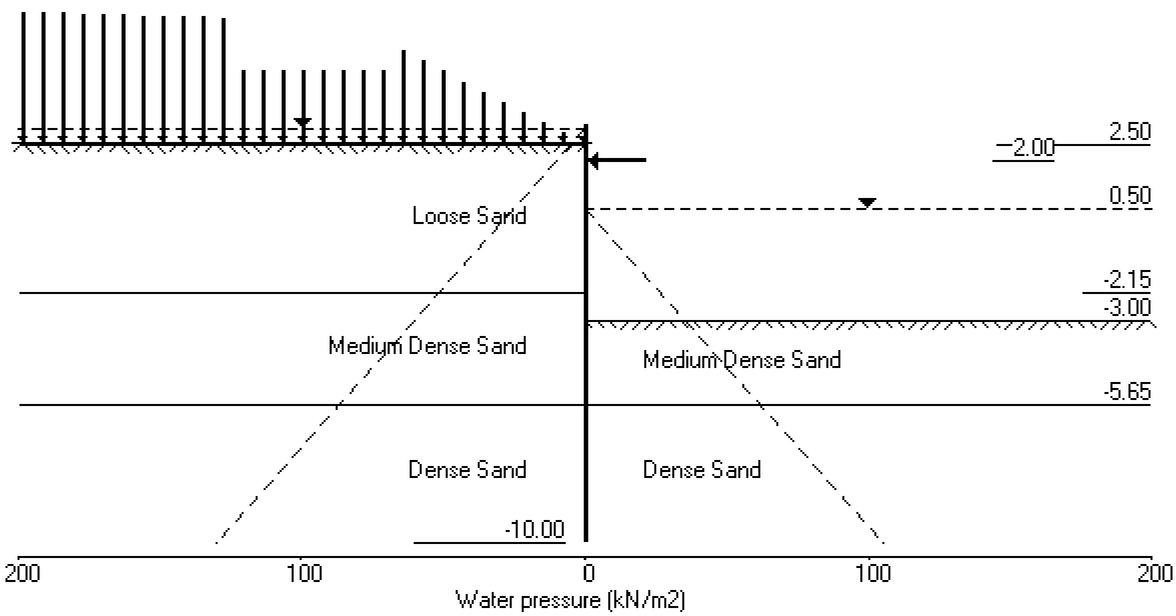
Distance to rigid boundary on active side = 100.00 m
Distance to rigid boundary on passive side = 100.00 m

OUTPUT OPTIONS

Stage no.	Stage description	Output options: Displacement Bending mom. Shear force	Output options: Active, Passive pressures	Output options: Graph. output
1	Apply surcharge no.1 at elev. 2.50	No	No	No
2	Apply surcharge no.2 at elev. 2.50	No	No	No
3	Apply surcharge no.3 at elev. 2.50	No	No	No
4	Apply surcharge no.4 at elev. 2.50	No	No	No
5	Apply surcharge no.5 at elev. 2.50	No	No	No
6	Excav. to elev. 1.50 on PASSIVE side	No	No	No
7	Install strut no.1 at elev. 2.00	No	No	No
8	Excav. to elev. -3.00 on PASSIVE side	No	No	No
*	Summary output	Yes	-	Yes

Units: kN,m

Stage No.8 Excav. to elev. -3.00 on PASSIVE side



Units: kN,m

Stage No. 8 Excavate to elevation -3.00 on PASSIVE side

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	FoS for toe elev. = -10.00: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.500: Toe elev.	Wall Penetr -ation
8	2.50	-3.00	2.00	2.144	n/a	-8.32	5.32

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Node no.	Y coord	Nett pressure kN/m2	Wall disp. m	Wall rotation rad.	Shear force kN/m	Bending moment kN.m/m	Strut forces kN/m
1	3.00	0.00	0.038	-1.68E-03	0.0	0.0	
2	2.50	5.00	0.039	-1.68E-03	1.3	0.2	
3	2.00	12.48	0.040	-1.69E-03	5.6	1.9	120.0
		12.48	0.040	-1.69E-03	-114.4	1.9	
4	1.50	19.92	0.041	-1.59E-03	-106.3	-53.3	
5	1.00	27.30	0.041	-1.28E-03	-94.5	-104.0	
6	0.50	34.61	0.042	-7.89E-04	-79.0	-147.4	
7	-0.15	37.46	0.042	7.17E-05	-55.6	-189.5	
8	-0.80	40.13	0.042	1.11E-03	-30.4	-217.6	
9	-1.48	42.73	0.041	2.29E-03	-2.4	-228.9	
10	-2.15	45.16	0.039	3.48E-03	27.3	-220.3	
		41.92	0.039	3.48E-03	27.3	-220.3	
11	-2.58	43.35	0.037	4.19E-03	45.4	-204.7	
12	-3.00	44.72	0.035	4.84E-03	64.1	-181.3	
13	-3.50	32.14	0.033	5.48E-03	83.3	-144.4	
14	-4.00	19.50	0.030	5.96E-03	96.2	-99.2	
15	-4.80	-0.82	0.025	6.33E-03	103.7	-19.1	
16	-5.23	-11.67	0.022	6.32E-03	101.0	24.5	
17	-5.65	-22.54	0.019	6.17E-03	93.8	65.9	
		-31.60	0.019	6.17E-03	93.8	65.9	
18	-6.43	-59.25	0.015	5.59E-03	58.6	126.0	
19	-7.20	-86.97	0.011	4.74E-03	1.9	150.3	
20	-8.00	-47.26	0.007	3.85E-03	-51.8	133.7	
21	-8.80	-9.48	0.005	3.18E-03	-74.5	78.4	
22	-9.40	32.91	0.003	2.92E-03	-67.5	32.0	
23	-10.00	191.93	0.001	2.85E-03	0.0	0.0	

Strut force at elev. 2.00 = 120.00 kN/m run = 359.99 kN/strut (horiz.)
= 509.11 kN/strut (incl.)

(continued)

Stage No.8 Excavate to elevation -3.00 on PASSIVE side

Units: kN,m

Summary of results

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

				FoS for toe elev. = -10.00		Toe elev. for FoS = 1.500	
Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	Factor of Safety	Moment equilib. at elev.	Toe elev.	Wall Penetr -ation
1	2.50	2.50	Cant.	4.831	-8.69	1.16	1.34
2	2.50	2.50	Cant.	4.020	-8.87	1.11	1.39
3	2.50	2.50	Cant.	3.906	-9.04	1.11	1.39
4	2.50	2.50	Cant.	3.862	-9.16	1.11	1.39
5	2.50	2.50	Cant.	3.855	-9.18	1.11	1.39
6	2.50	1.50	Cant.	2.552	-9.14	-4.53	6.03
7	2.50	1.50	2.00	4.753	n/a	-0.90	2.40
8	2.50	-3.00	2.00	2.144	n/a	-8.32	5.32

Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement maximum m	Displacement minimum m	Bending moment maximum kN.m/m	Bending moment minimum kN.m/m	Shear force maximum kN/m	Shear force minimum kN/m
1	3.00	0.038	0.000	0.0	-0.0	0.0	0.0
2	2.50	0.039	0.000	0.2	0.0	1.3	0.0
3	2.00	0.040	0.000	2.5	0.0	9.2	-114.4
4	1.50	0.041	0.000	7.2	-53.3	13.7	-106.3
5	1.00	0.041	0.000	15.8	-104.0	19.5	-94.5
6	0.50	0.042	0.000	25.2	-147.4	17.0	-79.0
7	-0.15	0.042	0.000	35.1	-189.5	9.9	-55.6
8	-0.80	0.042	0.000	39.8	-217.6	10.8	-30.4
9	-1.48	0.041	0.000	43.7	-228.9	13.8	-2.4
10	-2.15	0.039	0.000	49.5	-220.3	27.3	0.0
11	-2.58	0.037	0.000	52.6	-204.7	45.4	0.0
12	-3.00	0.035	0.000	52.8	-181.3	64.1	-2.5
13	-3.50	0.033	0.000	50.0	-144.4	83.3	-7.5
14	-4.00	0.030	0.000	45.5	-99.2	96.2	-10.4
15	-4.80	0.025	0.000	36.2	-19.1	103.7	-9.2
16	-5.23	0.022	0.000	32.9	0.0	101.0	-5.7
17	-5.65	0.019	0.000	65.9	0.0	93.8	-0.6
18	-6.43	0.015	0.000	126.0	0.0	58.6	-7.5
19	-7.20	0.011	0.000	150.3	0.0	1.9	-10.3
20	-8.00	0.007	0.000	133.7	0.0	0.0	-51.8
21	-8.80	0.005	0.000	78.4	0.0	0.0	-74.5
22	-9.40	0.003	0.000	32.0	0.0	0.0	-67.5
23	-10.00	0.003	0.000	0.0	-0.0	0.0	0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment maximum kN.m/m	elev.	Bending moment minimum kN.m/m	elev.	Shear force maximum kN/m	elev.	Shear force minimum kN/m	elev.
1	14.4	-3.50	-0.0	3.00	8.0	-2.15	-3.6	-7.20
2	16.8	-6.43	-5.7	-0.15	12.2	-2.15	-6.1	-8.00
3	16.6	-6.43	-7.5	-0.80	12.0	-2.15	-6.3	-8.00
4	17.0	-6.43	-7.8	-0.80	12.0	-2.15	-6.5	-8.00
5	17.1	-6.43	-7.8	-0.80	12.0	-2.15	-6.6	-8.00
6	52.8	-3.00	-0.0	3.00	19.5	1.00	-10.4	-4.00
7	37.5	-3.50	-9.6	1.00	18.5	-2.15	-26.2	2.00
8	150.3	-7.20	-228.9	-1.48	103.7	-4.80	-114.4	2.00

Summary of results (continued)

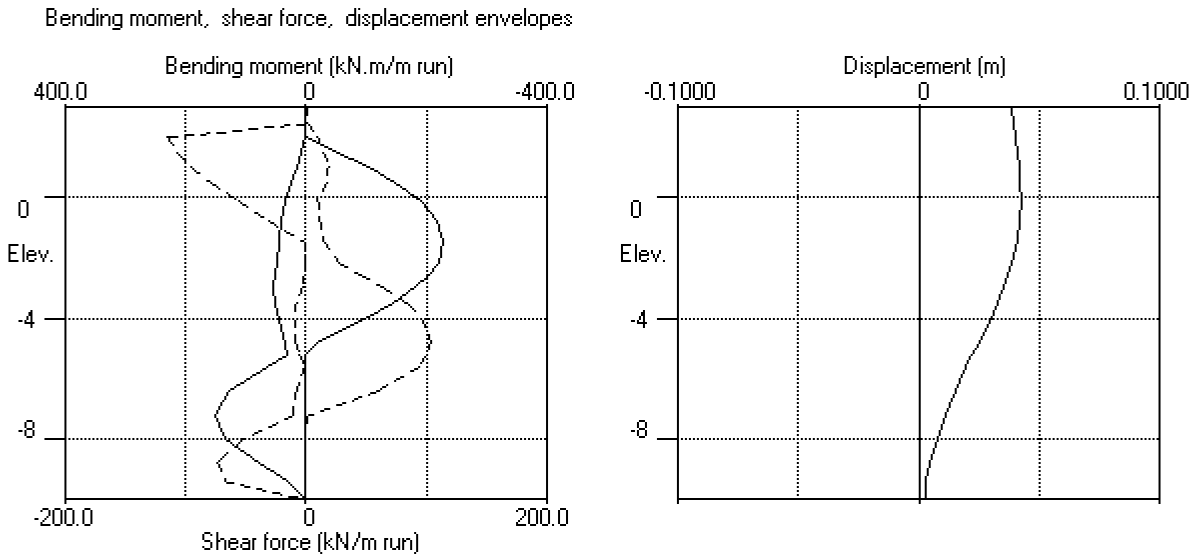
Maximum and minimum displacement at each stage

Stage no.	Displacement maximum m	elev.	minimum m	elev.	Stage description
1	0.006	3.00	0.000	3.00	Apply surcharge no.1 at elev. 2.50
2	0.009	3.00	0.000	3.00	Apply surcharge no.2 at elev. 2.50
3	0.009	3.00	0.000	3.00	Apply surcharge no.3 at elev. 2.50
4	0.009	3.00	0.000	3.00	Apply surcharge no.4 at elev. 2.50
5	0.009	3.00	0.000	3.00	Apply surcharge no.5 at elev. 2.50
6	0.021	3.00	0.000	3.00	Excav. to elev. 1.50 on PASSIVE side
7	0.017	3.00	0.000	3.00	Install strut no.1 at elev. 2.00
8	0.042	-0.15	0.000	3.00	Excav. to elev. -3.00 on PASSIVE side

Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 2.00 kN/m run	kN/strut
7	35.36	106.07
8	120.00	359.99

Units: kN,m



Bending moment, shear force, displacement envelopes
Bending moment (kN.m/m run)
400.0
0
-400.0
0
Elev.
-4
-8
-200.0
0
200.0
Shear force (kN/m run)
Displacement (m)
-0.1000
0
0.1000
0
Elev.
-4
-8

Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types: Active side	Soil types: Passive side
1	2.50	1 Loose Sand	1 Loose Sand
2	-2.15	2 Medium Dense Sand	2 Medium Dense Sand
3	-5.65	3 Dense Sand	3 Dense Sand

SOIL PROPERTIES

-- Soil type -- No. Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1 Loose Sand	16.00	20000	0.500	OC	0.292	3.000	
				(0.300)	(0.000)	(0.000)	
2 Medium	18.00	40000	0.600	OC	0.245	3.537	
Dense Sand				(0.300)	(0.000)	(0.000)	
3 Dense Sand	20.00	80000	0.500	OC	0.224	3.852	
				(0.300)	(0.000)	(0.000)	

Additional soil parameters associated with Ka and Kp

------- Soil type ------- No. Description	Ka: Soil friction angle	Ka: Wall adhesion coeff.	Ka: Back-fill angle	Kp: Soil friction angle	Kp: Wall adhesion coeff.	Kp: Back-fill angle
1 Loose Sand	30.00	0.500	0.00	30.00	0.000	0.00
2 Medium Dense Sand	34.00	0.500	0.00	34.00	0.000	0.00
3 Dense Sand	36.00	0.500	0.00	36.00	0.000	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	3.00	0.50

Automatic water pressure balancing at toe of wall : No

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = -8.00
Maximum finite element length = 0.60 m
Youngs modulus of wall E = 2.0000E+08 kN/m2
Moment of inertia of wall I = 3.6230E-04 m4/m run
E.I = 72460 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	2.00	3.00	0.000363	2.000E+08	3.50	45.00	150.0	No

SURCHARGE LOADS

Surch-arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Far edge	Equiv. soil type	Partial factor/ Category
1	2.50	5.80(A)	50.00	15.80	5.00	=	N/A	N/A
2	2.50	0.00(A)	50.00	5.80	0.00	100.00	N/A	N/A
3	2.50	5.80(A)	50.00	5.30	72.00	=	N/A	N/A
4	2.50	11.10(A)	50.00	6.70	128.00	134.40	N/A	N/A
5	2.50	17.80(A)	50.00	3.70	134.40	=	N/A	N/A

Note: A = Active side, P = Passive 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 no.	Stage description
1	Apply surcharge no.1 at elevation 2.50
2	Apply surcharge no.2 at elevation 2.50
3	Apply surcharge no.3 at elevation 2.50
4	Apply surcharge no.4 at elevation 2.50
5	Apply surcharge no.5 at elevation 2.50
6	Excavate to elevation 1.50 on PASSIVE side
7	Install strut or anchor no.1 at elevation 2.00
8	Excavate to elevation -3.00 on PASSIVE side

FACTORS OF SAFETY and ANALYSIS OPTIONS

Stability analysis:
Method of analysis - CP2
Factor on passive for calculating wall depth = 1.50

Parameters for undrained strata:
Minimum equivalent fluid density = 5.00 kN/m3
Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:
Method - Subgrade reaction model using Influence Coefficients
Open Tension Crack analysis? - No
Non-linear Modulus Parameter (L) = 0 m

Boundary conditions:
Length of wall (normal to plane of analysis) = 30.00 m

Width of excavation on active side of wall = 100.00 m
Width of excavation on passive side of wall = 100.00 m

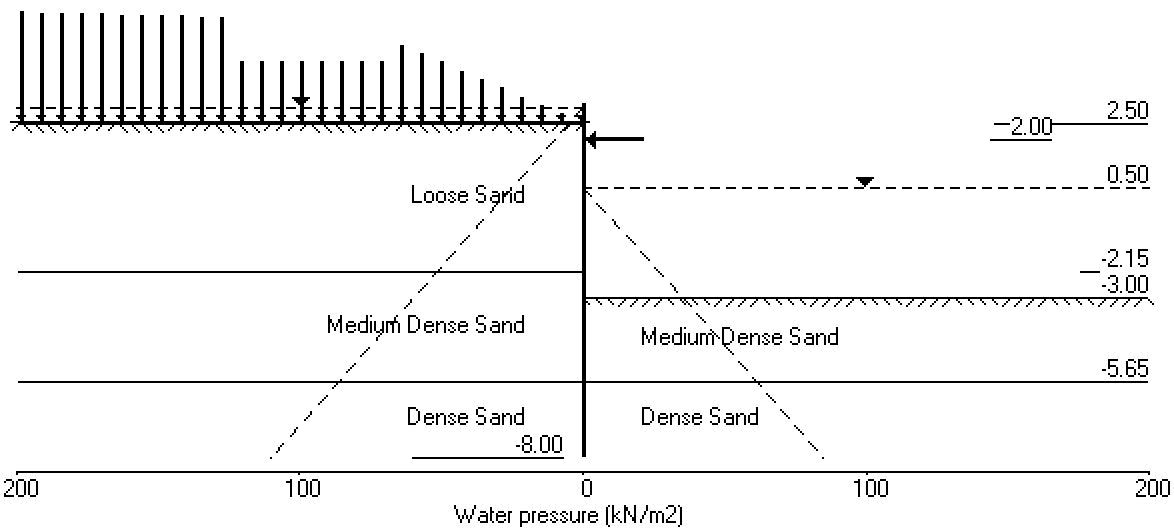
Distance to rigid boundary on active side = 100.00 m
Distance to rigid boundary on passive side = 100.00 m

OUTPUT OPTIONS

Stage no.	Stage description	Output options: Displacement Bending mom. Shear force	Active, Passive pressures	Graph. output
1	Apply surcharge no.1 at elev. 2.50	No	No	No
2	Apply surcharge no.2 at elev. 2.50	No	No	No
3	Apply surcharge no.3 at elev. 2.50	No	No	No
4	Apply surcharge no.4 at elev. 2.50	No	No	No
5	Apply surcharge no.5 at elev. 2.50	No	No	No
6	Excav. to elev. 1.50 on PASSIVE side	No	No	No
7	Install strut no.1 at elev. 2.00	No	No	No
8	Excav. to elev. -3.00 on PASSIVE side	No	No	No
*	Summary output	Yes	-	Yes

Units: kN,m

Stage No.8 Excav. to elev. -3.00 on PASSIVE side



Units: kN,m

Stage No. 8 Excavate to elevation -3.00 on PASSIVE side

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	FoS for toe elev. = -8.00: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.500: Toe elev.	Wall Penetr -ation
8	2.50	-3.00	2.00	1.375	n/a	***	***

Legend: *** Result not found

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Node no.	Y coord	Nett pressure kN/m2	Wall disp. m	Wall rotation rad.	Shear force kN/m	Bending moment kN.m/m	Strut forces kN/m
1	3.00	0.00	0.038	-7.46E-03	0.0	0.0	
2	2.50	5.00	0.042	-7.46E-03	1.3	0.2	
3	2.00	12.48	0.045	-7.46E-03	5.6	1.8	133.5
		12.48	0.045	-7.46E-03	-127.9	1.8	
4	1.50	19.92	0.049	-7.26E-03	-119.8	-60.1	
5	1.00	27.30	0.053	-6.65E-03	-108.0	-117.6	
6	0.50	34.61	0.056	-5.66E-03	-92.5	-167.6	
7	-0.05	37.03	0.059	-4.23E-03	-72.8	-211.6	
8	-0.60	39.33	0.060	-2.49E-03	-51.8	-246.0	
9	-1.20	41.69	0.061	-3.58E-04	-27.5	-270.0	
10	-1.68	43.47	0.061	1.43E-03	-7.2	-278.4	
11	-2.15	45.16	0.060	3.25E-03	13.8	-276.7	
		41.92	0.060	3.25E-03	13.8	-276.7	
12	-2.58	43.35	0.058	4.85E-03	31.9	-266.9	
13	-3.00	44.72	0.056	6.36E-03	50.6	-249.2	
14	-3.60	29.61	0.051	8.27E-03	72.9	-212.1	
15	-4.20	14.43	0.046	9.83E-03	86.2	-164.0	
16	-4.80	-0.82	0.040	1.09E-02	90.2	-111.0	
17	-5.23	-11.67	0.035	1.15E-02	87.6	-73.2	
18	-5.65	-22.54	0.030	1.18E-02	80.3	-37.4	
		-31.60	0.030	1.18E-02	80.3	-37.4	
19	-6.13	-48.54	0.024	1.19E-02	61.3	-3.6	
20	-6.60	-65.50	0.019	1.19E-02	34.2	19.2	
21	-7.20	-86.97	0.011	1.17E-02	-11.5	26.4	
22	-7.60	-43.47	0.007	1.16E-02	-37.6	17.3	
23	-8.00	231.66	0.002	1.15E-02	0.0	0.0	

Strut force at elev. 2.00 = 133.48 kN/m run = 400.43 kN/strut (horiz.)
= 566.29 kN/strut (incl.)

(continued)

Stage No.8 Excavate to elevation -3.00 on PASSIVE side

Units: kN,m

Summary of results

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

				FoS for toe elev. = -8.00		Toe elev. for FoS = 1.500	
Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	Factor of Safety	Moment equilib. at elev.	Toe elev.	Wall Penetr -ation
1	2.50	2.50	Cant.	4.365	-6.97	1.16	1.34
2	2.50	2.50	Cant.	3.624	-7.16	1.11	1.39
3	2.50	2.50	Cant.	3.536	-7.30	1.11	1.39
4	2.50	2.50	Cant.	3.504	-7.37	1.11	1.39
5	2.50	2.50	Cant.	3.499	-7.39	1.11	1.39
6	2.50	1.50	Cant.	2.165	-7.34	-4.53	6.03
7	2.50	1.50	2.00	4.032	n/a	-0.91	2.41
8	2.50	-3.00	2.00	1.375	n/a	***	***

Legend: *** Result not found

Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement maximum m	Displacement minimum m	Bending moment maximum kN.m/m	Bending moment minimum kN.m/m	Shear force maximum kN/m	Shear force minimum kN/m
1	3.00	0.038	0.000	0.0	0.0	0.0	0.0
2	2.50	0.042	0.000	0.2	0.0	1.3	0.0
3	2.00	0.045	0.000	2.5	0.0	9.9	-127.9
4	1.50	0.049	0.000	7.2	-60.1	13.7	-119.8
5	1.00	0.053	0.000	15.8	-117.6	19.5	-108.0
6	0.50	0.056	0.000	25.3	-167.6	17.0	-92.5
7	-0.05	0.059	0.000	33.8	-211.6	11.2	-72.8
8	-0.60	0.060	0.000	36.8	-246.0	9.1	-51.8
9	-1.20	0.061	0.000	37.6	-270.0	8.5	-27.5
10	-1.68	0.061	0.000	38.2	-278.4	9.6	-7.2
11	-2.15	0.060	0.000	40.0	-276.7	13.8	0.0
12	-2.58	0.058	0.000	40.7	-266.9	31.9	-2.1
13	-3.00	0.056	0.000	38.8	-249.2	50.6	-7.4
14	-3.60	0.051	0.000	32.3	-212.1	72.9	-12.4
15	-4.20	0.046	0.000	24.0	-164.0	86.2	-13.8
16	-4.80	0.040	0.000	16.1	-111.0	90.2	-10.7
17	-5.23	0.035	0.000	12.5	-73.2	87.6	-6.4
18	-5.65	0.030	0.000	11.5	-37.4	80.3	-0.6
19	-6.13	0.024	0.000	10.6	-3.6	61.3	-4.4
20	-6.60	0.019	0.000	19.2	0.0	34.2	-6.5
21	-7.20	0.011	0.000	26.4	0.0	0.0	-11.5
22	-7.60	0.007	0.000	17.3	0.0	0.0	-37.6
23	-8.00	0.002	0.000	0.0	0.0	0.0	0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment maximum kN.m/m	elev.	Bending moment minimum kN.m/m	elev.	Shear force maximum kN/m	elev.	Shear force minimum kN/m	elev.
1	10.0	-3.00	-0.4	-0.05	6.8	-2.15	-2.8	-4.20
2	7.6	-3.60	-7.0	-0.60	10.8	-2.15	-5.1	1.00
3	5.3	-6.13	-8.3	-0.60	10.8	-2.15	-5.5	1.00
4	5.2	-6.13	-8.5	-0.60	10.8	-2.15	-5.6	1.00
5	5.2	-6.13	-8.5	-0.60	10.8	-2.15	-5.6	1.00
6	40.7	-2.58	0.0	3.00	19.5	1.00	-13.8	-4.20
7	29.7	-3.00	-7.7	1.00	12.4	-2.15	-25.5	2.00
8	26.4	-7.20	-278.4	-1.68	90.2	-4.80	-127.9	2.00

Summary of results (continued)

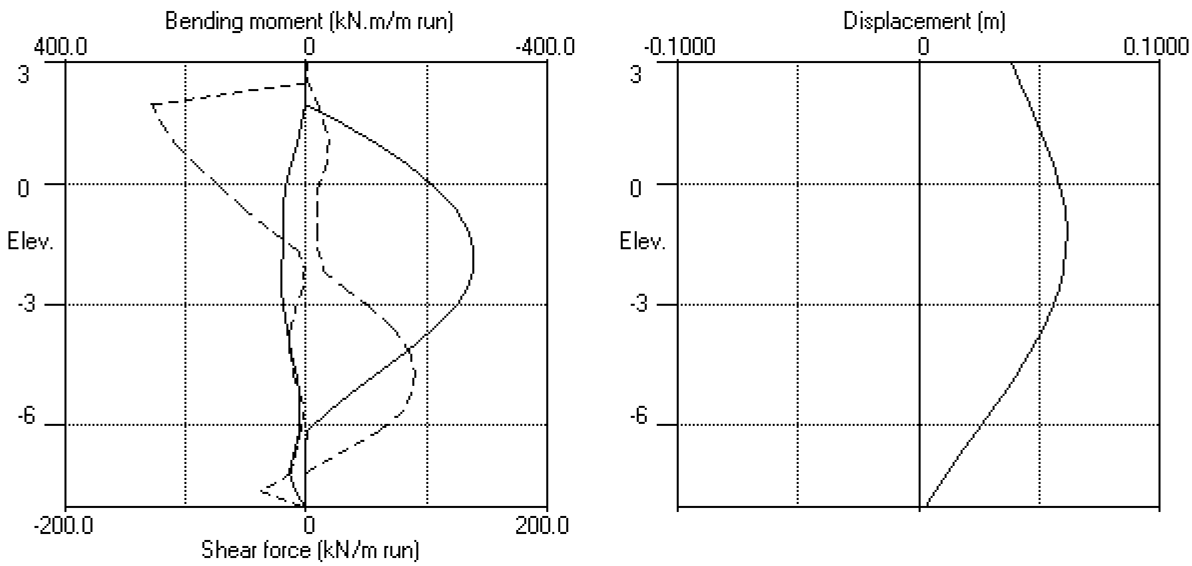
Maximum and minimum displacement at each stage

Stage no.	Displacement maximum (m)	elev.	Displacement minimum (m)	elev.	Stage description
1	0.006	3.00	0.000	3.00	Apply surcharge no.1 at elev. 2.50
2	0.009	3.00	0.000	3.00	Apply surcharge no.2 at elev. 2.50
3	0.009	3.00	0.000	3.00	Apply surcharge no.3 at elev. 2.50
4	0.009	3.00	0.000	3.00	Apply surcharge no.4 at elev. 2.50
5	0.009	3.00	0.000	3.00	Apply surcharge no.5 at elev. 2.50
6	0.024	3.00	0.000	3.00	Excav. to elev. 1.50 on PASSIVE side
7	0.019	3.00	0.000	3.00	Install strut no.1 at elev. 2.00
8	0.061	-1.20	0.000	3.00	Excav. to elev. -3.00 on PASSIVE side

Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 2.00 kN/m run	kN/strut
7	35.36	106.07
8	133.48	400.43

Bending moment, shear force, displacement envelopes



Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types: Active side	Soil types: Passive side
1	2.50	1 Loose Sand	1 Loose Sand
2	-2.15	2 Medium Dense Sand	2 Medium Dense Sand
3	-5.65	3 Dense Sand	3 Dense Sand

SOIL PROPERTIES

-- Soil type -- No. Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1 Loose Sand	16.00	20000	0.500	OC	0.292	3.000	
				(0.300)	(0.000)	(0.000)	
2 Medium Dense Sand	18.00	40000	0.600	OC	0.245	3.537	
				(0.300)	(0.000)	(0.000)	
3 Dense Sand	20.00	80000	0.500	OC	0.224	3.852	
				(0.300)	(0.000)	(0.000)	

Additional soil parameters associated with Ka and Kp

------- Soil type ------- No. Description	Ka: Soil friction angle	Ka: Wall adhesion coeff.	Ka: Back-fill angle	Kp: Soil friction angle	Kp: Wall adhesion coeff.	Kp: Back-fill angle
1 Loose Sand	30.00	0.500	0.00	30.00	0.000	0.00
2 Medium Dense Sand	34.00	0.500	0.00	34.00	0.000	0.00
3 Dense Sand	36.00	0.500	0.00	36.00	0.000	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	3.00	0.50

Automatic water pressure balancing at toe of wall : No

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = -8.00
Maximum finite element length = 0.60 m
Youngs modulus of wall E = 2.0000E+08 kN/m2
Moment of inertia of wall I = 1.1567E-03 m4/m run
(Arcelor AZ48) E.I = 231340 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	2.00	3.00	0.000363	2.000E+08	3.50	45.00	150.0	No

SURCHARGE LOADS

Surch-arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Surcharge kN/m2 Far edge	Equiv. soil type	Partial factor/ Category
1	2.50	5.80(A)	50.00	15.80	5.00	=	N/A	N/A
2	2.50	0.00(A)	50.00	5.80	0.00	100.00	N/A	N/A
3	2.50	5.80(A)	50.00	5.30	72.00	=	N/A	N/A
4	2.50	11.10(A)	50.00	6.70	128.00	134.40	N/A	N/A
5	2.50	17.80(A)	50.00	3.70	134.40	=	N/A	N/A

Note: A = Active side, P = Passive 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 no.	Stage description
1	Apply surcharge no.1 at elevation 2.50
2	Apply surcharge no.2 at elevation 2.50
3	Apply surcharge no.3 at elevation 2.50
4	Apply surcharge no.4 at elevation 2.50
5	Apply surcharge no.5 at elevation 2.50
6	Excavate to elevation 1.50 on PASSIVE side
7	Install strut or anchor no.1 at elevation 2.00
8	Excavate to elevation -3.00 on PASSIVE side

FACTORS OF SAFETY and ANALYSIS OPTIONS

Stability analysis:
Method of analysis - CP2
Factor on passive for calculating wall depth = 1.50

Parameters for undrained strata:
Minimum equivalent fluid density = 5.00 kN/m3
Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:
Method - Subgrade reaction model using Influence Coefficients
Open Tension Crack analysis? - No
Non-linear Modulus Parameter (L) = 0 m

Boundary conditions:
Length of wall (normal to plane of analysis) = 30.00 m

Width of excavation on active side of wall = 100.00 m
Width of excavation on passive side of wall = 100.00 m

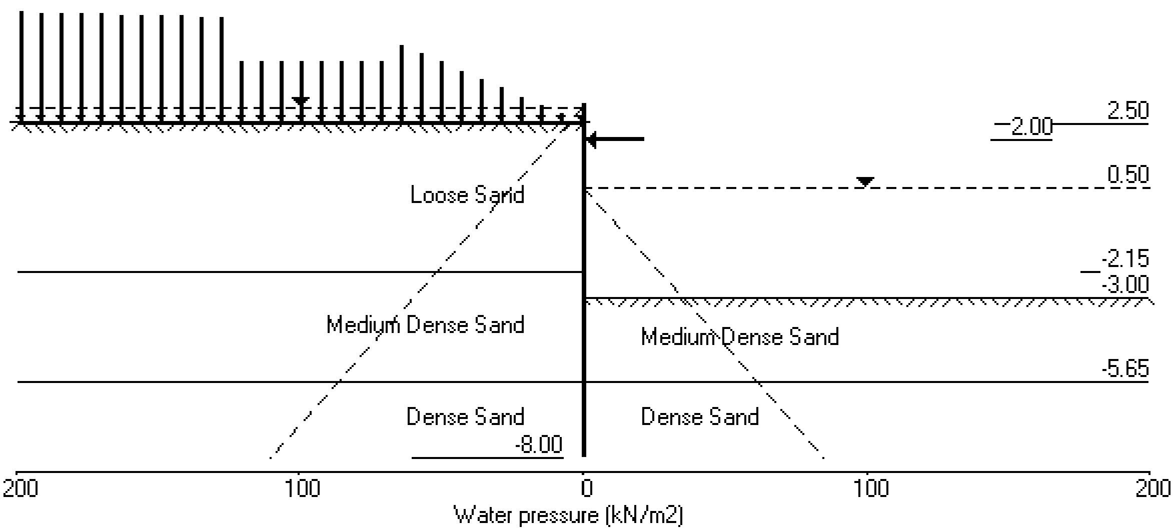
Distance to rigid boundary on active side = 100.00 m
Distance to rigid boundary on passive side = 100.00 m

OUTPUT OPTIONS

Stage no.	Stage description	Displacement Bending mom. Shear force	Active, Passive pressures	Graph. output
1	Apply surcharge no.1 at elev. 2.50	No	No	No
2	Apply surcharge no.2 at elev. 2.50	No	No	No
3	Apply surcharge no.3 at elev. 2.50	No	No	No
4	Apply surcharge no.4 at elev. 2.50	No	No	No
5	Apply surcharge no.5 at elev. 2.50	No	No	No
6	Excav. to elev. 1.50 on PASSIVE side	No	No	No
7	Install strut no.1 at elev. 2.00	No	No	No
8	Excav. to elev. -3.00 on PASSIVE side	No	No	No
*	Summary output	Yes	-	Yes

Units: kN,m

Stage No.8 Excav. to elev. -3.00 on PASSIVE side



Units: kN,m

Stage No. 8 Excavate to elevation -3.00 on PASSIVE side

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

Stage No.	G.L. Act.	G.L. Pass.	Strut Elev.	FoS for toe elev. = -8.00: Factor of Safety	FoS for toe elev. = -8.00: Moment equilib. at elev.	Toe elev. for FoS = 1.500: Toe elev.	Toe elev. for FoS = 1.500: Wall Penetr -ation
8	2.50	-3.00	2.00	1.375	n/a	***	***

Legend: *** Result not found

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Node no.	Y coord	Nett pressure kN/m2	Wall disp. m	Wall rotation rad.	Shear force kN/m	Bending moment kN.m/m	Strut forces kN/m
1	3.00	0.00	0.044	2.96E-04	0.0	-0.0	
2	2.50	5.00	0.044	2.96E-04	1.3	0.2	
3	2.00	12.48	0.044	2.94E-04	5.6	2.0	135.6
		12.48	0.044	2.94E-04	-130.0	2.0	
4	1.50	19.92	0.044	3.58E-04	-121.9	-61.1	
5	1.00	27.30	0.043	5.53E-04	-110.1	-119.6	
6	0.50	34.61	0.043	8.66E-04	-94.6	-169.9	
7	-0.05	37.03	0.042	1.32E-03	-74.9	-216.7	
8	-0.60	39.33	0.042	1.88E-03	-53.9	-252.2	
9	-1.20	41.69	0.040	2.56E-03	-29.6	-277.4	
10	-1.68	43.47	0.039	3.14E-03	-9.4	-286.7	
11	-2.15	45.16	0.037	3.73E-03	11.6	-286.3	
		41.92	0.037	3.73E-03	11.6	-286.3	
12	-2.58	43.35	0.036	4.25E-03	29.8	-277.2	
13	-3.00	44.72	0.034	4.74E-03	48.5	-260.4	
14	-3.60	29.61	0.031	5.37E-03	70.8	-224.5	
15	-4.20	14.43	0.027	5.89E-03	84.0	-177.5	
16	-4.80	-0.82	0.023	6.29E-03	88.1	-125.9	
17	-5.23	-11.67	0.021	6.48E-03	85.4	-89.0	
18	-5.65	-22.54	0.018	6.62E-03	78.1	-54.2	
		-31.60	0.018	6.62E-03	78.1	-54.2	
19	-6.13	-48.54	0.015	6.69E-03	59.1	-21.3	
20	-6.60	-65.50	0.012	6.71E-03	32.0	0.5	
21	-7.20	-50.31	0.008	6.70E-03	-2.7	11.9	
22	-7.60	-12.43	0.005	6.68E-03	-15.3	7.1	
23	-8.00	88.75	0.002	6.68E-03	0.0	0.0	

Strut force at elev. 2.00 = 135.64 kN/m run = 406.92 kN/strut (horiz.)
= 575.47 kN/strut (incl.)

(continued)

Stage No.8 Excavate to elevation -3.00 on PASSIVE side

Units: kN,m

Summary of results

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

				FoS for toe elev. = -8.00		Toe elev. for FoS = 1.500	
Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	Factor of Safety	Moment equilib. at elev.	Toe elev.	Wall Penetr -ation
1	2.50	2.50	Cant.	4.365	-6.97	1.16	1.34
2	2.50	2.50	Cant.	3.624	-7.16	1.11	1.39
3	2.50	2.50	Cant.	3.536	-7.30	1.11	1.39
4	2.50	2.50	Cant.	3.504	-7.37	1.11	1.39
5	2.50	2.50	Cant.	3.499	-7.39	1.11	1.39
6	2.50	1.50	Cant.	2.165	-7.34	-4.53	6.03
7	2.50	1.50	2.00	4.032	n/a	-0.91	2.41
8	2.50	-3.00	2.00	1.375	n/a	***	***

Legend: *** Result not found

Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement maximum m	Displacement minimum m	Bending moment maximum kN.m/m	Bending moment minimum kN.m/m	Shear force maximum kN/m	Shear force minimum kN/m
1	3.00	0.044	0.000	0.0	-0.0	0.0	0.0
2	2.50	0.044	0.000	0.2	0.0	1.3	0.0
3	2.00	0.044	0.000	2.5	0.0	8.5	-130.0
4	1.50	0.044	0.000	7.2	-61.1	13.7	-121.9
5	1.00	0.043	0.000	15.8	-119.6	19.5	-110.1
6	0.50	0.043	0.000	26.2	-169.9	17.6	-94.6
7	-0.05	0.042	0.000	34.2	-216.7	12.6	-74.9
8	-0.60	0.042	0.000	40.2	-252.2	10.1	-53.9
9	-1.20	0.040	0.000	45.8	-277.4	13.7	-29.6
10	-1.68	0.039	0.000	50.7	-286.7	17.2	-9.4
11	-2.15	0.037	0.000	56.6	-286.3	21.3	0.0
12	-2.58	0.036	0.000	61.1	-277.2	29.8	0.0
13	-3.00	0.034	0.000	62.3	-260.4	48.5	-0.8
14	-3.60	0.031	0.000	59.2	-224.5	70.8	-8.3
15	-4.20	0.027	0.000	52.5	-177.5	84.0	-13.4
16	-4.80	0.023	0.000	43.5	-125.9	88.1	-14.7
17	-5.23	0.021	0.000	37.4	-89.0	85.4	-12.8
18	-5.65	0.018	0.000	32.6	-54.2	78.1	-8.9
19	-6.13	0.015	0.000	27.0	-21.3	59.1	-14.5
20	-6.60	0.012	0.000	19.0	0.0	32.0	-17.3
21	-7.20	0.008	0.000	11.9	0.0	0.0	-15.6
22	-7.60	0.005	0.000	7.1	0.0	0.0	-15.3
23	-8.00	0.002	0.000	0.0	0.0	0.0	-0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment maximum kN.m/m	elev.	Bending moment minimum kN.m/m	elev.	Shear force maximum kN/m	elev.	Shear force minimum kN/m	elev.
1	16.7	-3.00	0.0	3.00	8.6	-2.15	-5.1	-6.60
2	12.4	-3.60	-5.4	-0.60	12.1	-2.15	-5.4	-6.60
3	8.4	-3.60	-7.4	-0.60	11.7	-2.15	-5.3	1.00
4	7.3	-3.60	-7.9	-0.60	11.6	-2.15	-5.4	1.00
5	7.2	-3.60	-7.9	-0.60	11.6	-2.15	-5.4	1.00
6	62.3	-3.00	-0.0	3.00	19.5	1.00	-17.3	-6.60
7	37.6	-3.60	-11.2	1.00	21.3	-2.15	-26.8	2.00
8	11.9	-7.20	-286.7	-1.68	88.1	-4.80	-130.0	2.00

Summary of results (continued)

Maximum and minimum displacement at each stage

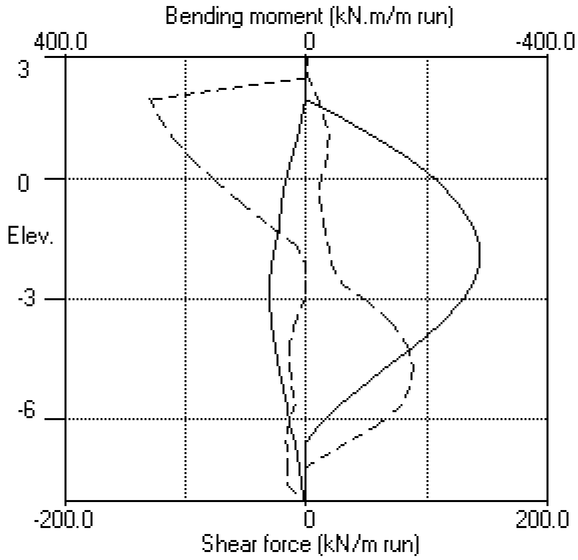
Stage no.	Displacement maximum m	elev.	minimum m	elev.	Stage description
1	0.006	3.00	0.000	3.00	Apply surcharge no.1 at elev. 2.50
2	0.009	3.00	0.000	3.00	Apply surcharge no.2 at elev. 2.50
3	0.010	3.00	0.000	3.00	Apply surcharge no.3 at elev. 2.50
4	0.010	3.00	0.000	3.00	Apply surcharge no.4 at elev. 2.50
5	0.010	3.00	0.000	3.00	Apply surcharge no.5 at elev. 2.50
6	0.020	3.00	0.000	3.00	Excav. to elev. 1.50 on PASSIVE side
7	0.016	3.00	0.000	3.00	Install strut no.1 at elev. 2.00
8	0.044	3.00	0.000	3.00	Excav. to elev. -3.00 on PASSIVE side

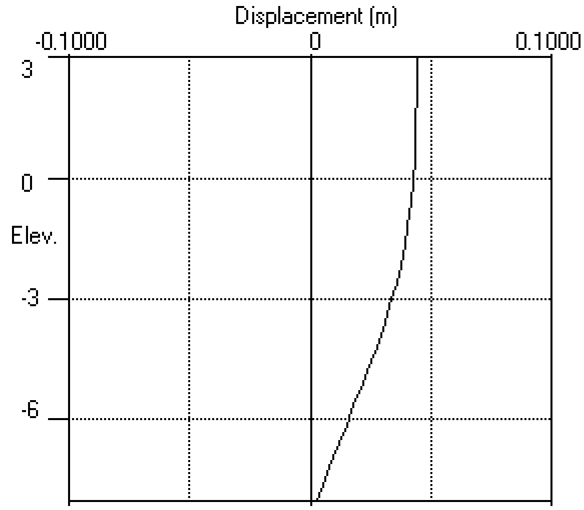
Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 2.00 kN/m run	kN/strut
7	35.36	106.07
8	135.64	406.92

Units: kN,m

Bending moment, shear force, displacement envelopes





Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types: Active side	Soil types: Passive side
1	2.50	1 Loose Sand	1 Loose Sand
2	-2.15	2 Medium Dense Sand	2 Medium Dense Sand
3	-5.65	3 Dense Sand	3 Dense Sand

SOIL PROPERTIES

-- Soil type -- No. Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1 Loose Sand	16.00	20000	0.500	OC (0.300)	0.292 (0.000)	3.000 (0.000)	
2 Medium Dense Sand	18.00	40000	0.600	OC (0.300)	0.245 (0.000)	3.537 (0.000)	
3 Dense Sand	20.00	80000	0.500	OC (0.300)	0.224 (0.000)	3.852 (0.000)	

Additional soil parameters associated with Ka and Kp

------- Soil type -------	--- parameters for Ka ---			--- parameters for Kp ---		
No. Description	Soil friction angle	Wall adhesion coeff.	Back-fill angle	Soil friction angle	Wall adhesion coeff.	Back-fill angle
1 Loose Sand	30.00	0.500	0.00	30.00	0.000	0.00
2 Medium Dense Sand	34.00	0.500	0.00	34.00	0.000	0.00
3 Dense Sand	36.00	0.500	0.00	36.00	0.000	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	3.00	0.50

Automatic water pressure balancing at toe of wall : No

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = -8.00
Maximum finite element length = 0.60 m
Youngs modulus of wall E = 2.0000E+07 kN/m2
Moment of inertia of wall I = 6.3600E-03 m4/m run
E.I = 127200 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	2.00	3.00	0.000363	2.000E+08	3.50	45.00	150.0	No

SURCHARGE LOADS

Surch -arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Far edge	Equiv. soil type	Partial factor/ Category
1	2.50	5.80(A)	50.00	15.80	5.00	=	N/A	N/A
2	2.50	0.00(A)	50.00	5.80	0.00	100.00	N/A	N/A
3	2.50	5.80(A)	50.00	5.30	72.00	=	N/A	N/A
4	2.50	11.10(A)	50.00	6.70	128.00	134.40	N/A	N/A
5	2.50	17.80(A)	50.00	3.70	134.40	=	N/A	N/A

Note: A = Active side, P = Passive 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 no.	Stage description
1	Apply surcharge no.1 at elevation 2.50
2	Apply surcharge no.2 at elevation 2.50
3	Apply surcharge no.3 at elevation 2.50
4	Apply surcharge no.4 at elevation 2.50
5	Apply surcharge no.5 at elevation 2.50
6	Excavate to elevation 1.50 on PASSIVE side
7	Install strut or anchor no.1 at elevation 2.00
8	Excavate to elevation -3.00 on PASSIVE side

FACTORS OF SAFETY and ANALYSIS OPTIONS

Stability analysis:
Method of analysis - CP2
Factor on passive for calculating wall depth = 1.50

Parameters for undrained strata:
Minimum equivalent fluid density = 5.00 kN/m3
Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:
Method - Subgrade reaction model using Influence Coefficients
Open Tension Crack analysis? - No
Non-linear Modulus Parameter (L) = 0 m

Boundary conditions:
Length of wall (normal to plane of analysis) = 30.00 m

Width of excavation on active side of wall = 100.00 m
Width of excavation on passive side of wall = 100.00 m

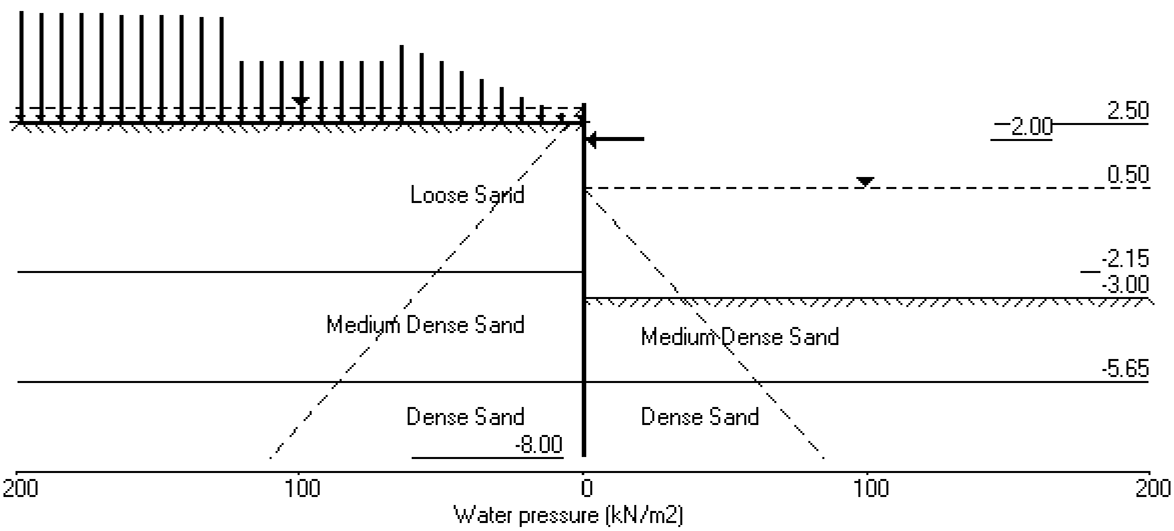
Distance to rigid boundary on active side = 100.00 m
Distance to rigid boundary on passive side = 100.00 m

OUTPUT OPTIONS

Stage no.	Stage description	Displacement Bending mom. Shear force	Active, Passive pressures	Graph. output
1	Apply surcharge no.1 at elev. 2.50	No	No	No
2	Apply surcharge no.2 at elev. 2.50	No	No	No
3	Apply surcharge no.3 at elev. 2.50	No	No	No
4	Apply surcharge no.4 at elev. 2.50	No	No	No
5	Apply surcharge no.5 at elev. 2.50	No	No	No
6	Excav. to elev. 1.50 on PASSIVE side	No	No	No
7	Install strut no.1 at elev. 2.00	No	No	No
8	Excav. to elev. -3.00 on PASSIVE side	No	No	No
*	Summary output	Yes	-	Yes

Units: kN,m

Stage No.8 Excav. to elev. -3.00 on PASSIVE side



Units: kN,m

Stage No. 8 Excavate to elevation -3.00 on PASSIVE side

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	FoS for toe elev. = -8.00: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.500: Toe elev.	Wall Penetr -ation
8	2.50	-3.00	2.00	1.375	n/a	***	***

Legend: *** Result not found

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Node no.	Y coord	Nett pressure kN/m2	Wall disp. m	Wall rotation rad.	Shear force kN/m	Bending moment kN.m/m	Strut forces kN/m
1	3.00	0.00	0.042	-2.66E-03	0.0	0.0	
2	2.50	5.00	0.043	-2.66E-03	1.3	0.2	
3	2.00	12.48	0.044	-2.67E-03	5.6	1.9	134.7
		12.48	0.044	-2.67E-03	-129.1	1.9	
4	1.50	19.92	0.046	-2.55E-03	-121.0	-60.7	
5	1.00	27.30	0.047	-2.20E-03	-109.2	-118.7	
6	0.50	34.61	0.048	-1.63E-03	-93.7	-169.4	
7	-0.05	37.03	0.048	-8.06E-04	-74.0	-214.4	
8	-0.60	39.33	0.049	1.95E-04	-53.0	-249.4	
9	-1.20	41.69	0.048	1.43E-03	-28.7	-274.1	
10	-1.68	43.47	0.047	2.47E-03	-8.5	-283.0	
11	-2.15	45.16	0.046	3.52E-03	12.6	-282.1	
		41.92	0.046	3.52E-03	12.6	-282.1	
12	-2.58	43.35	0.044	4.45E-03	30.7	-272.7	
13	-3.00	44.72	0.042	5.33E-03	49.4	-255.6	
14	-3.60	29.61	0.038	6.45E-03	71.7	-219.1	
15	-4.20	14.43	0.034	7.37E-03	84.9	-171.7	
16	-4.80	-0.82	0.030	8.06E-03	89.0	-119.5	
17	-5.23	-11.67	0.026	8.40E-03	86.3	-82.2	
18	-5.65	-22.54	0.023	8.61E-03	79.1	-47.0	
		-31.60	0.023	8.61E-03	79.1	-47.0	
19	-6.13	-48.54	0.018	8.72E-03	60.0	-13.7	
20	-6.60	-65.50	0.014	8.73E-03	33.0	8.4	
21	-7.20	-70.06	0.009	8.67E-03	-7.7	20.6	
22	-7.60	-27.17	0.006	8.62E-03	-27.2	11.8	
23	-8.00	162.93	0.002	8.60E-03	0.0	-0.0	

Strut force at elev. 2.00 = 134.71 kN/m run = 404.12 kN/strut (horiz.)
= 571.51 kN/strut (incl.)

(continued)

Stage No.8 Excavate to elevation -3.00 on PASSIVE side

Units: kN,m

Summary of results

STABILITY ANALYSIS of Fully Embedded Wall according to CP2 method

Factor of safety on gross pressure (excluding water pressure)

Stage No.	G.L. Act.	G.L. Pass.	Strut Elev.	FoS for toe elev. = -8.00: Factor of Safety	FoS for toe elev. = -8.00: Moment equilib. at elev.	Toe elev. for FoS = 1.500: Toe elev.	Toe elev. for FoS = 1.500: Wall Penetr -ation
1	2.50	2.50	Cant.	4.365	-6.97	1.16	1.34
2	2.50	2.50	Cant.	3.624	-7.16	1.11	1.39
3	2.50	2.50	Cant.	3.536	-7.30	1.11	1.39
4	2.50	2.50	Cant.	3.504	-7.37	1.11	1.39
5	2.50	2.50	Cant.	3.499	-7.39	1.11	1.39
6	2.50	1.50	Cant.	2.165	-7.34	-4.53	6.03
7	2.50	1.50	2.00	4.032	n/a	-0.91	2.41
8	2.50	-3.00	2.00	1.375	n/a	***	***

Legend: *** Result not found

Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 30.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 100.00 from wall
Passive side 100.00 from wall

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement maximum m	Displacement minimum m	Bending moment maximum kN.m/m	Bending moment minimum kN.m/m	Shear force maximum kN/m	Shear force minimum kN/m
1	3.00	0.042	0.000	0.0	-0.0	0.0	0.0
2	2.50	0.043	0.000	0.2	0.0	1.3	0.0
3	2.00	0.044	0.000	2.5	0.0	9.2	-129.1
4	1.50	0.046	0.000	7.2	-60.7	13.7	-121.0
5	1.00	0.047	0.000	15.8	-118.7	19.5	-109.2
6	0.50	0.048	0.000	25.3	-169.4	17.0	-93.7
7	-0.05	0.048	0.000	33.8	-214.4	10.6	-74.0
8	-0.60	0.049	0.000	38.1	-249.4	9.9	-53.0
9	-1.20	0.048	0.000	41.3	-274.1	11.7	-28.7
10	-1.68	0.047	0.000	43.9	-283.0	14.1	-8.5
11	-2.15	0.046	0.000	47.6	-282.1	17.3	0.0
12	-2.58	0.044	0.000	50.1	-272.7	30.7	0.0
13	-3.00	0.042	0.000	49.6	-255.6	49.4	-4.4
14	-3.60	0.038	0.000	44.6	-219.1	71.7	-10.8
15	-4.20	0.034	0.000	36.8	-171.7	84.9	-14.0
16	-4.80	0.030	0.000	28.1	-119.5	89.0	-13.1
17	-5.23	0.026	0.000	23.0	-82.2	86.3	-9.8
18	-5.65	0.023	0.000	19.8	-47.0	79.1	-4.8
19	-6.13	0.018	0.000	16.2	-13.7	60.0	-9.2
20	-6.60	0.014	0.000	11.1	0.0	33.0	-11.0
21	-7.20	0.009	0.000	20.6	0.0	0.0	-9.2
22	-7.60	0.006	0.000	11.8	0.0	0.0	-27.2
23	-8.00	0.002	0.000	0.0	-0.0	0.0	0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment maximum kN.m/m	elev.	Bending moment minimum kN.m/m	elev.	Shear force maximum kN/m	elev.	Shear force minimum kN/m	elev.
1	13.1	-3.00	-0.0	3.00	7.7	-2.15	-3.8	-6.60
2	9.8	-3.60	-6.3	-0.60	11.5	-2.15	-4.9	1.00
3	6.6	-3.60	-8.0	-0.60	11.2	-2.15	-5.4	1.00
4	6.1	-6.13	-8.3	-0.60	11.2	-2.15	-5.5	1.00
5	6.1	-6.13	-8.3	-0.60	11.2	-2.15	-5.5	1.00
6	50.1	-2.58	-0.0	-8.00	19.5	1.00	-14.0	-4.20
7	33.2	-3.00	-9.6	1.00	17.3	-2.15	-26.2	2.00
8	20.6	-7.20	-283.0	-1.68	89.0	-4.80	-129.1	2.00

Summary of results (continued)

Maximum and minimum displacement at each stage

Stage no.	Displacement maximum (m)	elev.	Displacement minimum (m)	elev.	Stage description
1	0.006	3.00	0.000	3.00	Apply surcharge no.1 at elev. 2.50
2	0.009	3.00	0.000	3.00	Apply surcharge no.2 at elev. 2.50
3	0.010	3.00	0.000	3.00	Apply surcharge no.3 at elev. 2.50
4	0.010	3.00	0.000	3.00	Apply surcharge no.4 at elev. 2.50
5	0.009	3.00	0.000	3.00	Apply surcharge no.5 at elev. 2.50
6	0.021	3.00	0.000	3.00	Excav. to elev. 1.50 on PASSIVE side
7	0.017	3.00	0.000	3.00	Install strut no.1 at elev. 2.00
8	0.049	-0.60	0.000	3.00	Excav. to elev. -3.00 on PASSIVE side

Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 2.00 kN/m run	Strut no. 1 at elev. 2.00 kN/strut
7	35.36	106.07
8	134.71	404.12

```
J.K. GEOTECHNICS                                                | Sheet No.
Program: WALLAP  Version 6.05  Revision A41.B56.R46             | Job No. 30243ZR
                             Licensed from GEOSOLVE             | Made by :   OBN
Data filename/Run ID: Section A-A Secant 600mm anchor Pile -8m
Wamberal Section A-A                                            | Date:27-02-2017
Secant 600mm Pile Wall with Anchor  Socketed at RL -8m          | Checked :
-----------------------------------------------------------------------------
                                                       Units: kN,m
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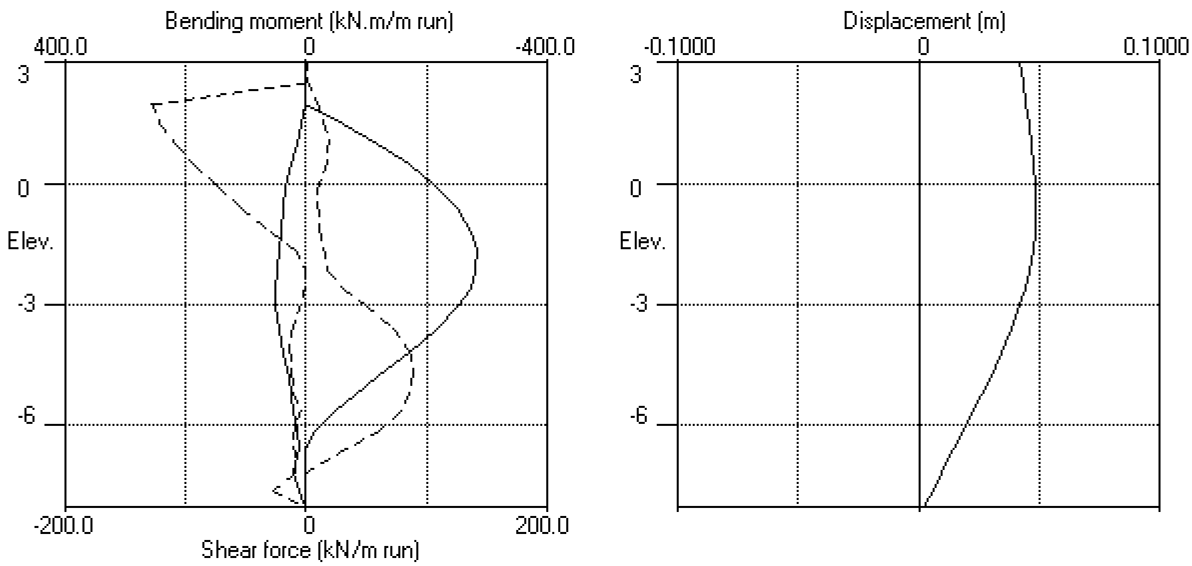
Bending moment, shear force, displacement envelopes



REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 to 2mm
Gravel	2 to 60mm

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N>30
  15, 30/40mm

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

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

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

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

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

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

The information provided on the charts comprise:

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

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

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

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

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

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

Two relatively similar tests are used:

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

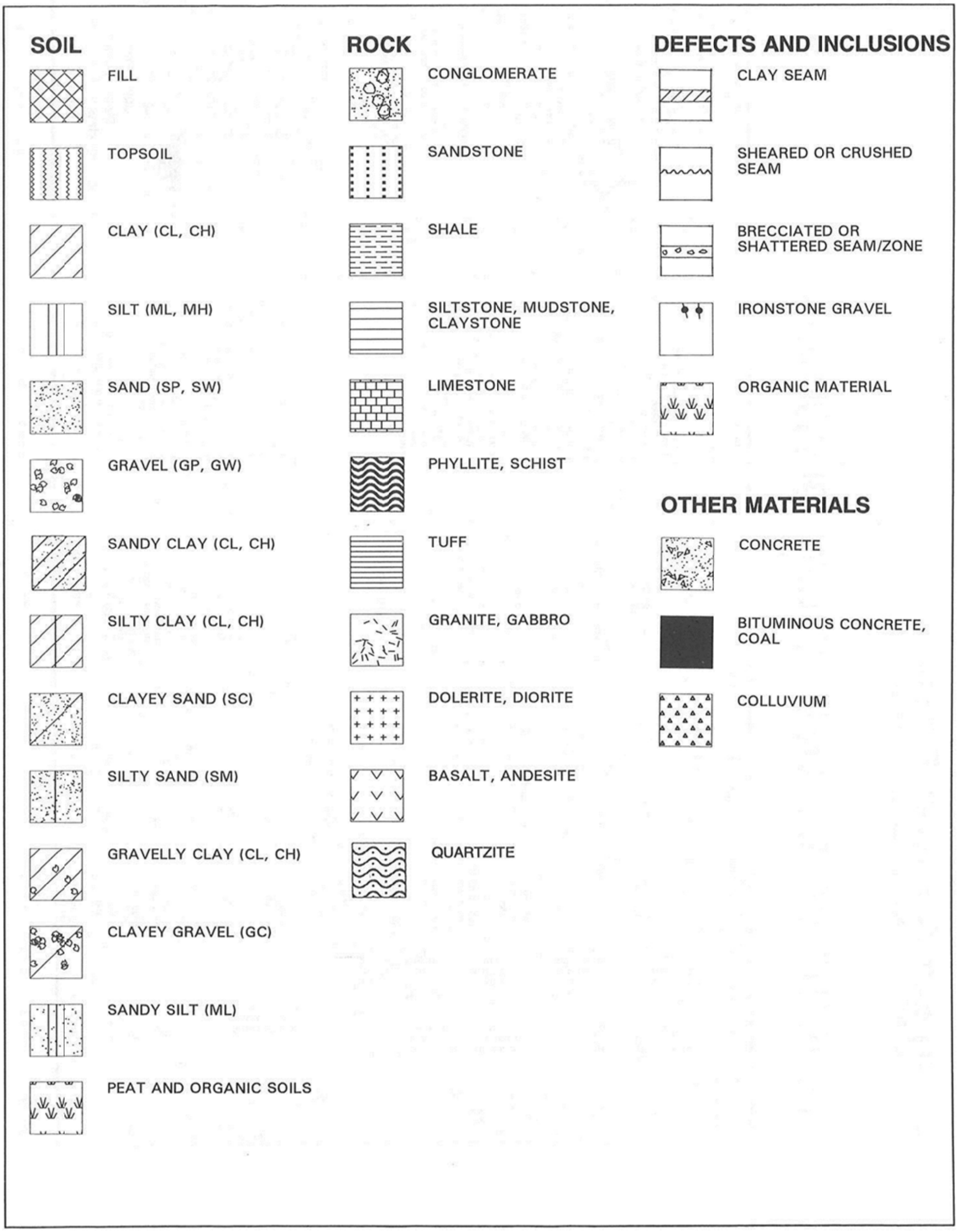
Requirements could range from:

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

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

iii) full time engineering presence on site.

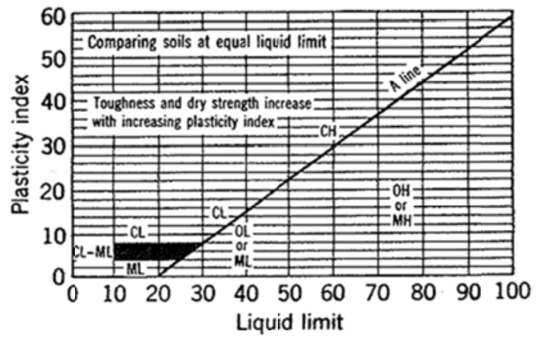
GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



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

Laboratory classification criteria (coarse-grained soils): Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols.

Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Identification Procedures on Fraction Smaller than 380 μm Sieve Size: Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
	Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
	Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	
Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
	High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	
	Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils	Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

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

LOG SYMBOLS

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

LOG SYMBOLS continued

ROCK MATERIAL WEATHERING CLASSIFICATION

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

ROCK STRENGTH

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

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

ABBREVIATIONS USED IN DEFECT DESCRIPTION

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