SOIL SURVEYS

EMBANKMENT STABILITY ASSESSMENT

PROJECT NO. 1-24684

MARCH, 2023

MULTI SPAN AUSTRALIA

PROPOSED STORAGE FACILITY &
RETAIL DEVELOPMENT

1 HOMESTEAD DRIVE, JIMBOOMBA

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1.0 INTRODUCTION

1.1 General

This report presents the results of the embankment slope stability assessment carried out by Soil Surveys Engineering Pty Limited (SSE) for the Proposed Storage Facility & Retail Development, at 1 Homestead Drive, Jimboomba.

The report was carried out following authorisation from Andrew Boyce of Multi Span Australia on 1st March, 2022.

The objectives of this investigation were to assess subsurface conditions at the site in accordance with the Scope of Services detailed in Section 2.0.

1.2 Proposed Development

It is understood that the proposed development is to consist of the construction of a Proposed Storage Facility & Retail Development at the above site.

The proposed building (Stage 1) is an ‘L’ shaped building approx. 70m x 60m and of two suspended levels with an on-grade ground floor.

General earthworks are expected to be minor (cuts and fills of < 0.5m) over the majority of the site, however excavation into the rear batter (embankment) will be required. The proposed ground floor slab in this area will be RL 50.635m. This suggests a cut of up to 2.0m (maximum).

2.0 SCOPE OF STUDY

The objective of this study would be to undertake a reanalysis of the embankment using the updated construction methodology with computer modelling.

3.0 GEOTECHNICAL INVESTIGATION

3.1 General

Soil Surveys undertook a geotechnical investigation and report in 2021 (ref 1-24684, 2021-11-25, BR VER 1). Refer to this report for borehole logs, laboratory test results and site plan. The following is a summary of the field investigation results.

3.2 Site Description

The site is situated at 1 Homestead Drive, Jimboomba (refer Figure 1). Access onto the site was off Commercial Circuit.

A significant fill embankment was noted along the northern side of the site. Homestead Drive is on top of the embankment.

At the time of the investigation, the site was free of any structures.

The site generally slopes to the south and east with a fall of between 5.5m and 6.0m from the north boundary to the toe of the northern embankment and 2.5m over the remainder of the site.

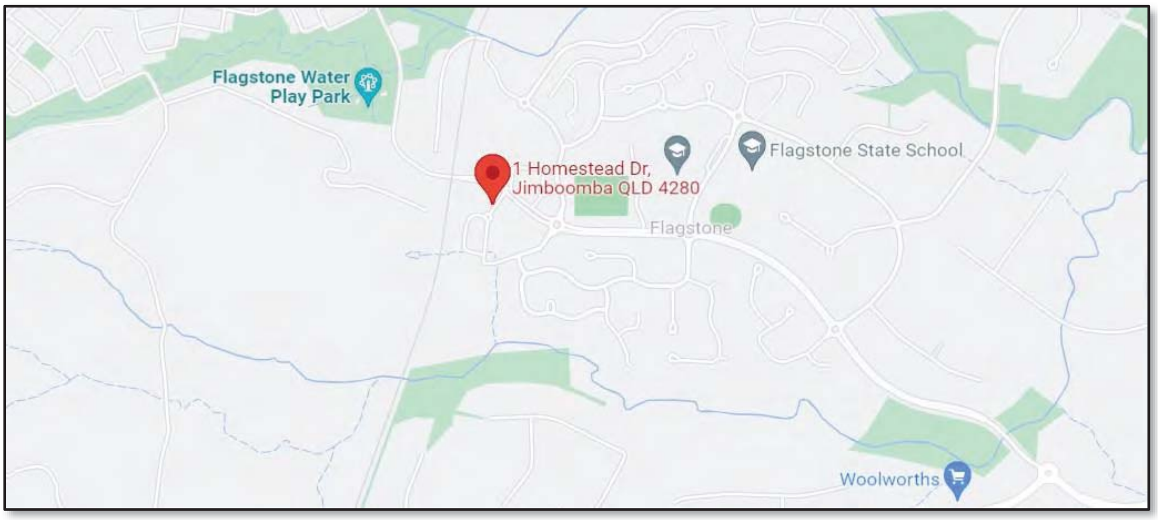

FIGURE 1 – SITE LOCATION

The area is well grassed with several small to medium sized trees on the embankment and long the western boundary.

Refer to following site photograph for typical site conditions.

PLATE 1 – SITE LOOKING NORTH SHOWING EMBANKMENT

3.3 Geotechnical Model

The subsurface profile intersected during the drilling program (in 2021) along the northern section of the site (at the base of the embankment) generally consisted of minor fill underlain by varying thickness of residual soils with weathered rock encountered at depths of between 0.7m (BH09) to 2.1m (BH06).

As access above the embankment was not possible due to the existing slope and road, three DCP's were undertaken upslope of the adjacent boreholes along the approximate alignment of the proposed cut. These reached refusal at depths of between 0.3m and 1.5m which was within the fill.

Groundwater was not encountered in the boreholes at the time of the investigation.

Refer to Soil Surveys Engineering's report 1-24684, 2021-11-25, BR VER 1 for further details.

4.0 EMBANKMENT STABILITY ASSESSMENT

4.1 General

A preliminary assessment of the potential effects of a temporary cut on the existing embankment at the northern end of the site was undertaken as part of the original geotechnical assessment.

Following more detailed design work a proposed construction methodology has been development and a review of the slope stability and the effect of the proposed works has been requested.

Sections from the original preliminary assessment have been included below where they are relevant to the recent assessment.

4.2 Embankment Information

As noted above, no drilling was possible into the embankment at the line of the proposed excavation. The design model was developed based on the following:-

- A review of a provided Level 1 report on the construction of the embankment (ref. (Morrison Geotechnic Ref. 10693 dated 16th June, 2016).
- The provided site contour survey (ref. Veris Dwg No. 30109-DP-25024 dated 22nd March, 2019).
- Three DCP tests (DCP1 to DCP3, refer Dwg. No. 1-24684-01) SSE has undertaken near the line of the proposed temporary cut.
- The geometry of the cut is based on the supplied drawings indicating the alignment and floor levels for the proposed structure in this area.
- A review of historical air photos of the site.

4.3 Embankment Assessment Data

Based on the above, the following information has been assessed:-

- Based on air photos and site observations, there may have been an area of elevated ground in the location of the embankment prior to earthworks, however this could not be confirmed.
- The road bridge is founded into weathered rock with shallow weathered rock exposed in the cutting under the bridge and around the bridge piers.
- Construction commenced on the embankment in late 2016/early 2017.
- The embankment appears to have been constructed in stages with the section to the east of BH09 completed by late 2016. The rest of the embankment to the west of this point was completed by mid-2017. Visual evidence would suggest that the construction methodology consisted of overfilling and cutting back to the final profile to ensure complete compaction to the final surface.
- A review of the contour survey suggests an even slope along the entire embankment with a slope of 1V:3H from the top to the base of the embankment.
- The DCP testing indicated good compaction from the surface and shallow refusal in DCP01 and DCP03. The upper 0.9m in DCP02 was lower in strength than expected but significantly improved below that to refusal at 1.53m. This appears to be close to the

change in construction stages noted above and may relate to the interface between these two stages. The point of maximum cut is to the west of this location closer to DCP3 which reached refusal at <0.5m.

- A review of the provided Level 1 report indicates:-
    - The embankment consisted of fill.
    - The fill was placed under Level One Inspection and testing as per AS3798 & Logan City Council Project Specifications.
    - Numerous test locations were marked on a supplied plan (but not able to refer to specific test results to a specific location).
    - Based on the colour coded test locations, the pink tests were between RL 45.0m and RL 49.99m and the light blue between RL 50.0m and RL 54.99m.
    - A review of the provided site contour suggests most tests were more than likely in the upper 1-2m of the embankment.
    - A review of all test results indicates Density Ratio (HILF) in the range 95% to 103%.
    - The report does not cover the fill immediately behind the abutments but this is likely to be outside the subject site.
    - All surfaces where fill was to be placed were stripped and cleared of all visible organic matter, deleterious, loose and unsuitable material to depths exposing competent ground. Based on the description in the report, the exposed surface was natural soils in the section of embankment on the subject site.
    - The fill material consisted of Sandy CLAY (CI) and Clayey SAND (SC).
    - The fill was placed in layers; however details of layer thickness was not provided.

4.4 Analysis Methodology of the Geotechnical Model

A geotechnical model was developed from the information noted above as well as the details of the site investigation undertaken for this report. A four-layer model was adopted consisting of the embankment fill, general site fill, residual soils and weathered rock. Material parameters were adopted based on an assessment of the provided information and experience with similar materials on other sites.

The model was analysed using a 2D slope stability computer program (Slide 2 V9). Worst likely parameters were adopted with respect the subsurface profile and water conditions.

The acceptance criteria for these types of assessment will depend upon the situation being assessed (short or long term), the parameters adopted and the consequences of any failure that may occur.

In the case of long-term assessments using working strength parameters, a Factor of Safety of 1.5 is generally accepted as being the minimum. In the case of shorter term (i.e., during construction), a lower value (say 1.25) can be adopted.

4.5 Construction Methodology

The following methodology is recommended:-

1. Excavate a 45-degree batter from the base of the final cut line
2. Excavate the proposed retaining wall footing (depth of retaining wall footing key assumed to be 1m below slab level)[1]
3. Construct the footing
4. Construct the retaining wall to the final level

Note 1 – This could be achieved by the adoption of a key along the wall side of the footing or by the use of a series of short bored piers structurally connected to the footing on a relatively close (say 3D centre to centre) spacing along the line of the wall. The bored pier option would not lower the short term factor of safety for this step (i.e. the Step 2 situation would apply for Step 3) and would be the preferable option.

4.6 Results of the Analysis

Using estimated parameters (based on the information and assumptions provided in Section 3.3 and 4.1 to 4.3), and assuming **drained** material parameters, the following factors of safety (FOS) have been calculated for each step of the construction (refer Appendix B):-

- Existing slope – 1.68 – refer Figure B1
- Step 1 – 1.03 – refer Figure B2
- Step 2 – 0.70[1] – refer Figure B3
- Step 3 – 1.03[2] – refer Figure B5
- Step 4 – 1.48 – refer Figure B6

Note 1 - The Step 2 analysis was recalculated using undrained strength parameters for the fill. To achieve a minimum FOS of 1.5 (refer Figure 4), the undrained cohesion of the fill would need to exceed 25kPa. Generally, the cohesion of the fill would be expected to exceed 50kPa to 75kPa. Based on this and assuming the excavation of the retaining wall footing, placement of reinforcement and filling with concrete can be undertaken quickly, it is unlikely that the fill mass strength parameters would change to drained parameters within that period (refer Appendix B). ***The difference in the results between drained and undrained does emphasise the importance of completing the construction of this wall (particularly between steps 2 and 3) as quickly as possible to minimise the risk of instability.***

Note 2 – The assessed failure in Step 3 is the same as Step 1, i.e., the toe of the failure passes through the toe of the slope which is above the footing. Failure circles lower in the slope intersected the footing which is modelled as much higher strength. This results in higher FOS values. The critical strength of the footing should more correctly model the sliding resistance "strength" of the footing.

In general, the FOS for Steps 1 to 3 are less than what would normally be considered acceptable (refer Section 4.4) for a model using drained parameters. However, given the expected time to undertake construction of the toe the use of undrained parameters (as was adopted in Step 2) would most likely be more appropriate.

Whilst we have noted the need for this step to be undertaken quickly, it is recommended that:-

- The weather prior to construction will have a significant effect on the slope. Construction following or during rainfall is not preferred as this may decrease the FOS calculated above.

- The reinforcement for the footing be tied out of the footing and lifted in the excavation immediately after following excavation (and any required inspection) and filled with concrete.
- The excavation should be done in stages, the total length of the footing is some 70m and as such, it is unlikely this will be done quickly. It is strongly recommended that the footing excavation not be left open for any longer than say 24hr. Should this occur, the likelihood of overstressing the fill mass may be greater than recommended.
- Construction of the retaining wall should also be undertaken as rapidly as possible so that backfilling can be undertaken.

4.7 Analysis Conclusions

The analysis undertaken used the assessed and assumed material parameters and excavation and construction methodology indicates that the existing and final overall slopes have calculated FOS's of over 1.5.

The temporary batters have calculated a FOS of 0.70 and 1.03 using drained fill mass parameters. As these cuts will be temporary and provided the works are undertaken as per the information contained in Sections 4.6 and 4.8, the use of undrained material parameters provided calculated FOS of >1.5. Based on this, assuming the cuts can be undertaken adopting the recommended construction methodology and as quickly as possible then the works should not affect the overall stability of the existing embankment.

4.8 Construction Recommendations

It is recommended that:-

- Excavation and retaining wall construction be undertaken as quickly as possible.
- The works should preferably be done following a period of dry weather.
- The backfill adopted be non-compressible and contain suitable voids to allow it to drain rapidly, i.e., no fines concrete (for at least the lower 1m of the backfill).
- The top of the backfill be sealed to minimise water ingress into the backfill.
- Drains be installed in the backfill to allow rapid drainage of any seepage out from behind the wall.
- The retaining wall be designed for the top slope (refer Section 3.3 of original report) and using the Design Ko value.

It should be noted that the present embankment has most likely been constructed using good engineering practice given its performance following construction. What is proposed will need to reinstate after the area of cut to ensure the fill embankment is as close as practicable to the original conditions. Hence the use of a non-compressible (and in this case) material that to some degree 'set' and is resistant to scour or slumping.

5.0 LIMITATIONS

We have prepared this report for the use of **Multi Span Australia**, for design purposes in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has not been prepared for use by parties other than **Multi Span Australia**; it may not contain sufficient information for purposes of other parties or for other uses. Please note that any third party relying on the information contained in this report for any purpose whatsoever does so entirely at its own risk, and any duty of care to that third party is excluded.

Any interpretation or recommendation given by Soil Surveys Engineering shall be understood to be based on judgement and experience and not on greater knowledge of the facts than the reported investigations would imply. The interpretation and recommendations are therefore opinions provided for our Client's sole use in accordance with the specific brief. As such they do not necessarily address all aspects of ground behaviour on the subject site. Information provided by others has been taken in good faith, but no liability can be accepted for information provided by others.

Your attention is drawn to 'Appendix A', 'Notes Relating to this Report'. Interpretation of factual data given in this report is based on judgement, not a greater knowledge of facts other than those reported.

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

Please note that should, following detailed design, footings or piers/piles be required to extend within 3B (B = footing width/pier diameter) above the termination level of the boreholes/pits/CPTU or any excavations extend below borehole/pit/CPTU termination levels, Soil Surveys Engineering should be contacted immediately. In this case, geotechnical data in this report should be considered preliminary only; additional investigation is likely to be required.

In the event that conditions encountered on site during construction appear to vary from those expected from the information contained in the report, the Company strongly recommends that it immediately be notified. Most problems are more readily resolved when conditions are exposed than at some later stage, after the event. Should Soil Surveys Engineering not be notified or if this notification is delayed, then Soil Surveys can not be held responsible for the affect that any variation has on any aspect of the development.

Soil Surveys Engineering consider that a documentation review service (during the design phase and prior to construction) to verify that the intent of geotechnical recommendations is properly reflected in the design, along with construction inspections, forms a very important component of the geotechnical engineering design service/process.

The geotechnical review ensures geotechnical risks to our Client and their project are minimised at the design and tender stage of the project. Further, with Soil Surveys Engineering being commissioned to carry out geotechnical construction inspections, an opportunity at the time of construction to confirm any assumptions made in the preparation of the report and allow the effect of any normally occurring variation in ground conditions to be assessed with respect to construction becomes available.

The above statements are not intended to reduce the level of responsibility accepted by Soil Surveys Engineering in accordance with our commission, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in doing so and the risks they accept should they decline to have Soil Surveys Engineering carry out a geotechnical documentation review and geotechnical construction inspections.

It is highly recommended that the Client avail themselves of these review and inspection services; our standard rates will apply.

N.T. PERKINS (RPEQ 7527)
PRINCIPAL GEOTECHNICAL ENGINEER

For and on behalf of
SOIL SURVEYS ENGINEERING PTY LIMITED

APPENDICES

APPENDIX A

NOTES RELATING TO THIS REPORT

INTRODUCTION

These notes are provided by Soil Surveys Engineering Pty Limited (the Company) to complement the geotechnical report in regard to classification methods and field procedures. 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 information 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 information 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 at the time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

Soils - The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726-2017 (Geotechnical Site Investigations), where appropriate. 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 dominant particle size and behaviour as set out in AS 1726-2017.

Cohesive soils are classified on the basis of strength (consistency) either by use of hand penetrometer, shear vane, laboratory testing or engineering examination. The strength terms are defined in AS 1726-2017 Table 11.

Non-cohesive soils are classified on the basis of relative density usually based on insitu testing or engineering examination (see AS 1726-2017 Table 12).

Rocks - Rock types are classified by their geological names (AS 1726-2017 Tables 15 to 18), together with descriptive terms regarding weathering (AS 1726-2017 Table 20), strength (AS 1726-2017 Table 19), defects (AS 1726-2017 Table 22), etc.

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 sample disturbance, (information on strength and structure).

Undisturbed samples are taken by pushing a thin walled sample tube, usually 50mm diameter (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, volume change potential 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.

SAMPLE STORAGE – SOIL, ROCK AND WATER SAMPLES

Soil samples (not subject to testing) are not stored beyond a period of 90 days of taking or receiving said soil sample. Rock core (not subject to testing) is not stored beyond a period of six months of taking or receiving said rock core.

Should any party require that soil samples (not subject to testing) be stored beyond 90 days, or rock core (not subject to testing) be stored beyond six months, please contact Soil Surveys Engineering.

Water samples (not subject to testing) are not stored beyond a period of seven days of taking or receiving water samples.

TEST LOCATIONS

Test locations (e.g. boreholes, CPT's, test pits etc.) were based on available access at the time of testing. Test locations may have been shifted if access was not suitable.

Unless noted otherwise, accuracy of test locations are to the accuracy of hand held GPS equipment.

INVESTIGATION METHODS

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

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 approximately 3.0m for a backhoe and up to 6.0m 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. Refusal of the augers can occur on a variety of materials such as hard clay, gravel or rock fragments and does not necessarily indicate rock level.

Continuous Spiral Flight Augers - The borehole is advanced using 75mm to 300 mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling or 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 augers. Information from the drilling (as distinct from specific sampling) is of relatively lower reliability due to remoulding, inclusion of cuttings from above or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table has a lower reliability than augering above the water table. Various drill bits are attached to the base of the augers during the drilling. The depth of refusal of the different bit types can provide information as to the strength of the material encountered. Generally the 'TC' bit (a tungsten carbide tipped screw type bit) is used.

Wash Boring - The borehole is usually advanced by a rotary bit with water or fluid pumped down the hollow drill rods and returned up in the space between the rods and the soil or casing, 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. More accurate information on soil strata is gained by regular testing and sampling using the Standard Penetration Test (SPT) and undisturbed thin walled tube samples (U50).

Mud Stabilized Drilling - Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilize 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 regular intact sampling (e.g. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling - A continuous core sample is obtained using a diamond or tungsten carbide tipped core barrel. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable method of investigation. In rocks, NMLC coring (nominal 52 mm 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 is determined on site by the supervisor. If the location of the loss is uncertain, it is placed at the top end of the run, when the core is placed in a storage tray and recorded on the log.

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. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" - Test 6.3.1.

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

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

4, 6, 7 N = 13

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

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, it is noted on the borehole logs.

A modification to the SPT test is where the same driving system is used with a solid 600 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 SPT are shown as "N_c" on the borehole logs, together with the number of blows per 150 mm penetration.

Cone Penetration Tests - Test Method - Cone Penetration Tests (CPT) are carried out in accordance with AS 1289 Test 6.5.1-1999, using an electrical friction-cone penetrometer.

The test essentially comprises the measurement of resistance to penetration of a cone of 35.7 mm diameter pushed into the soil at a rate of 10-20 mm per second by hydraulic force. The resistance to penetration is recorded in terms of pressure on the end area of the cone (cone resistance, q_c, in MPa) and friction on the side of the 135 mm long sleeve immediately above the top of the cone (friction resistance, f_s, in kPa). These forces are measured by electrical transducers (strain gauges) within the cone device. The ratio between friction resistance and cone resistance is also calculated as a percentage, i.e.-

$$\text{Friction Ratio (FR)} = \frac{\textit{Friction Resistance},\ f_s\ (kPa)\ x\ 100}{\textit{cone resistance},\ q_c\ (kPa)}$$

The friction ratio, FR, is generally low in sands (less than 1% or 2%) and generally higher in clays (say 3% or more). The interpretation of sandy clays, clayey sands and material with a high silt content is more difficult, but intermediate values (between 1% and 3%) would be expected. Highly organic clays and peats generally have a friction ratio in excess of 5%.

Static cone data is recorded in the field on disc for later presentation using computer aided drafting.

The equipment can be operated from any conventional drill rig. A total applied load in the range of 4 to 10 tonnes is required for practical purposes, although lighter loads may be used. The cone penetrometers are available with various capacities of cone resistance ranging up to 100 MPa for general purpose investigations, while a range of 0 to 10 MPa can be used where more sensitive investigations of soft clay are required.

The cone resistance value provides a continuous measure of soil strength or density, and together with the friction ratio, provide very useful indications of the presence of narrow bands of geotechnically significant layers such as thin, soft clay layers or lenses of sand which might otherwise be missed using conventional drilling methods.

The lithology of the encountered soils is interpreted from static cone data and is generally presented on the static cone log sheets.

It is important to note that the lithology is interpreted information and is based on research by Schmertmann (1970), Sanglerat (1972), Robinson and Campinalli (1986), modified to suit local conditions as indicated by borehole information and laboratory testing.

As soils generally change gradually it is sometimes difficult to accurately describe depths of strata changes, although greater accuracy is obtained with the static cone compared with conventional drilling. In addition, friction ratios decrease in accuracy with low cone resistance values, and in desiccated soils. As a result, some overlap and minor discrepancies may exist between static cone and nearby borehole information.

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

The DCP comprises a Cone of 20 mm diameter with 30 degree taper attached to steel rods of smaller section.

The cone end is driven with a 9 kg hammer falling 510 mm (AS 1289 Test 6.3.2). The test was developed initially for pavement subgrade investigations, and empirical correlations of the test results with California Bearing Ratio have been published by various Road Authorities. The Company has developed their own correlations with Standard Penetration tests and Density Index tests in sands.

LOGS

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

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

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

GROUNDWATER

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

- Although groundwater may be present in lower permeability soils, it may enter the hole slowly or perhaps not at all during the time the hole is open.
- A localized 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 bailed out of the bore and mud must be washed out of the hole or "reverted" if water observations are to be made.

More reliable measurements can be made by use of standpipes which are read after stabilizing at periods 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 (e.g. 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 important 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 and the attached explanatory notes summarize important aspects of the Laboratory Test Procedures adopted.

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. The information provided in Soil Surveys Engineering reports is opinion and interpretation and not factual. The client/contractor increases their risk by not retaining the person who authored the geotechnical report, to carry out site inspection and review (overseeing role) during construction, to confirm opinion and interpretation expressed in the report is accurate. Where the report has been prepared for a specific design proposal the information and interpretation may not be relevant if the design proposal is changed. 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. Since the test sites in any exploration represent a very small proportion of the total site and since the exploration only identifies actual ground conditions at the test sites, even under the best circumstances actual conditions may vary from those inferred to exist. No responsibility is taken for:-

- Unexpected variations in ground and/or groundwater conditions.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of other persons.
- Any work where the company is not given the opportunity to supervise the construction using the Companies designs/recommendations.

If differences 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 expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are more readily resolved when conditions are exposed than at some later stage, well after the event.

Extreme events including but not limited to the results of climate change, e.g. flood levels above previously identified levels, beach scour or erosion beyond normal expectations (as identified by local authorities) extreme rainfall events, war, espionage, sabotage may result in different conditions between time of investigation and time of construction.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document “Guidelines for the Provision of Geotechnical Information in Construction Contracts (1987)”, 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.

REVIEW OF DESIGN

Where major civil or structural developments are propose or where only a limited investigation has been completed or where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer. We would be happy to assist in this regard as an extension of our investigation commission. Construction drawings should be reviewed by Soil Surveys Engineering, with sufficient time to allow changes if required, prior to inspections. Otherwise Soil Surveys Engineering reserves the right to refuse to carry out inspections.

SITE INSPECTION

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

i. Site visits during construction to confirm reported ground conditions
ii. Site visits to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, the stability of a filled or excavated slope; or
iii. Full-time engineering presence on site.

In the vast majority of cases it is advantageous to the principal for the geotechnical engineer who wrote the investigation report to be involved in the construction stage of the project.

The geotechnical engineer cannot take responsibility for variations in encountered conditions, where he is not given the opportunity to review plans for the proposed development with sufficient time to allow review and make changes to the proposed development if required, and where he is not given the opportunity to inspect the site and oversee construction methods with regard to site conditions with sufficient time to observe all relevant site conditions and operations.

RESPONSIBLE USE OF GEOTECHNICAL INFORMATION

Recommendations in our report are for design purposes only and provided on the basis that inspections are carried out to allow finalisation of opinions and recommendations contained in our report.

The geotechnical investigation consisting of field and laboratory testing has been carried out to indicate typical conditions by indicating conditions and parameters at the specific locations of boreholes/test pits. Subsurface conditions are indicated at these locations only and the inference of conditions between or away from these locations (interpolation and extrapolation) involves a certain degree of risk. Persons inferring such conditions or carrying out such inferences should do so with a degree of caution and conservatism which is commensurate with the consequences of the risk of error.

Estimates of volumes based on our findings require interpolation and extrapolation between test locations and as such may be significantly different from actual volumes.

APPENDIX B

STABILITY PRINTOUTS

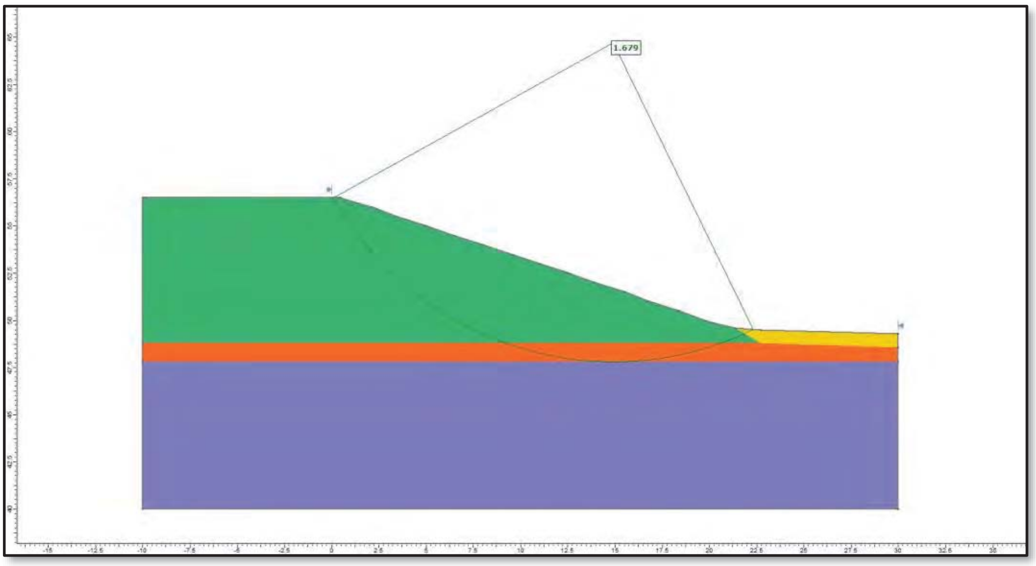

FIGURE B1 - EXISTING

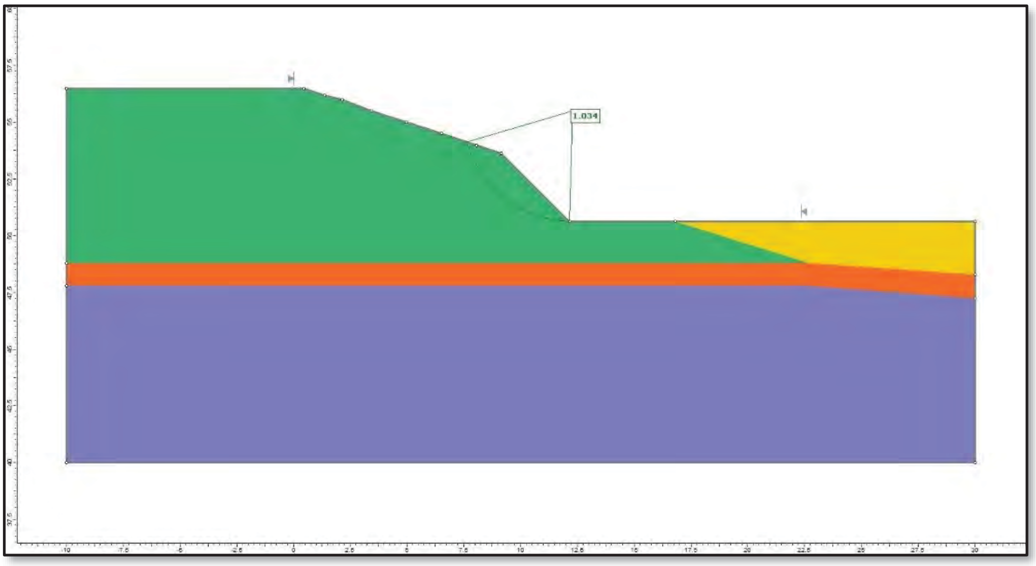

FIGURE B2 - STEP 1

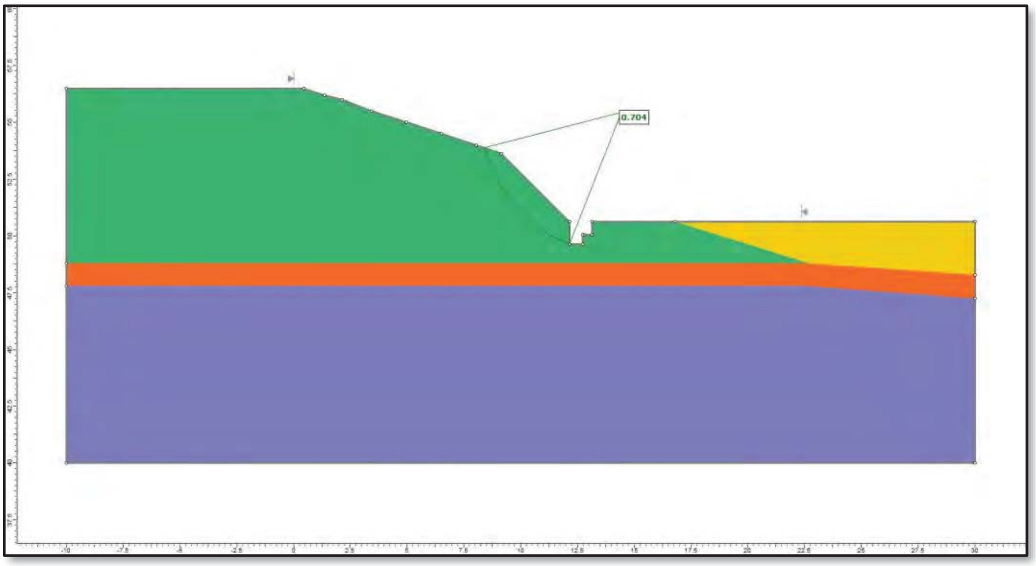

FIGURE B3 - STEP 2 - DRAINED

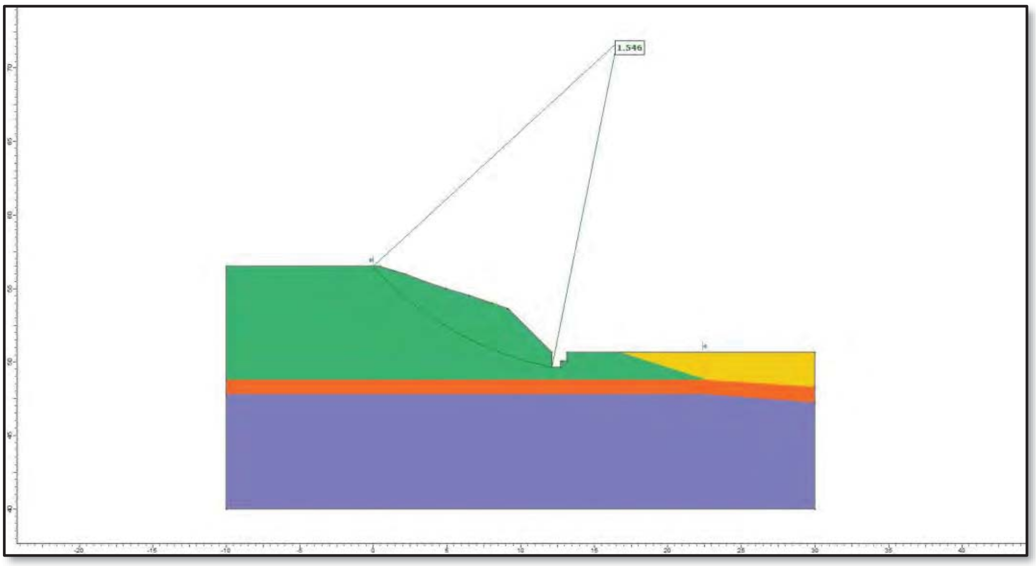

FIGURE B4 - STEP 2 - UNDRAINED

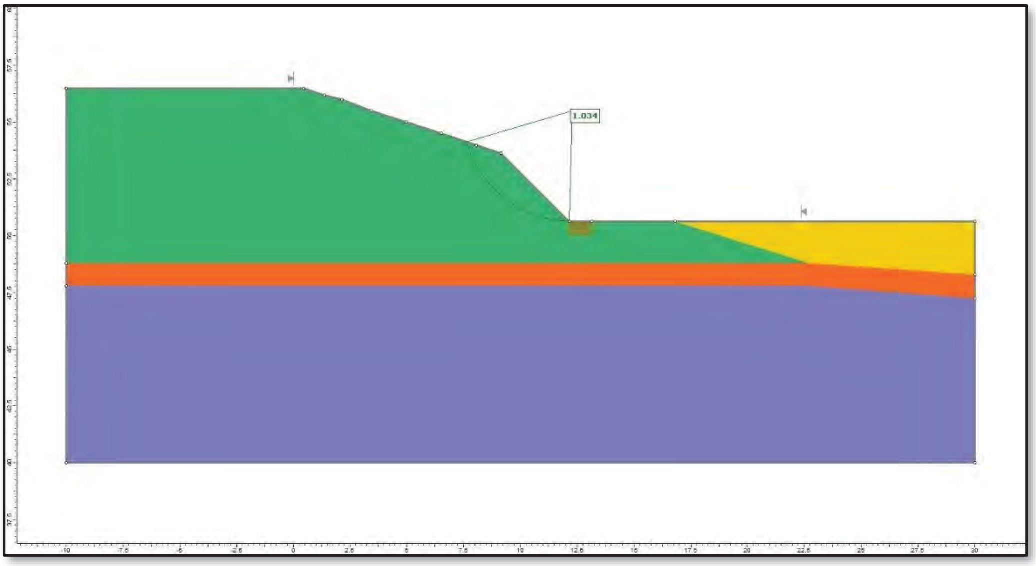

FIGURE B5 - STEP 3

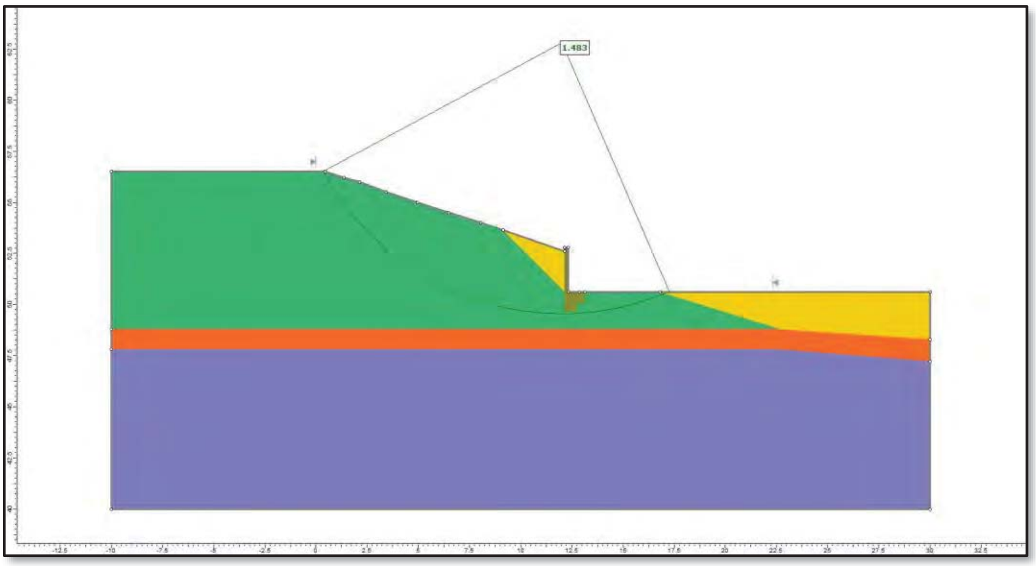

FIGURE B6 - STEP 4