DRAFT FINAL REPORT
on
GEOTECHNICAL ASSESSMENT

PROPOSED THIRD TERMINAL
PORT BOTANY

VOLUME 1

Prepared for
SYDNEY PORTS CORPORATION

Project 35224
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EXECUTIVE SUMMARY

This report presents the results of a geotechnical assessment undertaken for a proposed third terminal at Port Botany. The development will include reclamation of land and construction of wharf facilities in the area immediately north-west of the current north quay at Brotherson Dock.

It is understood that the development may be undertaken in two phases, with the first phase being reclamation of the bulk of the area supported by a series of rock berms, and the second phase being construction of the wharf facilities. Two options being considered for construction of the wharf are a caisson structure backfilled with sand, or a piled wharf structure with additional rock berms placed to improve the long term stability of the reclamation area.

The geotechnical assessment comprised a review of the available geotechnical information, preparation of a geological model and analysis of the proposed works. No additional field work was carried out. The information reviewed included the bores and some laboratory testing from a number of investigations carried out in the area over many years. Copies of all the relevant bores have been included in the report. In addition three interpretative reports prepared by other consultants were also reviewed.

The area is underlain by a deep valley within the sandstone bedrock which runs beneath the area of the proposed reclamation. This drowned valley has been filled with up to 70 m of sediments, essentially comprising a lower unit of clays of marine origin interbedded with sandy layers, an upper clay unit which includes numerous peat and organic silt and sand layers, and a relatively uniform sand unit near the surface.

The uniform sand unit has previously been dredged from within the area of the proposed works for use in construction of Sydney Airport's Third Runway. The sand is uniformly graded with very few fines and is present in the area to the west of the proposed reclamation. The sand is considered suitable for dredging using standard techniques, however, preliminary calculations indicate that there may be a slight shortfall of filling material if the caisson option is adopted for construction of the wharf.

Stability analyses undertaken on a series of small rock berms proposed to support the first phase of the development indicate that a slope of 1.5H:1V could be used for the temporary case if the sand backfill within a 20 m zone of the face of the slope is compacted using

vibrocompaction techniques. Alternatively a 2H:1V slope could be used without vibrocompaction.

If piles are used for the permanent wharf structure then additional rock berms would be required for long term stability at maximum slopes of 2H:1V with any additional sand filling well compacted. Construction of the piles should not be difficult with refusal likely to occur within the hard clays or very dense sands.

Stability analysis of a proposed caisson structure to support the wharf gave satisfactory factors of safety, however, settlement analysis indicates that the structure may undergo long term settlements of up to 200 mm. In order to limit these settlements to acceptable levels it may be necessary to remove some of the foundation soils beneath the caissons to depths of 20-30 m below sea bed levels.

FM:fm
Project 35224
14 October, 2002

GEOTECHNICAL ASSESSMENT
PORT BOTANY THIRD TERMINAL RECLAMATION

1. INTRODUCTION

This report presents the results of a geotechnical assessment undertaken for a proposed third terminal at Port Botany. The work was carried out at the request of Mr Tony Navaratne of Sydney Ports Corporation.

It is understood that Sydney Ports Corporation is considering further development of Port Botany to create additional land and wharf facilities in the area immediately north-west of the current north quay at Brotherson Dock (Patrick Container Terminal). This development will include reclamation of land over an area which has been previously dredged for construction of the Third Runway at Sydney (Kingsford-Smith) Airport. The geotechnical assessment was undertaken in order to provide information for preparation of an Environmental Impact Statement as well as recommendations for design and construction of the project.

The assessment comprised a review of all the available geotechnical information, preparation of a geological model for the site and analysis of the proposed works. Details are given in the report, together with recommendations for design and construction.

2. AVAILABLE INFORMATION

A number of investigations have been undertaken in the area over many years, originally mostly by the Maritime Services Board of NSW for development of the Port Botany area, and more recently for the construction of the Third Runway and for proposed extensions of the port area.

The geotechnical information held by Sydney Ports Corporation from previous investigations has been reviewed, together with some information from test bores and cone penetration tests (CPTs) undertaken for construction of the Third Runway. Test bore and CPT data which is located within the area of interest has been included in development of the geological model of the site.

The locations of all the test bores used in this assessment are shown on Drawing 2 in Appendix A, and copies of the original logs for each of these bores have been included in Appendix F (Volume 2) for completeness of information. In addition a series of sections have been prepared parallel and normal to the line of the proposed new wharf. These sections, the locations of which are shown on Drawing 3, include summary logs of most of the test bores and are included in Appendix B.

A detailed summary table of the included test bores is given in the front of Appendix F. This table presents the surveyed co-ordinates of each of the bores, the levels of the bores relative to Indian Springs Low Water (ISLW) datum, and gives summaries of the strata encountered. The sources of the test bores which have been used during this assessment are briefly described in Table 1 below.

In addition to the test bores, data from laboratory testing on samples from the bores and results of seismic investigations in the area have also been examined. The relevant laboratory test data has been compiled into a summary table of results. This summary table is given in Appendix C.

As well as the factual data obtained from bore logs and laboratory tests, three interpretative reports have been prepared on the geotechnical information in the area. These reports are:

- Coffey & Hollingsworth Pty Ltd - Botany Bay North Development. Report of Geotechnical Design Parameters. For Maritime Services Board of NSW. Report No. 5515 - AK, May 1976.
- Coffey Partners International Pty Ltd - Additional Port Facilities, Port Botany. Volume 2 Geotechnical Interpretation. For Connell Wagner Pty Ltd and Sydney Ports Corporation. Report No. S10526/2-CZ, 21 June 1999.
- GHD-Longmac Pty Ltd - Botany Bay Reclamation. Preliminary Geotechnical Review. For Sydney Ports Authority. Job No. 2710346, 29 January 2002.

In 1984 Colin Thorne (formerly of Coffey & Hollingsworth) published a paper "Strength assessment and stability analyses for fissured clays" in Geotechnique, Volume 34, No. 3, pp 305-322. The interpretations given in this paper were based on the investigations undertaken by Coffey & Hollingsworth at Port Botany.

Table 1 - Test Bores used in Geotechnical Assessment

Test Bores	Description
124 and 132	Maritime Services Board of NSW, July and August 1968. For development of Botany Bay North.
207-215, 217-221, 223-227	Maritime Services Board of NSW, January 1969 to May 1969. For reclamation and dredging programs for Botany Bay North area. Drilled to depths ranging from 14.3 m to 22.9 m. Seven terminated before intersecting bedrock.
338, 339, 341-346	Maritime Services Board of NSW, April and May 1974. For dredging of the approaches to the Brotherson Dock. In area currently dredged to about RL-16 for approaches to dock.
420, 422, 424, 425	Maritime Services Board of NSW, February to August 1975. For investigation of Brotherson Dock. Bores 420 and 422 were supervised by Coffey & Hollingsworth.
605	Maritime Services Board of NSW, February 1975. For investigation of Brotherson Dock. Supervised by Coffey & Hollingsworth.
901-919	Maritime Services Board of NSW, November 1975 to May 1976. For possible extensions to Brotherson Dock.
DM2-DM8, DM20, DM46, DM67-DM69, DM71, DM73	Dames & Moore, January 1991 to May 1991. Investigations for Airplan-GHD Joint Venture for Third Runway Project. In addition to these bores there were some additional bores drilled in the vicinity of the existing dredged area, however, there were no levels provided on the copies of the bore logs so these bores have not been included.
CP1-CP27	Coffey Partners International, July 1998 to October 1998. Investigations for Sydney Ports Corporation for possible extension to Brotherson Dock. Bores CP1 to CP12 were taken to bedrock. The remainder were drilled in proposed dredging and approach areas and were terminated after reaching the peat and clay layers beneath the sand layer.
S1-S13	Coffey Partners International, June 1998. Investigations for Sydney Ports Corporation for possible extension to Brotherson Dock. Surface sediment samples collected for environmental testing.
CW7 and CW8	Connell Wagner, December 2000. Two bores drilled through existing Brotherson Dock to investigate and monitor possible movements of dock.

3. GEOLOGICAL MODEL

3.1 Regional Geology

The regional geology of the area is shown on an extract of the Sydney 1:100, 000 Geological Series Sheet on Drawing 1 in Appendix A. Essentially the northern Botany Bay area comprises alluvial sediments deposited over Hawkesbury Sandstone bedrock with some significant areas of filling.

The investigations undertaken for the proposed new development and other developments in the area have shown that the depth to bedrock varies considerably from less than 20 m below ISLW to more than 70 m below ISLW. As shown on Drawing 4 in Appendix A, there appears to be a relatively narrow, deep channel running beneath the area of the proposed reclamation. The maximum depth to rock recorded in the bores within this area was in excess of 73 m below ISLW. This is consistent with the previous work for the reclamation of the existing port which showed a deep channel in the bedrock to about 80 m below ISLW, running in an easterly direction just to the south of the North Quay.

The sediments overlying the Hawkesbury Sandstone bedrock include sands, silts, clays, peats and various combinations of each of these. It is apparent from the stratigraphy of the site that the channel in the bedrock was a valley formed by erosion during a time when the sea level was much lower than it is at present and that the sediments have been deposited during various periods as the sea level rose.

There have been at least eight major fluctuations in sea level in the last 700,000 years, with a maximum sea level of about 6 m above present levels some 120,000 to 140,000 years ago. The most recent lowest sea level was about 70 m below current levels about 17,000 years ago with a rise to current levels about 6,000 to 6,500 years ago and most fluctuations since have been within 1 m of the current level. Between the major fluctuations in sea level there have been many minor fluctuations which have resulted in different depositional environments and have caused variability in the types of sediments deposited. The lower sediments in the area include shells which are indicative of a marine environment, while the upper clay layers include numerous organic and peaty layers which suggest a deltaic environment.

There are no known major faults or other structural features in the vicinity of the site. It is possible that there may be some igneous dykes within the bedrock as there are numerous dykes scattered throughout the Sydney Basin area and several are shown on the regional geological map to the east of the site trending in a westerly direction. Typically, however, any such dykes are less than 1 m wide and are normally vertical or very steeply dipping so would have very little impact on the proposed construction.

3.2 Geological Model for Site

Coffey & Hollingsworth prepared a geological model for the area in 1976 based on the investigations for the development of Botany Bay North. This model is summarised below:

Table 2 - Coffey's 1976 Geological Model

Horizon	Name	Description
D	Upper Sands	Medium dense to very dense, medium to fine grained sand. Few fines in upper sections but some peaty layers in lower sections. Possibly beach deposit.
C	Upper Clays	Interbedded clay, sandy clay, clayey sand, silty sand, peat and peaty clay. Fissures common in clays particularly near upper boundary. Probably lagoonal or estuarine origin
B	Lower Clays	Organic clays interbedded with silty sands and sandy clays. Shells found throughout and particularly abundant near upper surface. Fissuring of clays is common. Shells indicate probable marine origin.
A	Deep Sands	Mainly dense, medium to coarse grained sand with some interbedded lignite (consolidated peat) and organic clay. Possibly beach sand with peat layers deposited in shallow lagoons behind the beach.
Bedrock		Sandstone bedrock, sometimes overlain by up to 1 m thick extremely weathered rock

In 1999 Coffey Partners International Pty Ltd revised the geological model for the proposed new terminal area to the system outlined in Table 3. This system is essentially similar to the previous one with Unit 2 equivalent to Horizon D, Unit 3 equivalent to Horizon C and Unit 4 equivalent to Horizon B but the different types of soils within the main units have been separated into sub-units and separate units were given for the near surface disturbed sediments and the thin residual soil layer above the bedrock. Also the Horizon A - "Deep Sands" was not identified in any of the bores in this area.

Table 3 - Coffey's 1999 Geological Model

Main Unit	Name	Sub-Unit	Description	General Conditions
1	Seabed materials	1A	Silty sand and clayey sand	Very loose to loose
	Recent deposits or disturbed materials.	1B	Clayey silt, sandy silt, silty clay and sandy clay	Very soft to soft
	Associated with previous dredging and/or reclamation activities	1C	Sand and silty sand	Very loose to loose
		1D	Organic silty clay	Very soft to soft
2	Sands		Sand and silty sand, occasional peat lens	Dense to very dense
3	Interbedded peat, fissured clay and peaty sand	3A	Peat (lignite) often fissured	Very stiff to hard
		3B	Fissured clay, high plasticity	Very stiff to hard
		3C	Peaty sand, variable organic content	Dense to very dense
4	Fissured clay with silty sand and clayey sand interbeds	4A	Fissured clay, high plasticity	Very stiff to hard
		4B	Silty sand and clayey sand, cemented in some areas near base	Very dense
5	Residual soil		Clayey sand and sandy clay	Dense to very dense
6	Bedrock		Weathered Hawkesbury Sandstone, occasional siltstone	Low to high strength

Rather than developing yet another geological model for the area, it is proposed that the system devised by Coffey in 1999 be continued. It is noted, however, that while there is quite a distinct boundary between the base of the Unit 2 sands and the Unit 3 peats and clays, boundaries between Units 3, 4 and 5 are less clear and there is often no apparent correlation between the different sub-units within adjacent bores. This suggests that, while the Unit 2 sands were deposited in a relatively uniform depositional environment, the underlying sediments were laid down in more variable conditions, with discontinuous lenses of peat, silts and sands included within the clays. This variability in sediment type is very apparent from the summary logs shown on the sections drawn through the site (Appendix B). There may also be some variability in apparent soil type resulting from the different people logging the soil samples.

As can be seen on the summary logs there is considerable variation in the levels of the boundaries between the different units, although it is noted that the summary logs are shown on drawings with a vertical exaggeration of 10:1 which tends to emphasise changes in level. It is considered therefore that it would be unsatisfactory to try to simplify the geological model for the whole site into horizontal layers by assigning levels to the boundaries between the units. It is recommended instead that the design of the proposed works be undertaken by reviewing the sections through the site and identifying critical sections for analysis as appropriate.

It is noted that since the 1976 investigations there has been much interest in both the fissured clays and the amount of shell fragments within the different layers (to distinguish between the different horizons). Prior to this time there was no mention of any fissures in the clays in any of the bore logs and only very occasional references to shell fragments. More recent bore logs, such as the 900 series noted many fissured clay layers and numerous shell fragments. The recent Coffey bore logs (CP1 to CP27), however, only included a few references to fissured clays, although detailed descriptions of extruded undisturbed tube samples indicated that most of the clays sampled were fissured to some extent.

Coffeys 1976 interpretative report indicated that fissures were present in most of the clay beds with the origin of these fissures being caused by depositional processes, differential settlement over consolidating peat layers, or seasonal shrinkage and swelling of the clays.

While it is apparent from the descriptions on the bore logs that some layers, particularly the organic clays, are more highly fissured than others, there is no obvious pattern to this fissuring. It is therefore recommended that the design of the proposed new port facilities be undertaken assuming that all the clay layers are potentially fissured.

4. PROPOSED DEVELOPMENT

It is understood that the proposed development of the site requires reclamation of an approximately rectangular area of about 1400 m by 500 m wide immediately adjacent to the existing Patrick Container terminal. This area has been previously dredged and will require filling to an approximate surface level of 3.7 m above ISLW. It is intended that the bulk of the fill will be obtained from dredging of the area between the proposed new terminal and the existing Third Runway.

It is understood that Sydney Ports Corporation intends constructing a series of rock berms to contain the dredged sand fill and then, possibly at a later date, constructing a wharf to accommodate ships.

Two options under consideration for the wharf construction are:

- installing a concrete caisson structure around the edge of the proposed area. The caisson would then act as the permanent wharf structure and the area between the caisson and the rock berms would be backfilled, probably with sand.
- constructing a piled deck structure around the reclamation with additional rock berms placed to improve the long term stability of the temporary rock berms

The key geotechnical factors affecting the proposed development are:

- the stability of the proposed new reclamation area
- long term settlement of the filled area
- foundation types for the proposed structures
- the effects of earthquakes on the proposed development
- the effects of dredging on both the new and existing structures
- the quantity and quality of the material available for dredging
- environmental issues, such as turbidity during dredging

These factors are discussed in the following sections.

5. STABILITY ANALYSIS

5.1 Stability Design Parameters

In order to undertake stability analyses of the proposed development it has been necessary to assess the most appropriate values for each of the parameters required for stability analysis. The assessment has comprised a review of the available laboratory test data on samples from the bores on the site and adjacent to the site, together with consideration of previous experience in similar materials. A summary of the laboratory test data considered is included in Appendix C, together with plots of some of the data.

The design parameters assessed for each of the possible strata are given in Table 4 below.

Table 4 - Stability Design Parameters

Unit	Description	Bulk density γ (kN/m^3)	Effective cohesion c' (kPa)	Effective friction angle ϕ' (degrees)	Drained modulus E' (MPa)	Poisson's ratio ν'
1	Very loose to loose silty sands	14	0	27	5	0.3
2	Dense to very dense sands	20	0	37	100	0.3
3A	Very stiff to hard peats	15	5-10	18-25	20	0.3
3B	Very stiff to hard fissured clays and organic clays	19	5-10	18-25	20	0.3
3C	Dense to very dense peaty sands	19	0	32	40	0.3
4A	Very stiff to hard fissured clays	20	5-10	18-25	40	0.3
4B	Very dense silty sand and clayey sand	20	0	32	100	0.3
5	Dense to very dense residual clayey sands	21	0	34	100	0.3
6	Bedrock	23	500	45	200	0.2
Dredged sand fill	Not compacted	18	0	30	20	0.3
Dredged sand fill	Dredge and compacted to at least 70% relative density	18	0	35	40	0.3
Rock Berms	Imported 'sound' rock. Friction angle varies with rock size.	20	0	42-45	100	0.3

A range of shear strength values has been given for the clays in Units 3A, 3B and 4A. The lower values represent lower bound strengths which could only be achieved if a failure surface extended entirely along existing fissures within the clays. The higher values represent failure surfaces extending through a combination of intact clay and along fissures. It is considered that the upper values are more likely, but analysis has also been undertaken using the lower values to assess the sensitivity of stability to the lower shear strength values. It is possible that there may also be reduced shear strengths in the fissured clays beneath the previously dredged areas due to rebound after removal of the dredged soil, however the location of these soils relative to the proposed structures means that these layers do not affect the stability.

Similarly a range of friction angles has been given for the dredged sand fill and the imported rock fill. It is anticipated that, provided the sand fill is uniformly compacted to at least 70% relative density, then a friction angle of 35 degrees may be adopted. If, however, the compaction is not completely achieved or there is some poorer material included in the fill then the lower value may be more representative. For the rock fill the friction angle depends on the size of the rock particles and the manner in which it is placed.

5.2 Numerical Techniques and Assumptions

Slope stability analysis of the proposed reclamation has been undertaken using a computer program called PCSTABL5 developed by Purdue University in the USA. While most of the analyses have been undertaken using Bishop's simplified method to calculate the factors of safety for automatically generated circular slip surfaces, this program also has the ability to carry out analyses using several other different techniques.

The factor of safety is essentially the ratio of the stabilising forces to the destabilising forces. When the factor of safety is less than 1.0 it is likely that there will be some slope instability or deformation of the soils. For most engineering projects where there is a reasonable amount of data on the shear strength of the materials a factor of safety of 1.5 or above is normally considered acceptable for long term conditions. For short term conditions or unusual loads, such as earthquakes, a lower factor of safety may be adopted although there is some debate as to what is an acceptable value. For design of large dams, where the consequences of failure may be disastrous, factors of safety of 1.2-1.3 are usually required for short term conditions. For other structures where the consequences of failure are less severe lower values may be adopted, but a minimum value of 1.1 is usually suggested.

5.3 Models Analysed

It is understood that the proposed sequence of development is as follows:

- dredge along line of wharf as required for foundations, backfill existing dredged area and start to fill reclamation area using a series of small rock berms to retain the dredged sand fill;
- continue to construct the series of rock berms and fill the reclamation area using additional sand from new dredging areas and possibly other sources;
- the work may be stopped at this stage for a period of 1-3 years before work on the construction of the wharf is started. This would allow some consolidation of the underlying sediments to occur.
- options for construction of the wharf include construction of a caisson structure backfilled with sand, or a piled wharf structure with additional rock berms to increase the factor of safety of the reclaimed area.

Stability analysis was undertaken on three models as follows:

- temporary reclaimed slope using a series of small rock berms and sand filling
- permanent reclaimed slope with additional rock berms placed after construction of piled wharf structure
- caisson structure constructed next to temporary reclaimed slope and backfilled with sand.

The material properties used in the models were as defined in Table 4. For the dredged sand filling it is understood that vibrocompaction is to be limited to those areas that are necessary for stability only and hence the analysis was repeated for different widths of vibrocompaction to assess the minimum area required for treatment.

The loads applied to the permanent models have been based on the following assumptions:

- point loads under each side of the cranes
- distributed loads of 40 kPa on the wharf structure
- distributed loads of 60 kPa in the proposed container storage areas.

For the temporary reclaimed slope a distributed load of 20 kPa has been assumed for the zone within 30 m of the top of the slope and 60 kPa elsewhere.

For the earthquake loading cases a horizontal acceleration coefficient of 0.08g has been adopted as recommended in the Australian Standard for Earthquake Loads (AS1170.4), together with a vertical acceleration coefficient of 0.04g. The earthquake loads have been applied to the models in a pseudo-static analysis.

The various sections across the proposed wharf were assessed to determine which would be the most critical in terms of stability. Sections B4 and A6 were selected as being two representative sections. Section A6 was through the southern end of the proposed reclaimed area where there is an existing deep dredged channel just to the south of the line of the proposed wharf which could affect the stability. Section B4 was typical of the sections through the western side of the wharf where there is an existing deep dredged area which is to be backfilled and deep sediments including significant sand layers.

The detailed results of the analyses are given in Appendix D and are summarised in the following sections.

5.4 Slope Stability Analysis

5.4.1 Temporary Rock Berm

The following table lists the results of stability analysis undertaken for the proposed temporary reclamation comprising a series of rock berms and dredged sand fill. Detailed results are given in Appendix D1. The following two slope models were analysed:

- a series of five small rock embankments placed with an overall slope of 1.5H:1V, with dredged sand back fill
- a series of five small rock embankments placed with an overall slope of 2H:1V, with dredged sand backfill

Table 5 - Results of stability analysis on temporary rock berms

	Berm slope	Materials	Loads	Minimum Factor of Safety	Plus Earthquake
Section A6	1.5:1	Sand fill ϕ = 30	20 kPa - 3m to 30m from top of berm, then 60 kPa	1.22	0.89
		Sand fill ϕ = 35	"	1.44	1.05
	2:1	Sand fill ϕ = 30	"	1.34	0.96
		Sand fill ϕ = 35	"	1.55	1.11
Section B4	1.5:1	Sand fill ϕ = 30	"	1.20	0.88
		Sand fill ϕ = 35	"	1.38	1.01
	2:1	Sand fill ϕ = 30	"	1.45	1.02
		Sand fill ϕ = 35	"	1.74	1.23

During the analysis it became apparent that for these sections the critical material properties were those of the rock berm and the dredged sand fill, as all the critical failure surfaces occurred through the filling rather than through the underlying sands and clays. It had been assessed that the angle of friction of the sand fill could vary depending on the degree of compaction achieved during construction, therefore the analyses were repeated using lower bound and upper bound values of the angle of friction for the sand fill.

The results of the analysis indicate that the minimum factors of safety for slope instability of the 1.5H:1V series of rock berms are approximately 1.2 if it is assumed that the sand fill is not compacted (i.e. the friction angle of the sand is 30 degrees). The factor of safety reduces to about 0.9 when earthquake loads are applied to these slopes, indicating probable slope failure under these conditions. If it is assumed that the sand fill is compacted (friction angle of 35 degrees) then the factor of safety under normal loading increases to about 1.4 and about 1.0 when earthquake loads are applied.

For the 2H:1V series of rock berms, if the sand fill is not compacted the minimum factors of safety are about 1.4, decreasing to about 1.0 when the earthquake load is applied. If the sand fill is compacted then the minimum factor of safety increases to more than 1.55 with the factor of safety under earthquake loads more than 1.1.

Based on these analyses it is concluded that, provided factors of safety of 1.2 are considered acceptable for the temporary berm, a slope of 1.5H:1V may be used, however consideration needs to be given to whether possible slope failure during earthquake loading is acceptable. Higher factors of safety may be achieved either by flattening the slope or by compacting the sand fill. The analyses carried out indicate that most of the critical failure surfaces occur within 20 m of the face of the proposed rock berms. It would therefore be possible to improve the minimum factors of safety by ensuring that the sand fill placed within 20 m from the face of the slope is compacted.

The analysis was repeated using undrained, short term properties for the soils. The undrained properties of the sand layers were the same as the drained properties, however an undrained cohesion of 100 kPa was conservatively assumed for the consistently very stiff to hard clay layers. The results of the analysis were identical to the drained analyses as the failures predominantly occurred through the sand fill and the upper sand layers.

It should be noted that the stability analyses are based on two dimensional models assuming that the failure surface extends over a long length of the proposed reclamation. For smaller, circular failure surfaces the components of shear strength in the third dimension may increase the factors of safety by up to 30%.

5.4.2 Permanent Rock Berm

The analysis was repeated for Section A6 to model the permanent case where an additional rock berm is placed over the slope after construction of a piled wharf structure. It was assumed that a temporary rock berm slope of 1.5H:1V was selected for the first stage of the development and that additional rock was placed over this slope to a level of about RL-2.5 to increase the long term factors of safety with a 6 m high concrete retaining wall on top of the additional rock berm backfilled with sand.

For this model it was assumed that 60 kPa loads were distributed over the whole of the backfilled area with the loads of the piled structure transferred to the foundations at depth.

Two cases were analysed, one with the additional rock berm sloping at 1.4H:1V and the other sloping at 2H:1V. In the analysis the maximum friction angle that can be used for the rock fill is 45 degrees, although in reality higher friction angles may be achieved if large sized rock boulders are used. The results of the analysis are summarised below and are given in detail in Appendix D2.

Table 6 - Results of stability analysis on permanent rock berms

	Berm slope	Materials	Loads	Minimum Factor of Safety	Plus Earthquake
Section A6	1.4H:1V	Rock fill ϕ = 45	60 kPa	1.23	1.02
	2H:1V	"	"	1.42	1.16

Undrained, short term analyses undertaken by substituting an undrained cohesion of 100 kPa for the clay layers gave identical results to the drained analyses because the critical failure surfaces did not penetrate down to the clay layers.

These results indicate that, in order to achieve satisfactory factors of safety for the long term loading conditions, it will probably be necessary to slope the additional permanent rock berm at 2H:1V or flatter.

5.4.3 Caisson

Both sections A6 and B4 were analysed for a possible caisson structure backfilled with dredged sand, assuming that the caisson is founded at the depth of the adjacent dredging, that is about RL-16. The material properties adopted for the caisson ensured that no failure surfaces would pass through the caisson itself.

The analyses indicated that the material properties of the clay foundations were most critical to possible failures and lower bound properties and higher shear strength values were compared during the analysis. Detailed results of the analysis are given in Appendix D3.

Table 7 - Results of stability analysis on caisson

	Materials	Loads	Minimum Factor of Safety	Plus Earthquake
Section A6	Clays c = 5 kPa and ϕ = 18° Sand fill ϕ = 30°	60 kPa behind caisson	1.50	1.08
	Sand fill ϕ = 35°		1.54	1.10
	Clays c = 10 kPa and ϕ = 25° Sand fill ϕ = 30°	"	1.62	1.23
	Sand fill ϕ = 35°		1.79	1.35
Section B4	Clays c = 5 kPa and ϕ = 18° Sand fill ϕ = 30°	"	1.51	1.22
	Sand fill ϕ = 35°		1.60	1.25

Analyses repeated using undrained material properties for the clays gave higher factors of safety because the higher values of cohesion in the clay layers increased the factors of safety against instability.

The results of the drained analyses indicate that acceptable long term factors of safety are achieved for both sections even when lower bound material properties are assumed for the clays. For section A6, however, during earthquake loading the factors of safety were less than normally acceptable except when higher material properties were assumed for the clays.

5.4.4 Effects of Dredging

In order to assess the effects of dredging to deeper levels than RL-16, the analysis for a caisson structure at section B4 was repeated assuming that the adjacent sands had been dredged to RL-20 and that the caisson was founded at a similar level. The results of these analyses are given below and indicate that increasing the depth of dredging immediately adjacent to the proposed caisson significantly reduces the calculated factor of safety.

The factor of safety will increase if the deeper dredged area is kept at least 50 m from the toe of the caisson, as indicated in the table below. As Drawing 5 in Appendix A indicates, however, the greatest thickness of suitable sand available for dredging is immediately adjacent to the location of the proposed wharf and this restriction will reduce the amount of available sand for dredging.

Table 8 - Results of increasing dredging depth next to caisson

Section B4	Materials	Loads	Minimum Factor of Safety	Plus Earthquake
Dredge to RL-16	Clays c = 5 kPa and ϕ = 18° Sand fill ϕ = 30°	60 kPa behind caisson	1.51	1.22
	Sand fill ϕ = 35°		1.60	1.25
Dredge to RL-20	Clays c = 5 kPa and ϕ = 18° Sand fill ϕ = 30°	"	1.10	0.91
	Sand fill ϕ = 35°		1.14	0.94
	Clays c = 10 kPa and ϕ = 25° Sand fill ϕ = 30°	"	1.27	1.05
	Sand fill ϕ = 35°		1.34	1.10
Dredge to RL-20	Clays c = 5 kPa and ϕ = 18° Sand fill ϕ = 30°	"	1.47	1.12
	Sand fill ϕ = 35°		1.58	1.14
50 m from toe of caisson	Clays c = 10 kPa and ϕ = 25° Sand fill ϕ = 30°	"	1.47	1.20
	Sand fill ϕ = 35°		1.58	1.28

5.4.5 Third Runway

Analyses were also undertaken to assess the effects of the proposed dredging on the existing Third Runway reclamation and retaining walls. The results of these analyses are given in Appendix D4.

Documents provided by Sydney Airports Corporation Limited indicate that the majority of the Third Runway area is retained by a reinforced earth wall extending from about RL-3 to RL+2, with a 10 m wide horizontal berm at the toe of the wall which has been protected against scour by use of a concrete filled geofabric.

Analysis of the existing retaining wall and slope next to the Third Runway gave factors of safety of 1.6 for long term conditions and 1.2 under earthquake loading.

The analysis was repeated assuming that dredging of the sand layer would approach as close as possible to the Third Runway, assuming a maximum slope of 3H:1V for the dredged area starting at the edge of the scour protection layer, that is 10 m out from the retaining wall. The calculated factors of safety for failure into the proposed dredged area were 1.6 for long term conditions and 1.1 under earthquake loads. This indicates that extending the dredged area close to the Third Runway will not have any effect on the long term stability but that there may be a slight reduction in the factor of safety under earthquake loads.

Further analysis was undertaken to assess the distance from the retaining wall at which dredging would have no impact on the existing stability even under earthquake loading. The results of this analysis are given below in Table 9 and indicate that if the dredging is kept at least 35 m from the retaining wall along the Third Runway the stability of the wall and adjacent slope remains the same as the existing conditions.

Table 9 - Results of dredging next to Third Runway

Model (assumes 3H:1V slope into dredged area)	Minimum Factors of Safety	
	Long Term	With Earthquake
Existing slope	1.63	1.22
Dredge 10m from wall	1.63	1.05
Dredge 15m from wall	1.63	1.09
Dredge 20m from wall	1.64	1.12
Dredge 25m from wall	1.64	1.16
Dredge 30m from wall	1.64	1.20
Dredge 35m from wall	1.64	1.22

5.4.6 Effects of Varying Earthquake Loads

In order to assess the effects of varying the earthquake load, the stability analysis for the temporary rock berm comprising a series of five small rock berms at section A6 was repeated using different magnitudes of the horizontal acceleration coefficient. The results of these analyses are summarised below in Table 10, assuming upper bound values of ϕ = 45 degrees for the rock fill and ϕ = 35 degrees for the sand fill and an applied long term load of 60 kPa on the surface of the filled area.

Table 10: Results of varying earthquake coefficients

Horizontal Earthquake Coefficient	1.5H:1V slope	2H:1V slope
0 g	1.38	1.52
0.02 g	1.29	1.41
0.04 g	1.20	1.31
0.06 g	1.13	1.22
0.08 g	1.06	1.14
0.1 g	1.00	1.07

The value of 0.08 g recommended by the Australian Standard 1170.4 is based on a 10% chance of exceedance in 50 years, or a return period of about 475 years. The results above indicate that the calculated factors of safety for this earthquake loading are possibly only just acceptable for the 2H:1V slope.

It is noted, however, that pseudo-static analysis technique used to assess the effects of earthquake loading on the stability known to be conservative, particularly as the earthquake loads apply over a very short time period. Given that the analyses indicate that the seismic loading condition is critical for design it is recommended that more detailed analyses be undertaken in the design phase to assess the likely displacements under earthquake loads. The results can then be assessed to determine whether the structures can tolerate the predicted displacements under earthquake loads.

5.5 Summary

The results of the stability analysis indicate the following:

- for stability of the proposed rock berms the friction angles of the dredged sand fill and the imported rock fill are critical;
- for the temporary series of rock berms proposed for the first stage of the reclamation process adequate factors of safety can be achieved if either the rock berms are battered at 2H:1V or the sand fill is well compacted. The factors of safety for 1.5H:1V slopes with uncompacted backfilling are marginal, although it is considered that acceptable factors of safety could be achieved for these slopes by compacting the sand fill within a 20 m wide zone from the face of the rock berm. The selection of the batter slopes and the degree of compaction required will

ultimately depend on a comparison of the costs of each of the options together with an assessment of the risks and consequences of slope failure;

- for the permanent rock berm to be placed over the temporary berm, in conjunction with construction of a piled wharf structure, it is considered that a 2H:1V batter slope is required, together with compacted sand backfilling, in order to achieve adequate long term factors of safety;
- for analysis of the proposed caisson structures the properties of the clay layers became critical. Even assuming lower bound shear strength values for the clays the long term stability of the caisson was acceptable. For earthquake loading, however, the factors of safety were less than normally acceptable if lower bound properties were assumed;
- an assessment of the effects on increasing the proposed dredging depth from RL-16 to RL-20 indicated that the deeper dredged area should be kept at least 50 m from the toe of a proposed caisson structure;
- the analysis indicated that dredging close to the Third Runway will have no significant effect on the stability of the existing structure under long term conditions. Under earthquake loading the dredging could slightly reduce the factor of safety, however, if the dredging is kept at least 35 m from the existing retaining wall there would be no change to the existing stability.

6. SETTLEMENT ANALYSIS

6.1 Settlement Design Parameters

The information available for analysis of settlement at the site comprises a series of consolidation tests undertaken by Coffey Partners mainly on the upper (Units 3A and 3B) and lower (Unit 4A) clays. The results of these tests are summarised in Appendix C. The range of consolidation test results, together with average values, are given below:

Table 11 - Summary of Laboratory Test Results

Strata	Coefficient of volume change m_v (m^2/kN)	Coefficient of consolidation c_v (m^2/year)	Creep coefficient c_α
Units 3A and 3B - upper peaty clays and fissured clays			
Range	2×10^{-5} to 2×10^{-4}	0.04 to 32	0.001 to 0.013
Average	8.7×10^{-5}	5.3	0.004
Unit 4A - lower fissured clays			
Range	2×10^{-5} to 1×10^{-4}	0.2 to 3	0.001 to 0.01
Average	7.3×10^{-5}	1.2	0.003

It can be seen from these results that relatively consistent results have been obtained from all the tests on the clay samples. It should be noted, however, that laboratory tests of consolidation parameters are often unreliable, principally due to the small sample size and the disturbance of the soils during sampling and preparation for testing.

An alternative method for estimating the consolidation parameters can be derived by estimating modulus values from the SPT test results and deriving values for m_v using the following equations and assuming reasonable values for Poisson's ratio (ν).

$$m_v = \frac{(1 + \nu)(1 - 2\nu)}{(1 - \nu) E'}$$

Accordingly the following consolidation parameters can be estimated for some of the possible strata on the site.

Table 12 - Estimated Coefficients of Volume Change

Strata	Estimated Modulus E' (MPa)	Estimated Poisson's Ratio ν	Calculated mv (m^2/kN)	Laboratory mv (m^2/kN)
Loose sands	15	0.3	4.95×10^{-5}	
Medium dense sands	50	0.3	1.49×10^{-5}	
Dense sands	100	0.3	7.43×10^{-6}	
Very dense sands	150	0.3	4.95×10^{-6}	
Stiff clays	10	0.3	7.43×10^{-5}	
Very stiff clays	20	0.3	3.71×10^{-5}	8×10^{-5}
Hard clays	40	0.3	1.86×10^{-5}	8×10^{-5}

As indicated above there is a relatively small amount of data available on consolidation properties of the different strata on the site, in which case the best source of information should be monitoring of the settlement which has occurred on the existing port reclamation. This reclamation was undertaken in the late 1970s to the same level as the proposed new terminal over similar strata and it would be expected that settlement of the new reclamation area would be similar.

Connell Wagner undertook a study in December 2000 to investigate reported movements at the existing Patrick Terminal. They reviewed the available survey information and came to the conclusion that some settlement had occurred in some areas but that it was not possible to quantify this settlement. There also seemed to be a link between the areas of observed settlement and the development of sinkholes adjacent to stormwater drains, which means that some of the observed settlement areas had probably resulted from erosion rather than settlement.

Sydney Ports Corporation has monitored the levels of the rear crane rail on the Patrick Terminal in the period between 1979 and 2000. Copies of the surveyed results are included in Appendix E and indicate that there has been some variability in the magnitude of the measured settlement, ranging from about 20 mm at the eastern end of the wharf to about 140 mm near Chainage 450 along the wharf.

The Connell Wagner study included bores and CPTs at about Chainages 210 and 430 along the existing wharf which were drilled through the filling and the underlying natural soils down to bedrock. Using the soil profiles derived from the CPTs and bores, together with the measured settlements at nearby monitoring points it is possible to assess whether the material properties assumed for the different layers are reasonable.

A comparison has been made between the predicted and measured settlements on Patrick Terminal. For the analysis it has been assumed that filling was originally placed to a height of about 4 m above ISLW and an additional surface load of 40 kPa has been applied, from either a 2m high surcharge during construction or crane or vehicular loading since. The detailed results of the analysis are given in Appendix E.

The results of the analysis indicated that there was a good correlation between the measured and predicted settlements using the consolidation parameters assumed for the different layers. The analysis did indicate, however, that the values of coefficient of consolidation (c_v) for the clays were probably lower than had been originally assessed.

It is noted that the CPTs undertaken through the filling in the Patrick Terminal showed that there were some clayey layers within the filling, particularly near the base of the filling, which had a significant effect on the settlement behaviour of the profiles. Nevertheless it was considered that there was also a contribution to the settlement from the underlying natural soils. The results are summarised below:

Table 13 - Analysis of Settlement at Patrick Terminal

Profile	Thickness (m)	Predicted Settlement (mm)	
		1 to 10 years	10 to 50 years
Chainage 210			
Filling	19	32	17
Natural soils	41	34	20
Chainage 430			
Filling	21	51	35
Natural soils	22	16	14

Using a combination of laboratory testing, theoretical correlations, previous experience and back-analysis of the settlement of the rear crane rail on Patrick Terminal, the following consolidation parameters are assessed as being reasonable values for use in the analysis of settlement on the site.

Table 14 - Consolidation Design Parameters

Unit	Description	Coefficient of volume change m_v (m^2/kN)	Coefficient of consolidation c_v (m^2/year)	Creep coefficient c_α
1	Very loose to loose silty sands and clayey sands	5×10^{-5}	100	0
2	Dense to very dense sands	5×10^{-6}	100	0
3A	Very stiff to hard peats	8×10^{-5}	10	0.005
3B	Very stiff to hard fissured clays and organic clays	6×10^{-5}	0.5	0.003
3C	Dense to very dense peaty sands	5×10^{-6}	50	0
4A	Very stiff to hard fissured clays	4×10^{-5}	0.5	0.001
4B	Very dense silty sand and clayey sand	5×10^{-6}	50	0
5	Dense to very dense clayey sands	5×10^{-6}	20	0

6.2 Settlement Calculations

Estimations of possible settlement in different parts of the proposed reclamation area have been made using the strata intersected by selected bores to rock and the assumed consolidation parameters given above. The calculations have been made using one dimensional consolidation theory, which will be appropriate for most of the reclaimed area, and assuming that fill is placed to RL 3.7 (ISLW) with an applied surface load of 60 kPa. The detailed settlement calculations are given in Appendix E and are summarised below. The bores analysed were selected to give a range of depth of sediments as well as a range of proposed filling.

Table 15 - Estimated Total Settlements

Bore	Surface level (ISLW)	Rock level (ISLW)	Estimated total settlement (mm) after filling at		
			1 year	10 years	50 years
CP1	-1.65	-35.7	30	130	210
CP3	-15.10	-46.9	150	230	290
CP5	-18.40	-60.3	180	380	520
CP8	-21.95	-45.0	130	260	340
CP10	-15.95	-39.5	70	160	260
CP12	-11.70	-73.8	60	150	240

The above estimates show the possible range of settlements across the reclaimed area. It should be noted that due to the conservative assumptions used to derive the consolidation parameters these are probably upper estimates of the settlement. Nevertheless, the estimates give an indication of the possible differential settlements between different parts of the site over time. Essentially the settlements are directly related to the thicknesses of the clay layers, particularly Unit 3A peats and clayey peats, at each location. In the areas where there is a greater total thickness of sediments, i.e. the depth to rock is greater, there are often greater total thicknesses of clay and peat, resulting in higher estimated settlements.

Assuming that the construction takes at least one year to complete, the total settlement which is estimated to occur between 1 and 10 years is expected to be in the range of 80 - 200 mm, with a further 60 - 140 mm of settlement expected to occur in the following 40 years. Due to creep properties of the peats and clays and expected low coefficients of consolidation for the clay layers it is anticipated that settlement will continue for many years, gradually reducing with time.

The above estimates of settlement do not take into consideration the settlement of the dredged sand filling itself. The settlement of the filling will depend upon the degree of compaction which can be achieved during reclamation and the time over which the filling is placed. It is estimated that, provided the sand fill is adequately compacted, the settlement of the filling will be up to about 0.2% of the height of the fill. For example where the fill is only about 6 m high the estimated maximum settlement of the filling is about 10 mm, but where the fill is 26 m high the estimated maximum settlement of the filling is about 50 mm. Most of this settlement, however, is likely to happen during the construction phase of the project, i.e. within the first 1-2 years, as the filling is being placed below water level.

The following table lists the predicted settlements for each of the soil units at each of the analysed locations for the period between 1 year and 50 years.

Table 16 - Settlement of Different Units (1 to 50 years)

	Depth to seabed (m)		Unit 1	Unit 2	Unit 3	Unit 4	Unit 5
CP1	1.65	*Thickness (m)*	*2*	*4*	*21.8*	*5.9*	
		Settlement (mm)	0	0	151	31	
CP3	15.1	*Thickness (m)*	*1.5*	*6.3*	*8.2*	*14.8*	
		Settlement (mm)	0	0	57	69	
CP5	18.4	*Thickness (m)*	*3.2*	*0.8*	*11.5*	*24*	*2.3*
		Settlement (mm)	0	0	199	142	0
CP8	21.95	*Thickness (m)*	*0.7*	*1.3*	*7*	*13.2*	*0.8*
		Settlement (mm)	0	0	107	97	0
CP10	15.95	*Thickness (m)*	*0.8*	*3.2*	*3.8*	*9.5*	*0.5*
		Settlement (mm)	0	0	122	65	0
CP12	11.7	*Thickness (m)*	*0.6*	*10.9*	*14*	*36.3*	*0.2*
		Settlement (mm)	0	1	47	125	0

It is apparent from the above results that there is expected to be no significant settlement of Units 1, 2 or 5. Most of the settlement is expected to occur within the clay or peat layers within Units 3 and 4.

It is understood that a maximum acceptable settlement for a caisson type wharf structure is about 100 mm over 50 years. Depending on the actual soil conditions along the line of the proposed wharf, which should be checked during the detailed design phase, it is possible that it would be necessary to remove the Unit 3 material from beneath the foundation area of the caisson structure in order to ensure that the settlement does not exceed 100 mm. In some areas this could require dredging to 20-30 m below existing sea bed levels.

In the proposed new dredging area it is expected that the effective pressure on the deep clay layers will be reduced by the removal of 10-15 m of very dense sands. Accordingly it is likely that there will be some rebound, or upward movement, of the soils as the sand is removed. Unfortunately the rebound curves of the consolidation tests undertaken in previous investigations have possibly been affected by the use of distilled water, nevertheless it is expected that there may as much as 50-100 mm rebound movement when the sands are removed. This movement will occur over some time and it is unlikely to have any significant impact on the proposed structures, other than some possible component of additional side friction on piles if they are used.

6.3 Summary

There is only limited consolidation data available on the soils at this site and information on settlement of the existing port is also limited. Using assumed consolidation parameters it is estimated that there will be gradual settlement of the soils under the filled area by 80 - 200 mm in the period between 1 year and 10 years after construction is completed with ongoing creep settlement. The filling itself is likely to settle by a further 10 - 50 mm but this expected to occur either during the construction period or shortly afterwards.

Investigations at the existing Patrick Terminal indicated that there were some clayey layers included within the sand filling and calculations have indicated that these layers have probably had a significant effect on the long term settlement of the area. If clay or peat layers are included within the proposed filling of the new reclaimed area then the settlements may be greater than estimated above.

It is considered that preloading of the site would probably reduce the post-construction settlement of the filling, but would not have a significant effect on the long term settlement of the underlying clays. The disadvantage of preloads would be that the stability of the proposed reclamation would probably be reduced during the period of the preloading.

Other methods of reducing either the settlement or the time over which the settlement occurs such as wick drains or stone columns are considered not suitable for this site. Wick drains are conventionally pushed into soft clay soils using mandrels mounted on drilling rigs. Due to the thickness of sands and filling above the clay layers, as well as the very stiff to hard consistency of the clays, it would be necessary to predrill the holes for the wick drains. Both wick drains and stone columns, if used, would have to be installed to significant depths (possibly up to 30 m) to achieve reductions in the settlement and are likely to be extremely expensive. Wick drains theoretically reduce the time required for consolidation by shortening the drainage path for clay soils, however experience has shown that creep settlements may start earlier when wick drains are installed. Given that a large proportion of the predicted settlement is expected to be related to creep movements, it is considered that wick drains would not significantly improve the settlement on the site.

In order to limit the settlement of the soils beneath proposed caisson structures to acceptable levels it would probably be necessary to remove the Unit 3 soils, and possibly also some of the Unit 4 soils. This would require dredging to depths of 20-30 m below existing sea bed levels in some areas along the line of the proposed wharf.

7. FOUNDATIONS

At this stage the main types of possible structures on the site include filled areas, rock berms, caissons, piled wharves and footings for cranes.

An assessment of the potential for liquefaction of the soils at the site indicates that there is a very low potential for the dense and very dense sand layers (Units 2, 3C and 4B) to liquefy. There is, however, a reasonable probability that the very loose and loose silty sands and sands within the surface sediments (Unit 1) could liquefy. In such an event the liquefaction of these layers beneath structures could cause unacceptable settlements. It is therefore recommended that all the surface sediment layers (Unit 1) be removed from under proposed caisson structures and under the footprint of the rock berm.

An assessment of the liquefaction potential of the dredged sand filling indicates that this layer is unlikely to liquefy provided adequate compaction is achieved. It is suggested that vibrocompaction or similar techniques be undertaken during placement of this material to ensure that adequate compaction is achieved to reduce the settlement and minimise the risk of liquefaction, and the degree of compaction of the filled area should be confirmed by cone penetration tests during construction.

After the Unit 1 soils have been removed it is considered that the underlying very dense sands and very stiff to hard clays will provide adequate foundations for the proposed structures. The design of the footings for the structures will be governed by settlement under the different loads and will need to be considered during the detailed design phase. For preliminary design purposes, however, an allowable bearing pressure of 100 kPa may be assumed.

If a caisson structure is to be used then it is suggested that this structure should be founded on a uniform bed of compacted rock, possibly 0.5 m thick. The purpose of this rock bed would be to provide a uniform foundation for the caisson to try to limit the possible differential settlement under different parts of each caisson. A small amount of settlement under one corner of the caisson may lead to unacceptable rotations of the whole caisson, leading to increased bearing under parts of the caisson and then further settlement.

For a piled wharf structure it is considered that driven piles, either precast concrete or hollow tube steel tubes, are likely to provide the best options. For piles driven to refusal the loads which can be carried will probably be limited by the structural capacity of the piles rather than the strength of the soils. It is possible to design the piles to have minimal settlements under load, however, it will then be necessary to consider the possible differential settlements between the piled structure and other structures founded at shallow depth on the filling.

The driven piles are likely to reach refusal in the very dense sand layer where it is thicker than about 2-3 m and probably also in the hard clays, depending on the energy used during installation. The advantage of the hollow tube piles is that the centres of the piles may be mucked out if such refusal occurs above the depth required for protection against scour.

Standard bored piles could also be considered, however, these types of piles are less able to develop side friction and it may be necessary to drill down to rock to ensure sufficient load capacity. Given that the rock extends down to more than 70 m in places, this option is not considered viable. Another option which could be considered by the designers are enlarged base piles, however the equipment available to install these piles may not be able to drill to the depths required.

8. MATERIALS ASSESSMENT

The Unit 2 sands are considered the most suitable soils on the site for dredging purposes. These sands have low percentages of fines, as shown on the grading curves given in Appendix C, and the layer is relatively uniform across the site with a clear lower stratigraphic boundary marked by peaty layers.

Preliminary calculations of the volumes of fill required for the reclamation indicate that approximately 3.4 million cubic metres of fill is needed to fill the previously dredged area and a further 4.9 million cubic metres is needed to fill the area of the proposed new terminal to RL 3.7. Thus a total of about 8.3 million cubic metres of fill is required for the project, assuming that caisson type walls are used around the edge of the terminal. If rock berms are used then the required volume will be reduced slightly due to the slope of the rock berms and the volume of rock used in this construction.

An estimate of the sand available within the proposed dredged area is about 7.2 million cubic metres, assuming an average thickness of sand of 15 m over the whole area. In reality the thickness of the sand layer varies from about 9 m to 19 m over this area, so to maximise the extracted volume it will be necessary to dredge deeper in some areas than others. Approximate contours of the base level of the sand layer are shown on Drawing 5 in Appendix A.

Based on the above estimated volumes and considering that there will inevitably be material which is unsuitable due to inclusion of fines, and material which is lost during the dredging process, it is apparent that there may not be sufficient suitable sand in the proposed dredged area for use as fill. The options available for obtaining more fill include:

- extending the proposed dredging area closer to the Third Runway This option will be limited by ensuring that a stable dredged slope (maximum 3H:1V and preferable 5H:1V) is left beyond the existing scour protection for the retaining wall along the Third Runway. A review of the available bore logs indicates that there is probably an increase in the percentage of fines included in the sand layers closer to the Third Runway, and there may be operational restrictions caused by height limits of equipment which can operate next to the runway.
- extending the proposed dredging area further to the north. The bore logs indicate that there is a probable slight increase in fines within the sands towards the north which will mean a greater percentage of wasted material in this area. There may also be environmental impacts associated with dredging in shallower areas.

- importing suitable fill from other sites in Sydney.

9. DREDGING ASSESSMENT

Dredging has been satisfactorily undertaken within this area for the Third Runway project, using the sands from Unit 2 and it is considered that further similar dredging will also be successful.

The sands of Unit 2 are considered suitable material for dredging and subsequent compaction as fill, with low percentages of included fines. The sands do include relatively high percentages of quartz grains which will result in abrasion of the dredging tools, but there are no reports of cemented layers within the Unit 2 sands which could cause difficulties with dredging and it is anticipated that standard operational measures and precautions will control turbidity around the dredging area.

It is expected that there will be some peat and clay layers included within the sands and it is possible that there may be an increase in these materials towards the north and west. The quality and quantity of the sand layers in the proposed dredge areas should be confirmed by additional bores during the detailed investigation.

10. ADDITIONAL INVESTIGATIONS

It is recommended that during the detailed design stage the following additional investigations and tests be undertaken to confirm the design parameters:

- minimum of additional eight bores drilled to rock along the alignment of the proposed wharf to confirm the strata along this alignment and to obtain samples for additional laboratory testing for consolidation and shear strength parameters.
- minimum of eight cone penetration tests along the alignment of the wharf to provide additional information on the in-situ strength properties of the soils as well as giving detailed information for design of piles.
- a further eight cone penetration tests scattered over the proposed reclamation area to obtain estimates of in-situ consolidation parameters which would be used to check settlement estimates.
- additional twelve bores drilled to the base of Unit 2 sand layer within the proposed dredge area to confirm the quality and quantity of the available sands. In addition these bores could be used to obtain samples for environmental testing.
- a further review of settlements on the existing container terminal, comparing current levels to as constructed levels, to determine the magnitude of any settlement on the site.
- detailed seismic analyses to assess the likely displacements of structures under specified earthquake loads

11. CONCLUSIONS

A review of the available geotechnical information on the site and surrounding area indicates that there is a deep valley within the sandstone bedrock which essentially runs beneath the area of the proposed reclamation. This drowned valley has been filled with sediments, comprising mostly clays in the lower areas interbedded with some sandy layers. The lower clays contain abundant shell fragments indicating marine origins and the upper clays include numerous peat layers and organic silts indicating a deltaic depositional environment. There is a gradational change between the lower and upper clays indicating a gradual change in depositional environment. Fissuring has been noted through most of the clay layers.

Overlying the clay deposits there is a relatively uniform sand layer which has few fines and in places has been dredged for filling of the Third Runway area. The available bore data indicates that there may be a slight increase in the amount of fines in this layer towards the north and west. There have been no cemented zones recorded in this layer and it is considered that standard techniques can be used for dredging this material. Preliminary calculations of volumes, however, indicate that there may be a shortfall of filling material and additional fill may have to be obtained either by extending the dredged area or by importing fill from other sites.

The stability analyses of the temporary rock berm embankments indicated that a series of small rock berms could be used. The options available for achieving adequate factors of safety for short term loading conditions are to slope the berms at 2H:1V or to compact the sand filling and slope the rock berms at 1.5H:1V. An intermediate option of a 1.5H:1V slope with a 20 m wide zone of compacted sand immediately behind the face may provide a more economic option. In all cases the risks and consequences of failure need to be considered.

For a permanent rock berm to be constructed in conjunction with a piled wharf structure a maximum batter slope of 2H:1V is required to achieve adequate factors of safety. In addition the sand filling placed behind the permanent rock berm and in front of the temporary rock berm should be well compacted.

Detailed seismic analysis will be required to assess the likely deformations of the selected slope under earthquake loading. Based on the pseudo-static analysis undertaken to date the overall slope of a rock berm embankment should be limited to a maximum of 2H:1V in order to achieve satisfactory factors of safety.

Stability analyses of possible caisson structures indicate adequate factors of safety against slope failure may be obtained. However, if the proposed dredge area is to be deepened to about RL-20 this deeper dredged area should not be closer than 50 m to the toe of the caisson.

Due to the potential for liquefaction under earthquake loads and excessive settlement under loading it is recommended that all the Unit 1 sediments be removed from beneath proposed caisson structures and under the footprint of the rock berms. In addition, if a caisson structure is used then a bed of crushed rock is recommended to provide a uniform founding material to reduce the differential settlements which may occur under loading. In order to limit the settlement of the foundations under a caisson structure to acceptable levels it may be necessary to dredge out some of the clay and peat layers from Units 3 and 4, possibly to depths of 20-30 m below existing sea bed levels.

Piles may be used to support the wharf structure and driven piles are considered to be the most appropriate pile type for this site. Large diameter hollow tube steel piles have an advantage over precast concrete piles because they can be mucked out and driving continued if they reach refusal above the scour level. Driven piles are likely to reach refusal within both the very dense sands and the hard clays.

The piles may be designed to have minimal settlement under load, however, settlement of the reclaimed area may be as much as 200 mm over a ten year period and hence structures spanning between the piled structure and the filled area would need to be designed such that they can be adjusted when the differential settlement becomes excessive.

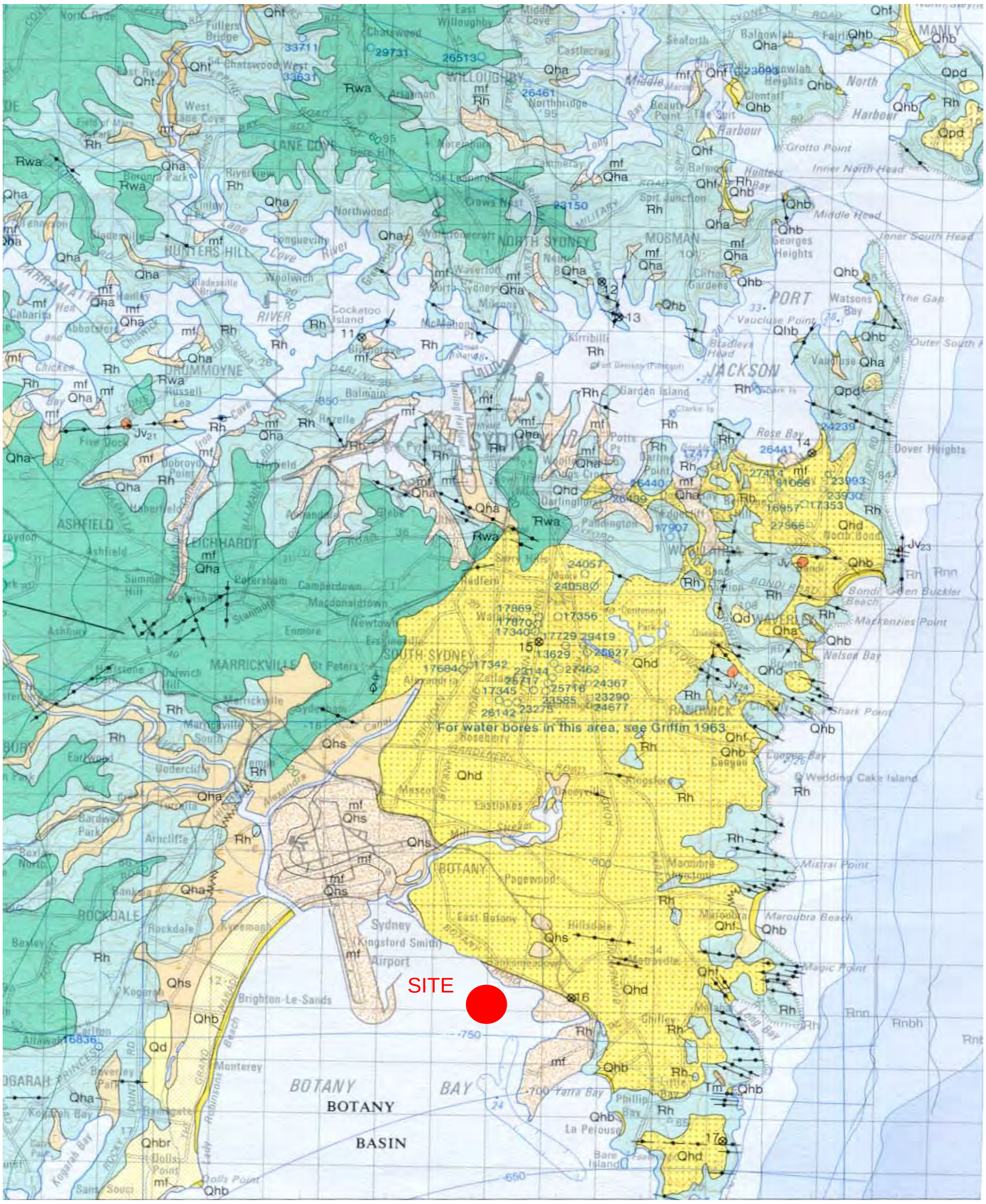
DOUGLAS PARTNERS PTY LTD

Reviewed by

Fiona MacGregor
Principal

Michael J Thom
Principal

APPENDIX A
Drawings



mf - Filling

Qhd - Sands

Rh - Hawkesbury Sandstone

Rwa - Ashfield Shale

Igneous dyke

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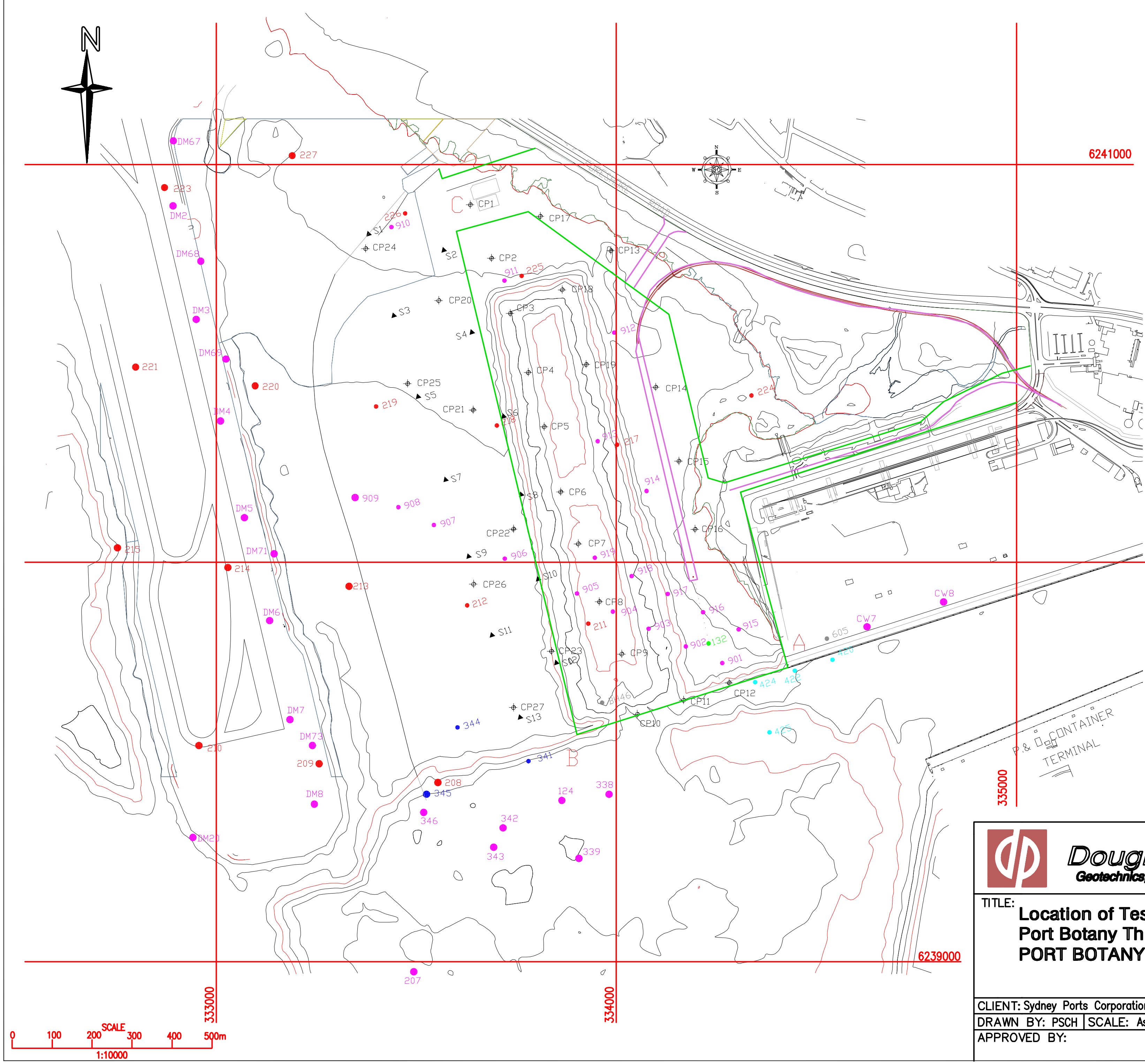
Melbourne, Perth, Wyong, Townsville, Cairns, Darwin

Title	**Regional Geology** **Proposed Third Terminal** **Port Botany**		
Client: Sydney Ports Corporation			Office: Sydney
Drawn by: FM	Scale: NTS	Project Number: 35224	Drawing No. 1
Approved by:		Date: August 2002	



LEGEND :

- BOREHOLE LOCATIONS OF COFFEY REPORT JUNE 1999
- ENVIRONMENTAL SEABED GRAB SAMPLES OF COFFEY REPORT JUNE 1999
- OTHER KNOWN BOREHOLE LOCATIONS
- PROPOSED QUAY FACE FOR RECLAMATION LAND AND WHARF FACILITIES

RECLAMATION LAND AND WHARF FACILITIES CO-ORDINATES

A	334429	6239738
B	333898	6239568
C	333571	6240933

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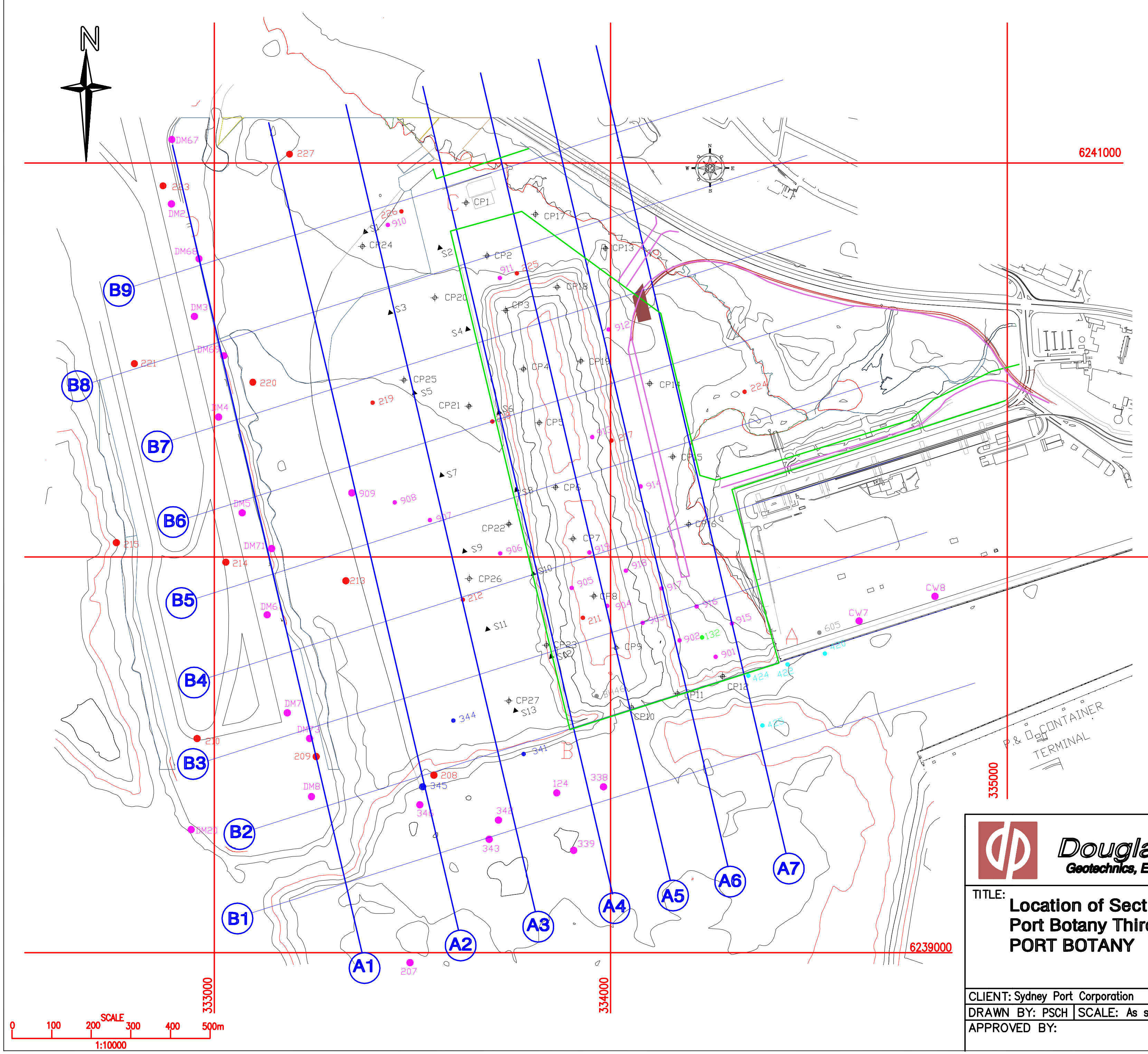
TITLE:
Location of Test Bores
Port Botany Third Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 25.6.2002	DRAWING No: 2



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- BOREHOLE LOCATIONS OF COFFEY REPORT JUNE 1999
- ENVIRONMENTAL SEABED GRAB SAMPLES OF COFFEY REPORT JUNE 1999
- OTHER KNOWN BOREHOLE LOCATIONS
- PROPOSED QUAY FACE FOR RECLAMATION LAND AND WHARF FACILITIES

RECLAMATION LAND AND WHARF FACILITIES CO-ORDINATES

A	334429	6239738
B	333898	6239568
C	333571	6240933

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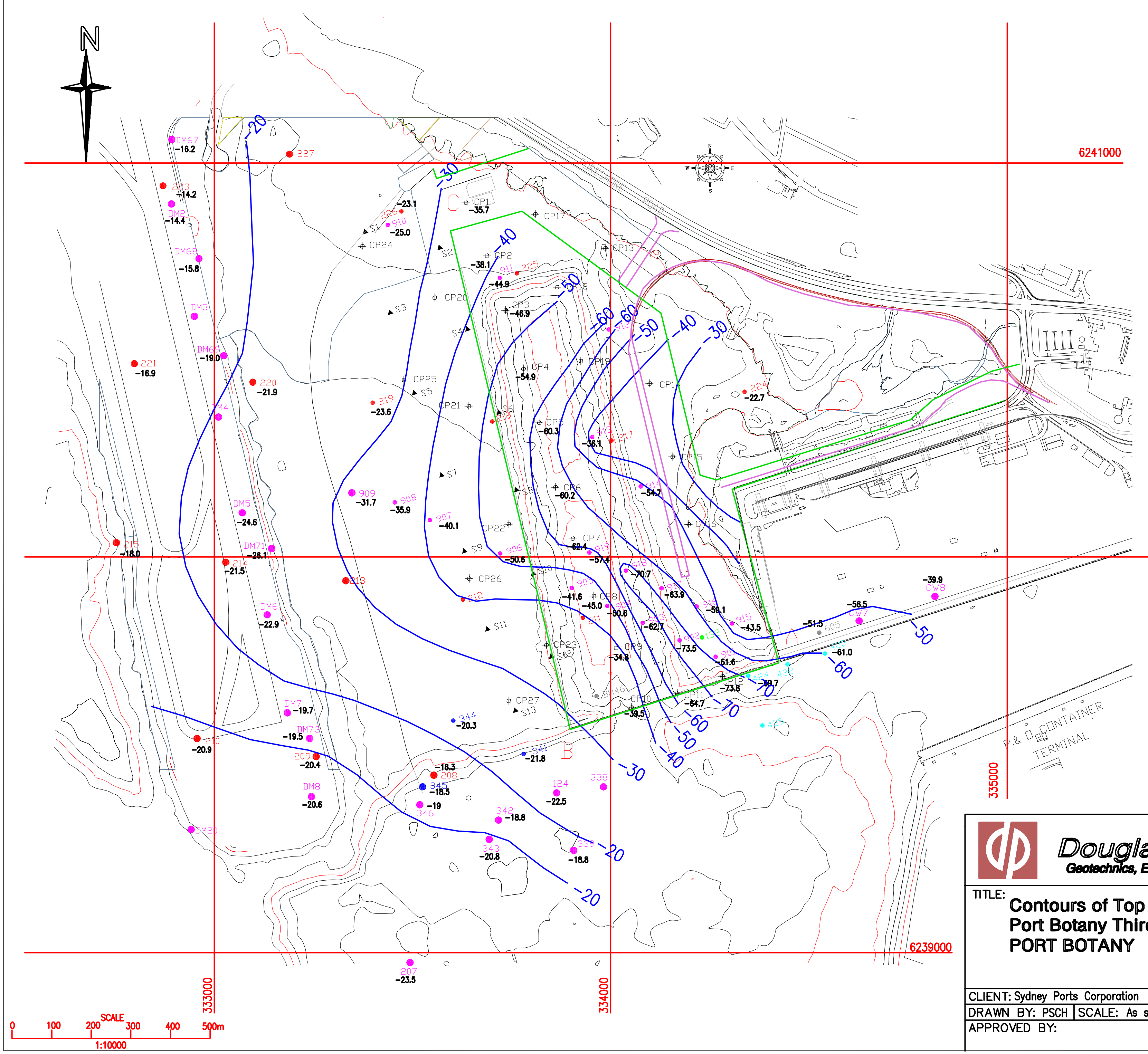
TITLE:
Location of Sections
Port Botany Third Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Port Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 25.6.2002	DRAWING No: 3



LEGEND :

- BOREHOLE LOCATIONS OF COFFEY REPORT JUNE 1999
- ENVIRONMENTAL SEABED GRAB SAMPLES OF COFFEY REPORT JUNE 1999
- OTHER KNOWN BOREHOLE LOCATIONS
- PROPOSED QUAY FACE FOR RECLAMATION LAND AND WHARF FACILITIES

RECLAMATION LAND AND WHARF FACILITIES CO-ORDINATES

A	334429	6239738
B	333898	6239568
C	333571	6240933

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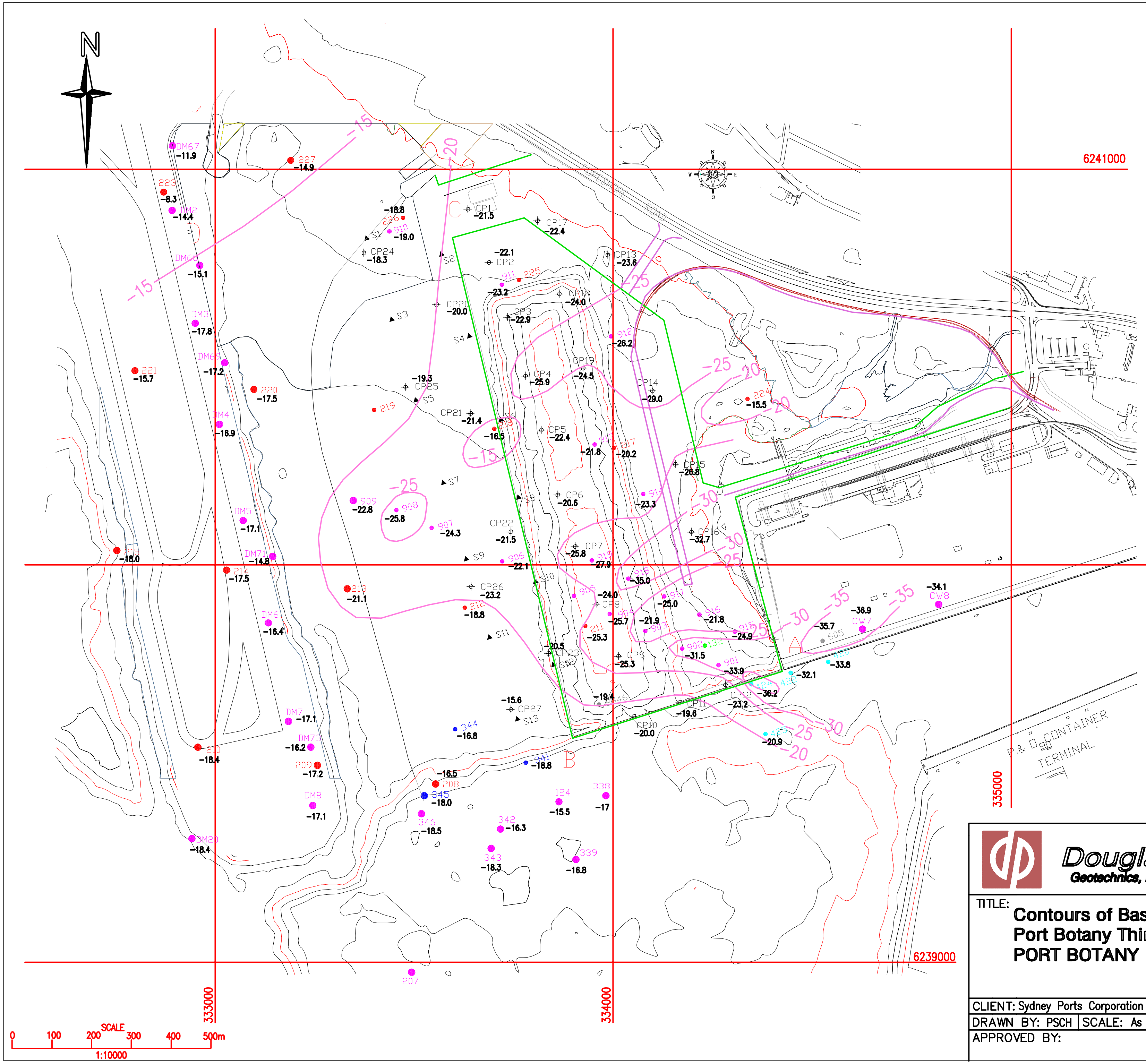
TITLE:
Contours of Top of Bedrock
Port Botany Third Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 26.6.2002	DRAWING No: 4



LEGEND :

- BOREHOLE LOCATIONS OF COFFEY REPORT JUNE 1999
- ENVIRONMENTAL SEABED GRAB SAMPLES OF COFFEY REPORT JUNE 1999
- OTHER KNOWN BOREHOLE LOCATIONS
- PROPOSED QUAY FACE FOR RECLAMATION LAND AND WHARF FACILITIES

RECLAMATION LAND AND WHARF FACILITIES CO-ORDINATES

A	334429	6239738
B	333898	6239568
C	333571	6240933

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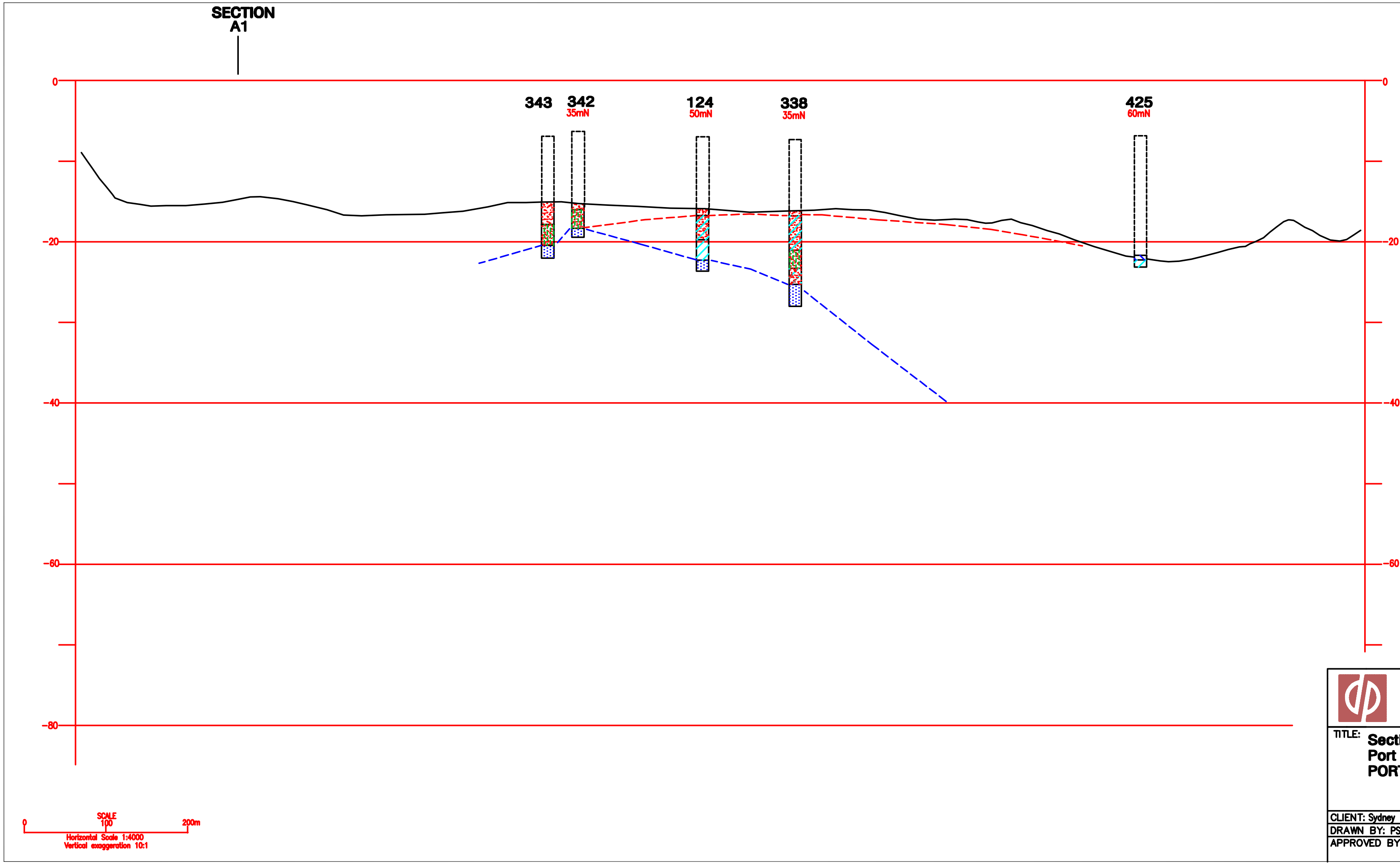
TITLE:
Contours of Base of Sand Layer
Port Botany Third Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 26.6.2002	DRAWING No: 5

APPENDIX B
Summary Sections



LEGEND

- SAND
- CLAY/SILTY CLAY
- SILT
- PEAT
- SILTY SAND
- SANDY CLAY/CLAYEY SAND
- ORGANIC CLAY/PEATY CLAY
- PEATY SAND
- ORGANIC SILT/PEATY SILT
- SANDSTONE/SILTSTONE

SOIL CONSISTENCY

VS	VERY SOFT
S	SOFT
F	FIRM
ST	STIFF
VST	VERY STIFF
H	HARD
VL	VERY LOOSE
L	LOOSE
MD	MEDIUM DENSE
D	DENSE
VD	VERY DENSE

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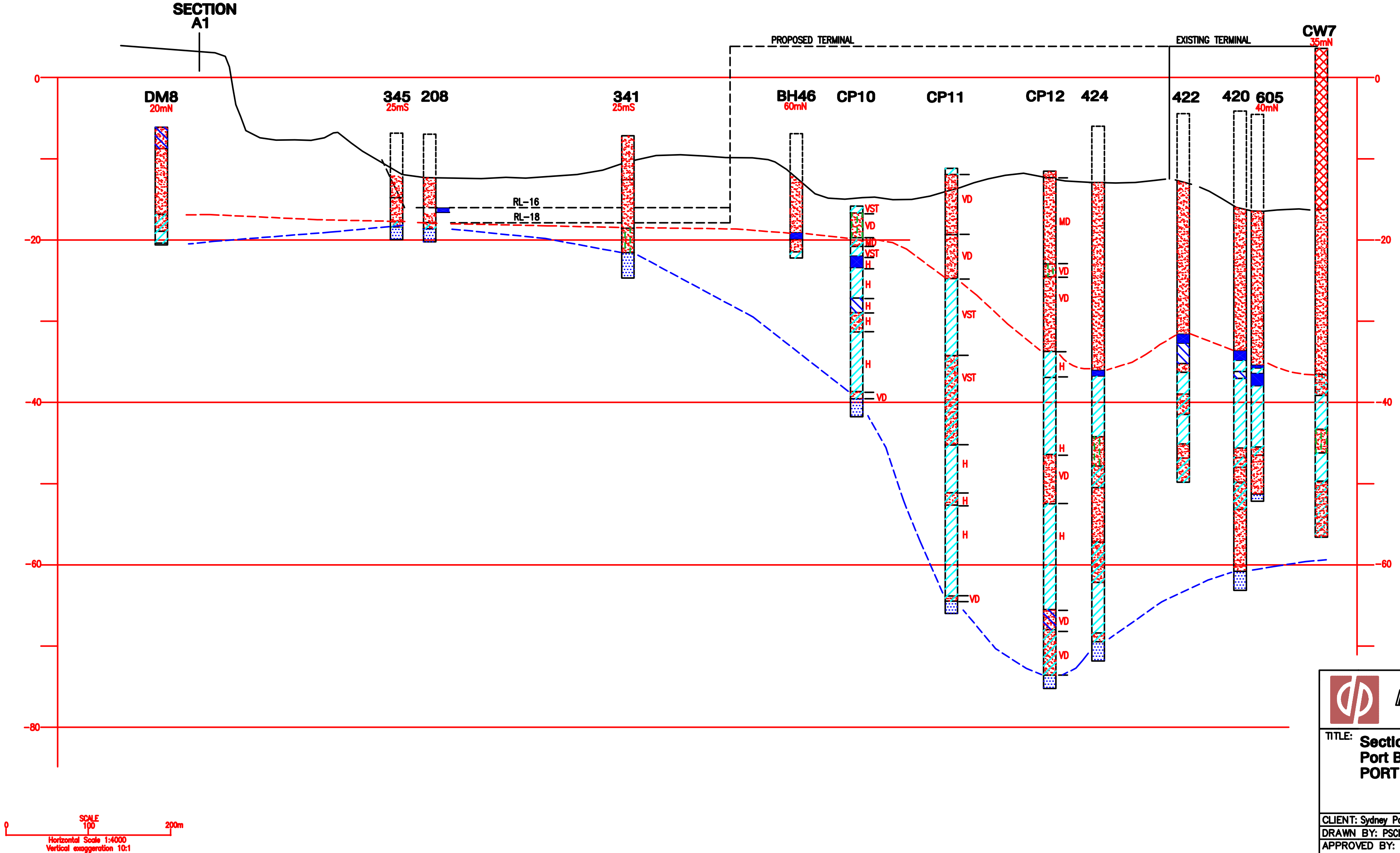
TITLE: **Section B1**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation

DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 26.6.2002	DRAWING No: 6



LEGEND

- SAND
- CLAY/SILTY CLAY
- SILT
- PEAT
- SILTY SAND
- SANDY CLAY/CLAYEY SAND
- ORGANIC CLAY/PEATY CLAY
- PEATY SAND
- ORGANIC SILT/PEATY SILT
- SANDSTONE/SILTSTONE

SOIL CONSISTENCY

VS	VERY SOFT
S	SOFT
F	FIRM
ST	STIFF
VST	VERY STIFF
H	HARD
VL	VERY LOOSE
L	LOOSE
MD	MEDIUM DENSE
D	DENSE
VD	VERY DENSE

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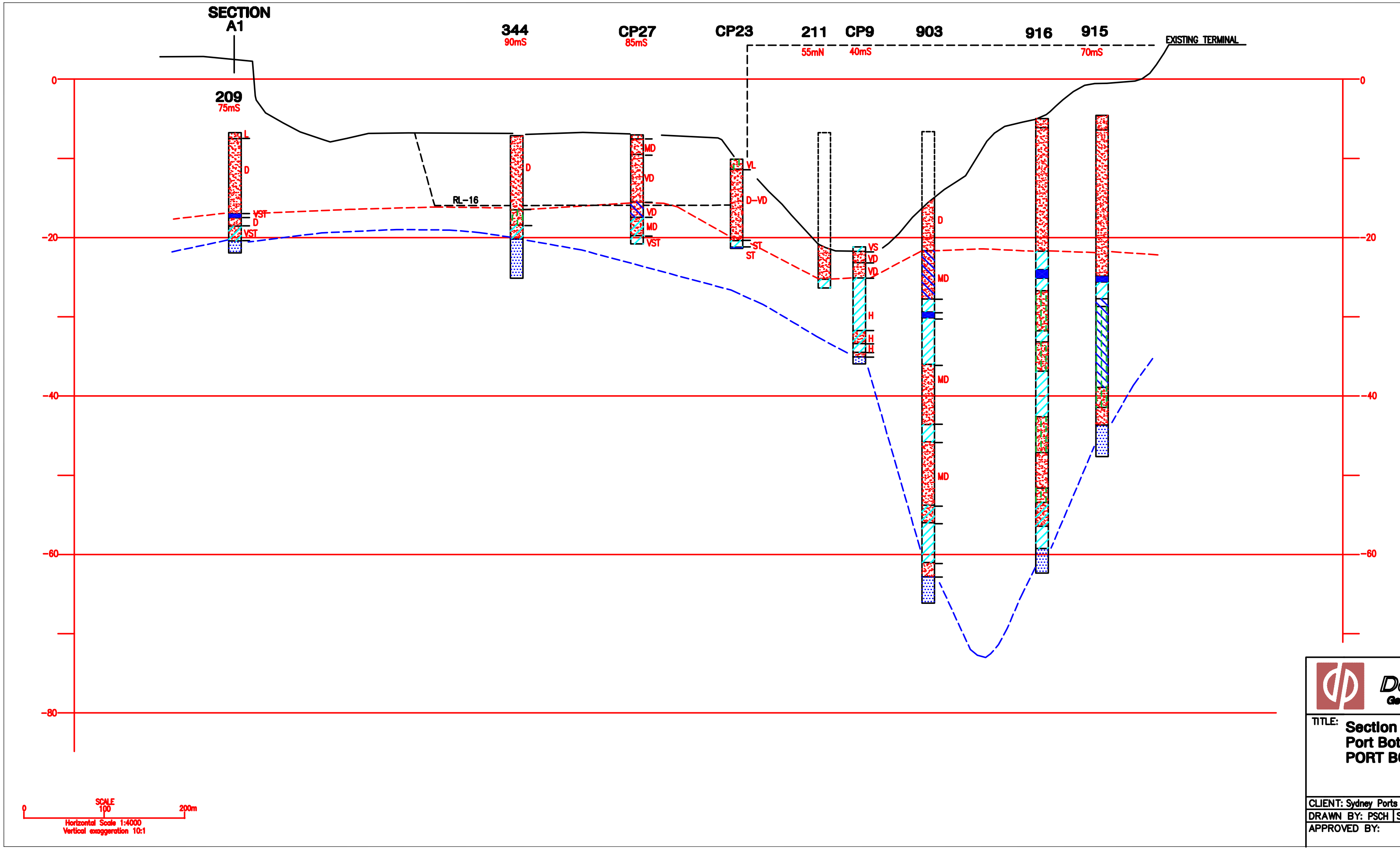
TITLE: **Section B2**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation
DRAWN BY: PSCH | SCALE: As shown | PROJECT No: 35224 | OFFICE: SYDNEY
APPROVED BY: | DATE: 26.6.2002 | DRAWING No: 7



LEGEND

- SAND
- CLAY/SILTY CLAY
- SILT
- PEAT
- SILTY SAND
- SANDY CLAY/CLAYEY SAND
- ORGANIC CLAY/PEATY CLAY
- PEATY SAND
- ORGANIC SILT/PEATY SILT
- SANDSTONE/SILTSTONE

SOIL CONSISTENCY

VS	VERY SOFT
S	SOFT
F	FIRM
ST	STIFF
VST	VERY STIFF
H	HARD
VL	VERY LOOSE
L	LOOSE
MD	MEDIUM DENSE
D	DENSE
VD	VERY DENSE

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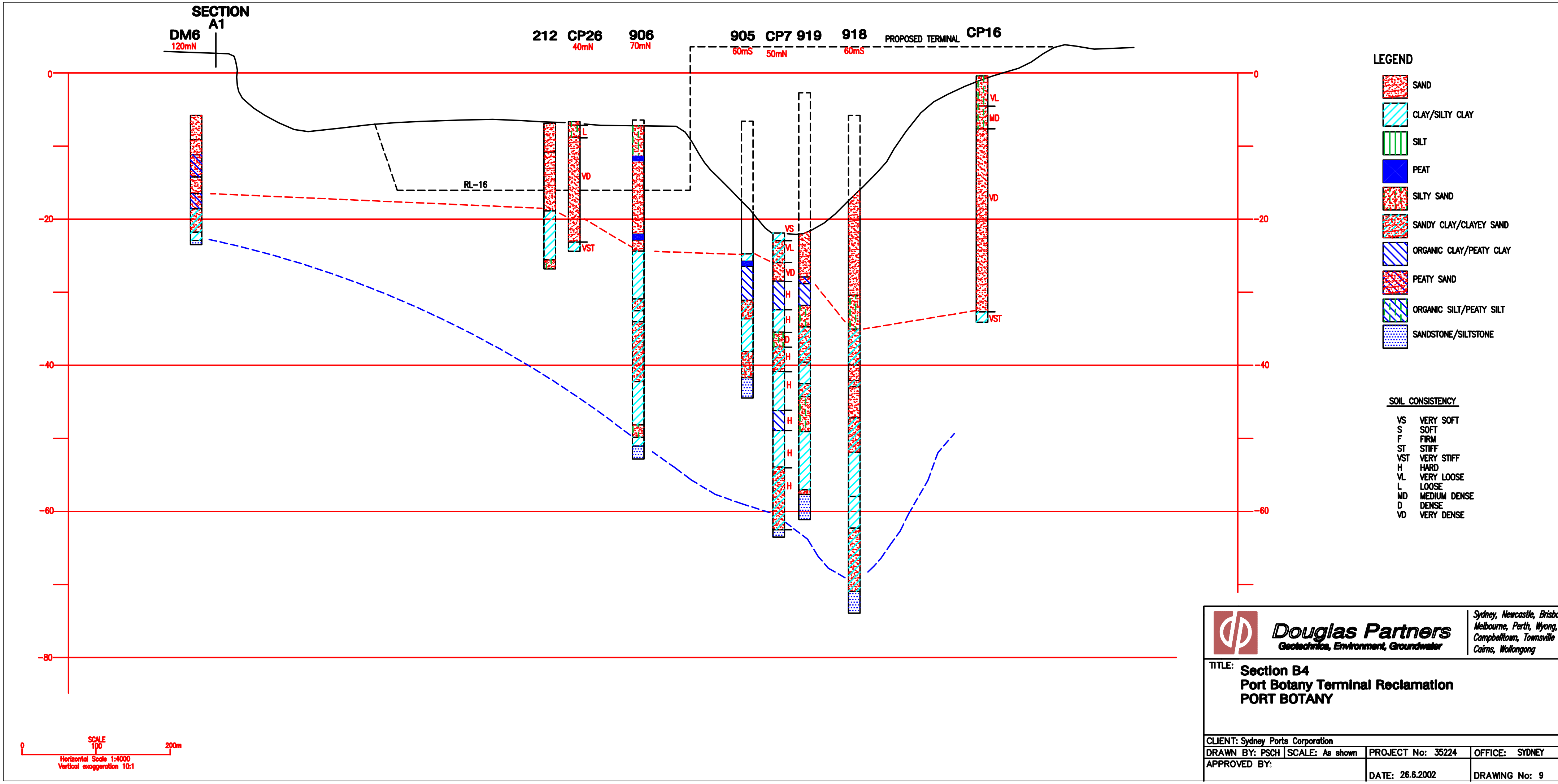
TITLE: **Section B3**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 26.6.2002	DRAWING No: 8



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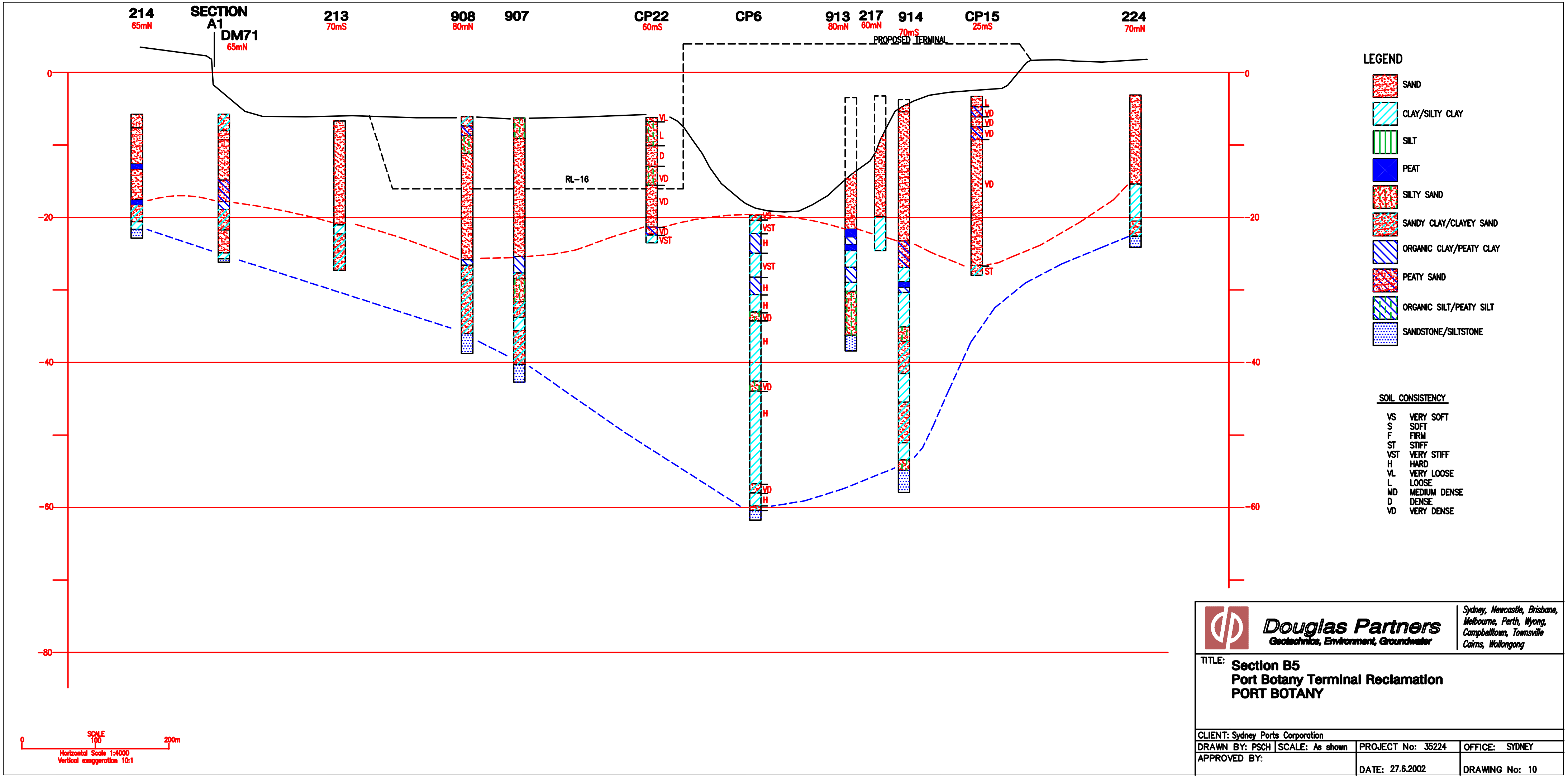
TITLE: **Section B4**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 26.6.2002	DRAWING No: 9



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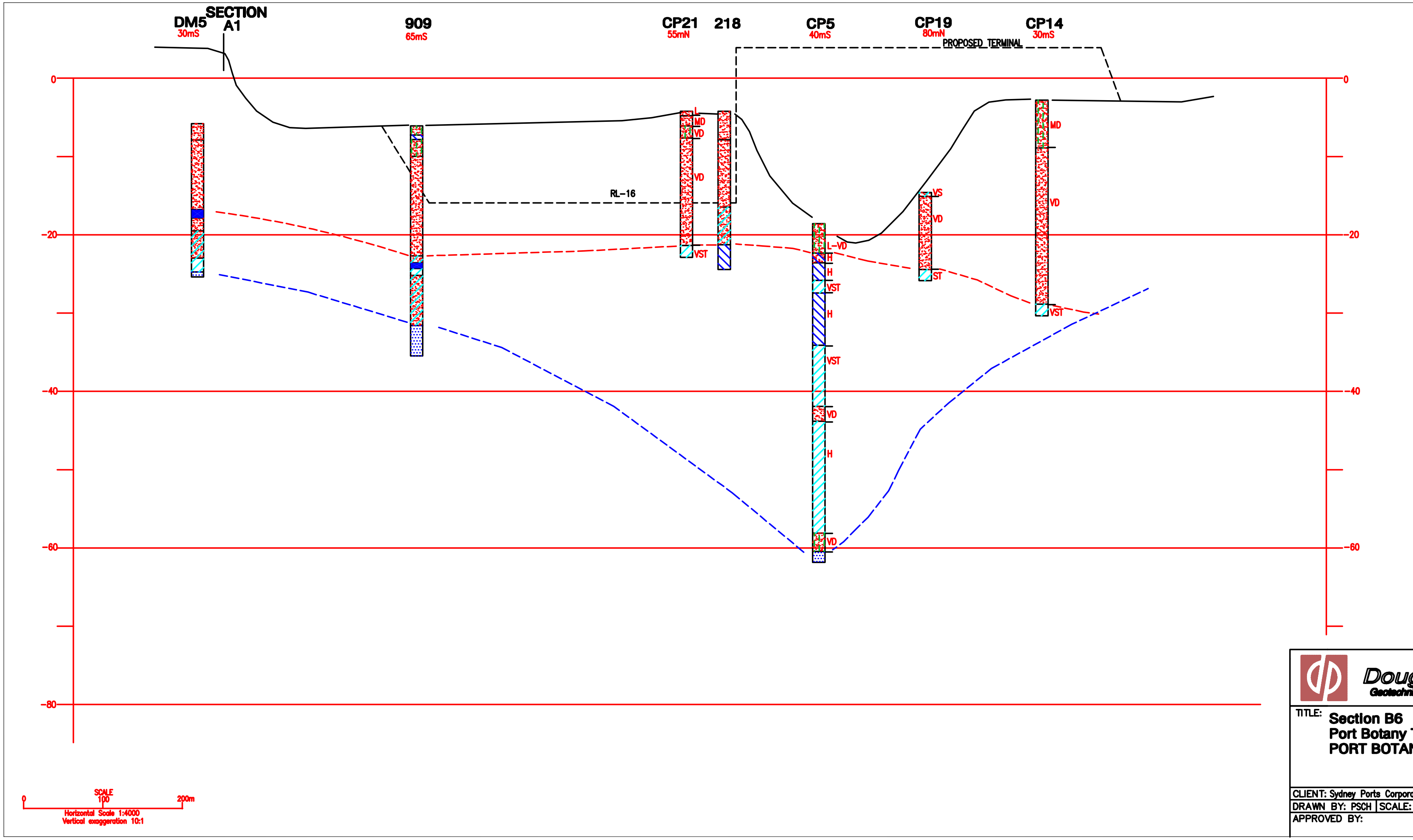
Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville, Cairns, Wollongong

TITLE: **Section B5**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 27.6.2002	DRAWING No: 10



LEGEND

- SAND
- CLAY/SILTY CLAY
- SILT
- PEAT
- SILTY SAND
- SANDY CLAY/CLAYEY SAND
- ORGANIC CLAY/PEATY CLAY
- PEATY SAND
- ORGANIC SILT/PEATY SILT
- SANDSTONE/SILTSTONE

SOIL CONSISTENCY

VS	VERY SOFT
S	SOFT
F	FIRM
ST	STIFF
VST	VERY STIFF
H	HARD
VL	VERY LOOSE
L	LOOSE
MD	MEDIUM DENSE
D	DENSE
VD	VERY DENSE

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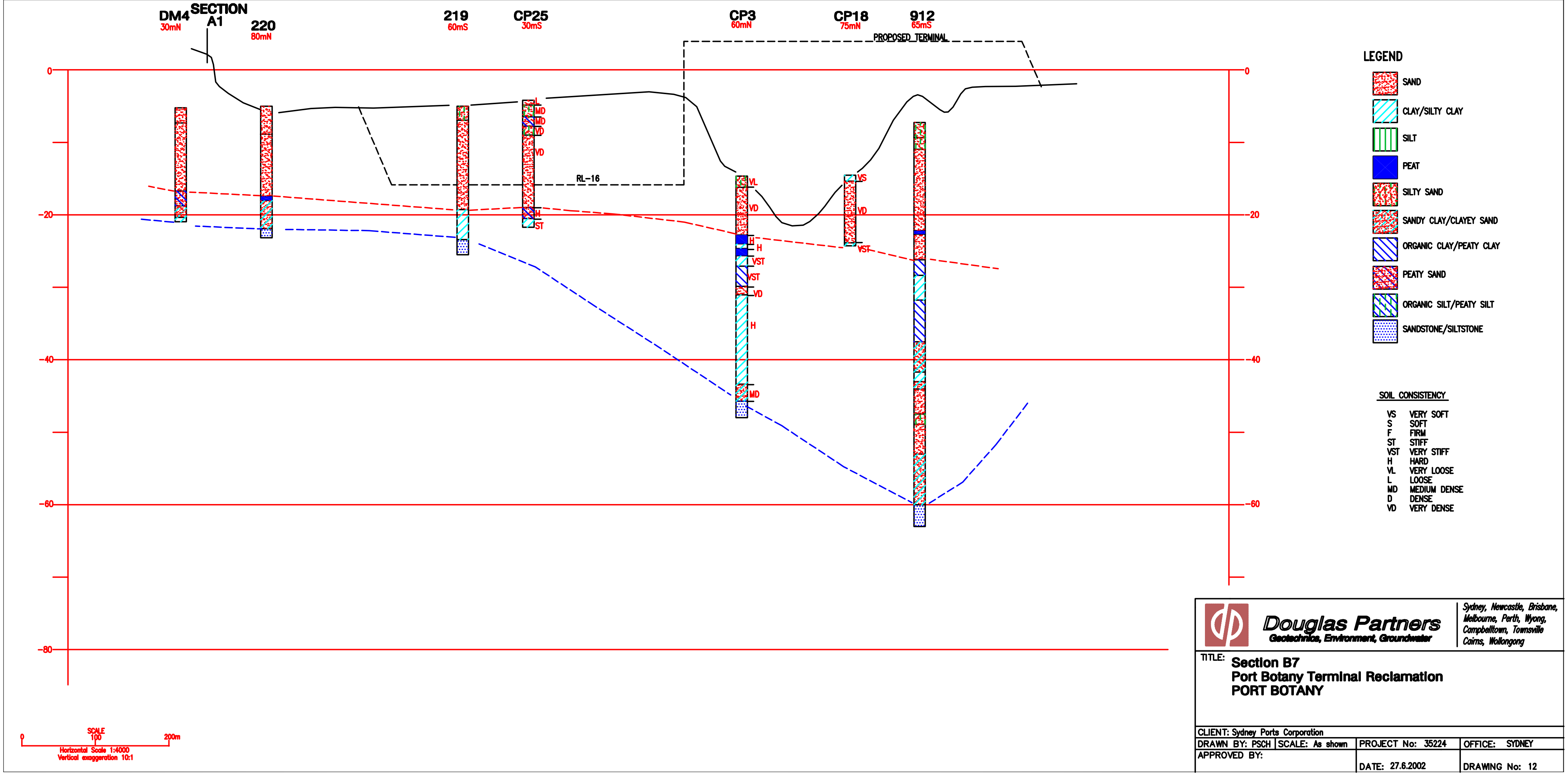
TITLE: **Section B6**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation

DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 27.6.2002	DRAWING No: 11



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TITLE: **Section B7**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
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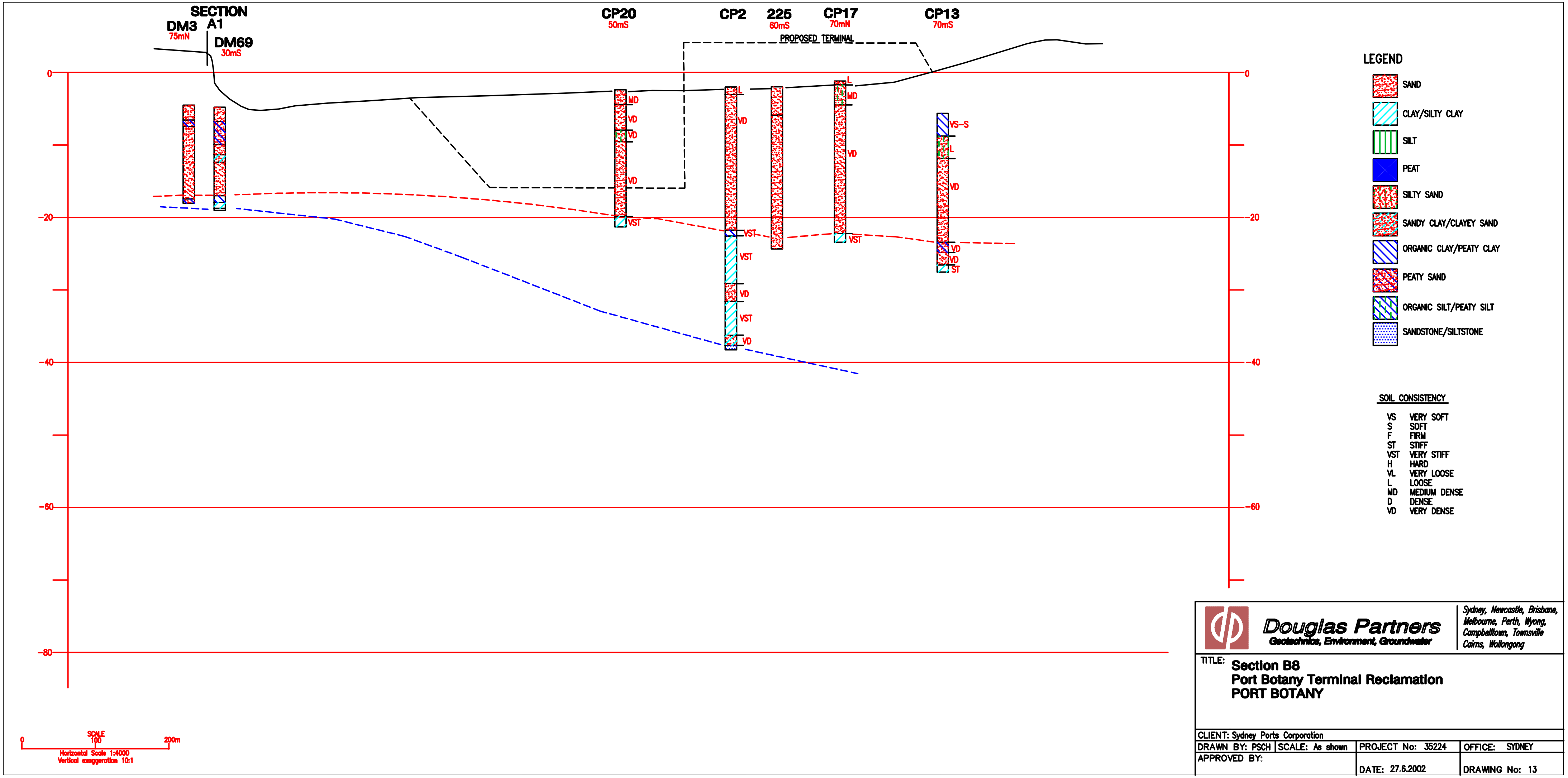
APPROVED BY:		DATE: 27.6.2002	DRAWING No: 12



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TITLE: **Section B8**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
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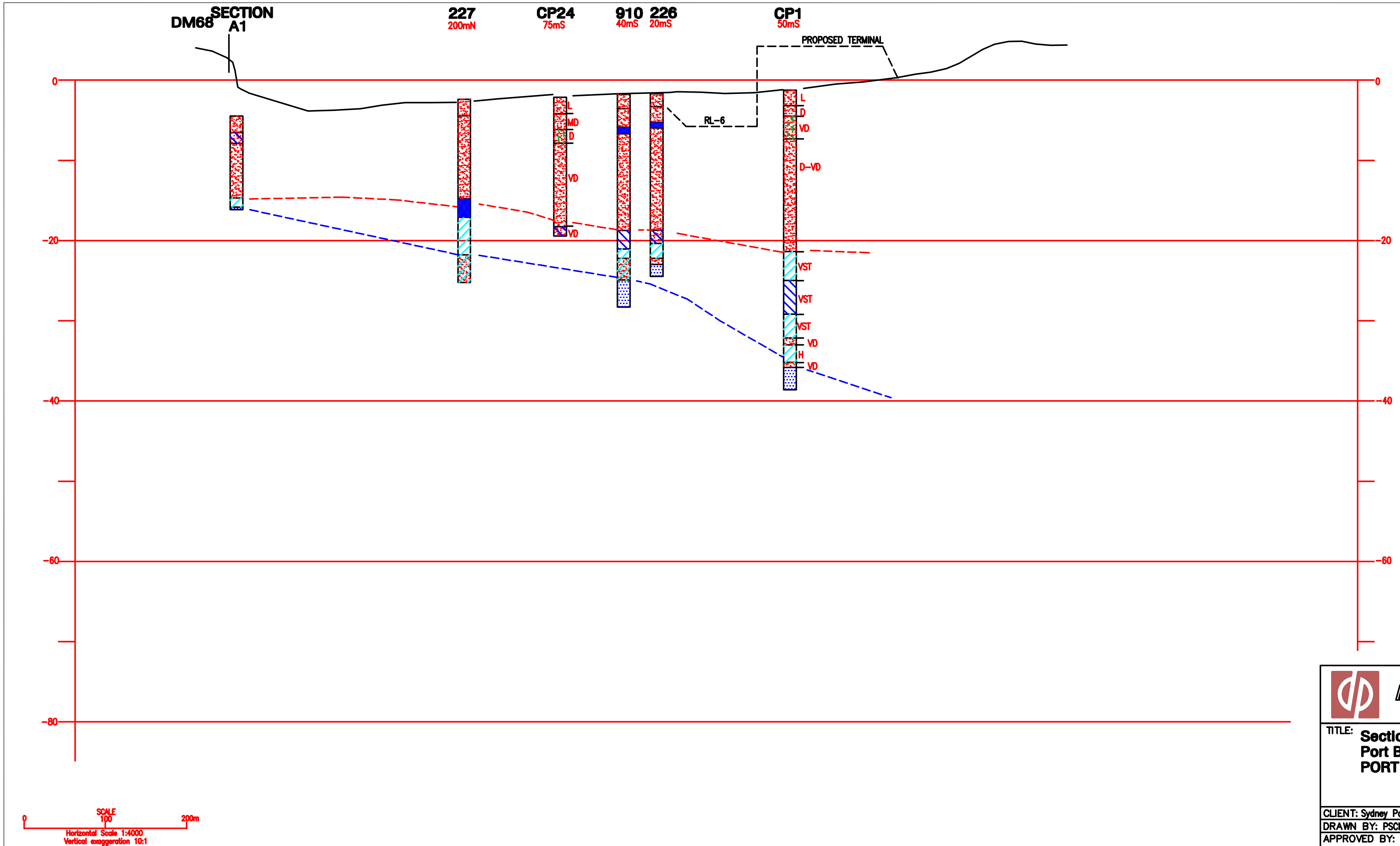
APPROVED BY:		DATE: 27.6.2002	DRAWING No: 13



LEGEND

- SAND
- CLAY/SILTY CLAY
- SILT
- PEAT
- SILTY SAND
- SANDY CLAY/CLAYEY SAND
- ORGANIC CLAY/PEATY CLAY
- PEATY SAND
- ORGANIC SILT/PEATY SILT
- SANDSTONE/SILTSTONE

SOIL CONSISTENCY

VS	VERY SOFT
S	SOFT
F	FIRM
ST	STIFF
VST	VERY STIFF
H	HARD
VL	VERY LOOSE
L	LOOSE
MD	MEDIUM DENSE
D	DENSE
VD	VERY DENSE

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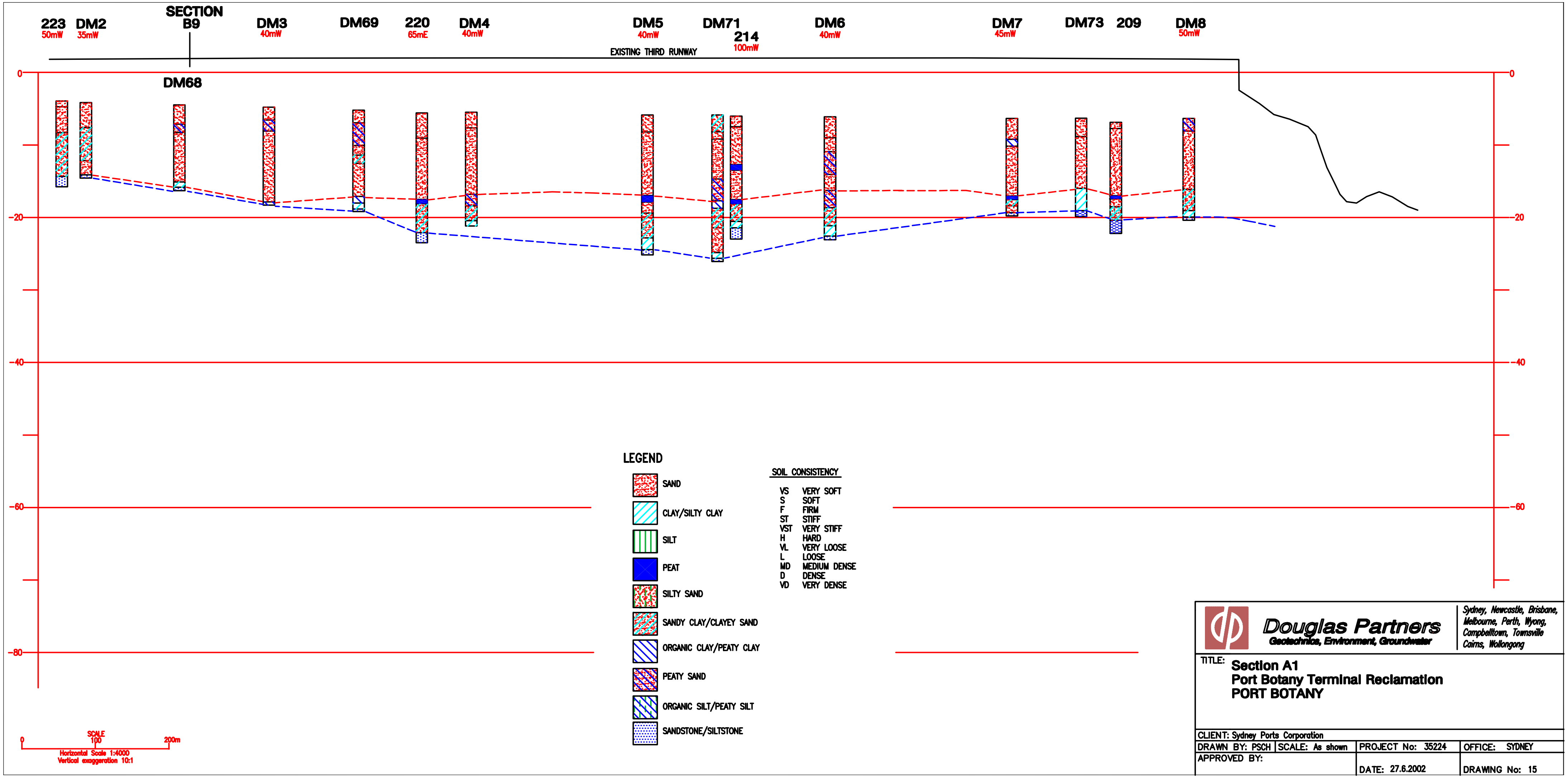
TITLE: **Section B9**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 27.6.2002	DRAWING No: 14



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TITLE: **Section A1**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
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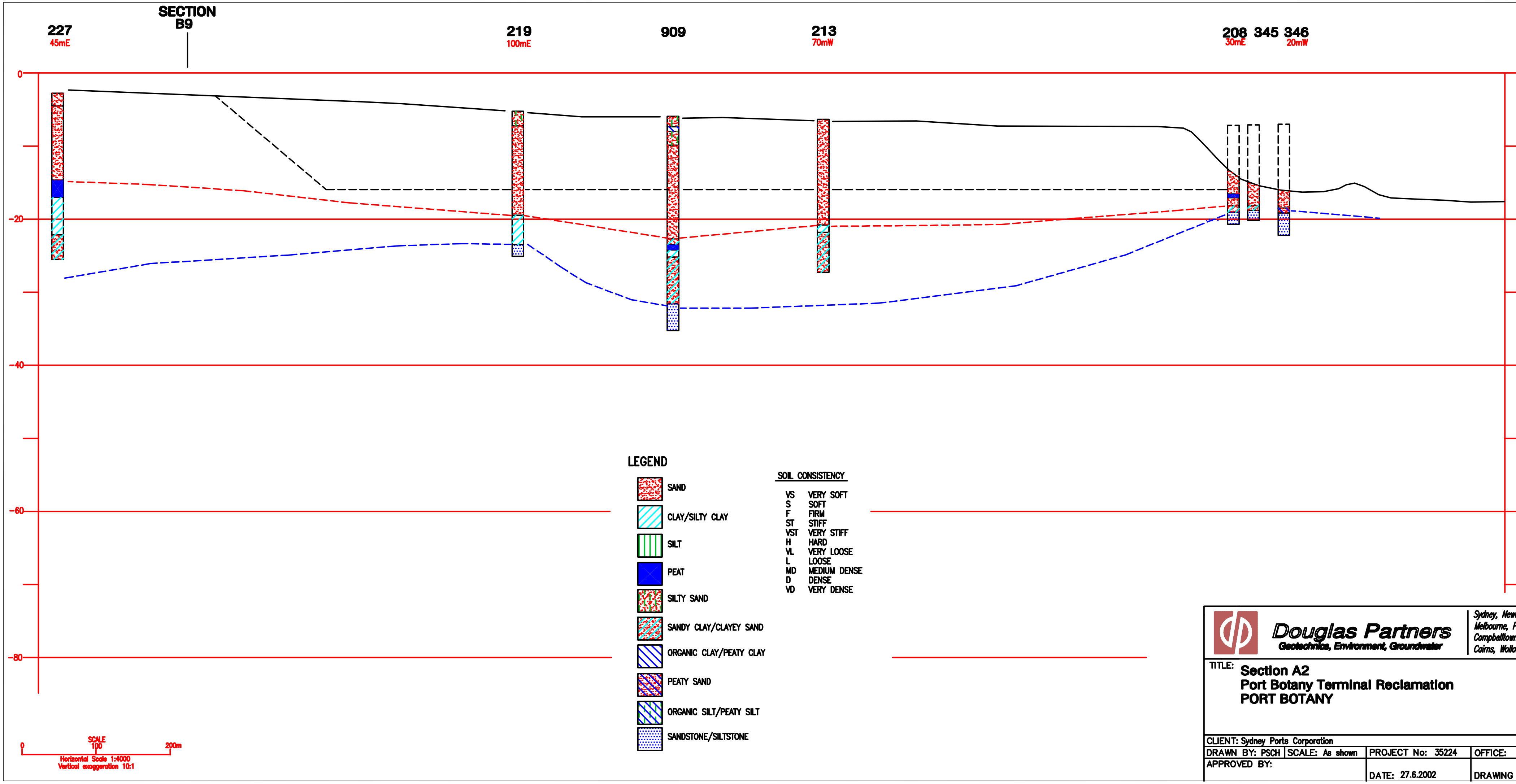
APPROVED BY:		DATE: 27.6.2002	DRAWING No: 15



Douglas Partners — Geotechnics, Environment, Groundwater			Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville Cairns, Wollongong
TITLE: Section A2 Port Botany Terminal Reclamation PORT BOTANY			
CLIENT: Sydney Ports Corporation			
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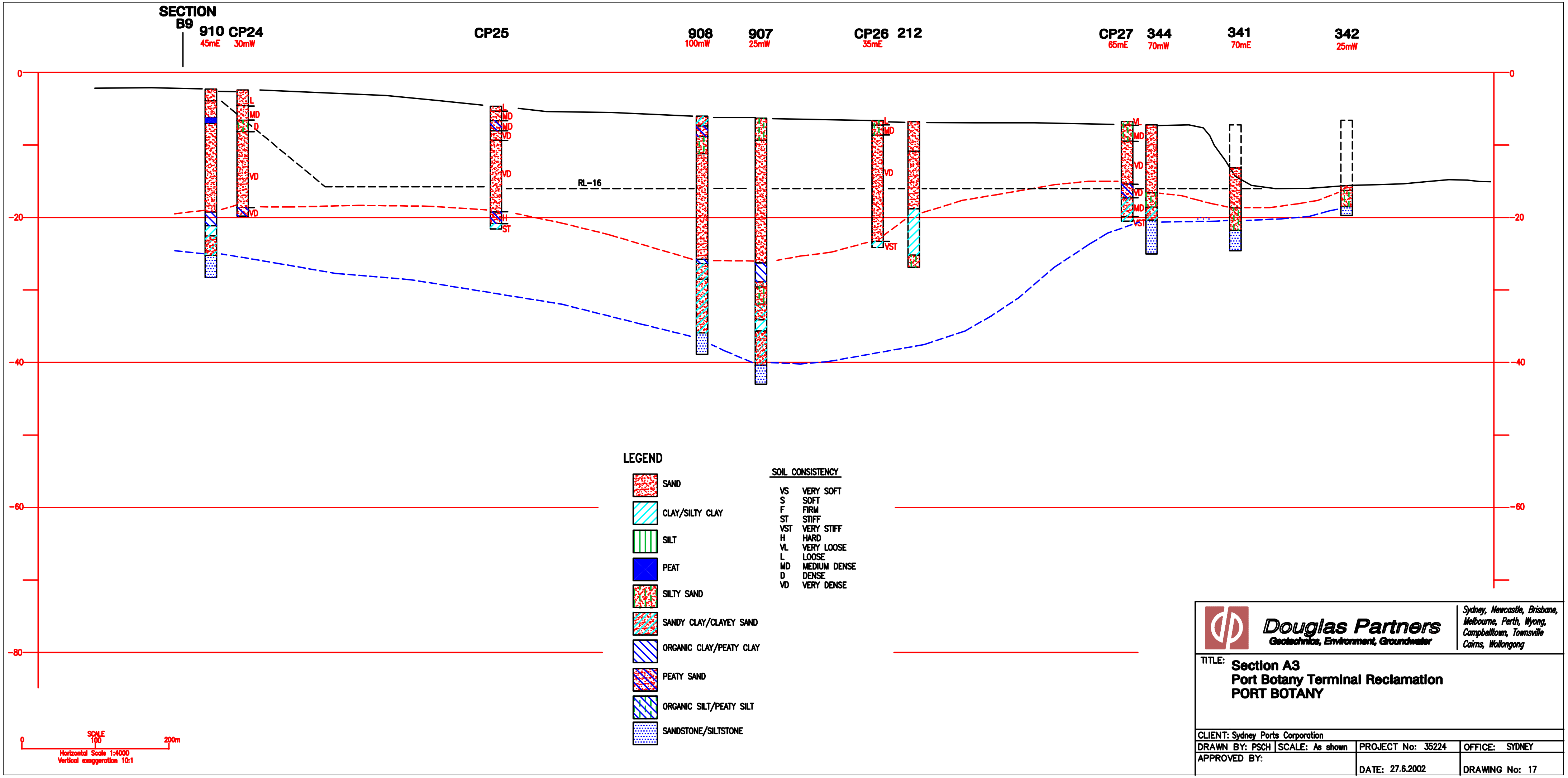

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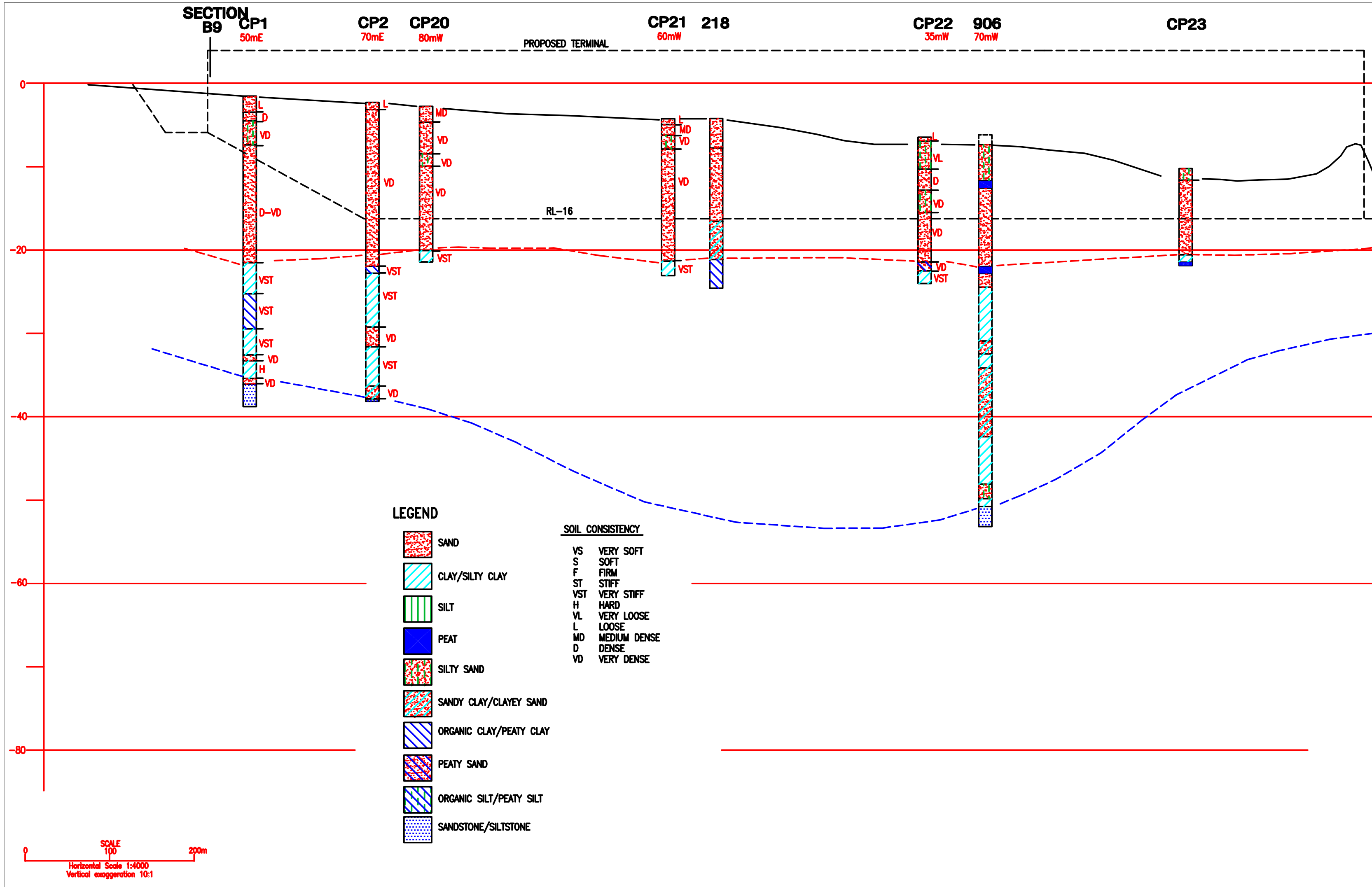
Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville, Cairns, Wollongong

TITLE: **Section A3**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 27.6.2002	DRAWING No: 17



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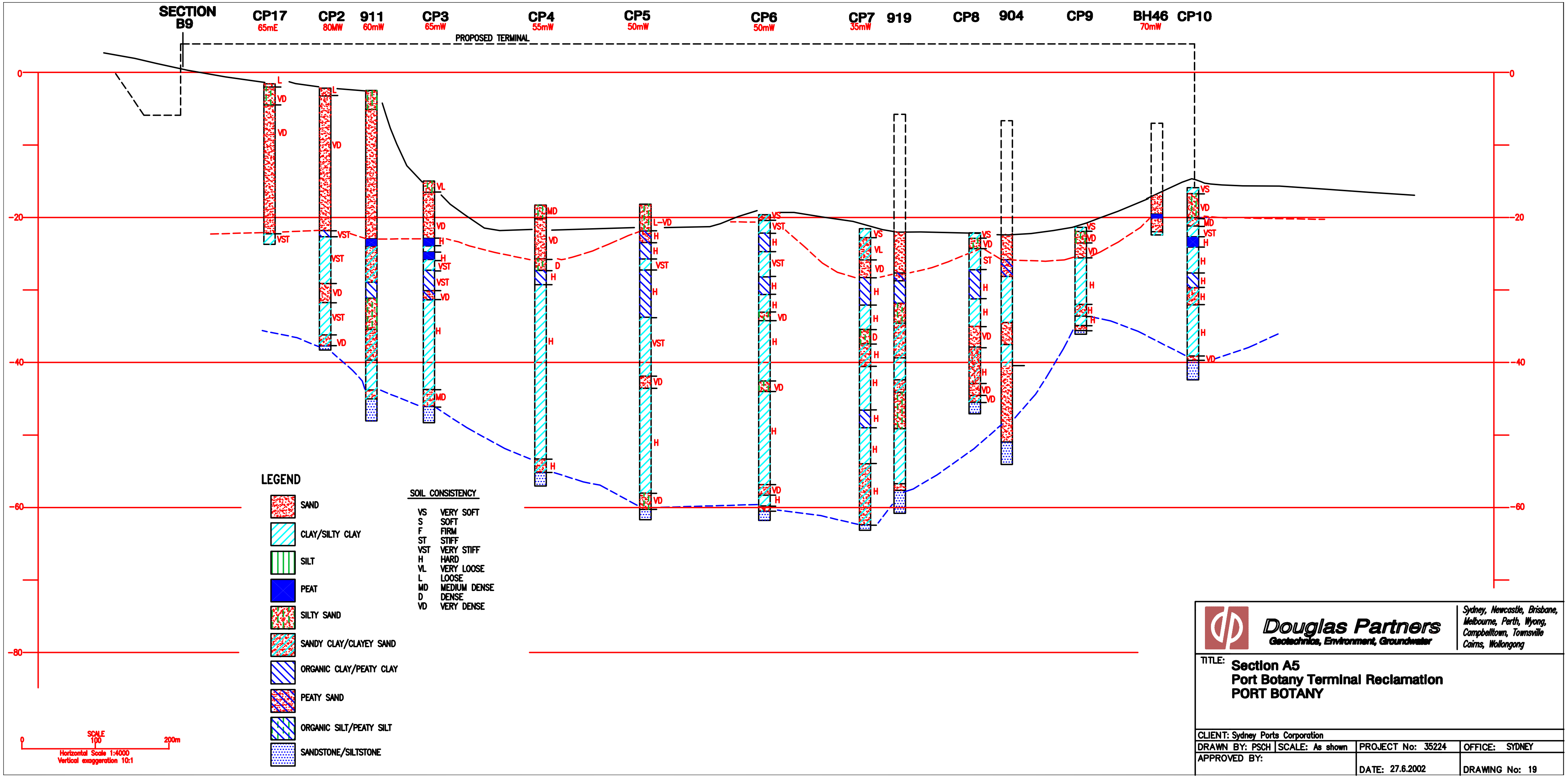
Geotechnics, Environment, Groundwater

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

TITLE: **Section A4**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 27.6.2002	DRAWING No: 18

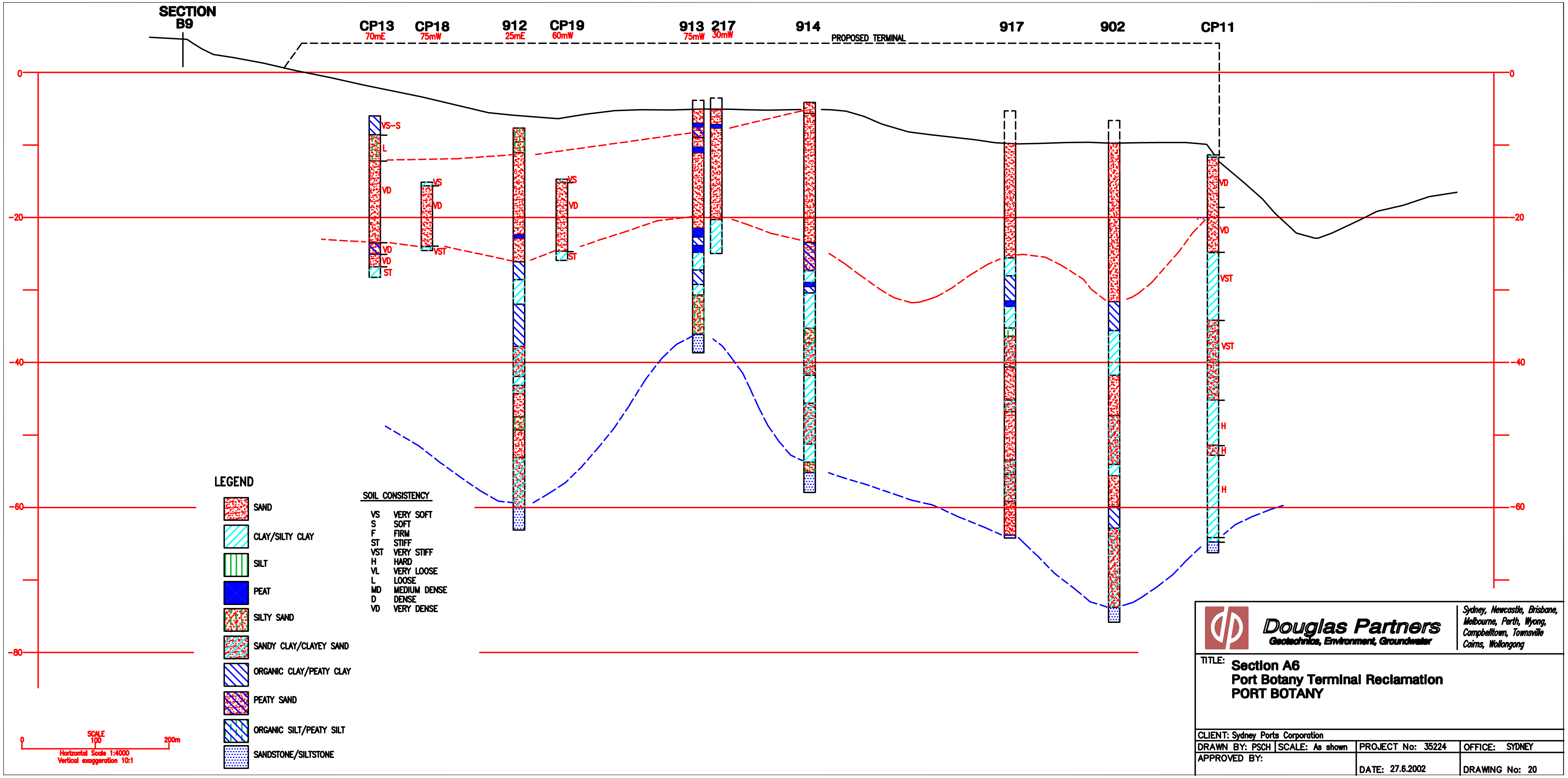


Douglas Partners
Geotechnics, Environment, Groundwater

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

TITLE: **Section A5**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 27.6.2002	DRAWING No: 19



Douglas Partners
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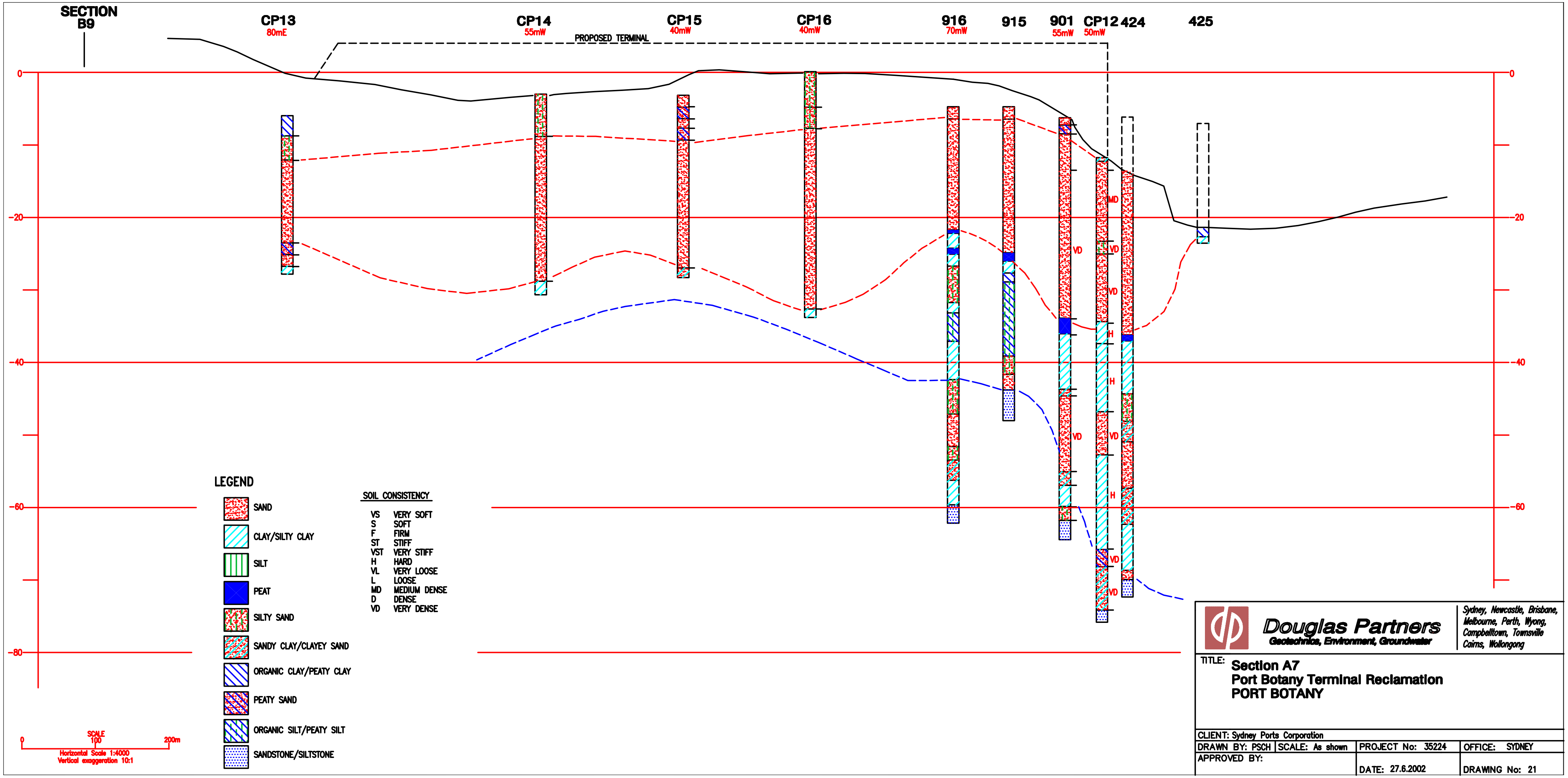
Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville, Cairns, Wollongong

TITLE: **Section A6**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 27.6.2002	DRAWING No: 20



Douglas Partners
Geotechnics, Environment, Groundwater

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

TITLE: **Section A7**
Port Botany Terminal Reclamation
PORT BOTANY

CLIENT: Sydney Ports Corporation			
DRAWN BY: PSCH	SCALE: As shown	PROJECT No: 35224	OFFICE: SYDNEY
APPROVED BY:		DATE: 27.6.2002	DRAWING No: 21

APPENDIX C
Summaries of Laboratory Data

Port Botany Third Terminal - Summary of Laboratory Data

Bore	Seabed RL (ISLW)	Depth From (m)	Depth To (m)	Top sample RL	Soil Unit	Moisture Content (%)	Wet Density (t/m3)	Dry Density (t/m3)	Atterberg Limits Wl (%)	Wp (%)	Ip (%)	LS (%)	Percent Passing 75um (%)	Percent Sand (%)	Percent Silt (%)	Percent Clay (%)	Unconfined Compressive Strength (kPa)	Confined Compressive Strength (kPa)	Confining Pressure (kPa)	Effective Strength Cohesion (kPa)	Effective Strength Friction Angle (deg)	Cv3 (m2/yr)	Direct Shear - Peak Cohesion (kPa)	Direct Shear - Peak Friction Angle (deg)	Direct Shear - Residual Cohesion (kPa)	Direct Shear - Residual Friction Angle (deg)	Consolidation In-situ Pressure (kPa)	Preconsolidation Pressure (kPa)	eo	Cc	Cr	Ca
CP01	-1.65	3.4	3.85	-5.05	2								9.9																			
CP01	-1.65	6.25	6.8	-7.9	2								3.9																			
CP01	-1.65	9.3	9.55	-10.95	2								5.5																			
CP01	-1.65	24.25	24.95	-25.9	3B	44.2	1.67	1.16															182	2	48	6.3						
CP01	-1.65	24.25	24.95	-25.9	3B	59.8	1.49	0.93																								
CP02	-2.46	2.45	2.9	-4.91	2								9.5																			
CP02	-2.46	6.15	6.6	-8.61	2								5.2																			
CP02	-2.46	7.85	8.05	-10.11	2								9.1																			
CP02	-2.46	10.65	10.85	-13.11	2								4.3																			
CP02	-2.46	21.6	21.9	-24.06	3B	47.7	1.76	1.19																			185	300	1.04	0.28	0.058	0.004
CP02	-2.46	25.1	25.6	-27.56	3B	82	1.47	0.81										188	630													
CP02	-2.46	25.1	25.6	-27.56	3B	53.8	1.57	1.02																			215	530	1.416	0.16	0.03	0.002
CP02	-2.46	30	30.5	-32.46	4A	28	1.97	1.54																								
CP03	-15.1	7.65	8.2	-22.95	3A	117.2	1.26	0.58												0	71.5											
CP03	-15.1	7.65	8.2	-22.95	3A	102.5	1.32	0.65																			68	290	1.946	0.086	0.017	0.005
CP03	-15.1	11	11.4	-26.1	3B	31.7	1.9	1.44															17.5	15.5	0	9.5						
CP03	-15.1	11	11.4	-26.1	3B	32.3	1.81	1.37																								
CP03	-15.1	11	11.4	-26.1	3B	32.3	1.81	1.37																								
CP03	-15.1	11	11.4	-26.1	3B	31.8	1.92	1.46																								
CP03	-15.1	20.8	21.2	-35.9	4A	15.2	2.15	1.87	32	13	19	9.5						1040	900													
CP04	-18.35	11.8	12.2	-30.15	3B	36.3	1.63	1.34										80	750													
CP04	-18.35	13.3	13.7	-31.65	3B	19.6	2.14	1.79												0	30											
CP04	-18.35	14.7	15.1	-33.05	3B				83	31	52	18				34																
CP04	-18.35	17.6	17.9	-35.95	3B	52.6	1.4	0.92															152	13	36	9.5						
CP04	-18.35	17.6	17.9	-35.95	3B	52.6	1.37	0.9																								
CP04	-18.35	26	26.4	-44.35	4A	28.2	1.96	1.53																			222	506	0.761	0.329	0.06	0.004
CP04	-18.35	29.85	30.25	-48.2	4A	33.2	1.88	1.41	63	27	36	13.5						460	800													
CP05	-18.4	6.2	6.5	-24.6	3B	125.7	1.02	0.45																			54	323	2.843	0.352		0.004
CP05	-18.4	7.8	8.2	-26.2	3B	31.6	2	1.52												5	23											
CP05	-18.4	12.9	13.3	-31.3	3A																											
CP05	-18.4	14.3	14.7	-32.7	3A	118.5	1.18	0.54															65	16.5	0	9.5						
CP05	-18.4	14.3	14.7	-32.7	3A	117.6	1.24	0.57																								
CP05	-18.4	16.55	16.95	-34.95	4A	30.4	1.92	1.47										355	450													
CP05	-18.4	20	20.4	-38.4	4A	22.3	2.02	1.65																			172	480	0.666	0.115	0.048	0.002
CP05	-18.4	20	20.4	-38.4	4A	23	2	1.63																								
CP05	-18.4	25.9	26.3	-44.3	4A	29.2	1.93	1.49								36							63	14	0	9.5						
CP05	-18.4	35.4	35.8	-53.8	4A	33.7	1.86	1.39															90	15.5	0	10						
CP05	-18.4	35.4	35.8	-53.8	4A	34.9	1.89	1.4																			303	400	0.88	0.395	0.108	0.01
CP06	-19.75	3.4	3.7	-23.15	3A	142.9	1.53	0.63												1.5	49.5											
CP06	-19.75	3.4	3.7	-23.15	3A	142.9	1.07	0.44																			29	230	3.102	0.127	0.018	0.013
CP06	-19.75	5	5.35	-24.75	3B	33.7	1.82	1.36															31	11.4	0	12.1						
CP06	-19.75	5	5.35	-24.75	3B	34.1	1.86	1.39																								
CP06	-19.75	6.8	7.1	-26.55	3B	28.3	2.01	1.57										105	300													
CP06	-19.75	10.1	10.45	-29.85	3A	79.8	1.35	0.75															26	19.5	0	7.5						
CP06	-19.75	10.1	10.45	-29.85	3A	42.9	1.74	1.22																								
CP06	-19.75	10.1	10.45	-29.85	3A	69.2	1.42	0.84																								
CP06	-19.75	17.4	17.7	-37.15	4A	29.3	1.98	1.53																			150	300	0.747	0.179	0.059	0.001
CP06	-19.75	26.5	26.8	-46.25	4A	27.6	1.99	1.56										120	550													
CP06	-19.75	29.65	30	-49.4	4A				60	25	35	16																				
CP07	-21.8	7.45	7.75	-29.25	3A		1.62																									
CP07	-21.8	9.65	9.9	-31.45	3A		1.5																									
CP07	-21.8	9.65	9.9	-31.45	3A		1.41																									
CP07	-21.8	11.2	11.55	-33	4A	21.1	2.06	1.7										86	750													
CP07	-21.8	11.2	11.55	-33	4A	21.1	2.05	1.69																			96	445	0.707	0.053	0.043	<0.001
CP07	-21.8	20	20.35	-41.8	4A	15.7	2.14	1.85	35	15	20	10.5						765	600													
CP07	-21.8	22.9	23.2	-44.7	4A	38	1.81	1.31												0	30											
CP07	-21.8	22.9	23.2	-44.7	4A	35.9	1.86	1.37																			196	730	1.03	0.118	0.063	<0.001
CP07	-21.8	28.25	28.6	-50.05	4A	26.7	1.94	1.53	60	21	39	16						220	750													
CP08	-21.95	4.1	4.45	-26.05	3B	48	1.64	1.24												5	28											
CP08	-21.95	4.1	4.45	-26.05	3B	71.3	1.35	0.79																			38	320	1.706	0.229	0.04	0.001
CP08	-21.95	10.3	10.6	-32.25	4A				45	17	28	12																				

Port Botany Third Terminal - Summary of Laboratory Data

Bore	Seabed RL (ISLW)	Depth From (m)	To (m)	Top sample RL	Soil Unit	Moisture Content (%)	Wet Density (t/m3)	Dry Density (t/m3)	Atterberg Limits Wl (%)	Wp (%)	Ip (%)	LS (%)	Percent Passing 75um (%)	Percent Sand (%)	Percent Silt (%)	Percent Clay (%)	Unconfined Compressive Strength (kPa)	Confined Compressive Strength (kPa)	Confining Pressure (kPa)	Effective Strength Cohesion (kPa)	Effective Strength Friction Angle (deg)	Cv3 (m2/yr)	Direct Shear - Peak Cohesion (kPa)	Direct Shear - Peak Friction Angle (deg)	Direct Shear - Residual Cohesion (kPa)	Direct Shear - Residual Friction Angle (deg)	Consolidation In-situ Pressure (kPa)	Preconsolidation Pressure (kPa)	eo	Cc	Cr	Ca
CP09	-21.25	6.3	6.6	-27.55	3B	39.3	1.85	1.33										145	325													
CP09	-21.25	12.6	12.9	-33.85	4A	18.4	2.11	1.78												0	39											
CP10	-15.95	8.55	8.9	-24.5	3B	18.5	2.1	1.77												2	25.5											
CP10	-15.95	8.55	8.9	-24.5	3B	18.5	2.12	1.79																			74	380	0.523	0.081	0.016	<0.001
CP10	-15.95	15.35	15.7	-31.3	4A	27.2	1.9	1.49	64	18	45	16				12							25	17.5	0	9						
CP10	-15.95	15.35	15.7	-31.3	4A	25.9	1.89	1.5																								
CP10	-15.95	15.35	15.7	-31.3	4A	26.6	1.92	1.62																								
CP11	-11.4	15.65	16	-27.05	3B	32.2	1.75	1.32										175	375													
CP11	-11.4	23.75	24.1	-35.15	4A	20.6	2.06	1.71																			203	320	0.613	0.112	0.029	0.002
CP11	-11.4	29.95	30.3	-41.35	4A	26.3	1.93	1.53										220	500													
CP11	-11.4	38.1	38.45	-47.5	4A				34	12	22	8				14																
CP11	-11.4	42.75	43.1	-54.15	4A	21.5	1.87	1.54										35	550													
CP11	-11.4	48.8	49.1	-60.2	4A	27.4	2	1.57																			416	1200	0.698	0.321	0.086	0.002
CP12	-11.7	24.05	24.3	-35.75	3B	24.6	1.92	1.54										225	530													
CP12	-11.7	26.95	27.3	-38.65	4A	22.5	2.01	1.84																								
CP12	-11.7	26.95	27.3	-38.65	4A	19.8	2.01	1.68																								
CP12	-11.7	29.3	29.65	-41	4A	32.3	1.85	1.4												0	27.5											
CP12	-11.7	29.3	29.65	-41	4A	31.7	1.94	1.47																			250	670	0.868	0.253	0.138	0.003
CP12	-11.7	49.6	49.9	-61.3	4A	18.1																										
CP20	-2.8	1	1.2	-3.8	1C								0.9																			
CP20	-2.8	2.75	3.2	-5.55	2								5.4																			
CP20	-2.8	4.35	4.8	-7.15	2								5																			
CP20	-2.8	5.9	6.35	-8.7	2								5.9																			
CP20	-2.8	7.5	7.7	-10.3	2								3.4																			
CP20	-2.8	9.05	9.2	-11.85	2								3.6																			
CP20	-2.8	10.6	10.75	-13.4	2								4.7																			
CP20	-2.8	12.15	12.6	-14.95	2	22.4							6.1																			
CP21	-4.35	1	1.2	-5.35	2								1.2																			
CP21	-4.35	2.9	3.35	-7.25	2								8.6																			
CP21	-4.35	3.95	4.4	-8.3	2	20.8							6.6																			
CP21	-4.35	5.5	5.8	-9.85	2								8.4																			
CP21	-4.35	7.15	7.3	-11.5	2								4.9																			
CP21	-4.35	8.7	9	-13.05	2								6.3																			
CP21	-4.35	10.25	10.4	-14.6	2								4.4																			
CP22	-6.45	1	1.2	-7.45	1A	20.7							7.2																			
CP22	-6.45	3.95	4.4	-10.4	2								6.3																			
CP22	-6.45	5.45	5.9	-11.9	2								7.2																			
CP22	-6.45	6.9	7.2	-13.35	2	22.4							14.4																			
CP22	-6.45	8.35	8.65	-14.8	2								9.1																			
CP24	-2.3	1	1.2	-3.3	1C								1.7																			
CP24	-2.3	2.75	3.2	-5.05	2								5.7																			
CP24	-2.3	4.3	4.75	-6.6	2								5																			
CP24	-2.3	5.8	6.25	-8.1	2	20.1							8			6.4																
CP24	-2.3	7.35	7.8	-9.85	2								5.7	24																		
CP24	-2.3	8.85	9.3	-11.15	2								5.3																			
CP24	-2.3	10.4	10.7	-12.7	2								1.7																			
CP24	-2.3	12	12.3	-14.3	2								5.6																			
CP25	-4.75	1	1.2	-5.75	2								4.2																			
CP25	-4.75	2.75	3.2	-7.5	2	52.1							8.4																			
CP25	-4.75	3.6	4.05	-8.35	2								6			6.7																
CP25	-4.75	5.05	5.5	-9.8	2								5	25																		
CP25	-4.75	6.4	6.85	-11.15	2	20.2							13.9																			
CP25	-4.75	7.85	8.3	-12.6	2								4.8																			
CP25	-4.75	9.25	9.7	-14	2								4.7																			
CP26	-6.7	1	1.2	-7.7	2								2.5																			
CP26	-6.7	3	3.45	-9.7	2								3.8	26		4.9																
CP26	-6.7	5.7	5.85	-12.4	2								7																			
CP26	-6.7	7.15	7.45	-13.85	2								6.1																			
CP27	-7.05	3	3.4	-10.05	2								6.8																			
CP27	-7.05	4.85	5.3	-11.9	2	22.2							5			5.7																
CP27	-7.05	6.3	6.8	-13.35	2								3.7																			
CP27	-7.05	7.75	7.95	-14.8	2								7.1																			

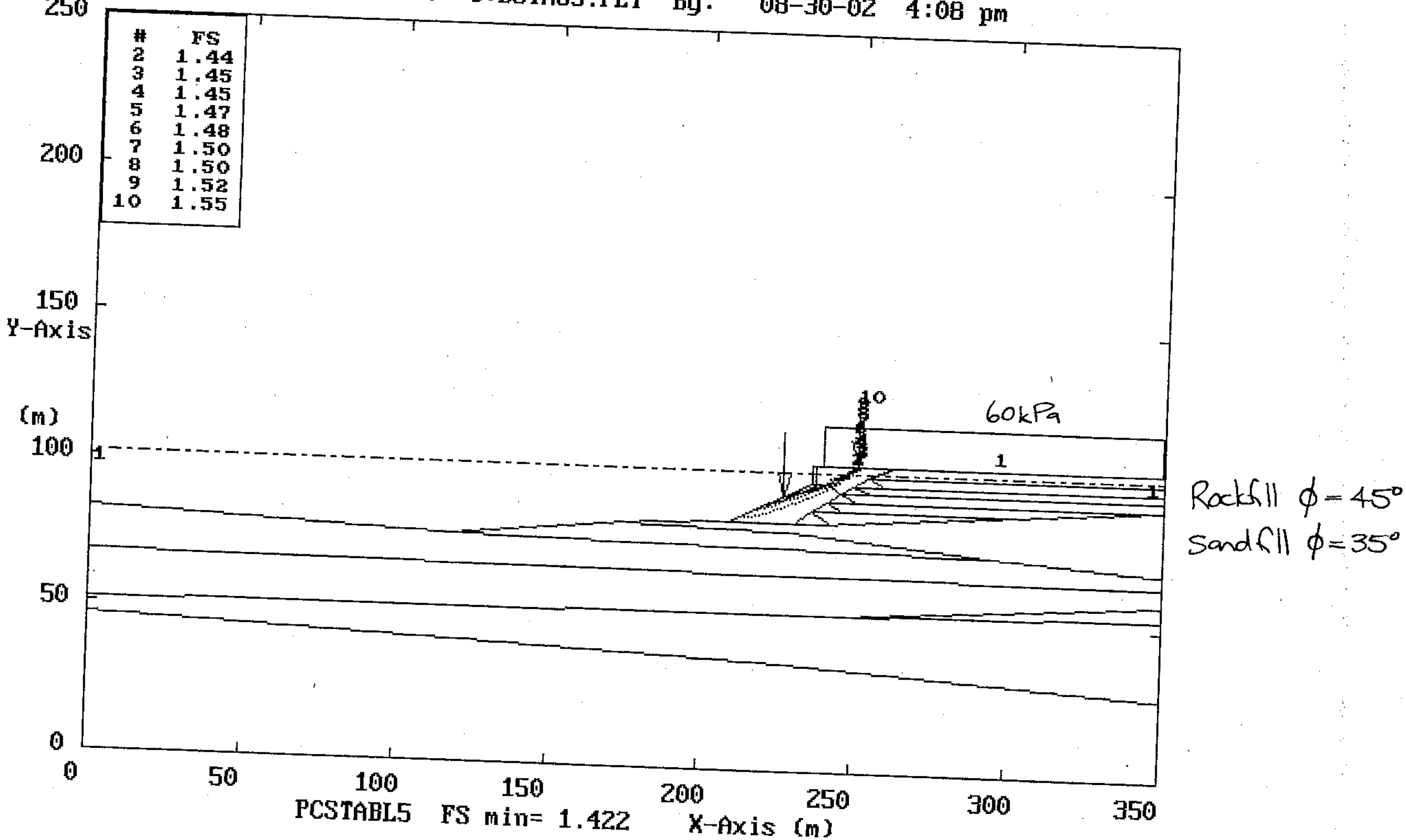
Port Botany Third Terminal - Summary of Laboratory Data

Bore	Seabed RL (ISLW)	Depth From (m)	To (m)	Top sample RL	Soil Unit	Moisture Content (%)	Wet Density (t/m3)	Dry Density (t/m3)	Atterberg Limits Wl (%)	Wp (%)	Ip (%)	LS (%)	Percent Passing 75um (%)	Percent Sand (%)	Percent Silt (%)	Percent Clay (%)	Unconfined Compressive Strength (kPa)	Confined Compressive Strength (kPa)	Confining Pressure (kPa)	Effective Strength Cohesion (kPa)	Effective Strength Friction Angle (deg)	Cv3 (m2/yr)	Direct Shear - Peak Cohesion (kPa)	Direct Shear - Peak Friction Angle (deg)	Direct Shear - Residual Cohesion (kPa)	Direct Shear - Residual Friction Angle (deg)	Consolidation In-situ Pressure (kPa)	Consolidation Preconsolidation Pressure (kPa)	eo	Cc	Cr	Ca
901	-6.4	33.6	33.9	-40	3B	28														70	24											
901	-6.4	35		-41.4	3B	34	1.91	1.47	60	22	38			2																		
902	-6.6	31		-37.6	3B	34			41	26	15			12						50	25											
902	-6.6	31		-37.6	3B	22			39	15	24			23						10	18.5											
902	-6.6	33		-39.6	3B	33			57	21	38			5						50	25											
903	-6.7	24.5		-31.2	3	65			np	np	np			2						25	20											
903	-6.7	50		-56.7	4A	40			60	25	35			5						18	21											
904	-6.7	27		-33.7	4B				np	np	np			79						0	37											
904	-6.7	31		-37.7	4A	24	2.05	1.67	43	16	27			17						49	25											
904	-6.7	33		-39.7	4A	23	2.04	1.66	41	18	23			29						48	29											
905	-6.6	23		-29.6	3B	69	1.56	0.97	np	np	np			3						240	24											
905	-6.6	33		-39.6	4B	29	1.87	1.59	36	17	19			37						36	34											
906	-6.45	18.4		-24.85	3B	38			40	27	13			6						20	29											
906	-6.45	18.45		-24.9	3B	29	1.96	1.49	40	19	21			9						2	33											
906	-6.45	26		-32.45	3B	32	1.98	1.59	56	24	32			10						15	27											
906	-6.45	36		-42.45	4A	26	2.02	1.65	72	22	50			8						32	18											
907	-6.25	20.25		-26.5	3B	90	1.38	0.73	np	np	np			6						37	34											
907	-6.25	32		-38.25	4A	31	1.87	1.51	47	20	27			13						59	36											
908	-6.2	20		-26.2	3B	45	1.71	1.15	46	32	14			5						12	17											
909	-6.1	21		-27.1	4B	16	2.15	1.88	30	13	17			63						20	31											
910	-2.1	18		-20.1	3A	50	1.75	1.17	42	21	21			9						32	26											
911	-2.55	24		-26.55	3B	33	1.99	1.55	59	19	40			11						20	28											
911	-2.55	28		-30.55	3B	68	1.52	0.94	49	39	10			4						119	26											
911	-2.55	36		-38.55	4A	40	1.86	1.43	56	26	30			16						36	24											
914	-3.9	28		-31.9	3B	28	1.95	1.59	48	18	30			7						35	20											
914	-3.9	30		-33.9	4B	62	1.52	0.94	np	np	np			2						18	13											
915	-4.7	30		-34.7	3B	53	1.68	1.11	48	33	15			1						0	24											
916	-5	23		-28	3B	86	1.44	0.84	np	np	np			15						45	48											
916	-5	25		-30	3B	45	1.53	1.01	60	41	19			3						0	16											
916	-5	25.2		-30.2	3B	42	1.66	1.13	52	27	25			3						17	41											
916	-5	27		-32	3B	106	1.37	0.7	np	np	np			0.2						90	21											
916	-5	31		-36	3B	59	1.63	1.04	52	37	15			0.2						14	26											
917	-5.4	26		-31.4	3B	80	1.39	0.73	np	np	np			3						172	25											
917	-5.4	32		-37.4	3B	26	2.01	1.64	48	18	30			14						37	27											
918	-5.7	29		-34.7	2	20	1.93	1.63	np	np	np			80						15	36											
918	-5.7	31		-36.7	3B	29	2	1.59	51	22	29			10						28	24											
918	-5.7	31.2		-36.9	3B	27	2.07	1.69	51	21	30			9						19	22											
919	-5.85	27		-32.85	4B		2.1	1.64	18	15	3			66						4	37											
919	-5.85	29		-34.85	4B	22	2.06	1.7	41	15	26			27						56	28											
919	-5.85	29.2		-35.05	4B	22	2.02	1.65	37	14	23			28						50	27											
919	-5.85	35		-40.85	4A	32	1.9	1.43	75	22	53			3						17	21											
208	-7.16	9.5		-16.66	3A		1.84		80	36	44			28	42	30	168															
208	-7.16	12.9		-20.06	6	12.5	2.22	1.98									330															
209	-6.86	10.4		-17.26	3A	21.3	2.01	1.66						5	50	45	19															
209	-6.86	10.4		-17.26	3A	51.2	1.6	1.06										155	103													
209	-6.86	12.1		-18.96	4B	12.3	2.24	1.99						53	25	22	275	251	138													
209	-6.86	12.9		-19.76	4B	12.7	2.26	2						63	22	15	250															
210	-6.40	12.3		-18.7	3B	14	2.21	1.94									354															
212	-6.86	17.2		-24.06	3B	8.2	2.14	1.98	76	25	51			14	51	35		180	172													
213	-6.55	6.95		-13.5	2	27.4	1.89	1.49									34															
214	-6.10	11.9		-18	3A	78	1.37	0.77										297	138													
214	-6.10	11.9		-18	3A	79	1.39	0.77						58	31	11	144															
214	-6.10	13.1		-19.2	4B	12.6	2.21	1.97						49	11	40	115															
214	-6.10	14.8		-20.9	4A	14.8	2.19	1.91									383															
215	-5.79	6.4		-12.19	2	45.1	1.78	1.23	64	19	45			37	32	31	55															
215	-5.79	7		-12.79	2	35.5	1.81	1.34						81	12	7	14															

Port Botany Third Terminal - Summary of Laboratory Data

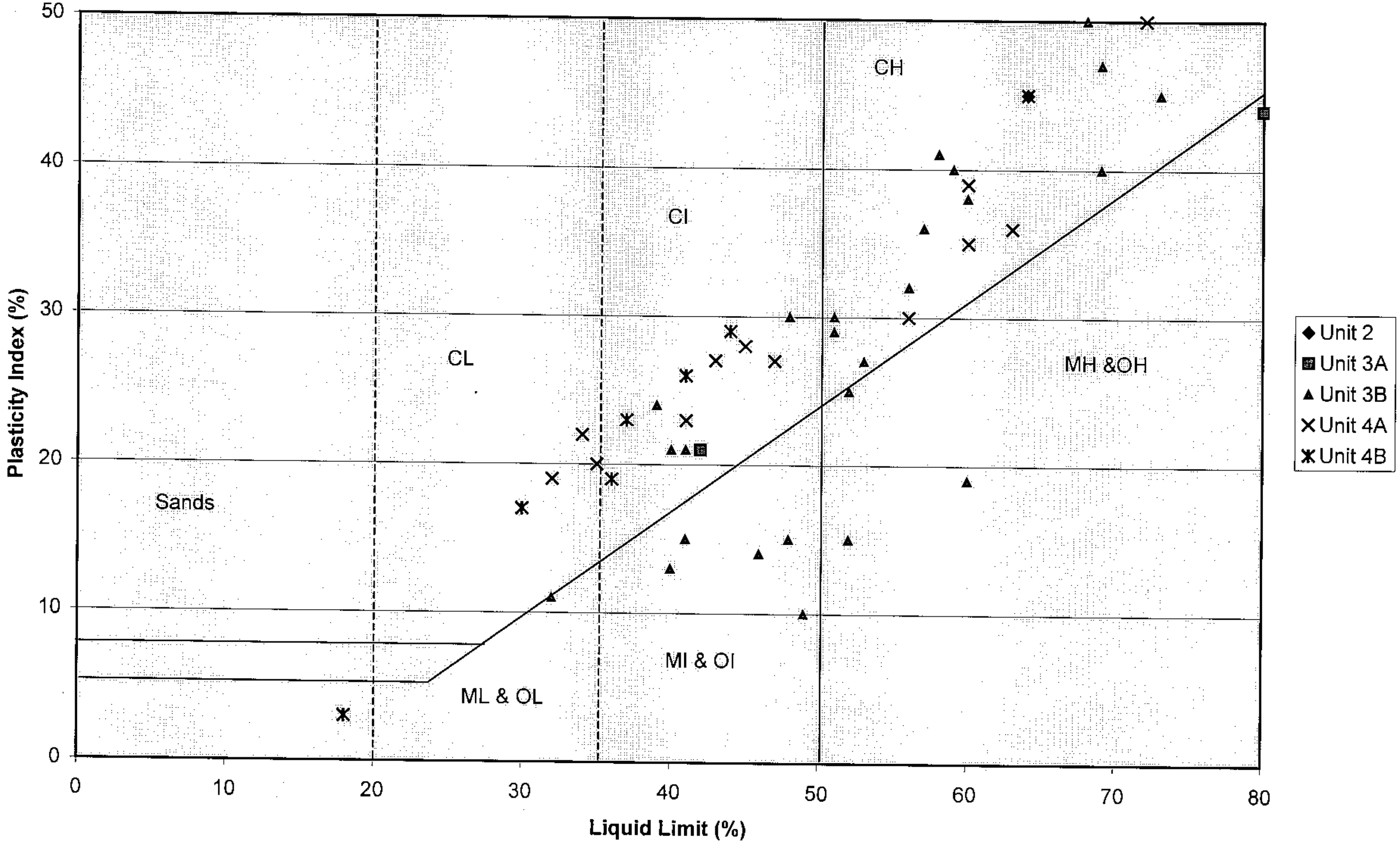
Bore	Seabed RL (ISLW)	Depth From (m)	To (m)	Top sample RL	Soil Unit	Moisture Content (%)	Wet Density (t/m3)	Dry Density (t/m3)	Atterberg Limits Wl (%)	Wp (%)	Ip (%)	LS (%)	Percent Passing 75um (%)	Percent Sand (%)	Percent Silt (%)	Percent Clay (%)	Unconfined Compressive Strength (kPa)	Confined Compressive Strength (kPa)	Confining Pressure (kPa)	Effective Strength Cohesion (kPa)	Friction Angle (deg)	Cv3 (m2/yr)	Direct Shear - Peak Cohesion (kPa)	Friction Angle (deg)	Direct Shear - Residual Cohesion (kPa)	Friction Angle (deg)	Consolidation In-situ Pressure (kPa)	Preconsolidation Pressure (kPa)	eo	Cc	Cr	Ca
217	-3.35	16.9		-20.25	3B		2.01	1.59	58	17	41							155	138													
217	-3.35	18.6		-21.95	3B	27.3	1.96	1.54	69	22	47							208	172													
217	-3.35	20.2		-23.55	3B	21.8	2.24	1.84	68	18	50			3	19	68		170	172													
217	-3.35	21.3		-24.65	3B	20.2	1.73	2.08										196	207													
220	-5.41	12.5		-17.91	3A	121.5	1.26	0.57	98	86	12			5	40	55		254	138													
220	-5.41	14.7		-20.11	4B	13.8	2.21	1.94						63	13	24		230	138													
220	-5.41	16.1		-21.51	4B	14	2.26	1.98						43	31	26	306															
221	-5.33	10.7		-16.03	4B	23.6	2	1.62						28	27	45	93	106	103													
223	-3.65	4.95		-8.6	4B	14.6	2.16	1.89						77	18	15	72	132	69													
223	-3.65	9.5		-13.15	4B	36.3	1.82	1.33						35	35	30	67															
223	-3.65	9.7		-13.35	4B	15.8	2.11	1.82						86	6	8	14															
226	-1.83	19		-20.83	3B		1.91		73	28	45			7	40	53		141	207													
227	-2.59	12.8		-15.39	3A	87	1.42	0.76	123	50	73			18	62	20	77	65	138													
227	-2.59	15.1		-17.69	4B	26.2	1.98	1.57	44	15	29			43	28	29	84	196	172													
227	-2.59	17.2		-19.79	4B	15.8	2.18	1.88						24	19	57	124	160	207													
227	-2.59	19.6		-22.19	4B	13.9	2.18	2.03						77	9	14	29															
DM4	-5.47	11.5	11.8	-16.97	3B	30	1.89		69	29	40									35	25	0.7										
DM5	-5.85	14.35	14.65	-20.2	3B	16.8	2.16						13							40	45	1										
DM5	-5.85	17.4	17.7	-23.25	3B	18.9	2.12										140															
DM8	-6.35	10.85	11.15	-17.2	3B	43.4	1.74		53	26	27		48					45	100								88	420	1.297	0.059	0.033	0.0012
DM20	-7.15	14.1	14.4	-21.25	3B	19.4	2.13										75															
DM67	-2.88	11.65	12.15	-14.73	3B	15.9			32	21	11									5	34.5											
DM71	-5.99	12	12.3	-17.99	3B	19	2.09										102															
DM73	-6.45	11	11.3	-17.45	3B	17.6	2.03		41	20	21									0	33											
CW8	3.65	38.65		-35	3B	59.9		1.27	112	48	64									25	29	19					345	150	1.585	0.392	0.056	0.001

Port Botany - Section A6 Permanent Rock Berm 2:1 batter
Ten Most Critical. C:BOTA65.PLT By: 08-30-02 4:08 pm

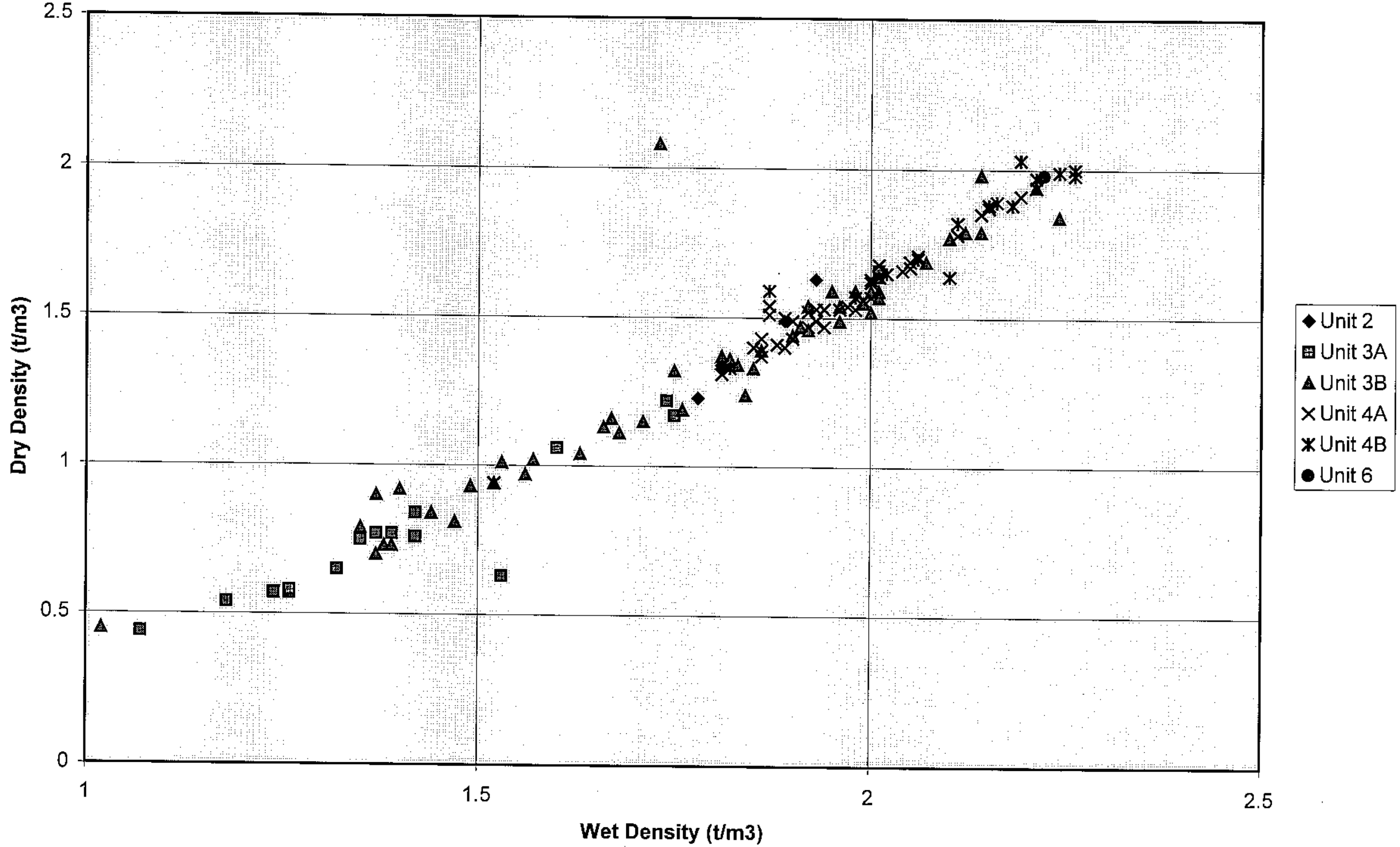


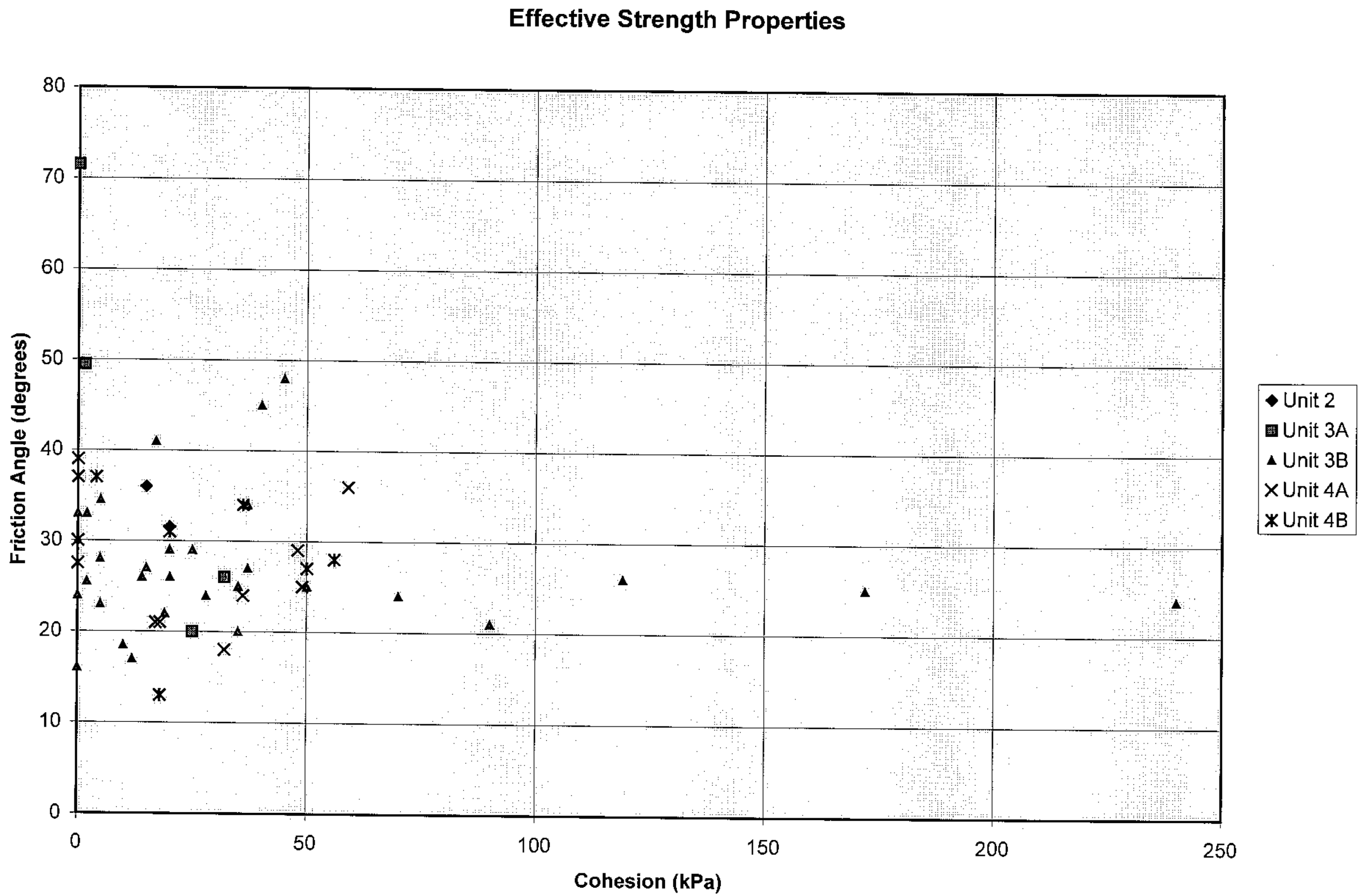
Permanent batter 2:1

Port Botany - Atterberg Limits



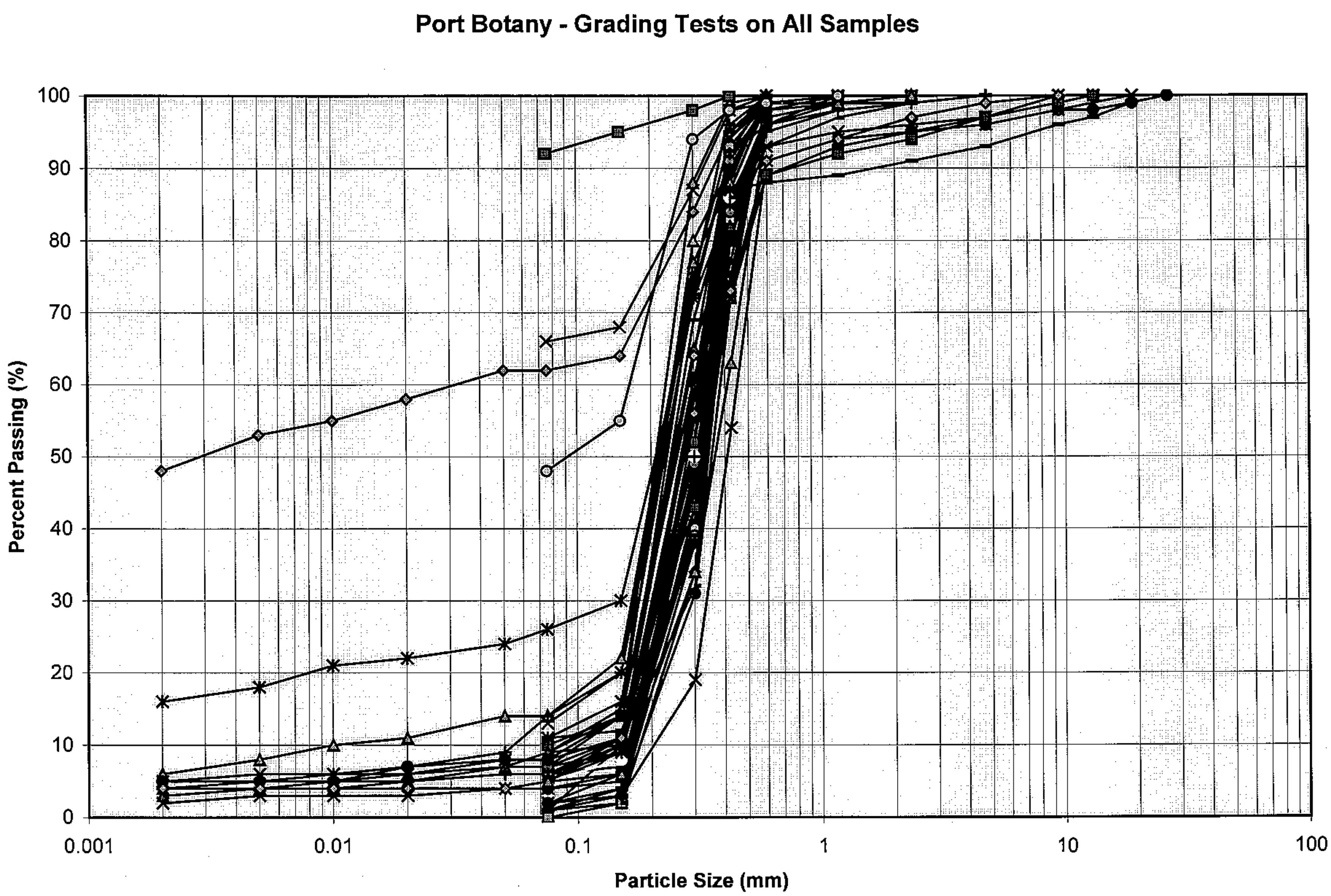
Port Botany - Densities



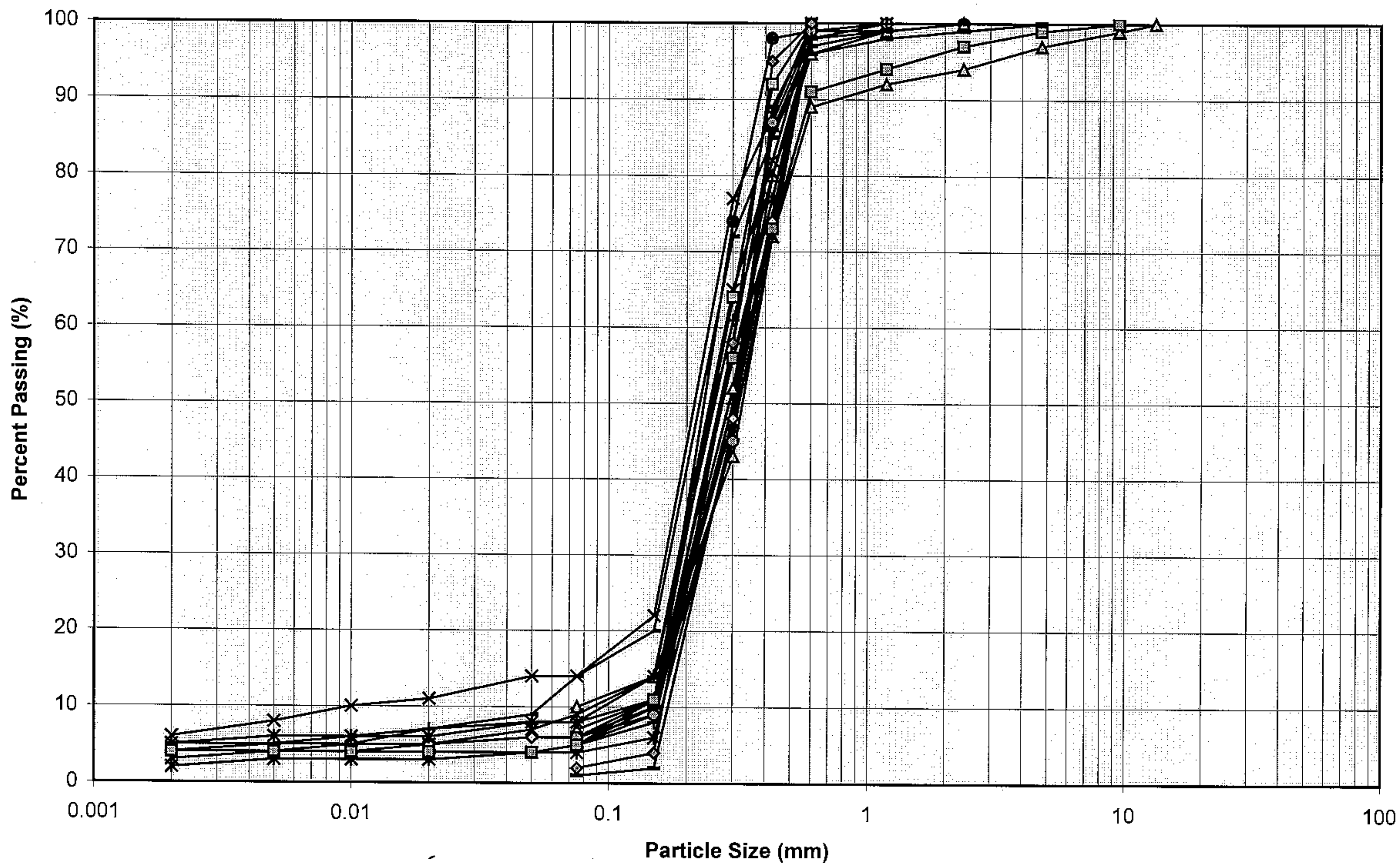

Effective Strength Properties
Friction Angle (degrees)
80
70
60
50
40
30
20
10
0
0
50
100
150
200
250
Cohesion (kPa)
Unit 2
Unit 3A
Unit 3B
Unit 4A
Unit 4B

Port Botany Third Terminal - Summary of Grading results

Bore	Depth from	Depth to	Unit	Percent Passing Particle Size (mm) 26.5	19	13.2	9.5	4.75	2.36	1.18	0.6	0.43	0.3	0.15	0.075	0.05	0.02	0.01	0.005	0.002
CP02	2.5	2.9	2			100	99	97	94	92	89	72	43	14	10					
CP02	7.65	8.05	2								100	82	65	14	9	7	5	4	4	3
CP01	6.25	6.8	2							100	99	72	46	6	4	4	3	3	3	2
CP20	2.75	3.2	2						100	99	99	98	74	9	5					
CP20	4.35	4.8	2							100	98	86	50	8	5					
CP20	12.15	12.6	2						100	99	98	75	50	10	6	6	5	4	4	4
CP21	1	1.2	2							100	99	89	47	2	1					
CP21	3.95	4.4	2							100	99	74	48	10	6	6	5	5	4	4
CP21	8.7	9	2								100	92	64	11	6					
CP22	5.45	5.9	2						100	99	96	80	52	11	7					
CP22	6.9	7.2	2								100	88	77	22	14	14	11	10	8	6
CP24	5.8	6.25	2							100	99	77	56	11	8	8	6	6	6	5
CP24	7.35	7.8	2							100	99	87	45	9	6					
CP25	2.75	3.2	2							100	98	80	61	14	8	8	7	5	5	5
CP25	6.4	6.85	2						100	99	97	85	72	20	14	9	7	6	5	4
CP25	9.25	9.7	2					100	99	98	96	86	58	10	5					
CP26	1	1.2	2								100	95	58	4	2					
CP27	4.85	5.3	2				100	99	97	94	91	73	56	11	5	4	4	4	4	4
DM7	12.55	12.85	3B									100	98	95	92					
DM5	12	12.25	3C						100	99	96	63	34	6	5					
DM5	14.35	14.65	3C							100	99	94	75	20	13					
DM5	15.85	16.15	3C							100	96	92	73	16	11					
DM8	10.85	11.15	3C							100	99	98	94	55	48					
CP22	1	1.2	1A						100	99	99	90	80	14	7	7	6	5	4	4
CP24	1	1.2	1C		100	99	98	96	95	94	91	54	19	3	2					
RC1	10.0	11.0	2						100	99	98	92	72	30	26	24	22	21	18	16
RC2	0.0	1.0	2	100	99	98	98	96	95	93	89	72	31	3	2					
RC2	4.0	5.0	2							100	99	84	46	3	2					
RC2	11.0	12.0	2							100	98	80	32	10	9					
RC4	1.0	2.0	2					100	99	99	97	82	41	5	4					
RC4	5.0	6.0	2							100	98	84	34	2	1					
RC4	8.0	9.0	2							100	98	83	40	9	7					
RC4	14.0	15.0	2								100	94	52	9	6					
RC5	1.0	2.0	2				100	97	96	95	93	85	41	7	6					
RC5	6.0	7.0	2							100	98	85	43	3	2					
RC6	7.0	8.0	2							100	99	91	49	2	0					
RC7	0.0	1.0	2					100	99	98	95	80	47	4	1					
RC7	6.0	7.0	2								100	96	55	3	1					
RC7	14.0	15.0	2	100	99	97	96	93	91	89	88	87	69	10	1					
RC8	13.0	14.0	2							100	99	95	84	64	62	62	58	55	53	48
RC9	3.0	4.0	2							100	99	93	51	2	0					
RC9	8.0	9.0	2								100	99	88	15	9					
RC9	14.0	15.0	2								100	98	87	68	66					
RC10	1.0	2.0	2			100	99	97	95	93	89	74	39	6	2					
RC10	7.0	8.0	2							100	99	84	39	6	4					
RC10	15.0	16.0	2								100	96	47	4	2					
RC11	0.0	1.0	2					100	99	97	93	81	52	3	2					
RC11	5.0	6.0	2							100	98	81	38	3	2					
RC11	10.0	11.0	2							100	99	86	39	12	11					
RC11	15.0	16.0	2							100	99	90	60	6	4					


Port Botany - Grading Tests on All Samples
Percent Passing (%)
100
90
80
70
60
50
40
30
20
10
0
0.001
0.01
0.1
1
10
100
Particle Size (mm)

Port Botany - Unit 2 Sands Only

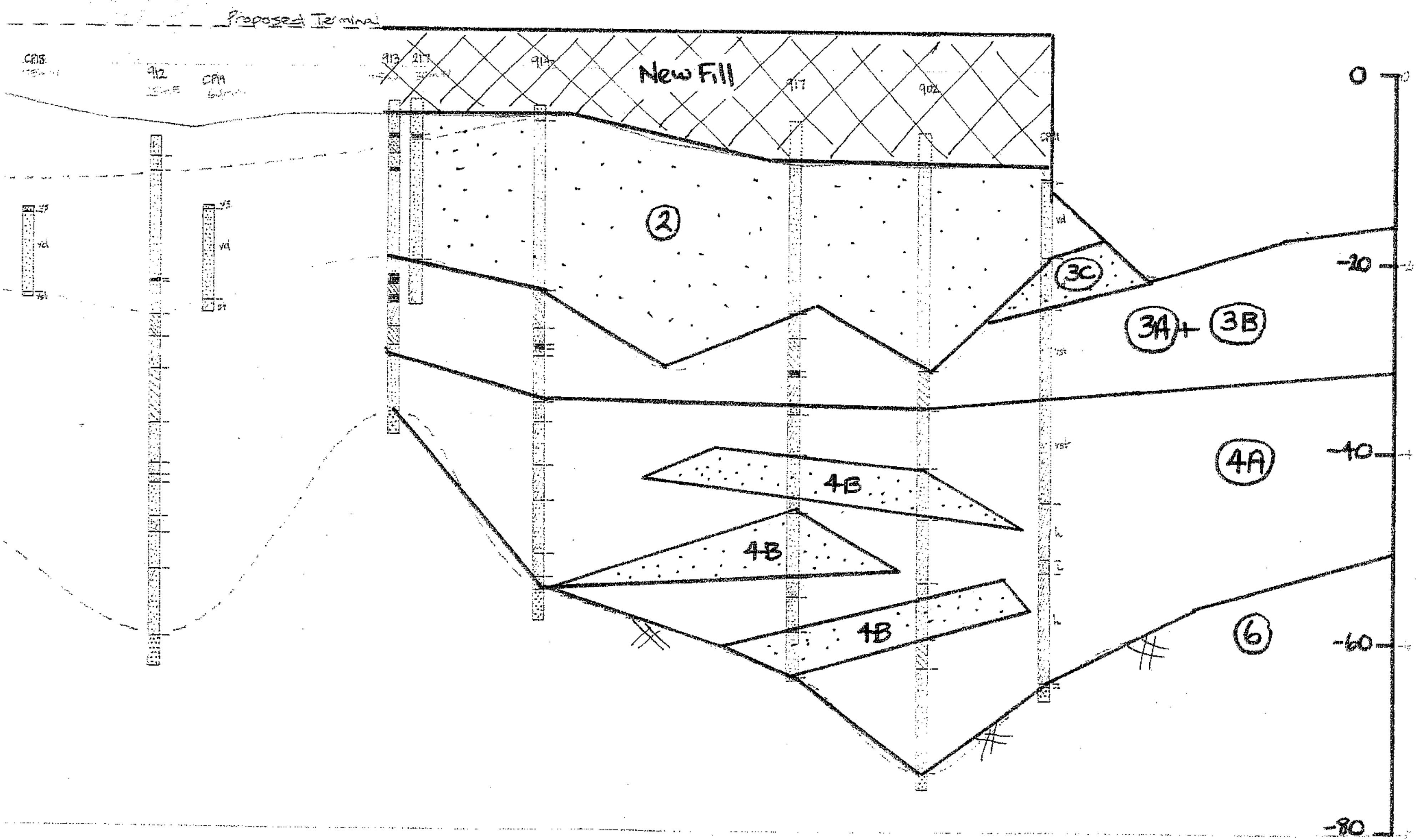


Port Botany Third Terminal - Summary of Consolidation Data

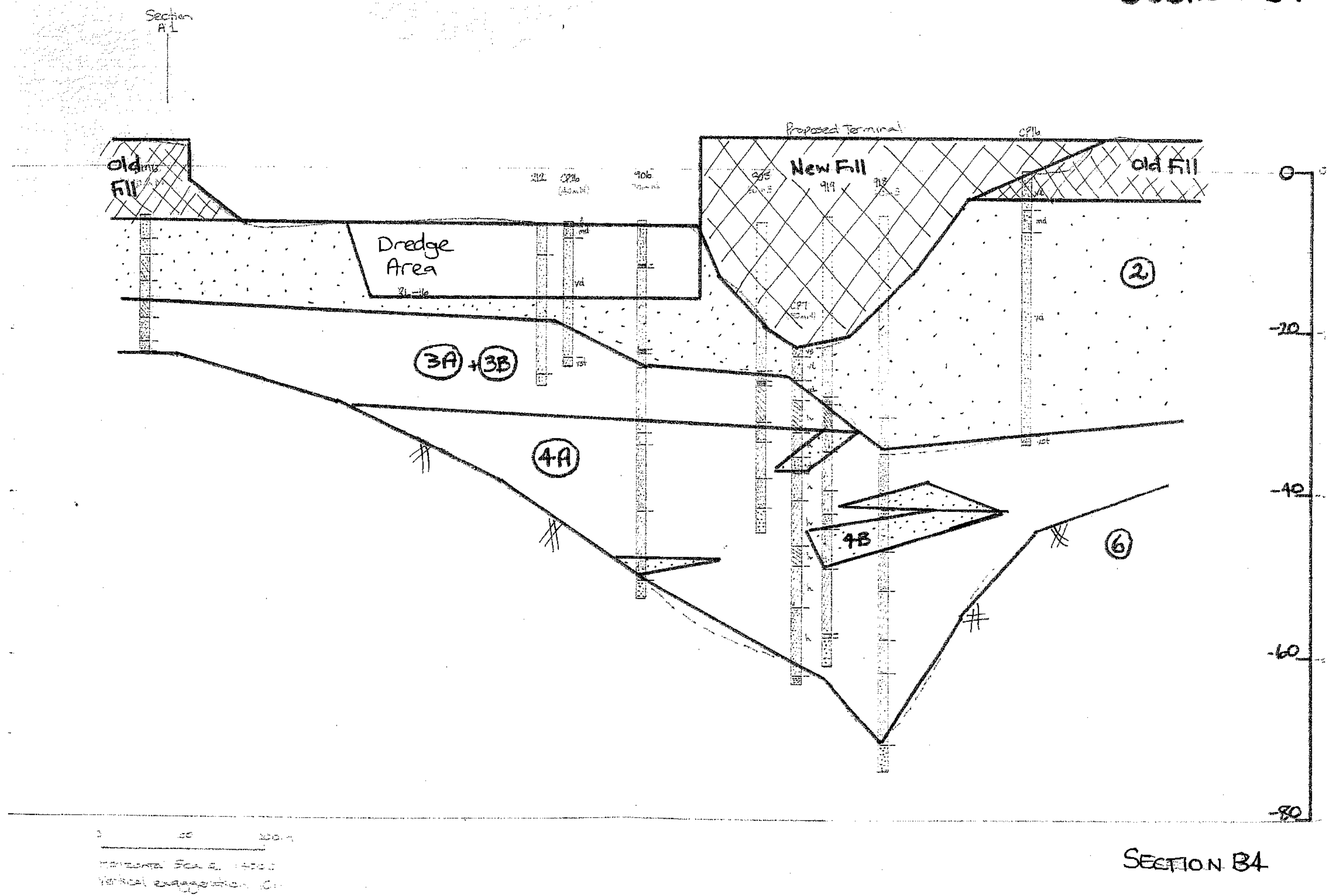
Bore	Seabed RL (ISLW)	Depth From (m)	To (m)	Top sample RL (ISLW)	Soil Unit	In-situ Pressure (kPa)	Preconsolidation Pressure (kPa)	OCR	Maximum Applied Pressure (kPa)	Consolidation eo	stress range (kPa)	Cc	mv (m2/kN)	Cv (m2/yr)	Cr	Ca	Soil Description	Cc/1+eo	Cr/1+eo
CP03	-15.1	7.85	8.2	-22.95	3A	68	290	4.3	300	1.946	155-310	0.0311	2.04E-05	31.9	0.0170	0.005	Black silty clay	0.011	0.006
CP06	-19.75	3.4	3.7	-23.15	3A	29	230	7.9	555	3.102	190-400	0.3772	1.45E-04	0.61	0.0180	0.013	Black silty clay	0.092	0.004
CP02	-2.46	21.6	21.9	-24.06	3B	185	300	1.6	305	1.04	190-300	0.1284	1.14E-04	0.44	0.0580	0.004	Dark grey-black silty clay	0.063	0.028
CP02	-2.46	25.1	25.6	-27.56	3B	215	530	2.5	335	1.416	310-400	0.0960	4.98E-05	0.54	0.0300	0.002	Dark grey-black silty clay	0.040	0.012
CP05	-18.4	6.2	6.5	-24.6	3B	54	323	6.0	310	2.843	155-310	0.1161	5.87E-05	0.04		0.004	Black silty clay	0.030	0.000
CP08	-21.95	4.1	4.45	-26.05	3B	38	320	8.4	320	1.706	190-300	0.1061	7.13E-05	0.23	0.0400	0.001	Black silty clay	0.039	0.015
CP10	-15.95	8.55	8.9	-24.5	3B	74	380	5.1	310	0.523	185-330	0.0306	3.49E-05	0.7	0.0180	0.001	Grey silty clay, trace sand	0.020	0.012
CW8	3.65	38.65		-35	3B	345	150	0.4	390	1.585	200-400	0.3922	2.16E-04	5.69	0.0560	0.001	Dark grey fissured clay	0.152	0.022
DM8	-6.35	10.85	11.15	-17.2	3B	88	420	4.8	250	1.297	100-200	0.0586	7.40E-05	7.1	0.0330	0.0012	Organic sandy clay	0.026	0.014
							Averages	*4.6*				*0.1485*	*8.71E-05*	*5.25*	*0.0338*	*0.004*		*0.148*	*0.034*
CP04	-18.35	26	26.4	-44.35	4A	222	506	2.3	470	0.761	190-380	0.1202	1.07E-04	0.81	0.0600	0.004	Sandy clay	0.068	0.034
CP05	-18.4	20	20.4	-38.4	4A	172	480	2.8	420	0.666	190-380	0.0320	3.04E-05	2.43	0.0480	0.002	Light grey silty clay, some sand	0.019	0.029
CP05	-18.4	35.4	35.8	-53.8	4A	303	400	1.3	545	0.88	380-850	0.3504	1.40E-04	0.51	0.1080	0.010	Dark grey-black silty clay	0.186	0.057
CP06	-19.75	17.4	17.7	-37.15	4A	150	300	2.0	410	0.747	190-380	0.1169	1.06E-04	0.2	0.0590	0.001	Grey silty clay, trace sand	0.067	0.034
CP07	-21.8	11.2	11.55	-33	4A	96	445	4.6	375	0.707	275-370	0.0303	2.43E-05	0.81	0.0430	0.001	Grey silty clay, trace sand	0.018	0.025
CP07	-21.8	22.9	23.2	-44.7	4A	196	730	3.7	470	1.03	190-380	0.1180	9.19E-05	0.64	0.0630	0.001	Grey silty clay, trace sand	0.058	0.031
CP11	-11.4	23.75	24.1	-35.15	4A	203	320	1.6	395	0.613	190-380	0.0530	5.19E-05	3.17	0.0290	0.002	Grey silty clay, trace sand	0.033	0.018
CP11	-11.4	48.8	49.1	-60.2	4A	416	1200	2.9	595	0.698	760-1000	0.0841	2.51E-05	1.53	0.0880	0.002	Grey silty clay, trace sand	0.050	0.052
CP12	-11.7	29.3	29.65	-41	4A	250	670	2.7	440	0.868	190-380	0.0927	7.79E-05	0.44	0.1380	0.003	Grey silty clay, trace sand	0.050	0.074
							Averages	*2.7*				*0.1108*	*7.27E-05*	*1.17*	*0.0707*	*0.003*		*0.111*	*0.071*
						Combined	*Averages*	*3.6*				*0.1297*	*7.99E-05*	*3.21*	*0.0533*	*0.003*		*0.130*	*0.053*

APPENDIX D
Stability Analysis

Section A6



SECTION A6


Section A1
Proposed Terminal
Old Fill
New Fill
Old Fill
Dredge Area
2
3A + 3B
4A
4B
6
0
-20
-40
-60
-80
212
CP16
906
905
919
918
CPT

SECTION B4

APPENDIX D1
Temporary Rock Berms

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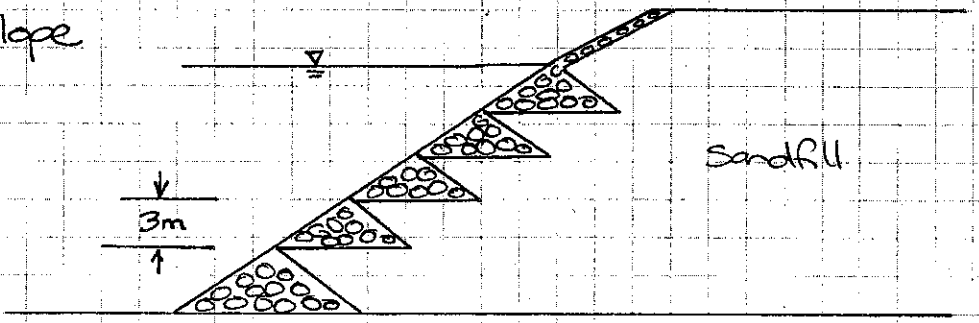
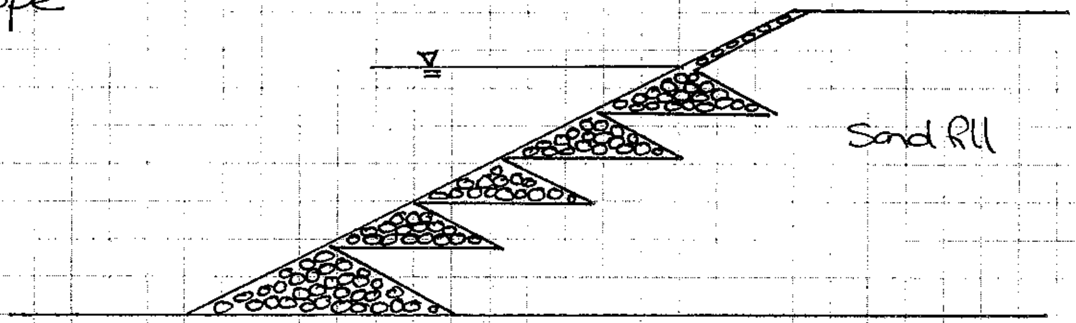
PROJECT Port Botany Job No. 35224 Page No. ______ of ______

ROCK BERM ANALYSIS

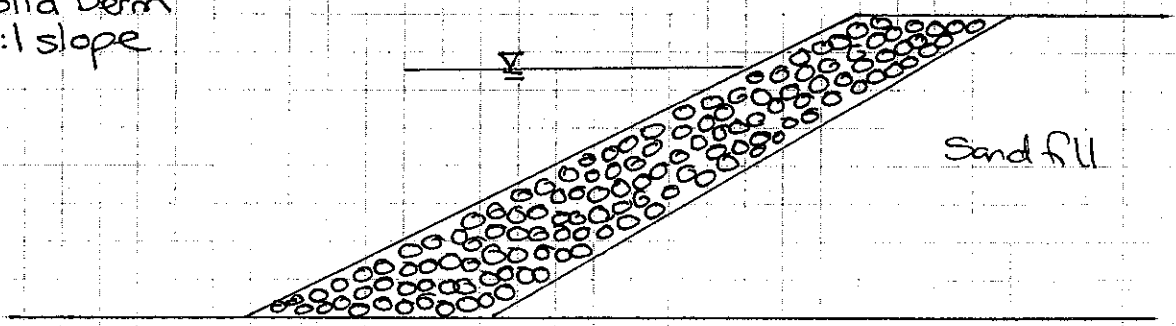
1·5:1 slope



2:1 slope



Solid berm
2:1 slope



Comps By / /	Checked By / /	Approved By / /

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PROJECT Port Botany Job No. Page No. of

CASE : SECTION A6 — Temporary Rock Berm

SERIES OF 5 ROCKBERMS

OVERALL SLOPE 1.5H:1V

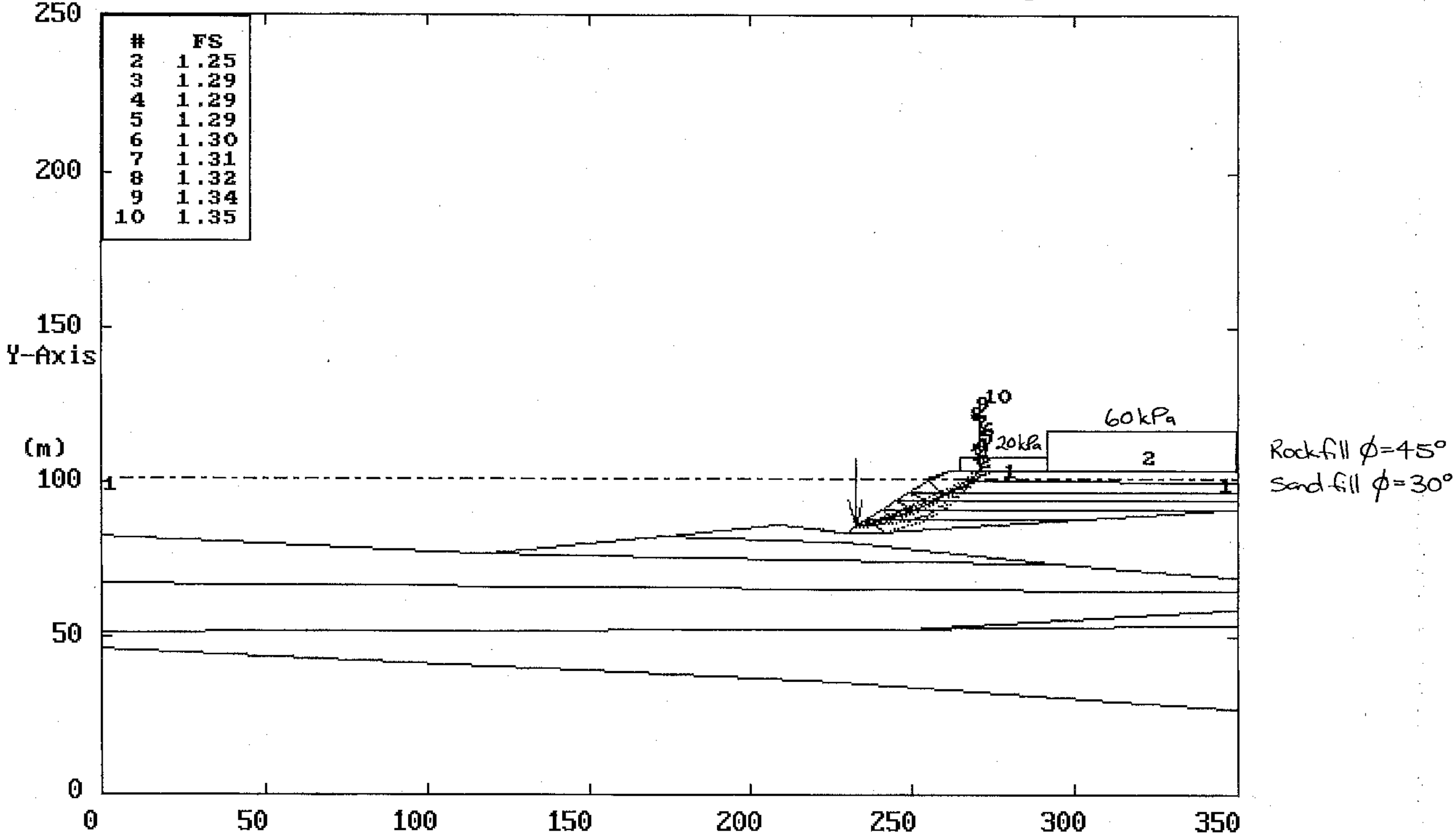
	Minimum Factor of Safety	
Sand Fill	20 & 60 kPa loads	Plus earthquake
$\phi = 30°$	1.22	0.89
$\phi = 35°$	1.44	1.05

OVERALL SLOPE 2H:1V

	Minimum Factor of Safety	
Sand Fill	20 + 60 kPa loads	Plus earthquake
$\phi = 30°$	1.34	0.96
$\phi = 35°$	1.55	1.11

Comps By / /	Checked By / /	Approved By / /

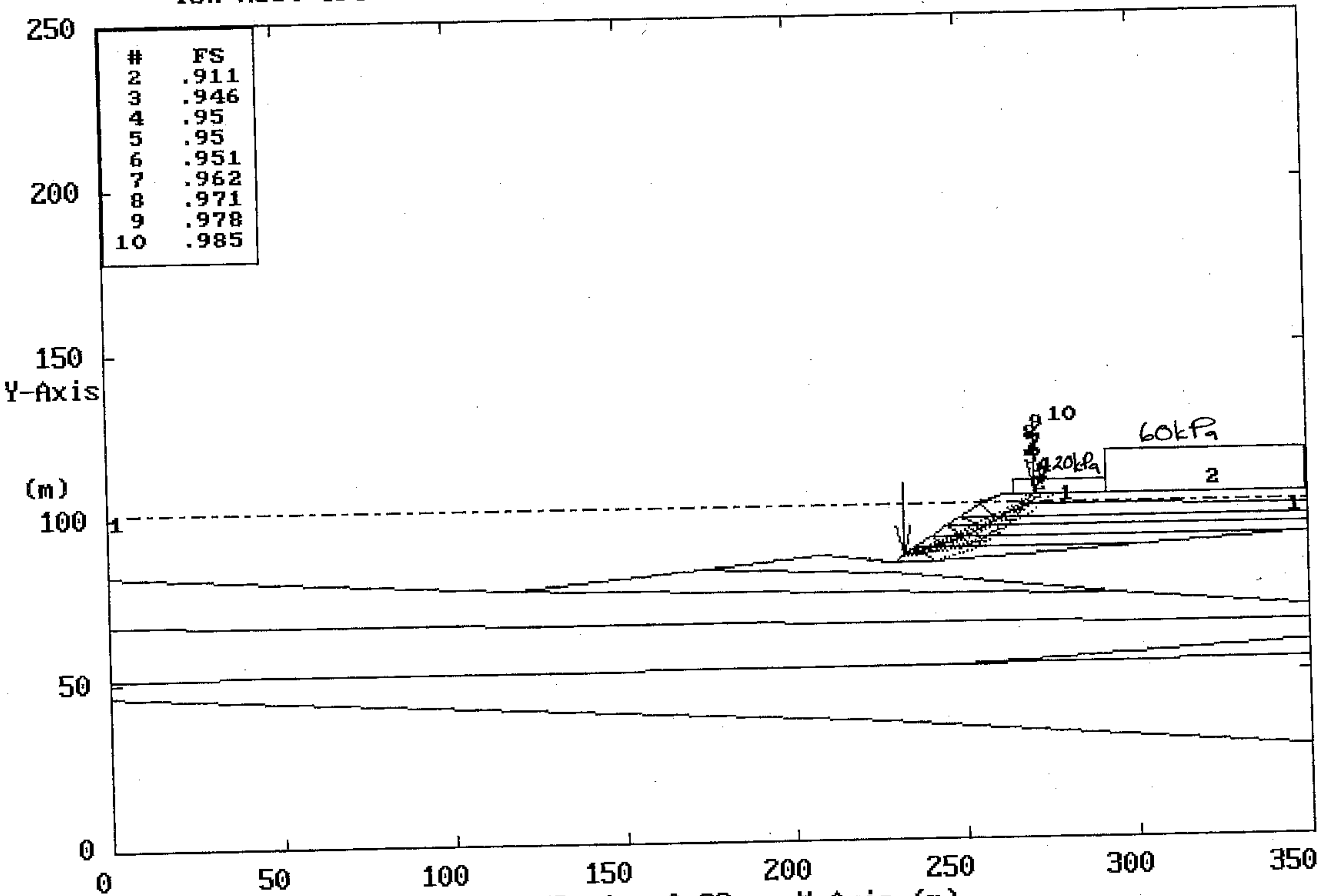
Port Botany - Section A6 Rock Berm 1.5:1 slope
Ten Most Critical. C:BOTA62.PLT By: 08-30-02 1:29 pm



PCSTABL5 FS min= 1.219 X-Axis (m)

Temporary Rock Berm 1.5:1 slope

Port Botany - Section A6 Rock Berm 1.5:1 slope
Ten Most Critical. C:BOTA62.PLT By: 08-30-02 1:35 pm



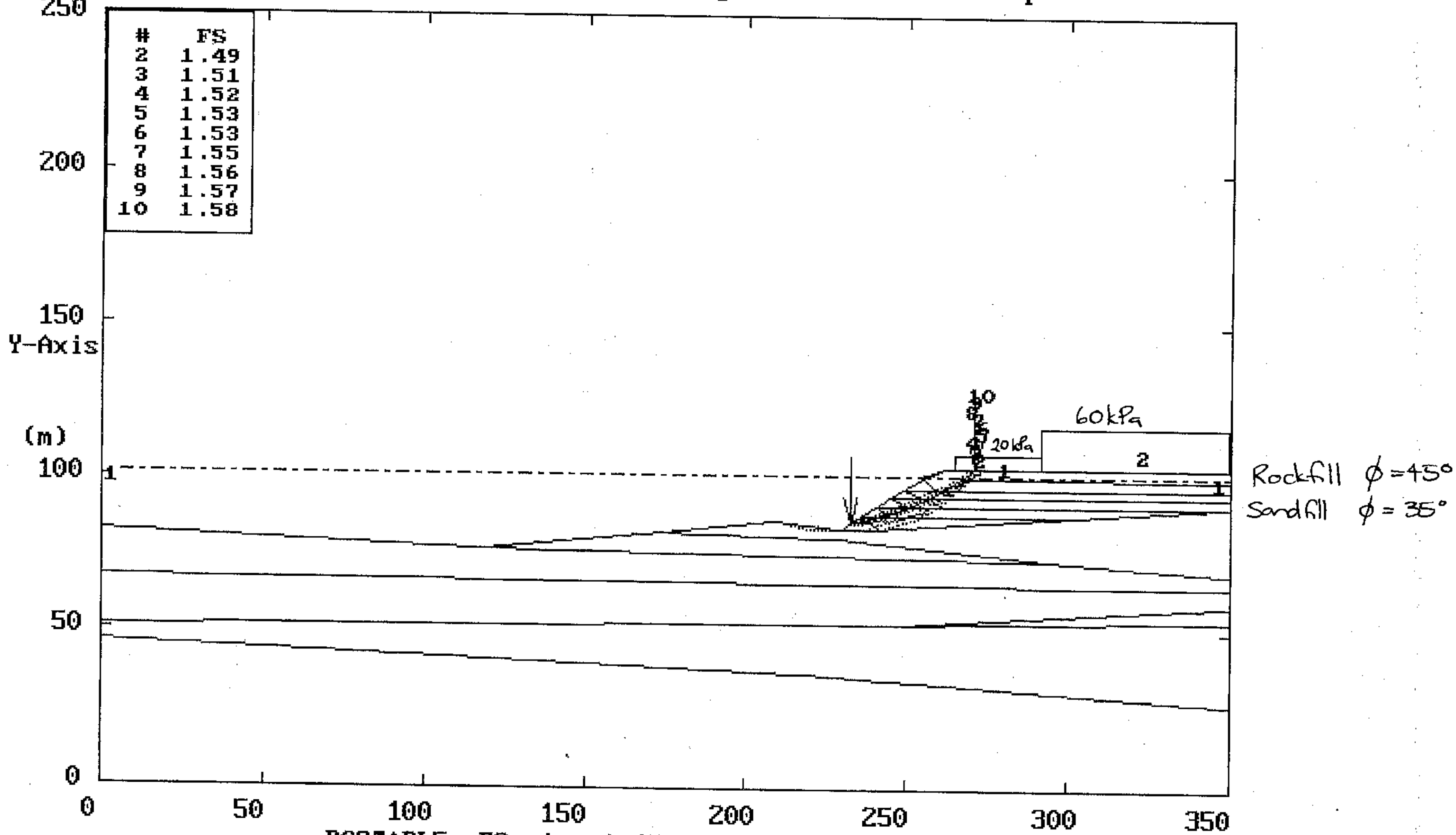
PCSTABL5 FS min= 0.89

Rockfill $\phi = 45°$
Sandfill $\phi = 30°$

Plus Earthquake
0.08g Horiz.
0.04g Vert.

Temporary Rock Berm 1.5:1 slope

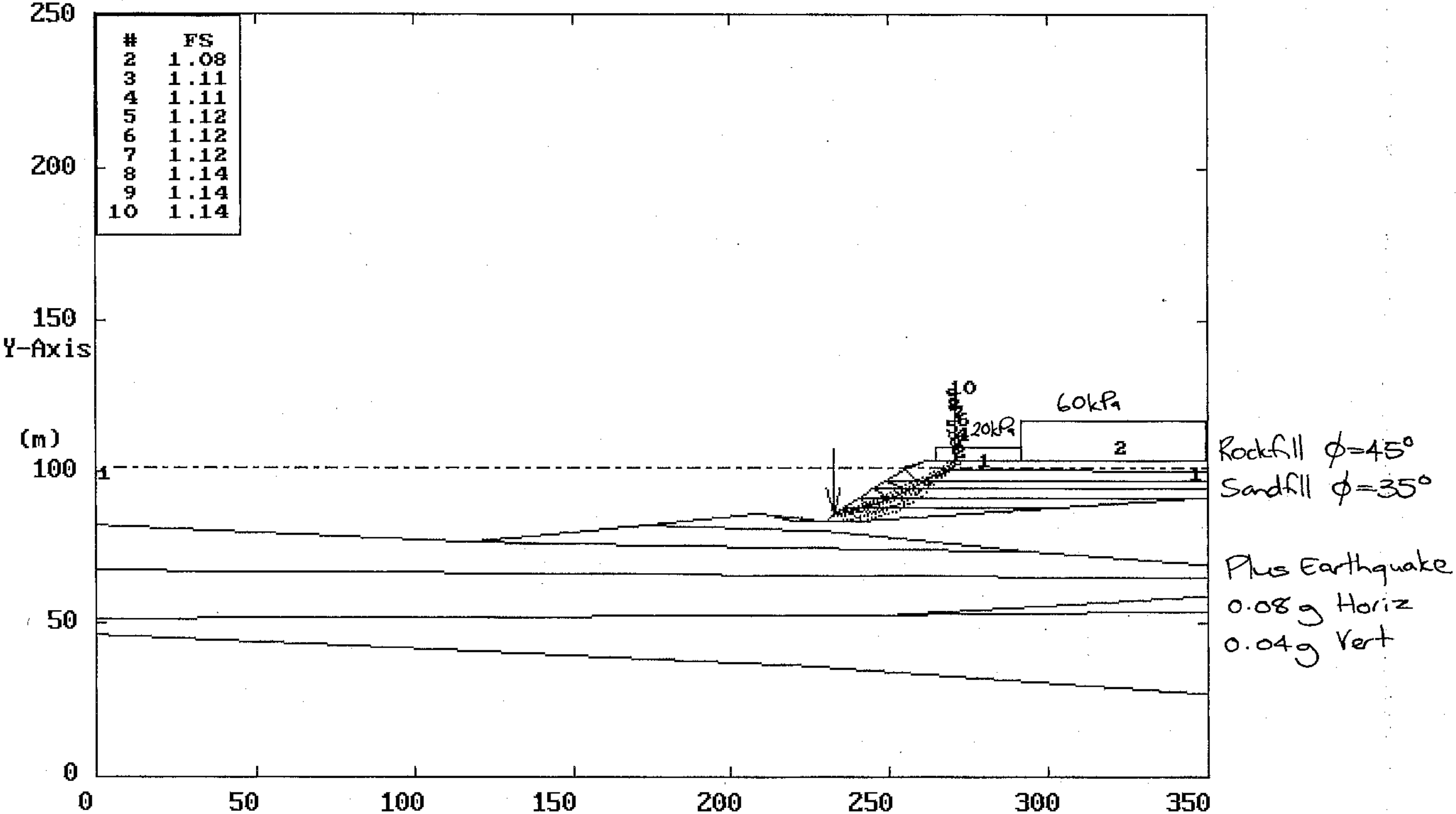
Port Botany - Section A6 Rock Berm 1.5:1 slope
Ten Most Critical. C:BOTA62.PLT By: 08-30-02 1:30 pm



PCSTABL5 FS min= 1.439 X-Axis (m)

Temporary Rock Berm 1.5:1 slope

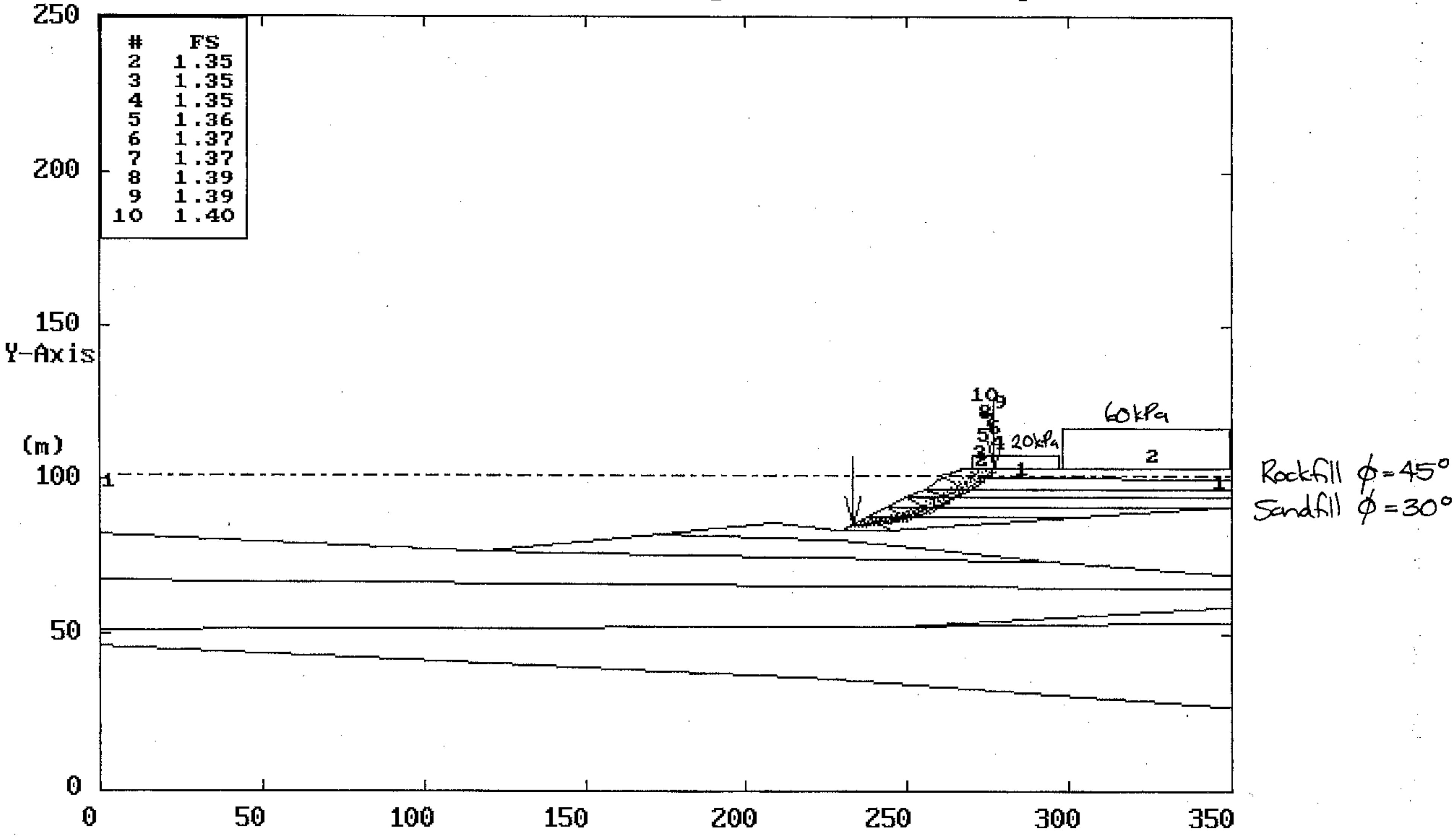
Port Botany - Section A6 Rock Berm 1.5:1 slope
Ten Most Critical. C:BOTA62.PLT By: 08-30-02 1:33 pm



PCSTABL5 FS min= 1.05 X-Axis (m)

Temporary Rock
Berm 1.5:1 slope

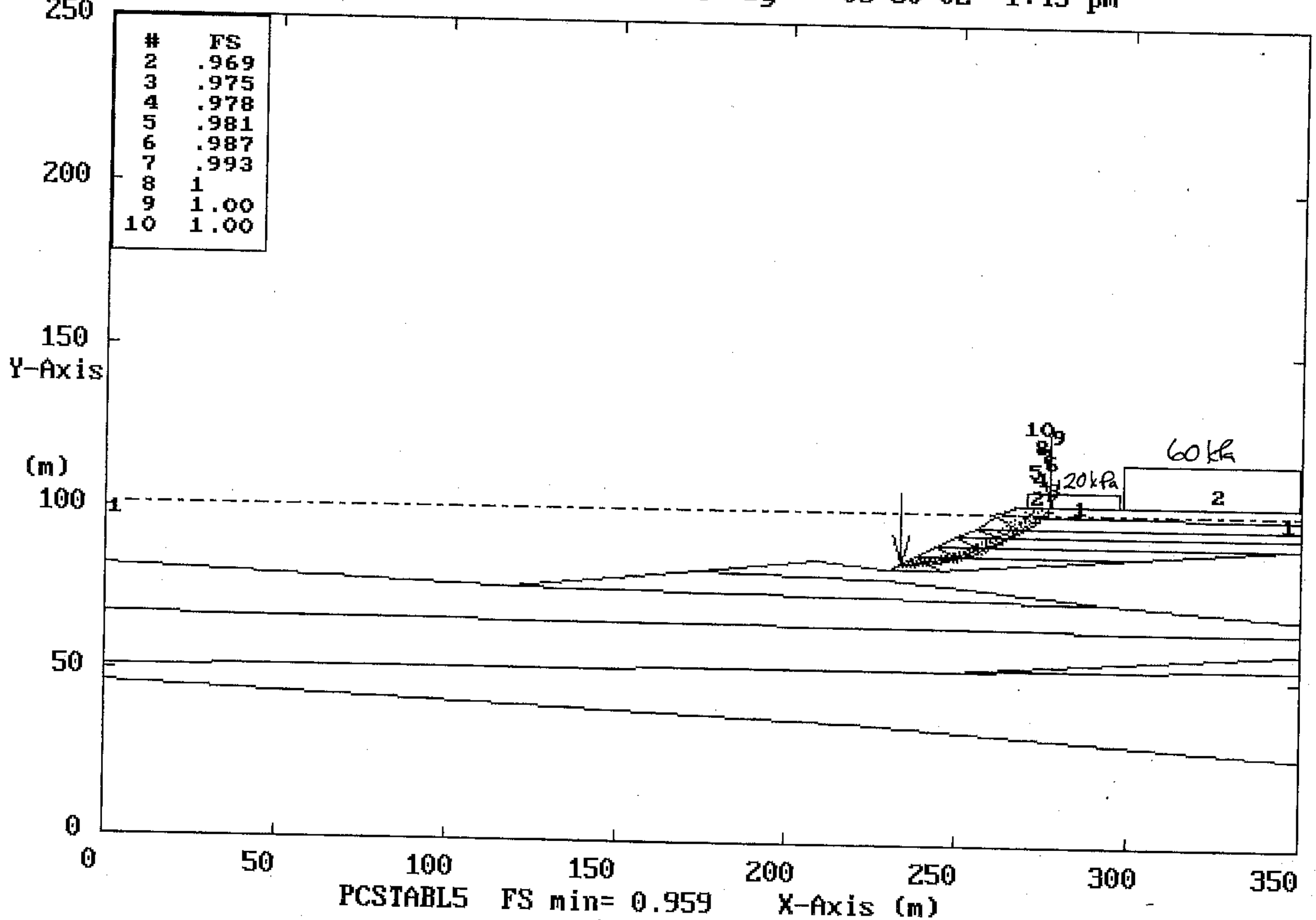
Port Botany - Section A6 Rock Berm 2:1 slope
Ten Most Critical. C:BOTA63.PLT By: 08-30-02 1:44 pm



PCSTABL5 FS min= 1.336 X-Axis (m)

Rock Berm 2:1

Port Botany - Section A6 Rock Berm 2:1 slope
Ten Most Critical. C:BOTA63.PLT By: 08-30-02 1:45 pm



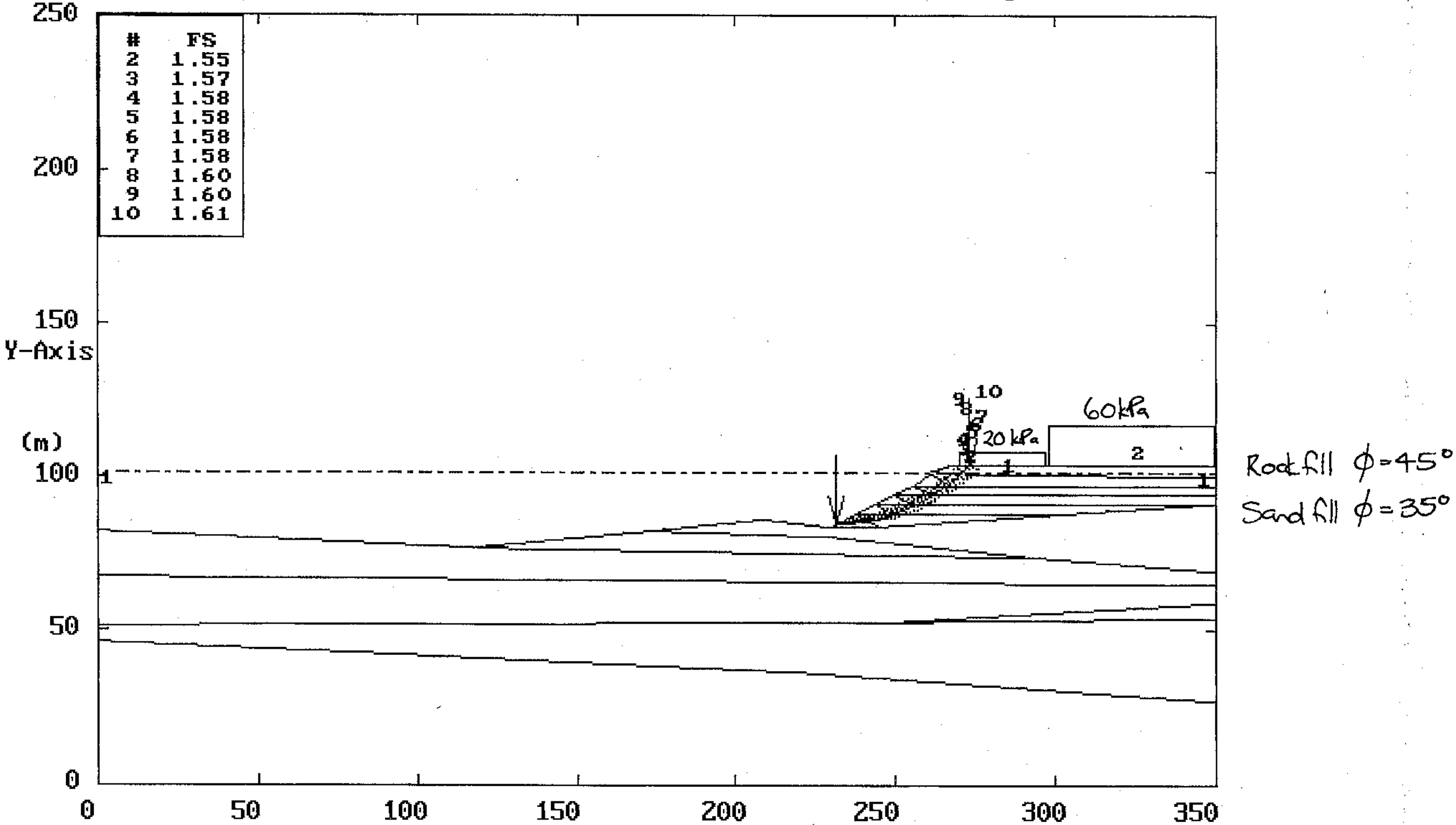
Rockfill $\phi = 45°$
Sandfill $\phi = 30°$

Plus earthquake
+0.08g Horiz
+0.04g Vertical

Rock Berm 2:1

Port Botany - Section A6 Rock Berm 2:1 slope

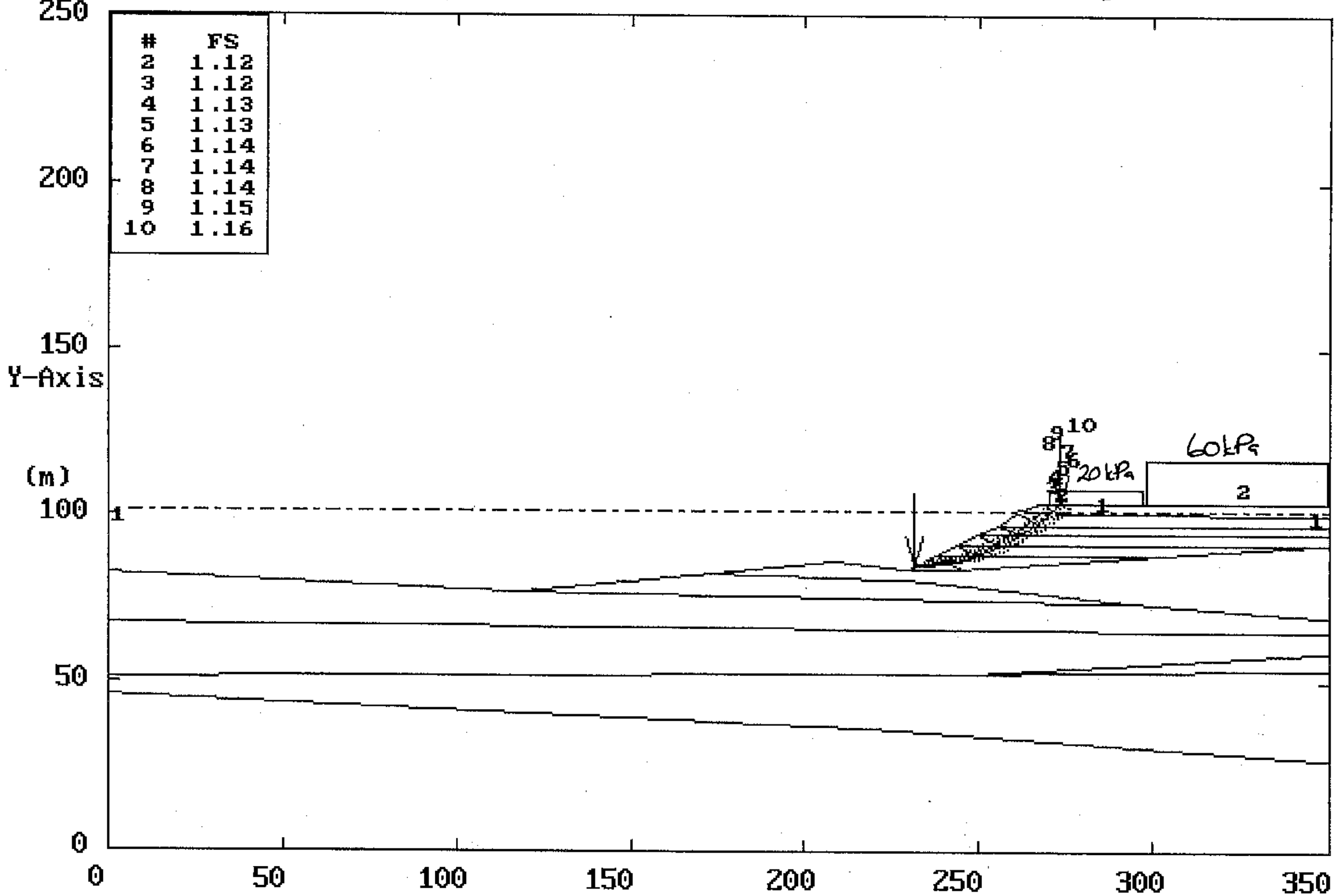
Ten Most Critical. C:BOTA63.PLT By: 08-30-02 1:44 pm



PCSTABL5 FS min= 1.553

Rock Berm 2:1

Port Botany - Section A6 Rock Berm 2:1 slope
Ten Most Critical. C:BOTA63.PLT By: 08-30-02 1:46 pm



PCSTABL5 FS min= 1.112 X-Axis (m)

Rockfill $\phi = 45°$
Sandfill $\phi = 35°$

Plus earthquake
+0.08g Horiz
0.04g Vertical

Rock Berm 2:1

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PROJECT Port Botany Job No.______ Page No.____ of____

CASE: SECTION B4 – Temporary Rock Berm

SERIES OF 5 ROCK BERMS

OVERALL SLOPE 1.5H:1V

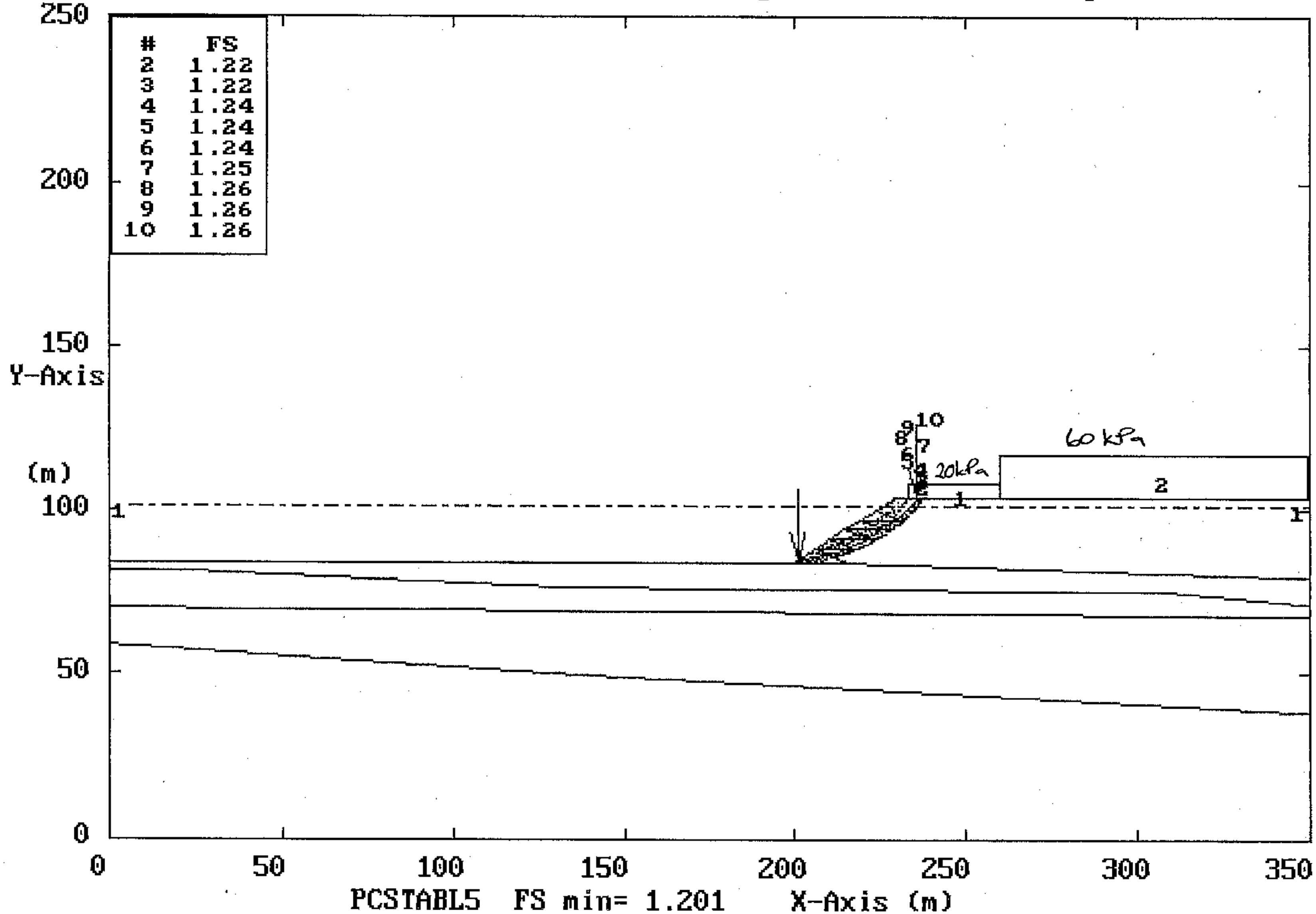
	Minimum Factor of Safety	
Sand Fill	20+60kPa loads	Plus earthquake
$\phi = 30°$	1.20	0.88
$\phi = 35°$	1.38	1.01

OVERALL SLOPE 2H:1V

	Minimum Factor of Safety	
Sand Fill	20+60kPa loads	Plus earthquake
$\phi = 30°$	1.45	1.02
$\phi = 35°$	1.74	1.23

Comps By / / Checked By / / Approved By / /

Port Botany Section B4 Rock Berm 1.5:1 slope
Ten Most Critical. C:BOTB46.PLT By: 08-30-02 2:34 pm

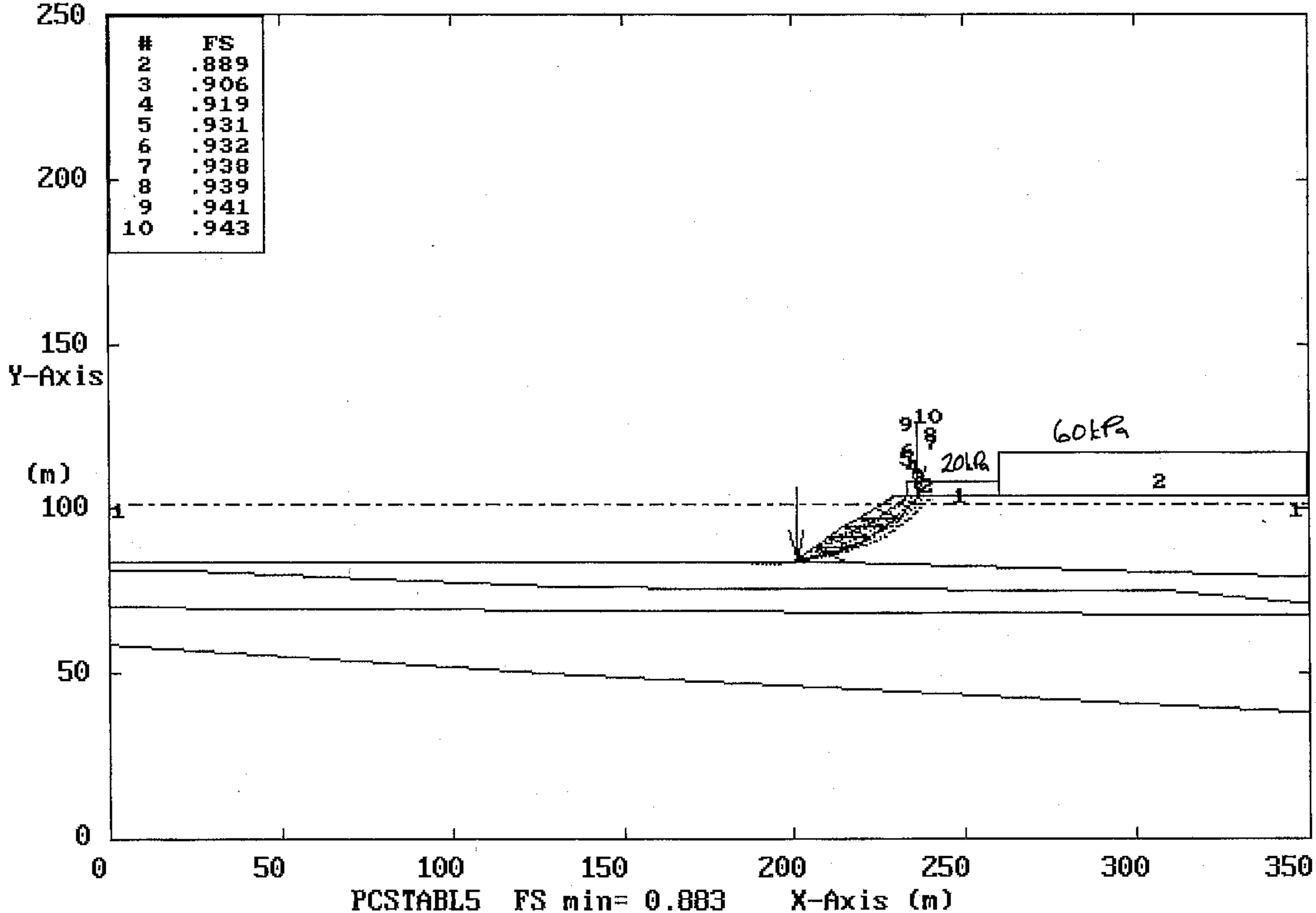


Rockfill $\phi = 45°$
Sandfill $\phi = 30°$

Rock Berm
1.5:1 slope

Port Botany Section B4 Rock Berm 1.5:1 slope

Ten Most Critical. C:BOTB46.PLT By: 08-30-02 2:36 pm



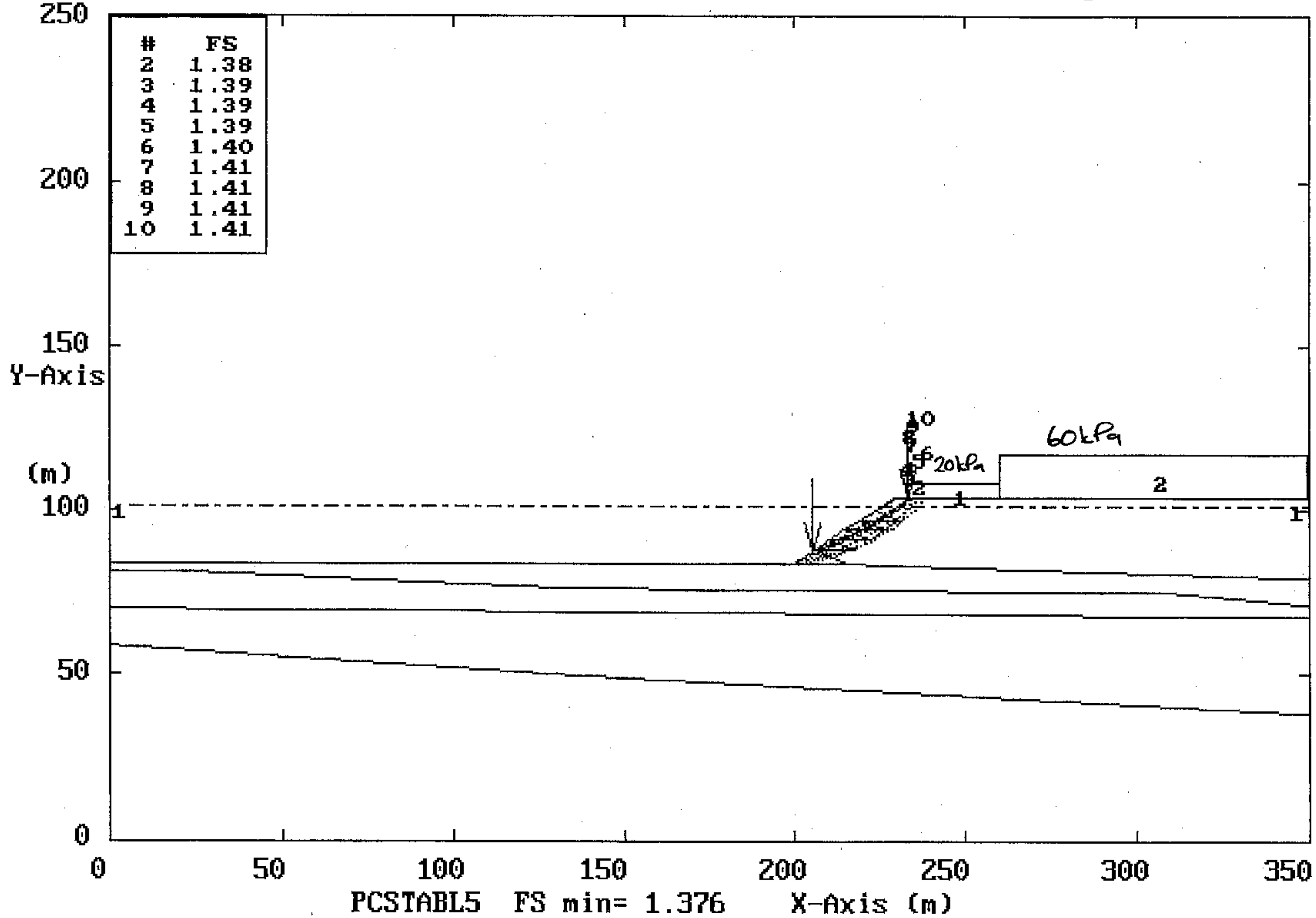
Rockfill $\phi=45°$
Sandfill $\phi=30°$

Plus Earthquake
0.08g Horiz
0.04g Vertical

Rock Berm
1.5:1 slope

Port Botany Section B4 Rock Berm 1.5:1 slope

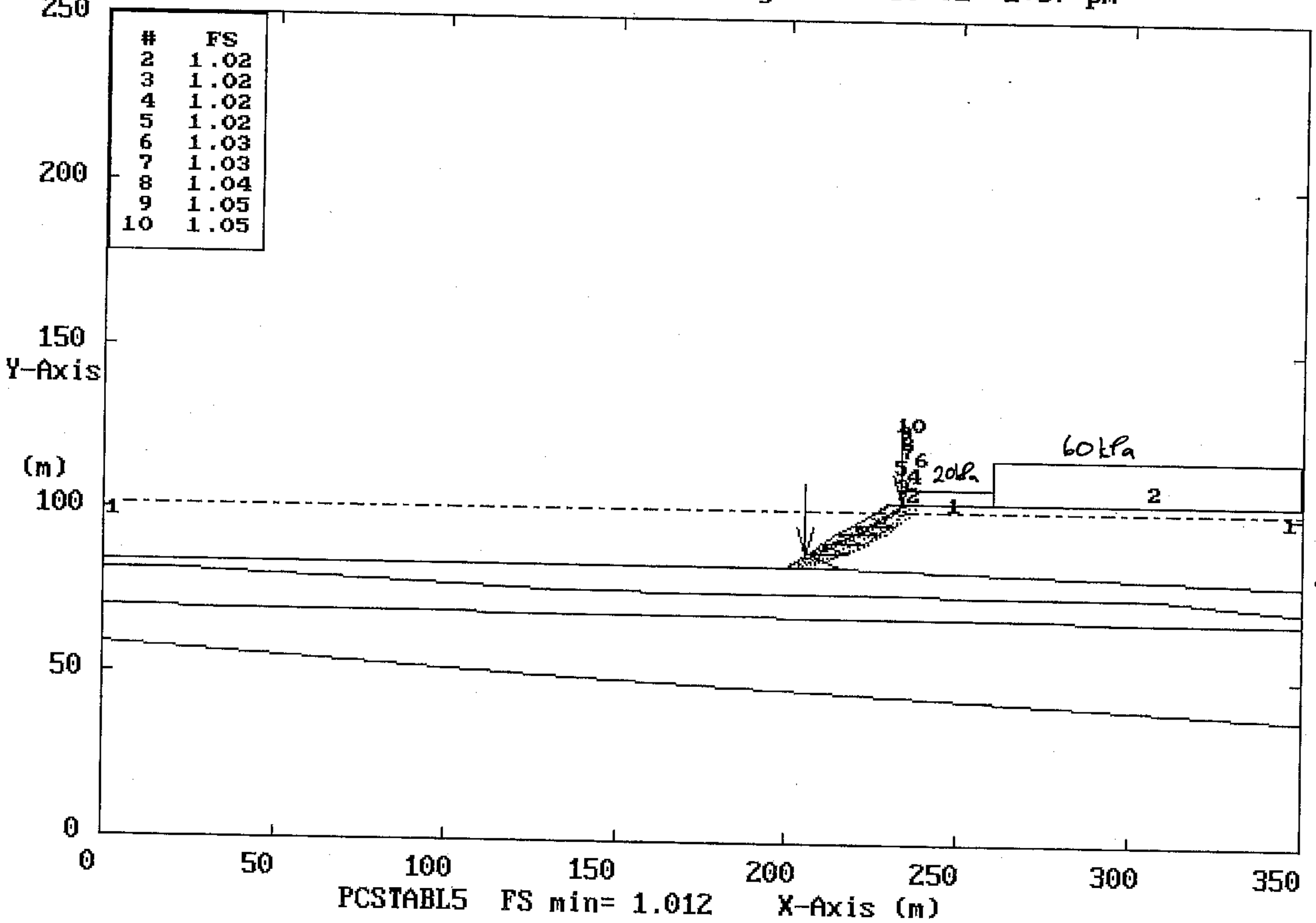
Ten Most Critical. C:BOTB46.PLT By: 08-30-02 2:35 pm



Rockfill $\phi = 45°$
Sandfill $\phi = 35°$

Rock Berm
1.5:1 slope

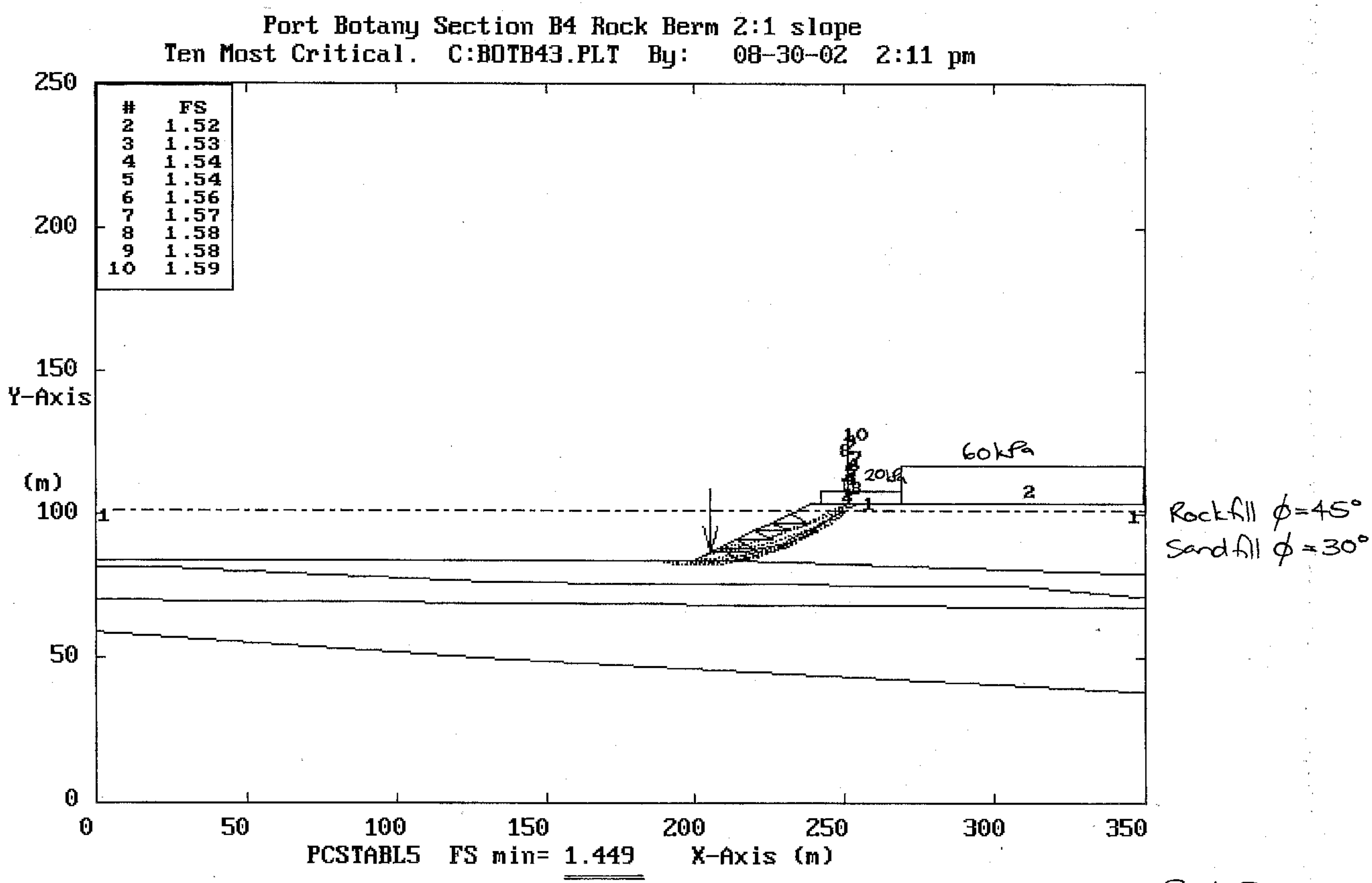
Port Botany Section B4 Rock Berm 1.5:1 slope
Ten Most Critical. C:BOTB46.PLT By: 08-30-02 2:37 pm



PCSTABL5 FS min= 1.012

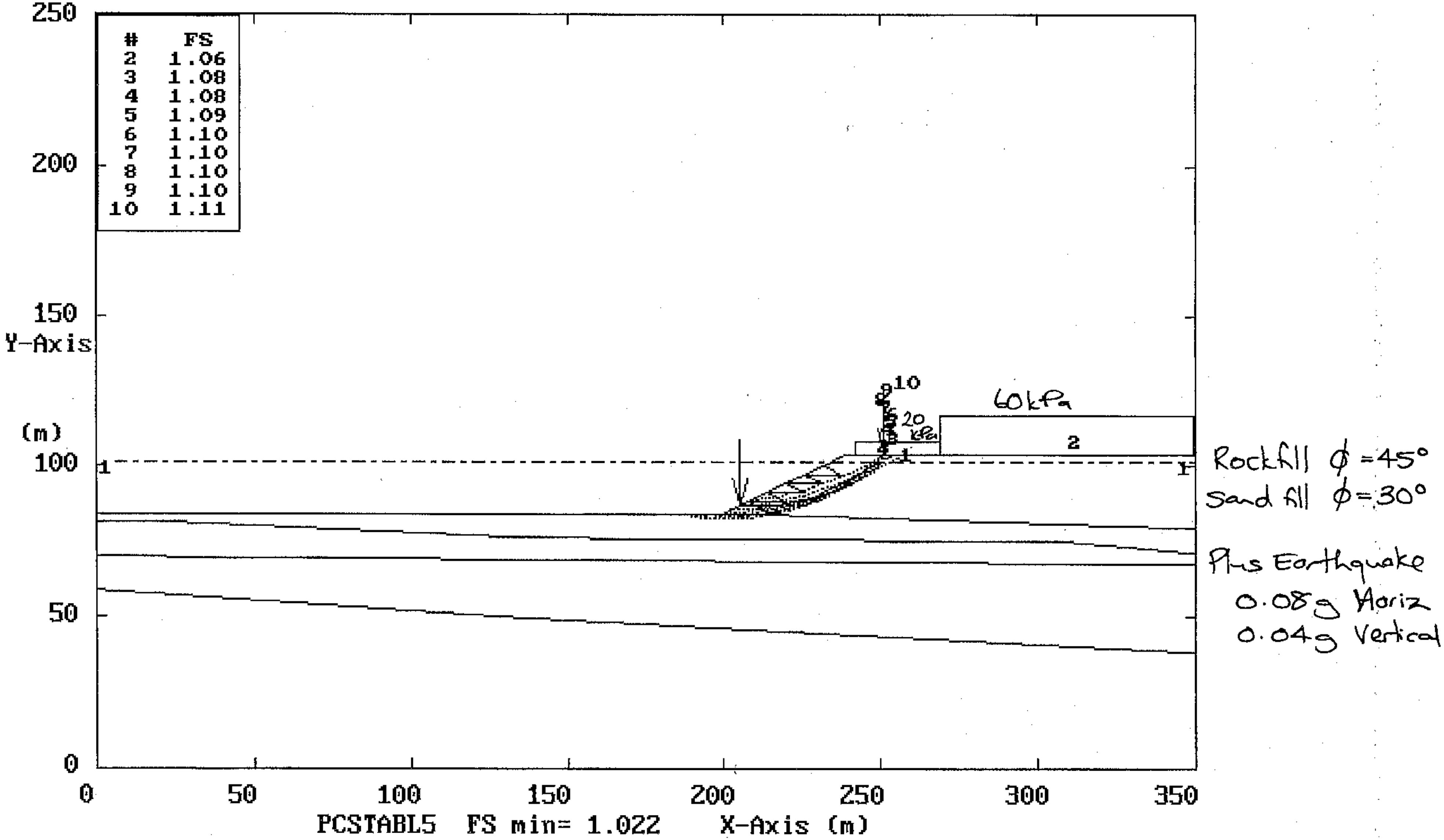
Rockfill $\phi = 45°$
Sandfill $\phi = 35°$
Plus Earthquake
0.08g Horiz
0.04g Vertical

Rock Berm



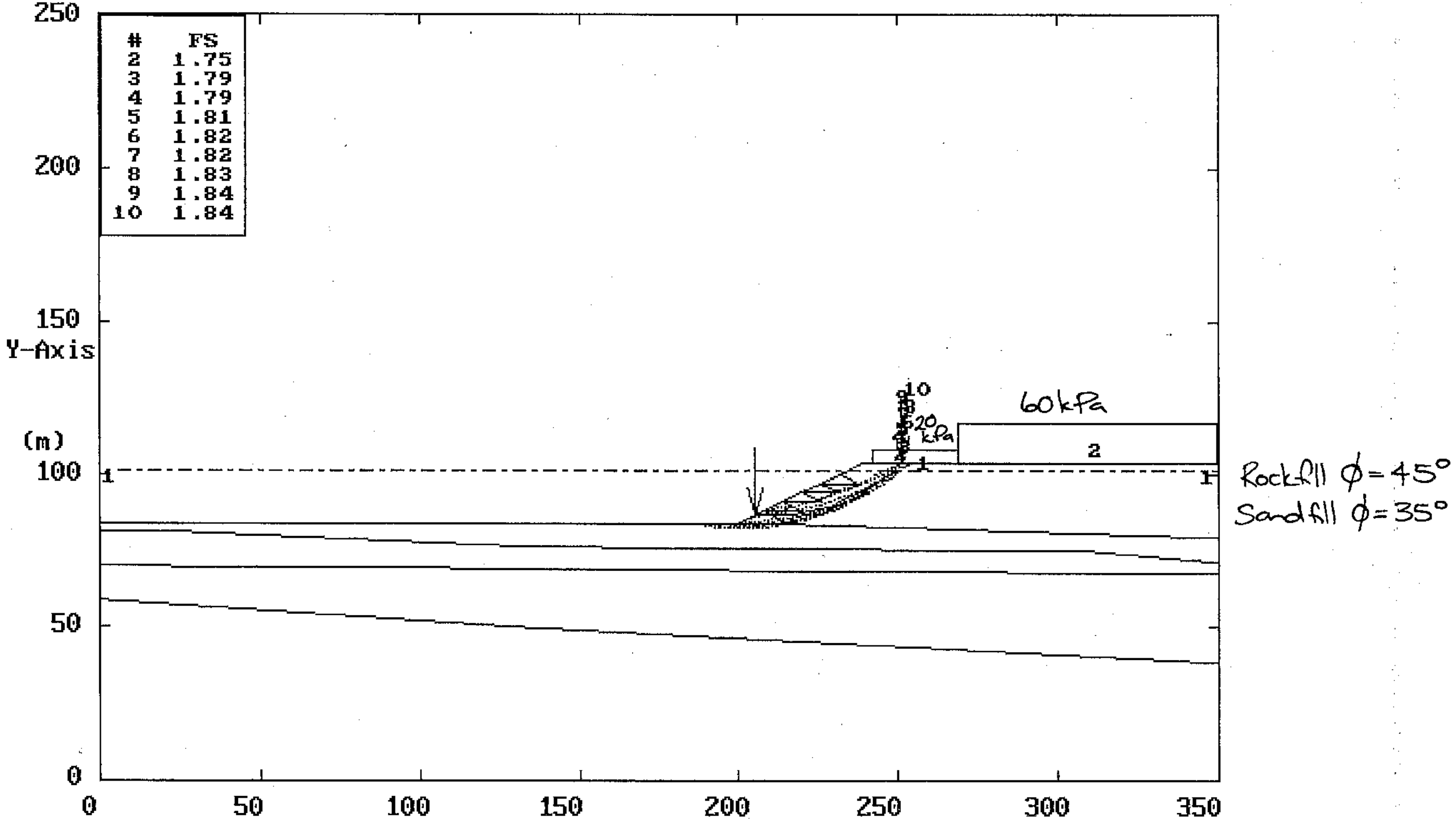
Rock Berm
2:1 slope

Port Botany Section B4 Rock Berm 2:1 slope
Ten Most Critical. C:BOTB43.PLT By: 08-30-02 2:12 pm



Rock Berm
2:1 slope

Port Botany Section B4 Rock Berm 2:1 slope
Ten Most Critical. C:BOTB43.PLT By: 08-30-02 2:12 pm

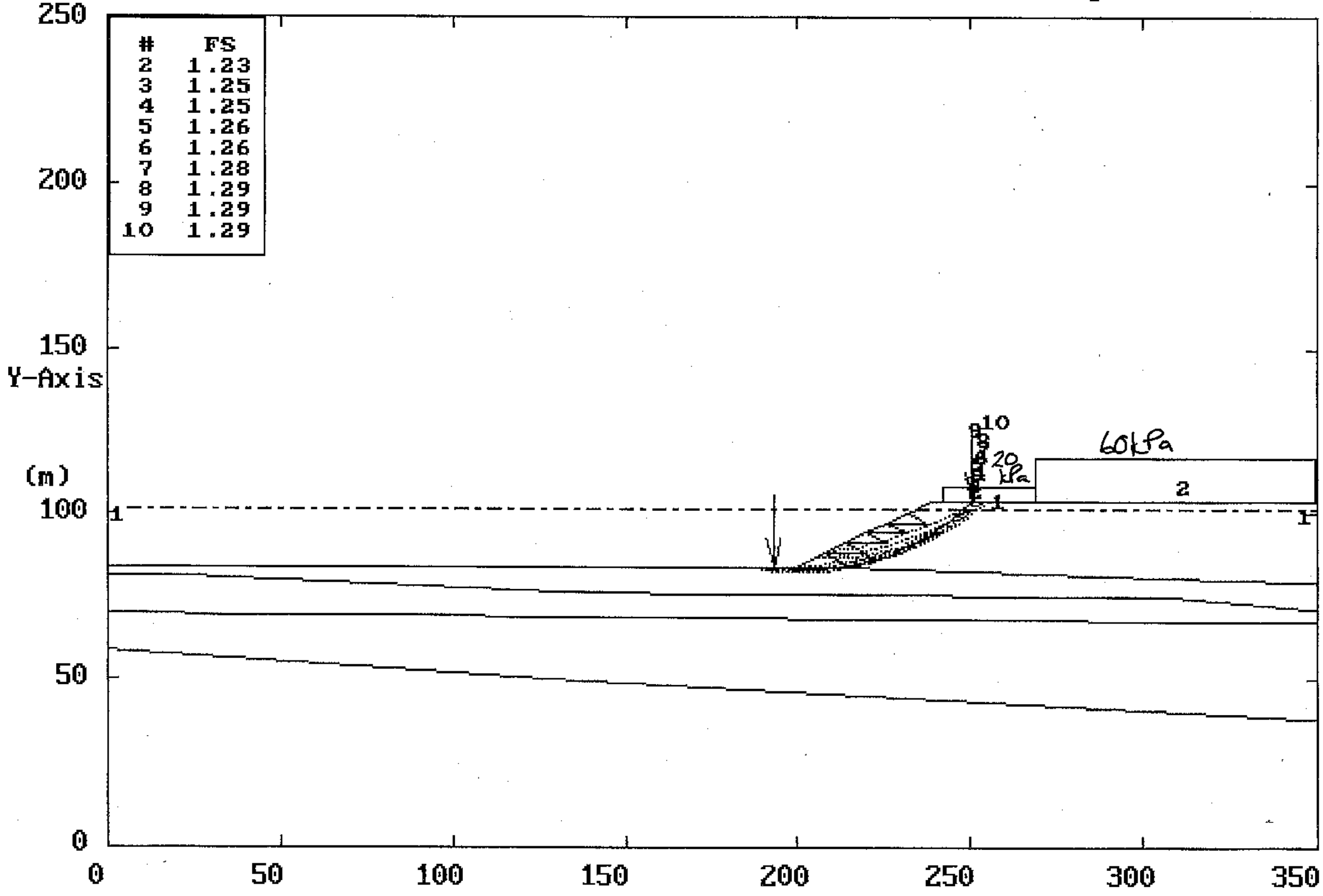


PCSTABL5 FS min= 1.739 X-Axis (m)

Rock Berm
2:1 slope

Port Botany Section B4 Rock Berm 2:1 slope

Ten Most Critical. C:BOTB43.PLT By: 08-30-02 2:13 pm



PCSTABL5 FS min= 1.225 X-Axis (m)

Rockfill $\phi = 45°$

Sandfill $\phi = 35°$

Plus Earthquake

0.08g Horiz

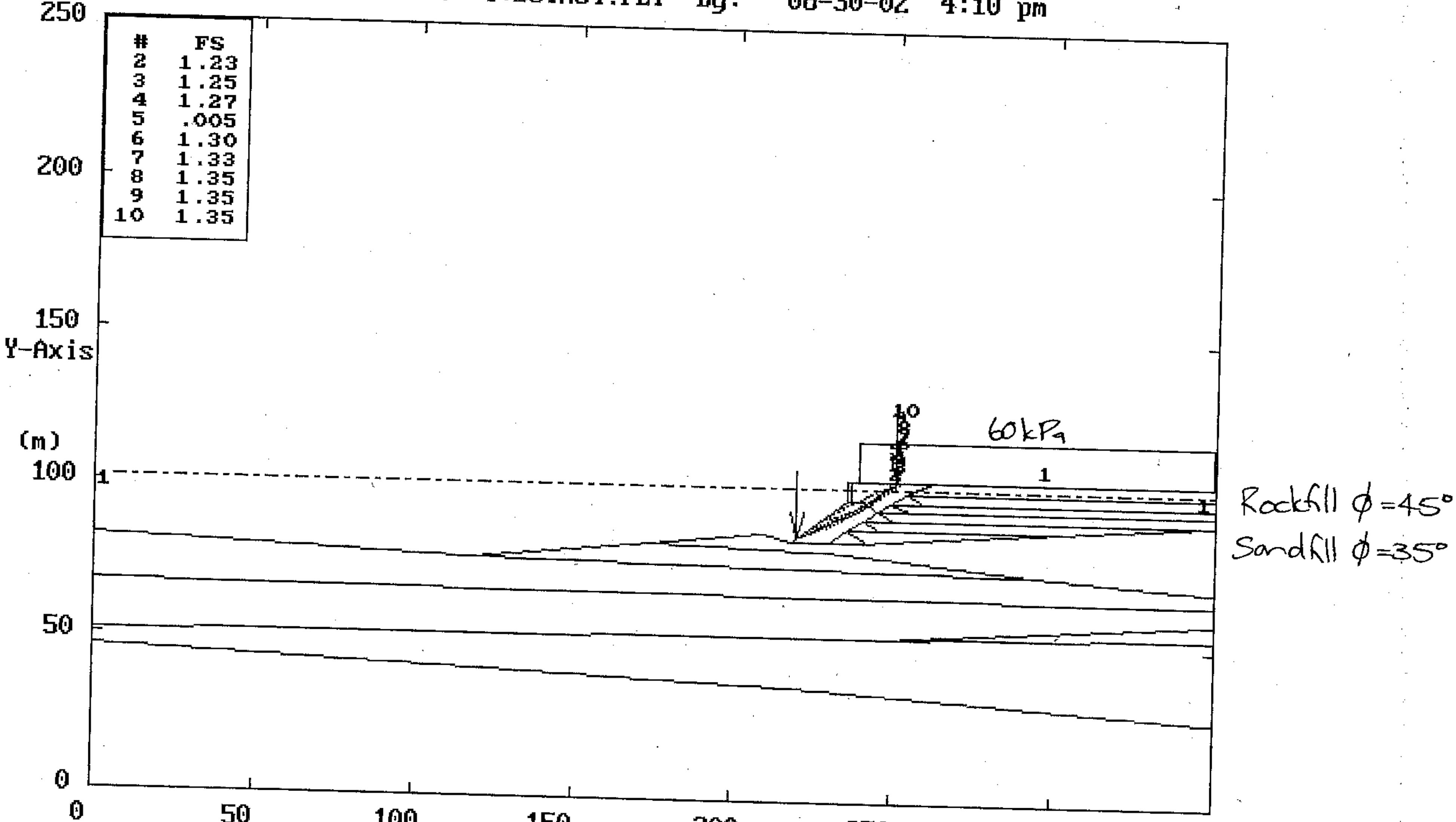
0.04g Vertical

Rock Berm

2:1 slope

APPENDIX D2
Permanent Rock Berms

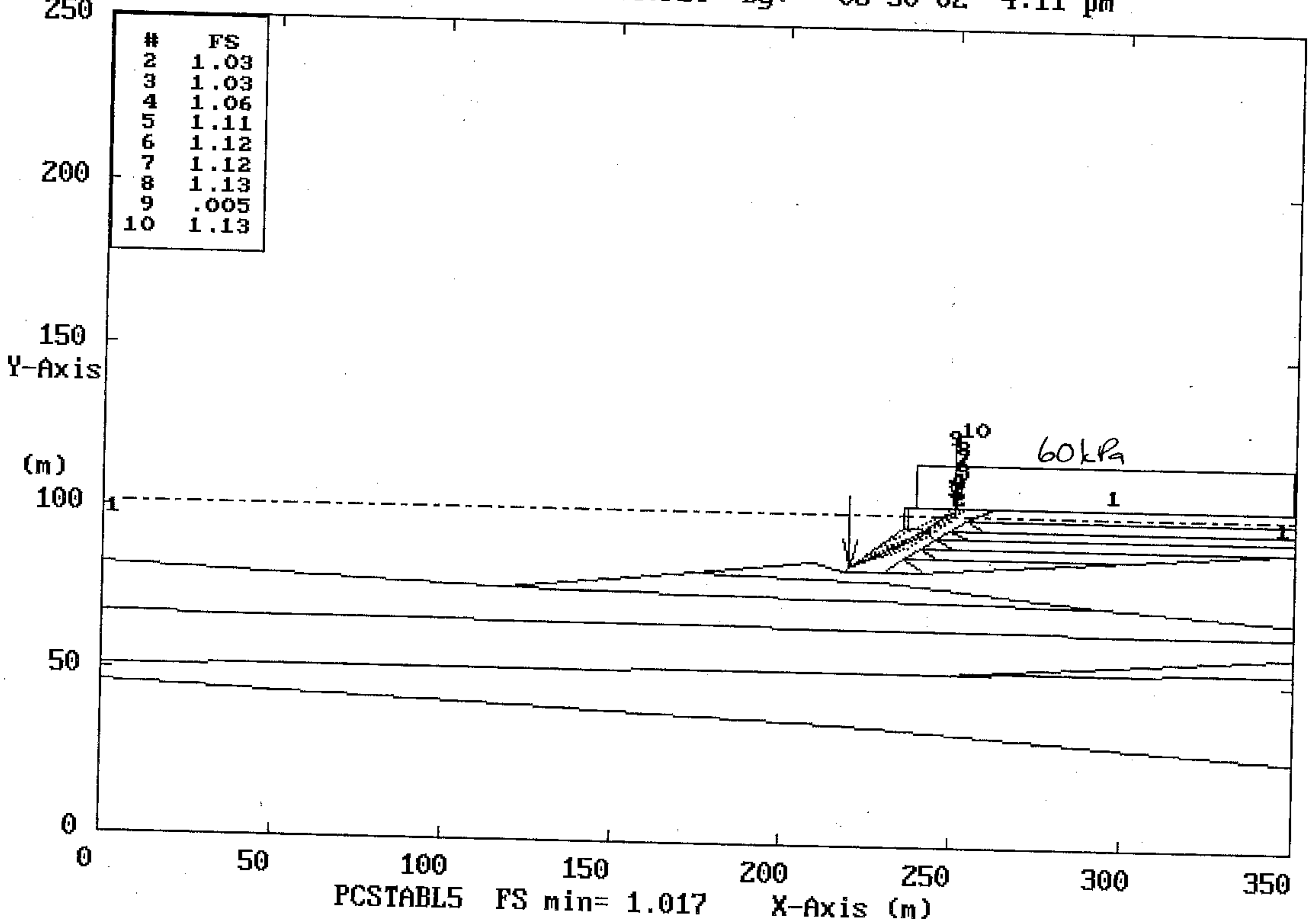
Port Botany - Section A6 Permanent Rock Berm for Piled Wharf
Ten Most Critical. C:BOTA64.PLT By: 08-30-02 4:10 pm



PCSTABL5 FS min= 1.226

Permanent batter
1.4.1

Port Botany - Section A6 Permanent Rock Berm for Piled Wharf
Ten Most Critical. C:BOTA64.PLT By: 08-30-02 4:11 pm

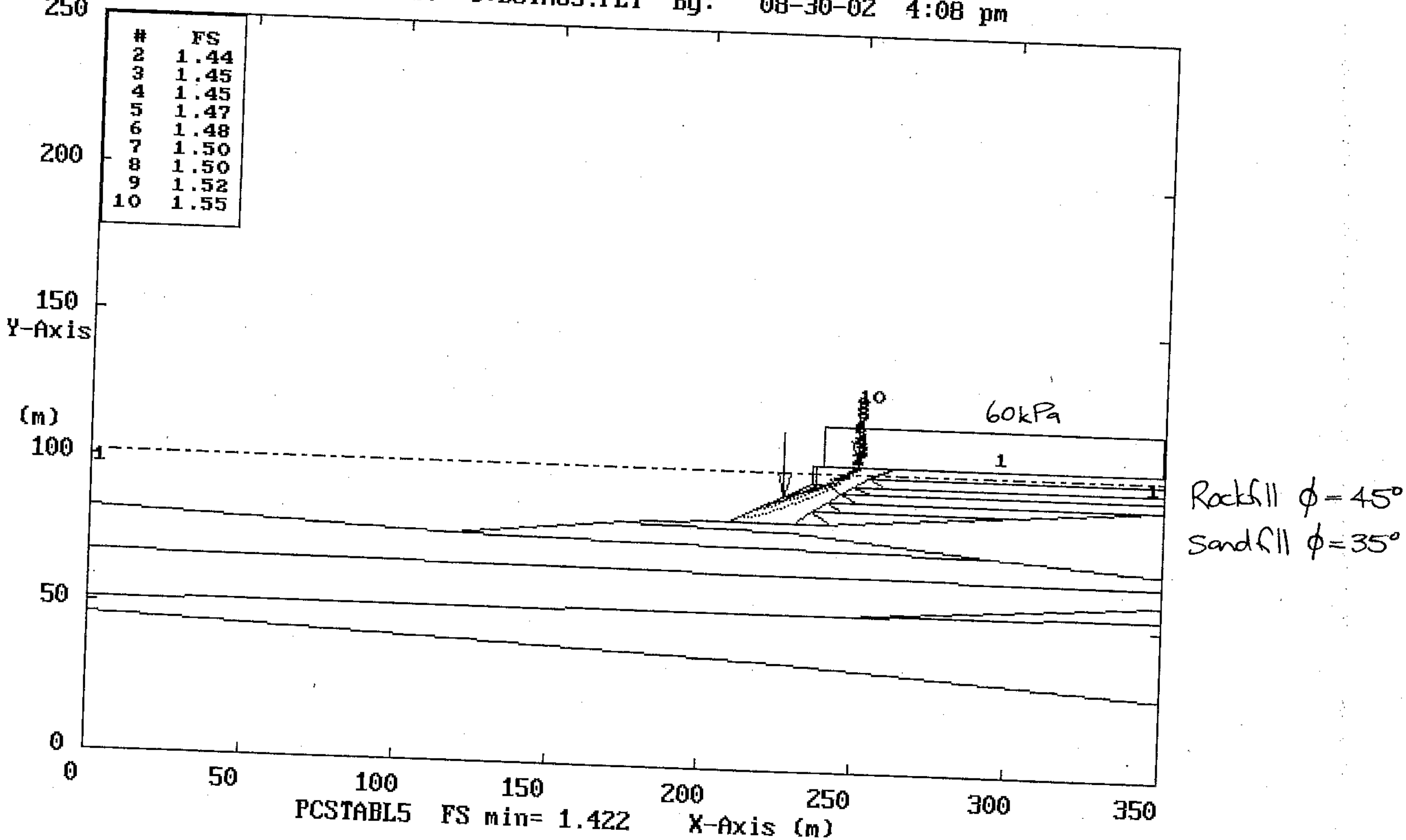


Rockfill $\phi = 45°$
Sandfill $\phi = 35°$

Plus Earthquake
0.08g Horiz
0.04g Vertical

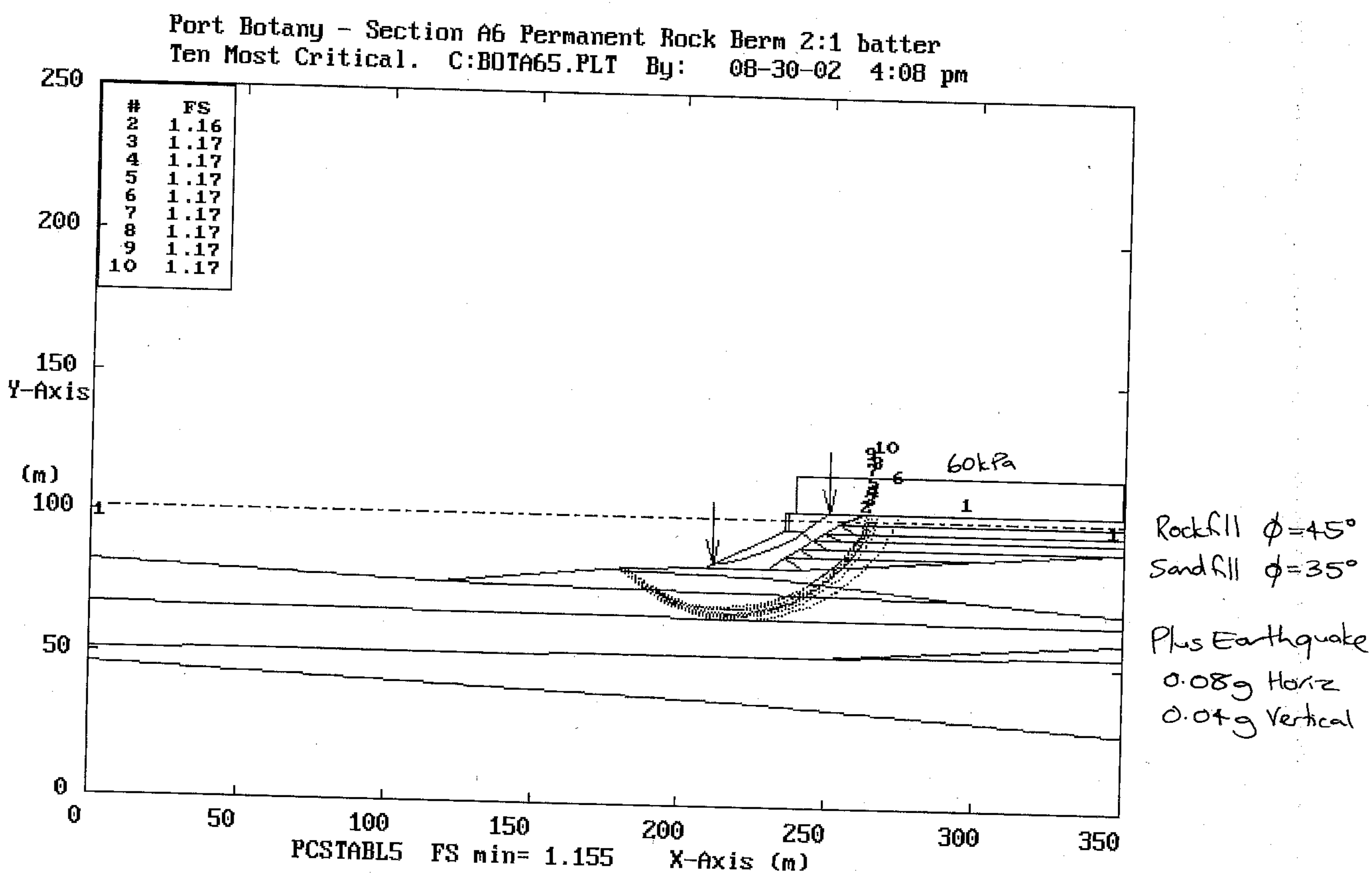
Permanent batter
1.4:1

Port Botany - Section A6 Permanent Rock Berm 2:1 batter
Ten Most Critical. C:BOTA65.PLT By: 08-30-02 4:08 pm



PCSTABL5 FS min= 1.422

Permanent batter 2:1


Port Botany - Section A6 Permanent Rock Berm 2:1 batter
Ten Most Critical. C:BOTA65.PLT By: 08-30-02 4:08 pm
FS
2 1.16
3 1.17
4 1.17
5 1.17
6 1.17
7 1.17
8 1.17
9 1.17
10 1.17
Y-Axis (m)
250
200
150
100
50
0
0
50
100
150
200
250
300
350
60kPa
PCSTABL5 FS min= 1.155 X-Axis (m)
Rockfill ϕ=45°
Sandfill ϕ=35°
Plus Earthquake
0.08g Horiz
0.04g Vertical

APPENDIX D3
Caisson Structure

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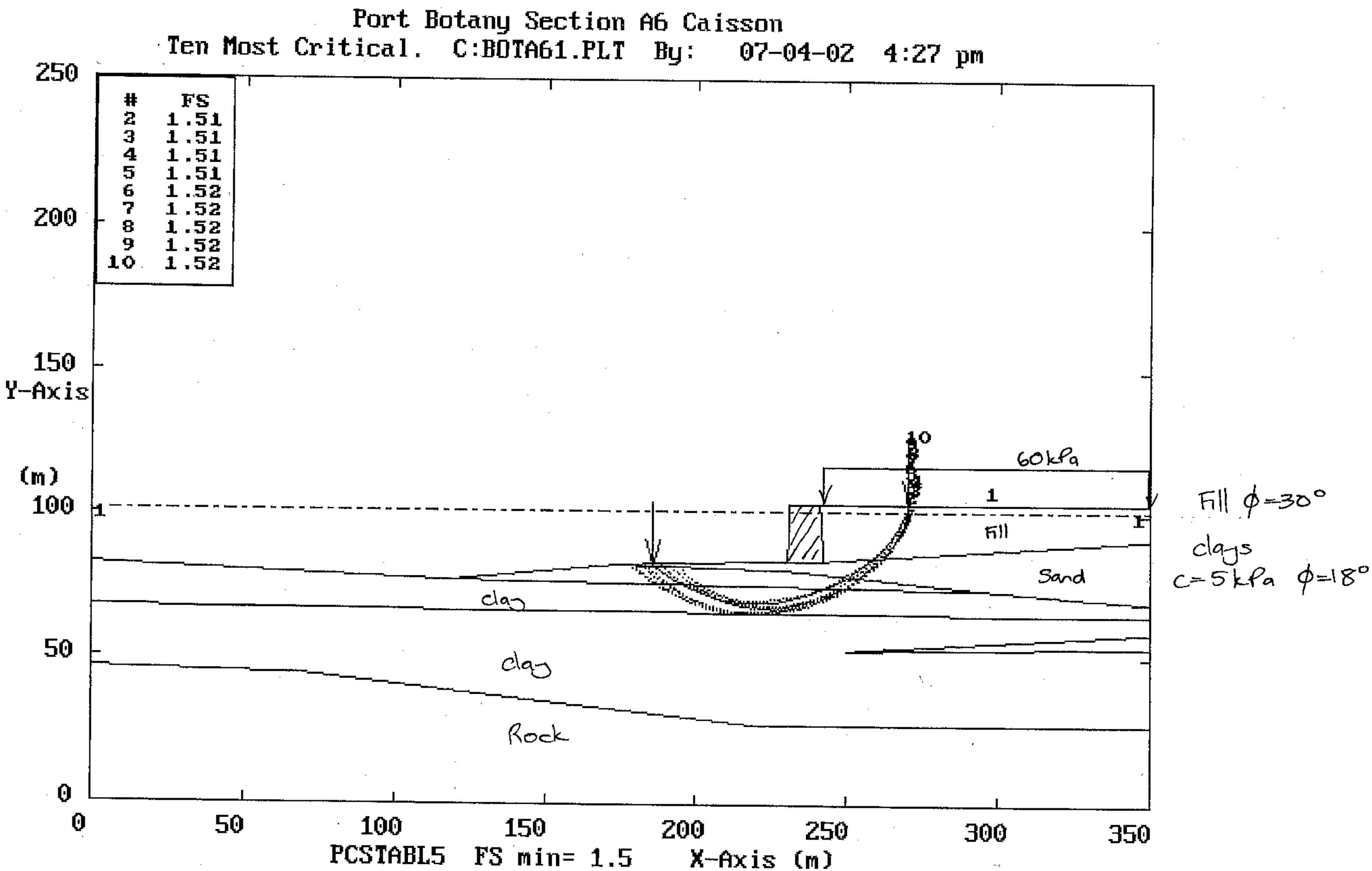
CASE: SECTION A6

CAISSON

DREDGING COMPLETED PREVIOUSLY
NO FURTHER DREDGING

Materials		Minimum Factor of Safety		
Clay	Sand Fill	Long term 60kPa	60kPa + Earthquake	
$c = 5kPa$ $\phi = 18°$	$\phi = 30°$	1.50	1.08	
	$\phi = 35°$	1.54	1.10	
$c = 10kPa$ $\phi = 25°$	$\phi = 30°$	1.62	1.23	
	$\phi = 35°$	1.79	1.35	

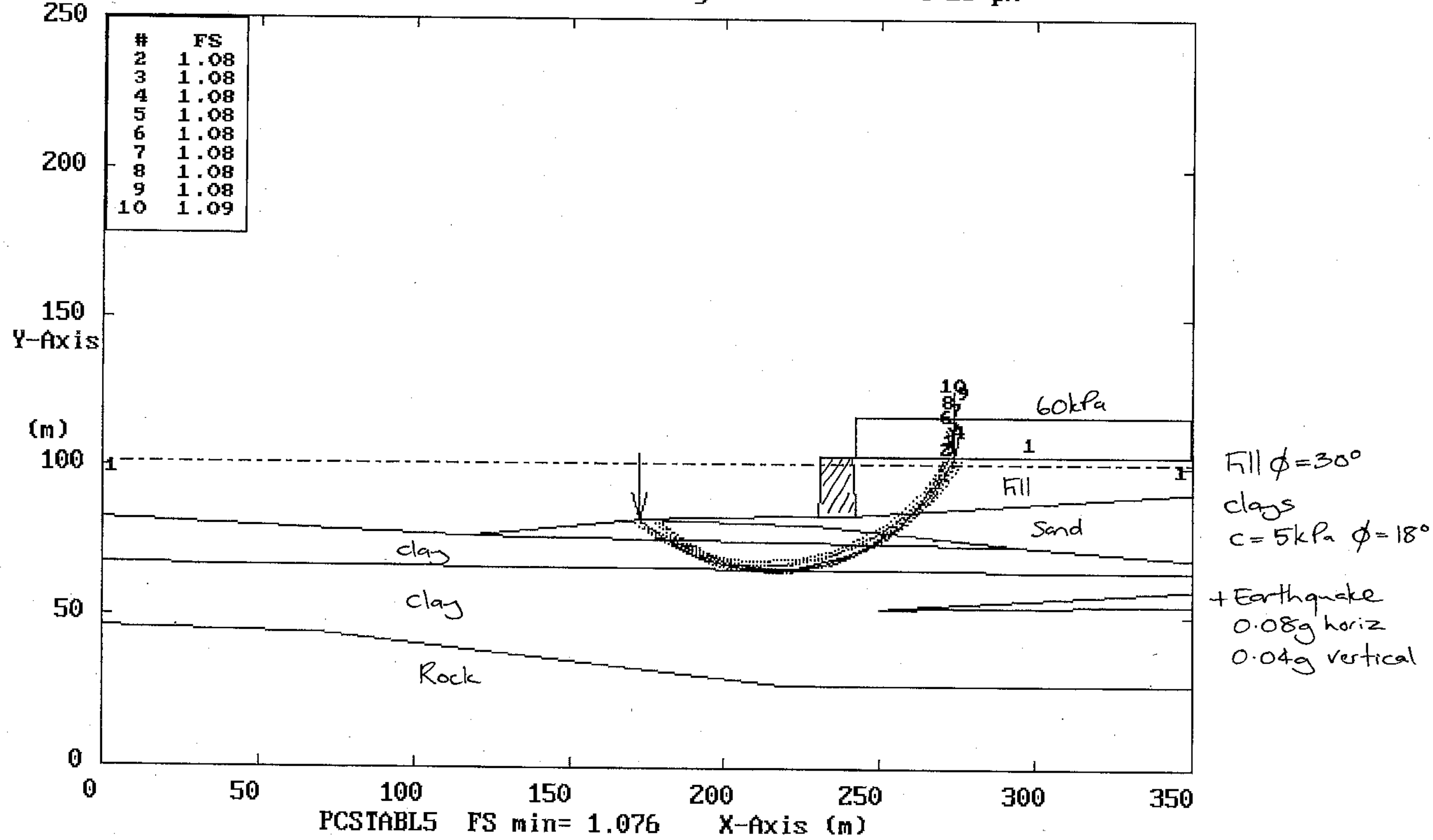
Comps By / /	Checked By / /	Approved By / /



Port Botany Section A6 Caisson
Ten Most Critical. C:BOTA61.PLT By: 07-04-02 4:27 pm
FS
2 1.51
3 1.51
4 1.51
5 1.51
6 1.52
7 1.52
8 1.52
9 1.52
10 1.52
250
200
150
Y-Axis
(m)
100
50
0
0
50
100
150
200
250
300
350
PCSTABL5 FS min= 1.5 X-Axis (m)
60kPa
Fill
Sand
clay
clay
Rock
Fill ϕ=30°
clays
c=5kPa ϕ=18°

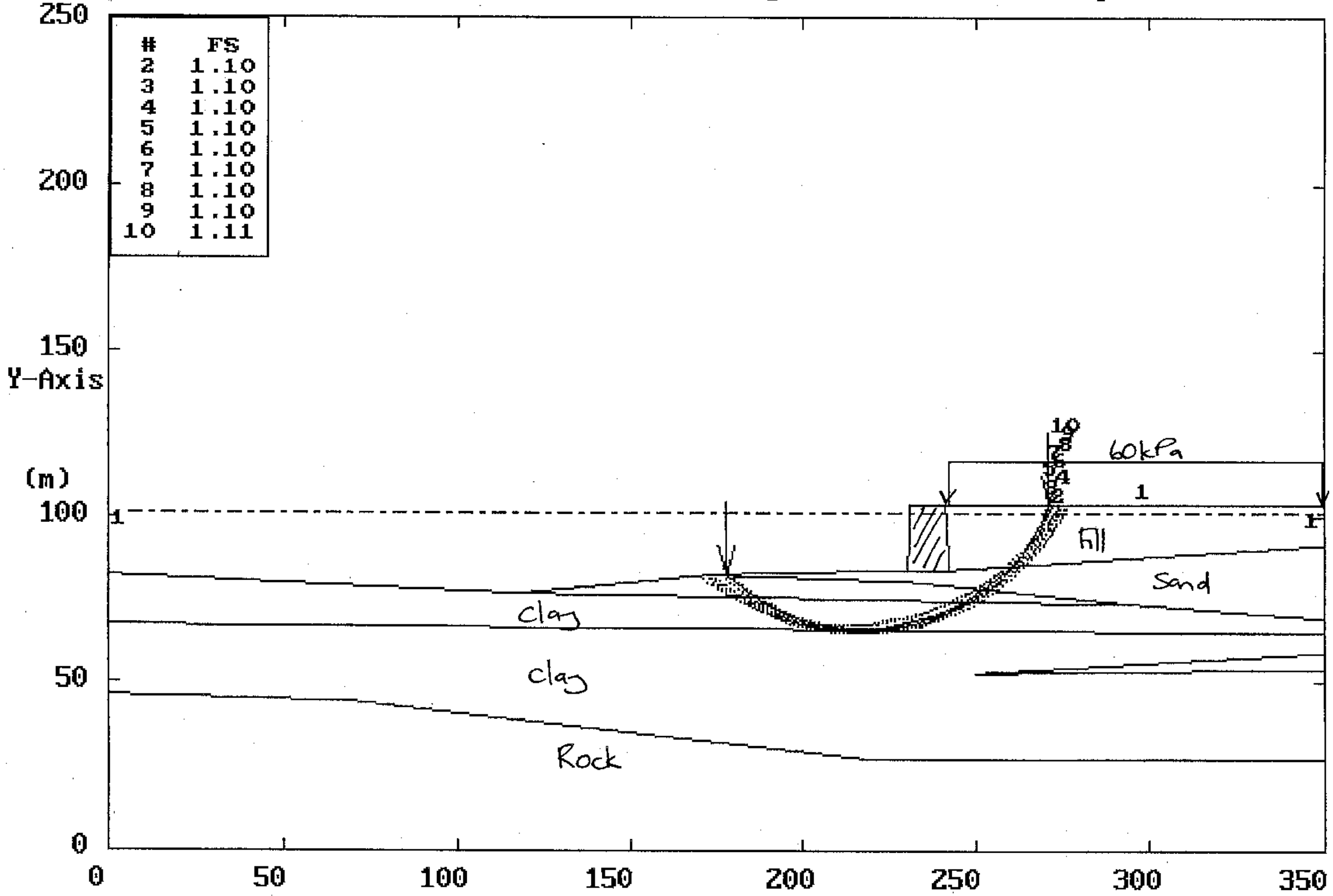
CAISSON

Port Botany Section A6 Caisson
Ten Most Critical. C:BOTA61.PLT By: 07-04-02 4:28 pm



CAISSON

Port Botany Section A6 Caisson
Ten Most Critical. C:BOTA61.PLT By: 07-05-02 5:42 pm



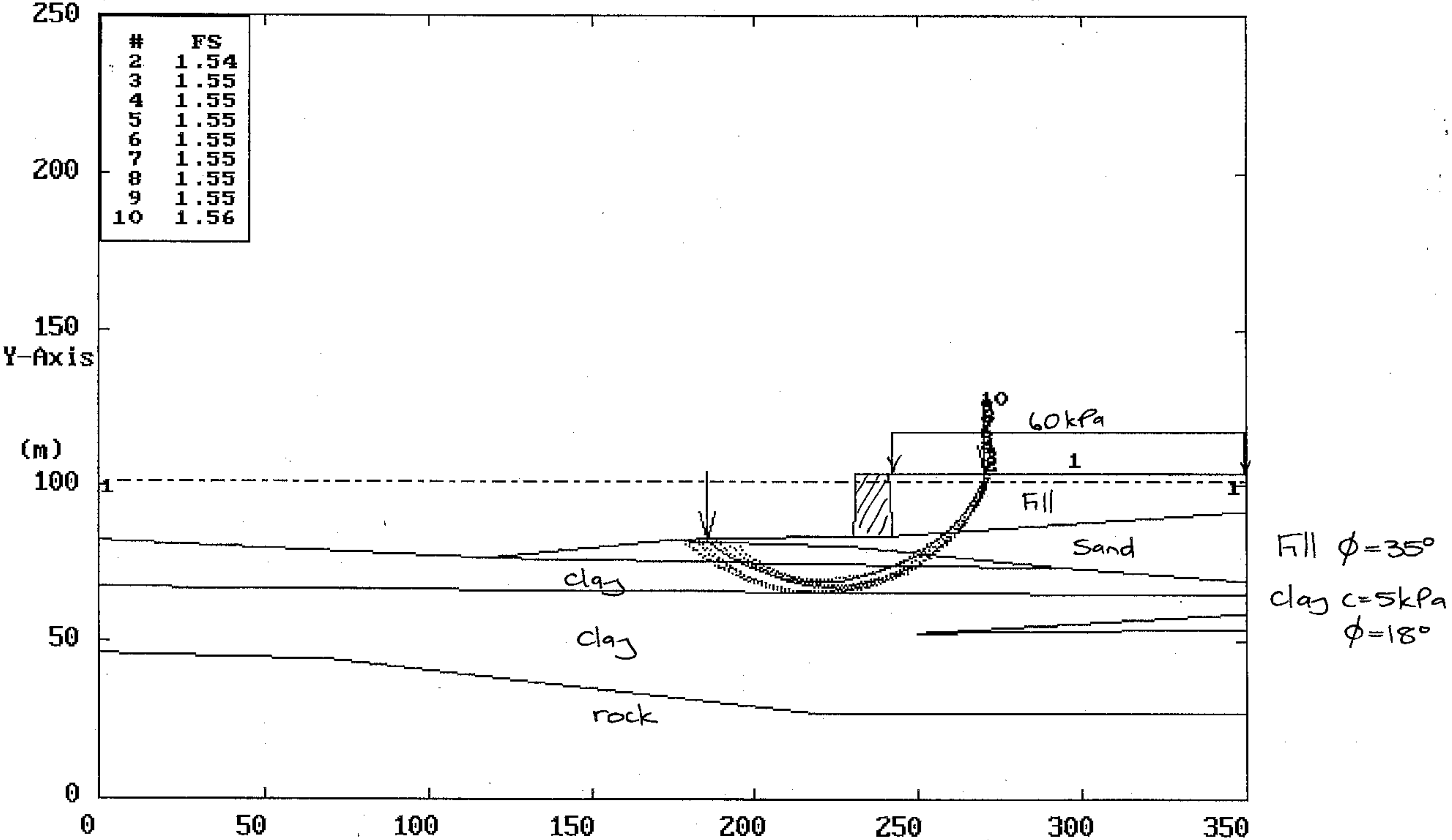
PCSTABL5 FS min= 1.095 X-Axis (m)

Fill $\phi = 35°$
Clay $c = 5kPa$
$\phi = 18°$

Plus Earthquake

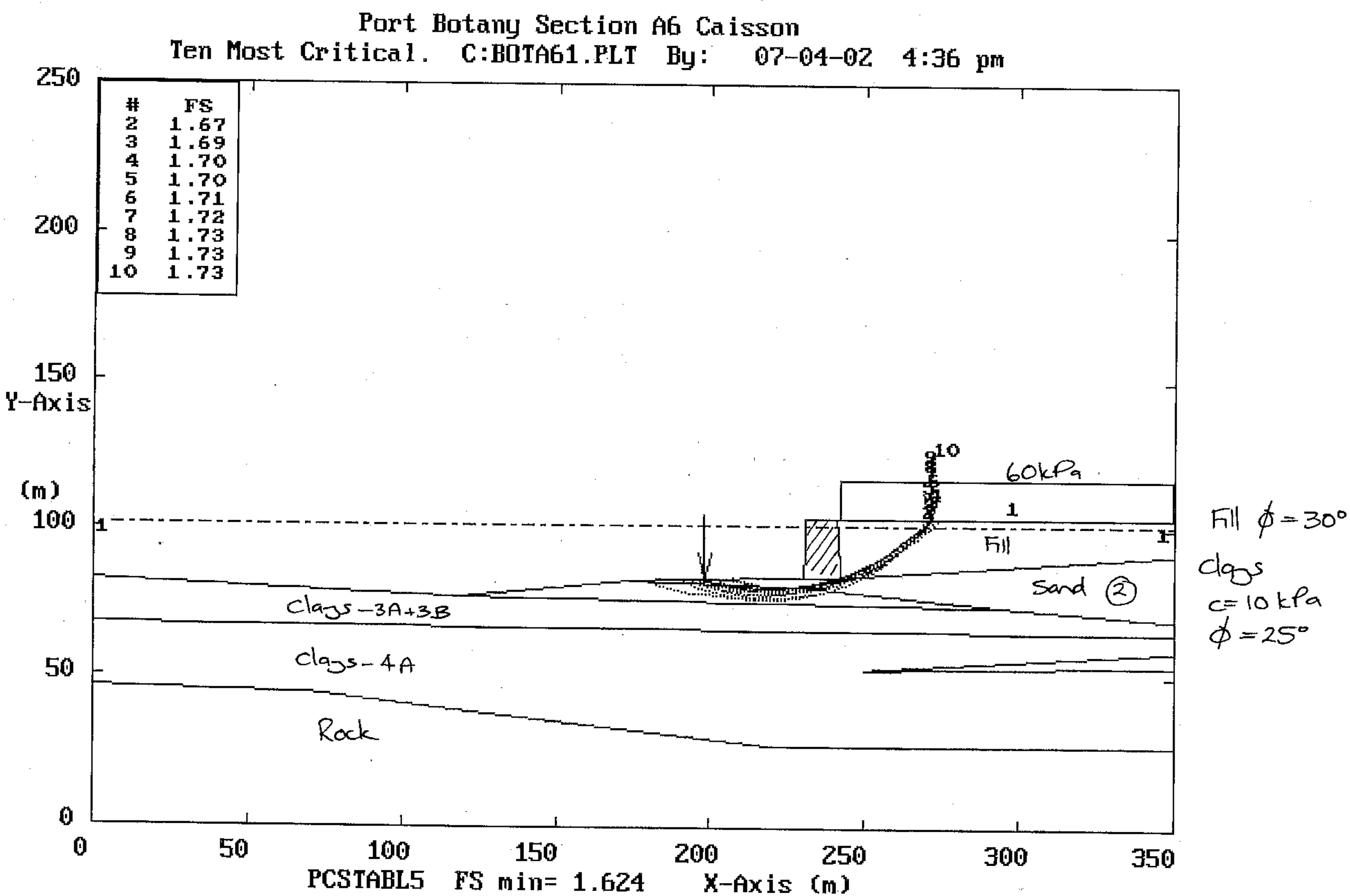
CAISSON

Port Botany Section A6 Caisson
Ten Most Critical. C:BOTA61.PLT By: 07-05-02 5:41 pm



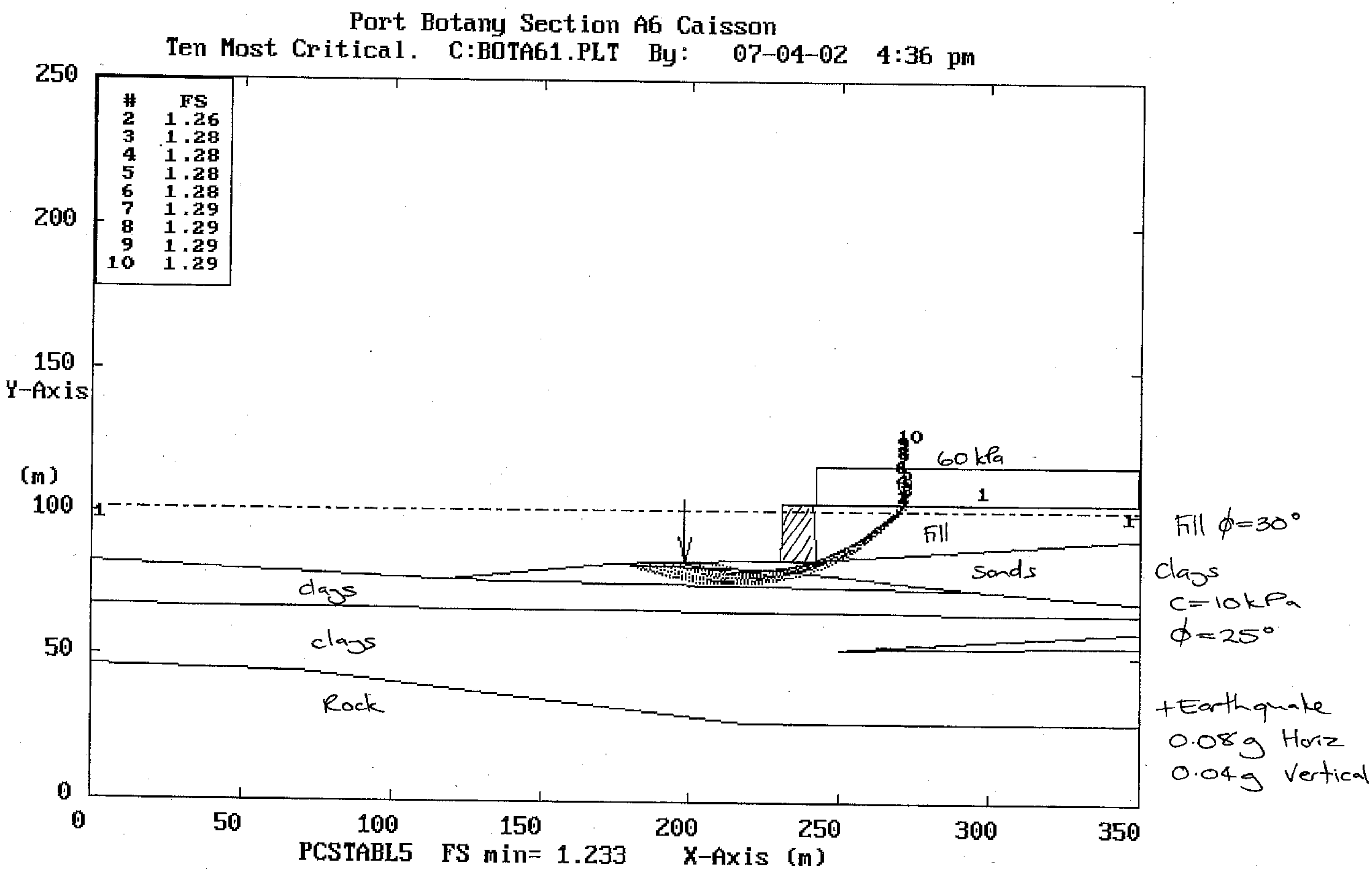
PCSTABL5 FS min= 1.54 X-Axis (m)

CAISSON

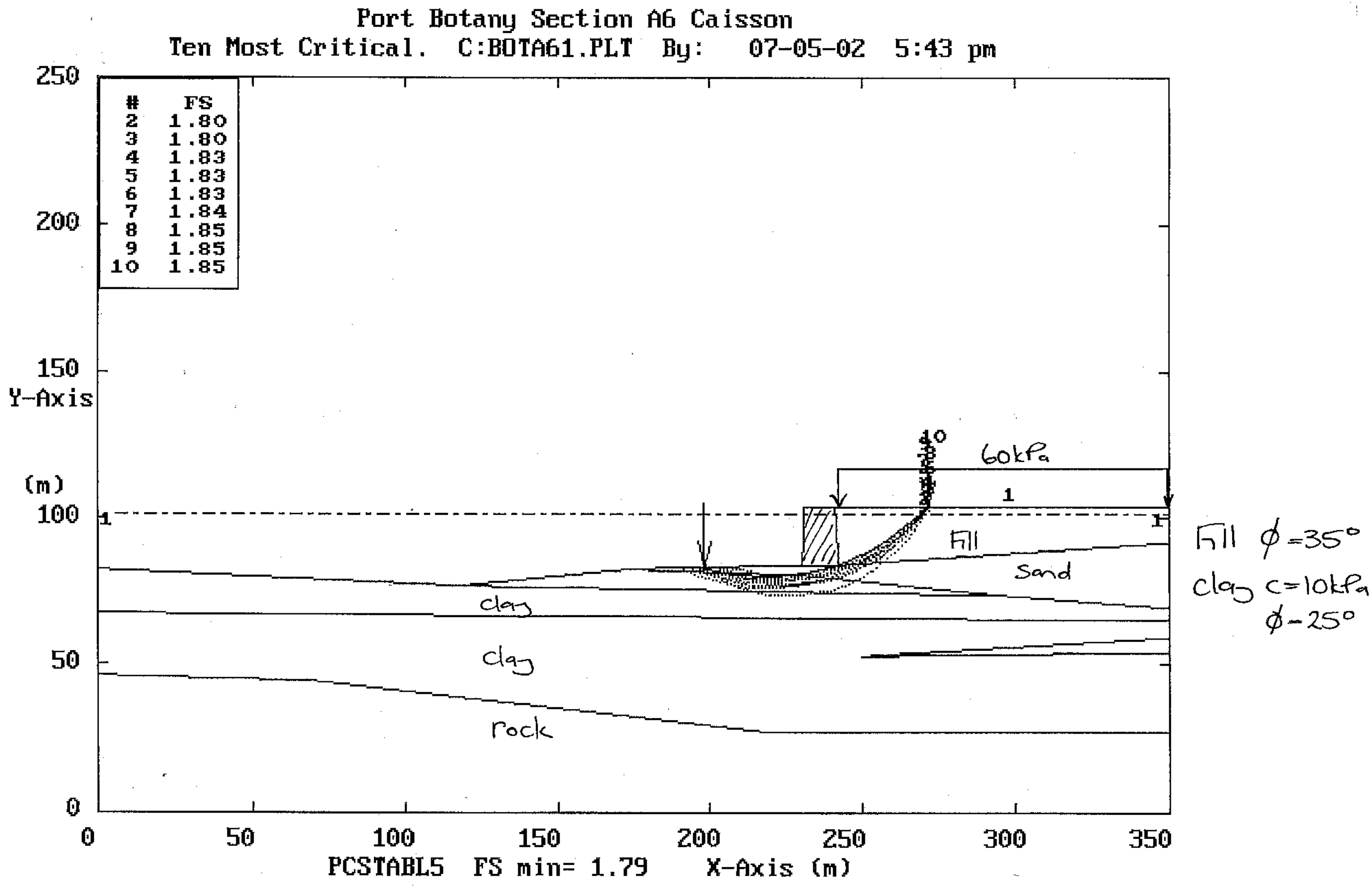


Port Botany Section A6 Caisson
Ten Most Critical. C:BOTA61.PLT By: 07-04-02 4:36 pm
FS
2 1.67
3 1.69
4 1.70
5 1.70
6 1.71
7 1.72
8 1.73
9 1.73
10 1.73
Y-Axis (m)
X-Axis (m)
PCSTABL5 FS min= 1.624
60kPa
Fill
Sand ②
Clays – 3A+3B
Clays – 4A
Rock
Fill $\phi = 30°$
Clays
c = 10 kPa
$\phi = 25°$

CASSON

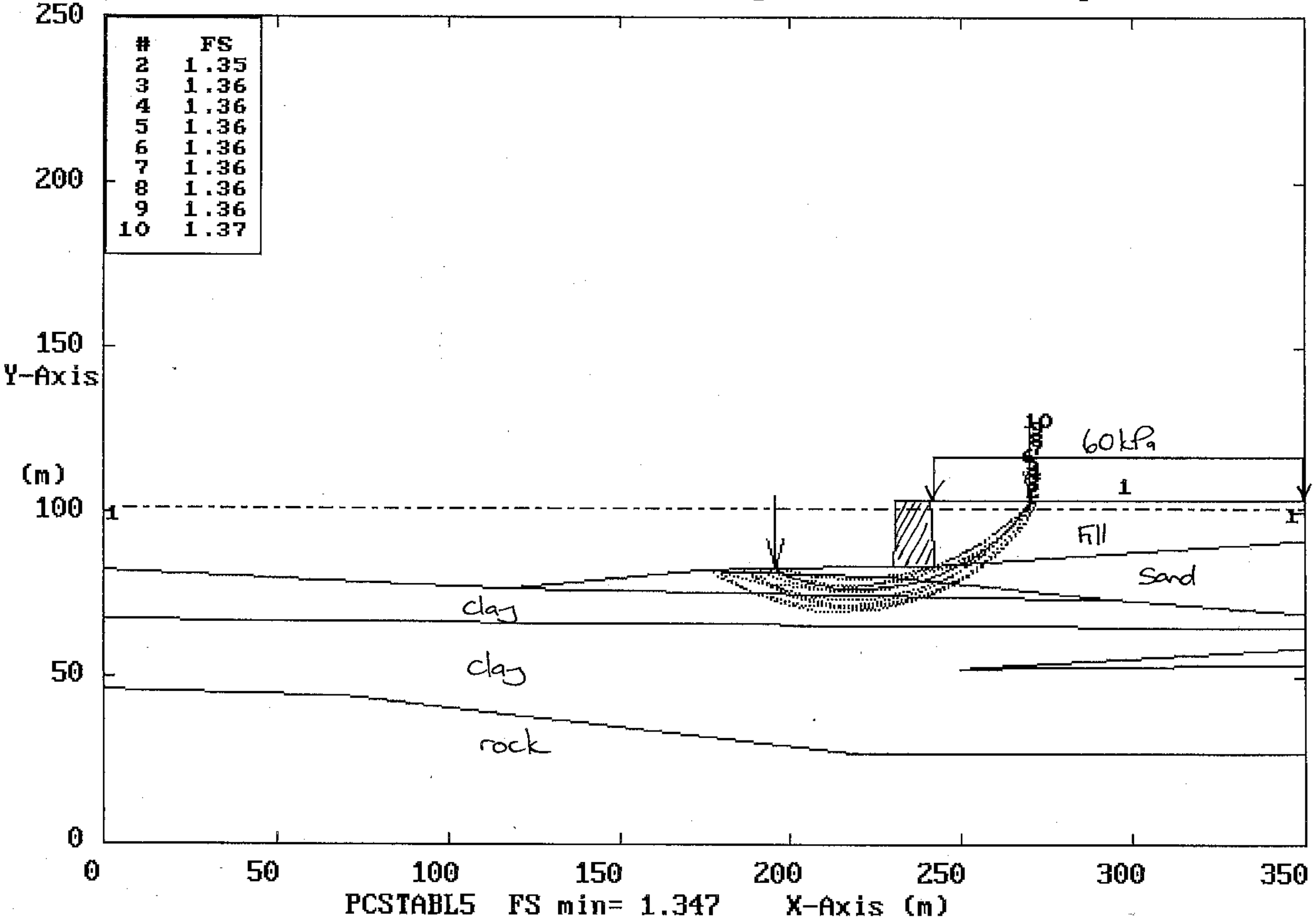

Port Botany Section A6 Caisson
Ten Most Critical. C:BOTA61.PLT By: 07-04-02 4:36 pm
FS
2 1.26
3 1.28
4 1.28
5 1.28
6 1.28
7 1.29
8 1.29
9 1.29
10 1.29
Y-Axis (m)
X-Axis (m)
PCSTABL5 FS min= 1.233
60 kPa
Fill
Sands
clays
clays
Rock
Fill ϕ=30°
Clays
C=10kPa
ϕ=25°
+Earthquake
0.08g Horiz
0.04g Vertical

CAISSON


Port Botany Section A6 Caisson
Ten Most Critical. C:BOTA61.PLT By: 07-05-02 5:43 pm
FS
2 1.80
3 1.80
4 1.83
5 1.83
6 1.83
7 1.84
8 1.85
9 1.85
10 1.85
Y-Axis (m)
250
200
150
100
50
0
0
50
100
150
200
250
300
350
PCSTABL5 FS min= 1.79
X-Axis (m)
10
60kPa
1
Fill
Sand
clay
clay
rock
Fill ϕ =35°
clay c=10kPa
ϕ =25°

CAISSON

Port Botany Section A6 Caisson
Ten Most Critical. C:BOTA61.PLT By: 07-05-02 5:43 pm



Fill $\phi=35°$
clay $c=10kPa$
$\phi=25°$

Plus Earthquake

CAISSON

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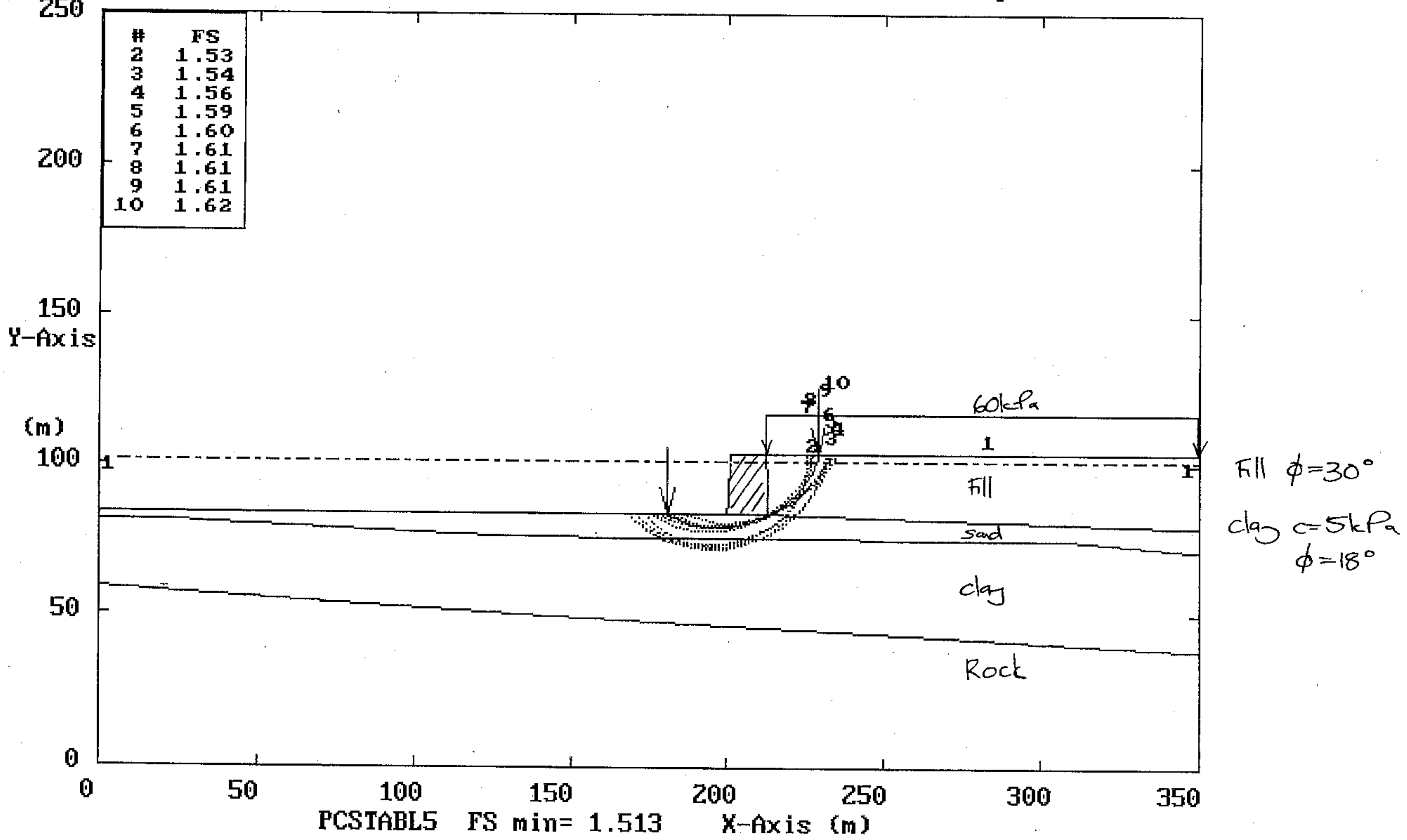
CASE: SECTION B4
CAISSON
DREDGE TO RL-16

Materials		Minimum Factors of Safety		
Clay	Sand Fill	Longterm 60kPa	60kPa + Earthquake	
c=5kPa ϕ=18°	ϕ=30°	1.51	1.22	
	ϕ=35°	1.60	1.25	

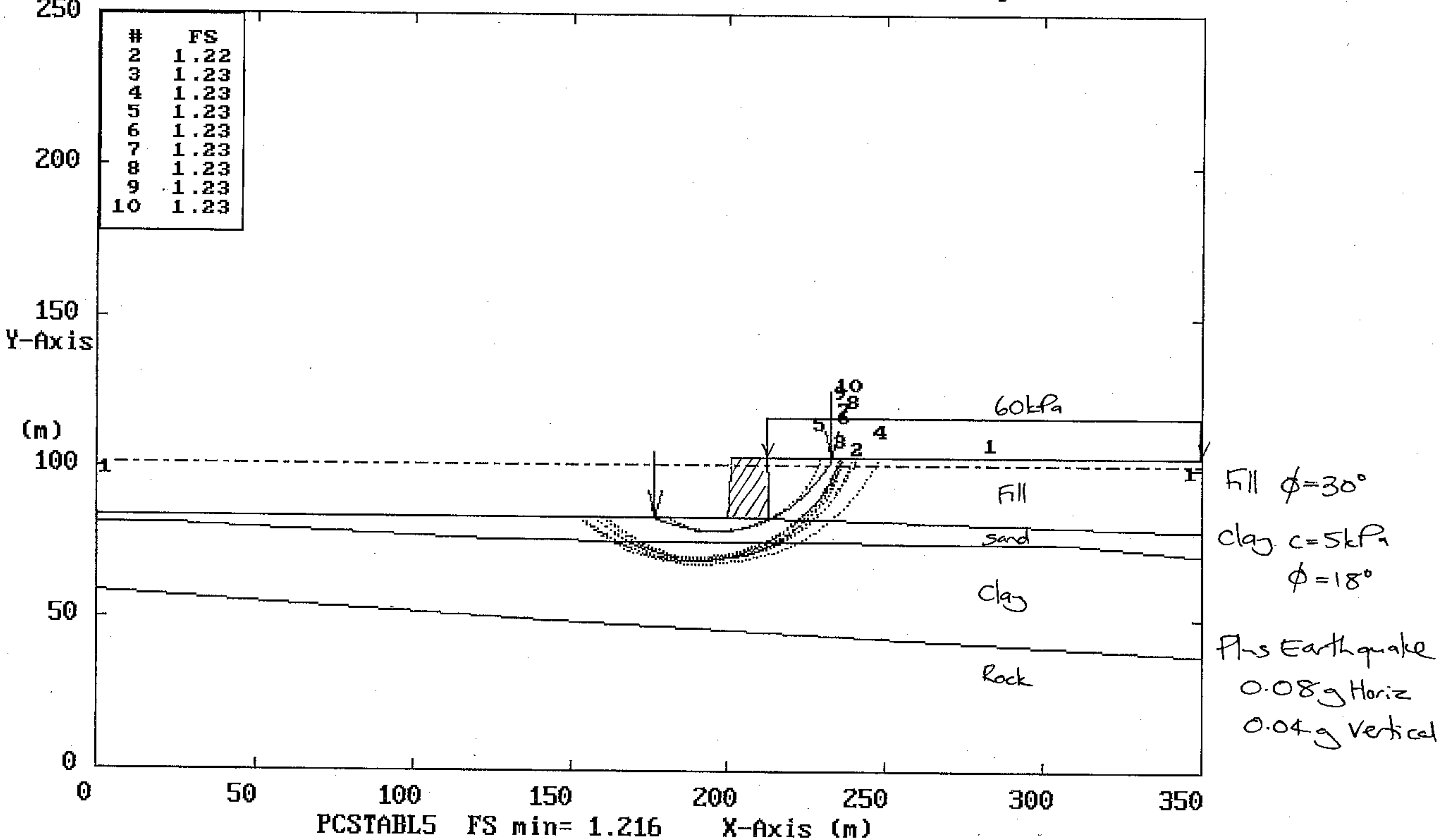
Comps By / / Checked By / / Approved By / /

Port Botany Section B4 Caisson
Ten Most Critical. C:BOTB41.PLT By: 07-04-02 5:58 pm

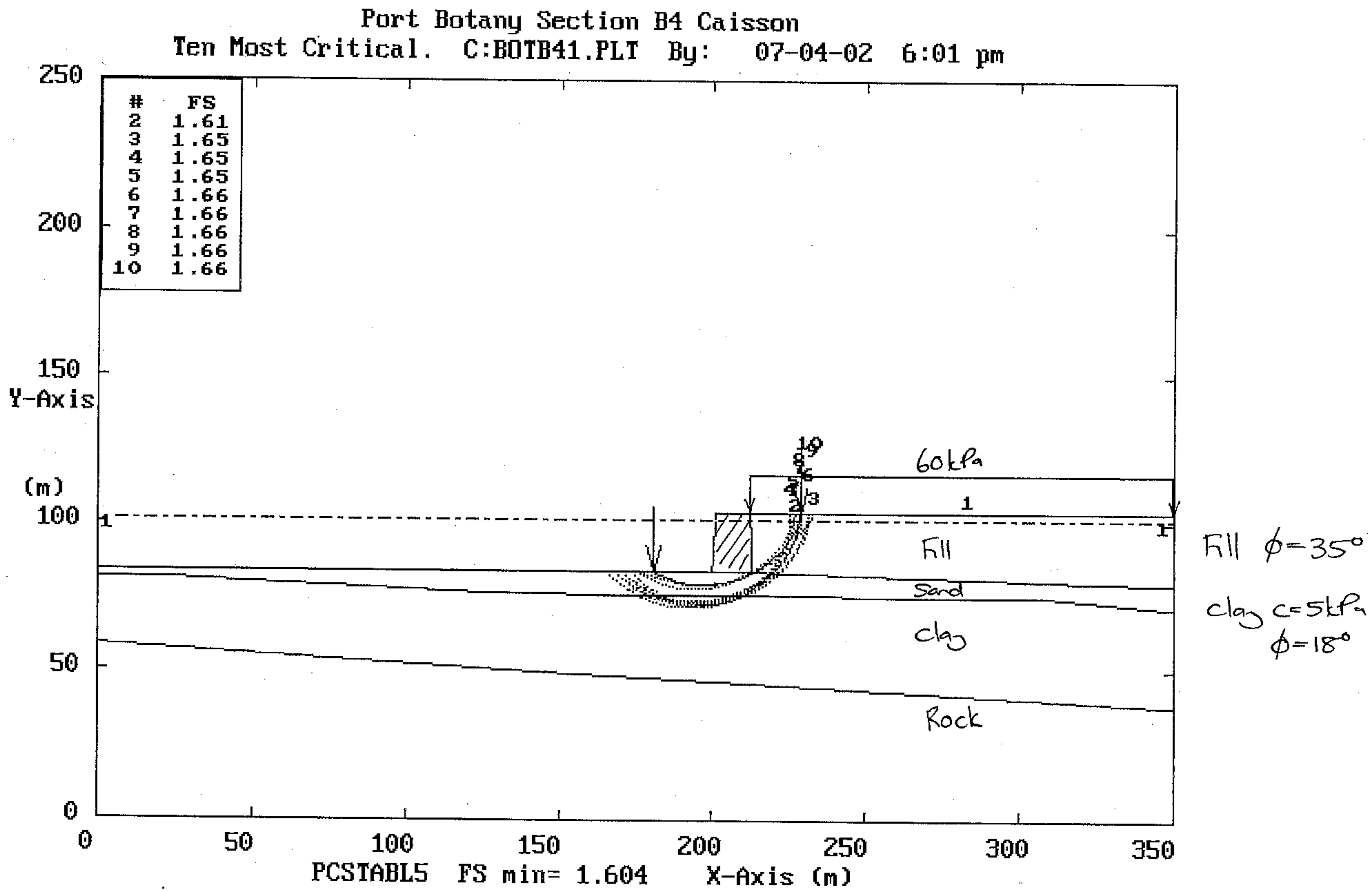


Caisson

Port Botany Section B4 Caisson
Ten Most Critical. C:BOTB41.PLT By: 07-04-02 5:59 pm



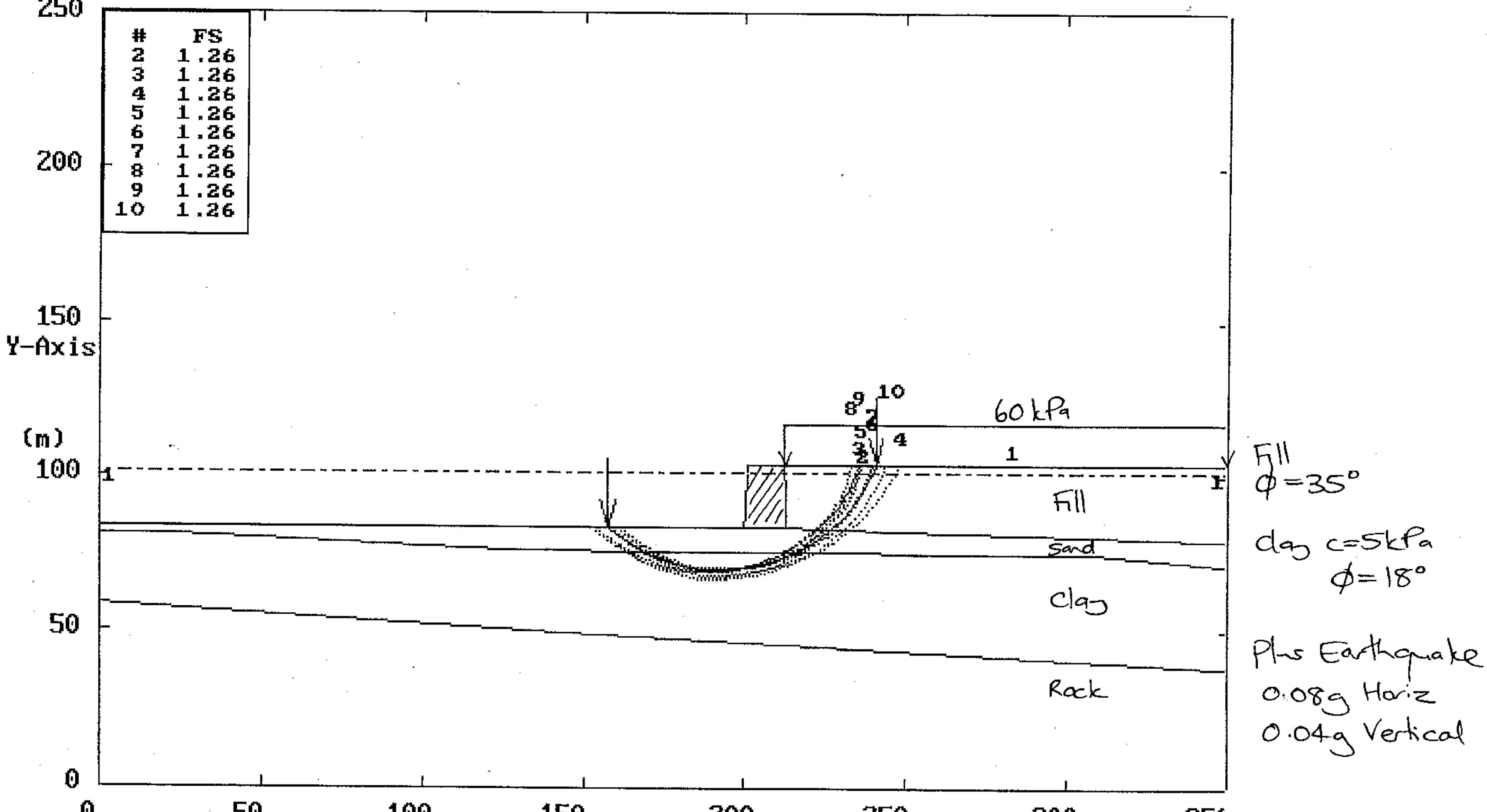
Caisson



Port Botany Section B4 Caisson
Ten Most Critical. C:BOTB41.PLT By: 07-04-02 6:01 pm
FS
2 1.61
3 1.65
4 1.65
5 1.65
6 1.66
7 1.66
8 1.66
9 1.66
10 1.66
Y-Axis (m)
250
200
150
100
50
0
0
50
100
150
200
250
300
350
PCSTABL5 FS min= 1.604 X-Axis (m)
60kPa
Fill
Sand
Clay
Rock
Fill φ=35°
clay c=5kPa φ=18°

Caisson

Port Botany Section B4 Caisson
Ten Most Critical. C:BOTB41.PLT By: 07-04-02 6:01 pm



PCSTABL5 FS min= 1.253 X-Axis (m)

Caisson

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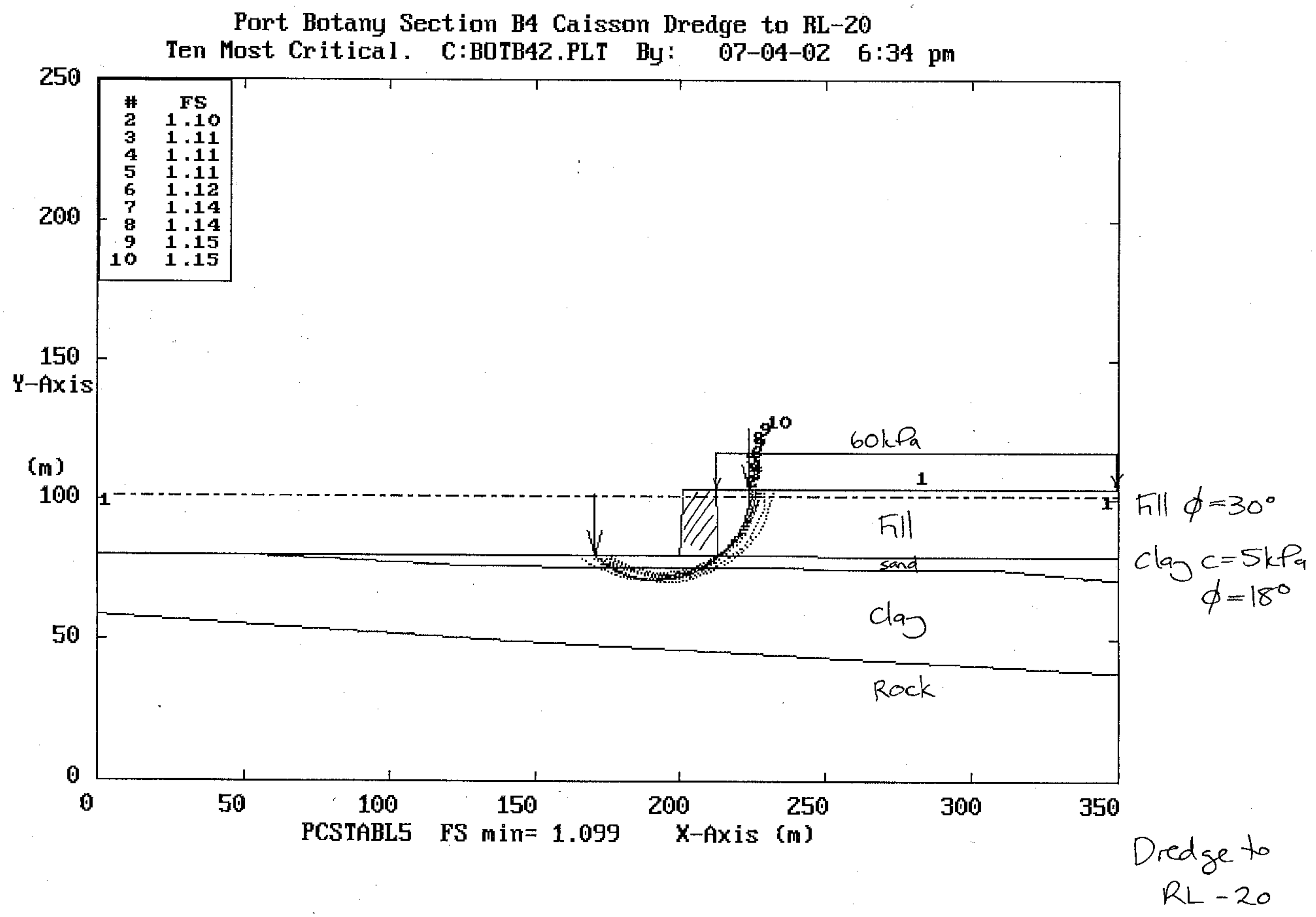
CASE : SECTION B4

CAISSON

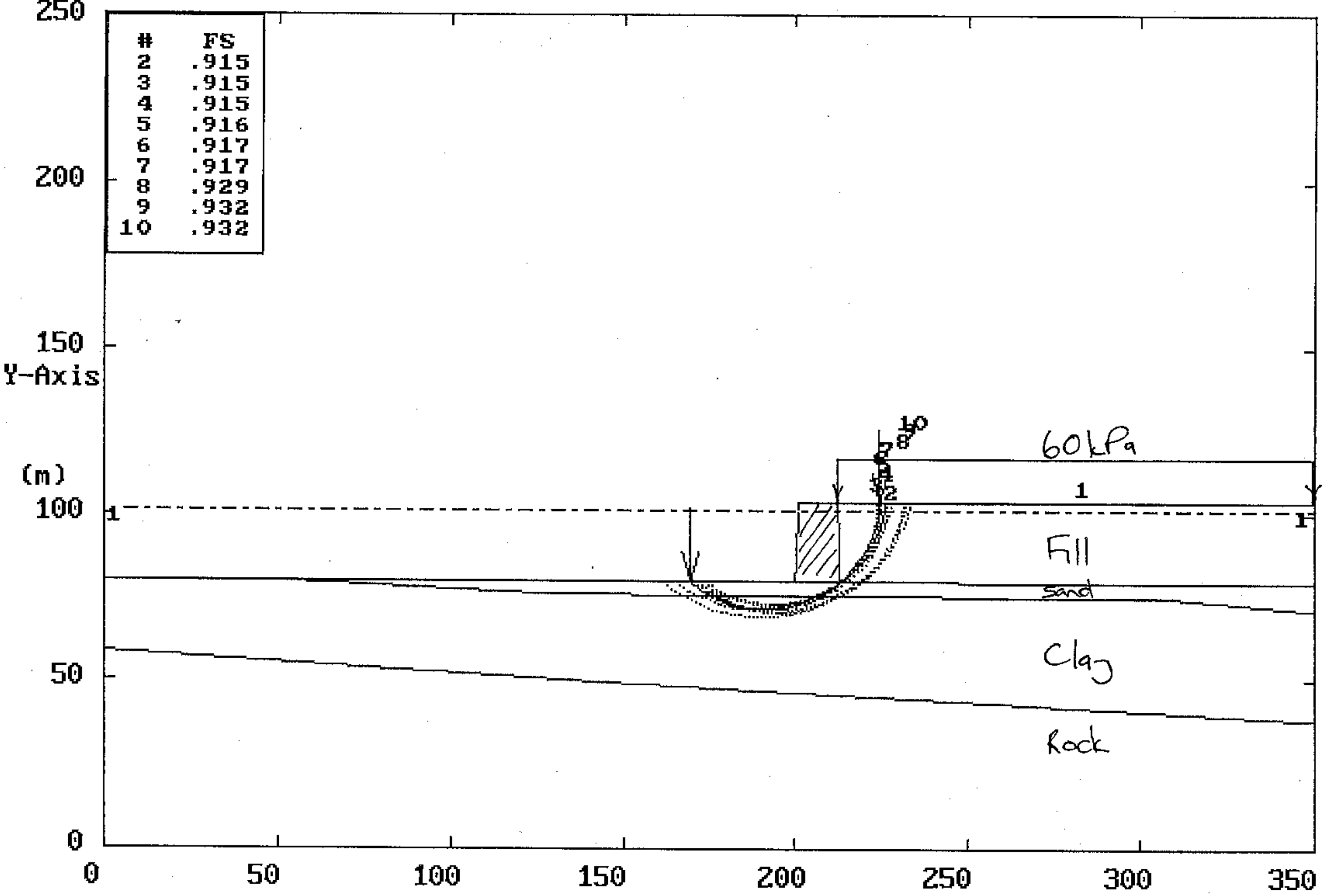
DREDGE TO RL -20m (next to Caisson)

Materials		Minimum Factors of Safety		
Clays	Sand Fill	Longterm 60kPa	60kPa + Earthquake	
C=5kPa $\phi=18°$	$\phi=30°$	1.10	0.91	
	$\phi=35°$	1.14	0.94	
C=10kPa $\phi=25°$	$\phi=30°$	1.27	1.05	
	$\phi=35°$	1.34	1.10	

Comps By / / | Checked By / / | Approved By / /


Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB42.PLT By: 07-04-02 6:34 pm
FS
2 1.10
3 1.11
4 1.11
5 1.11
6 1.12
7 1.14
8 1.14
9 1.15
10 1.15
Y-Axis (m)
60kPa
Fill
sand
Clay
Rock
Fill ϕ=30°
Clay c=5kPa
ϕ=18°
PCSTABL5 FS min= 1.099 X-Axis (m)
Dredge to RL-20

Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB42.PLT By: 07-04-02 6:33 pm



PCSTABL5 FS min= 0.908

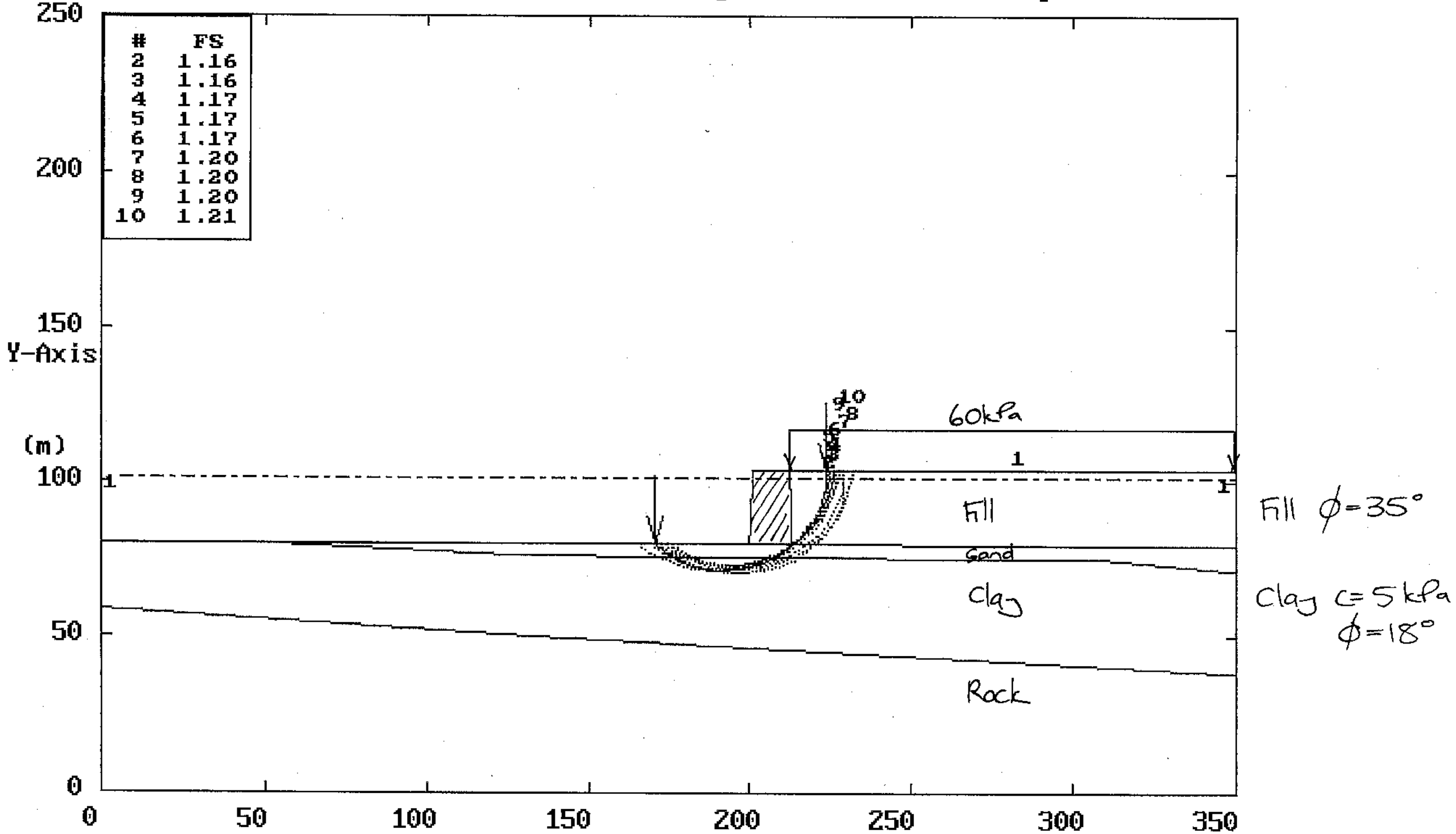
Fill $\phi = 30°$

Clay $c = 5kPa$
$\phi = 18°$

Plus Earthquake

Dredge to RL-20

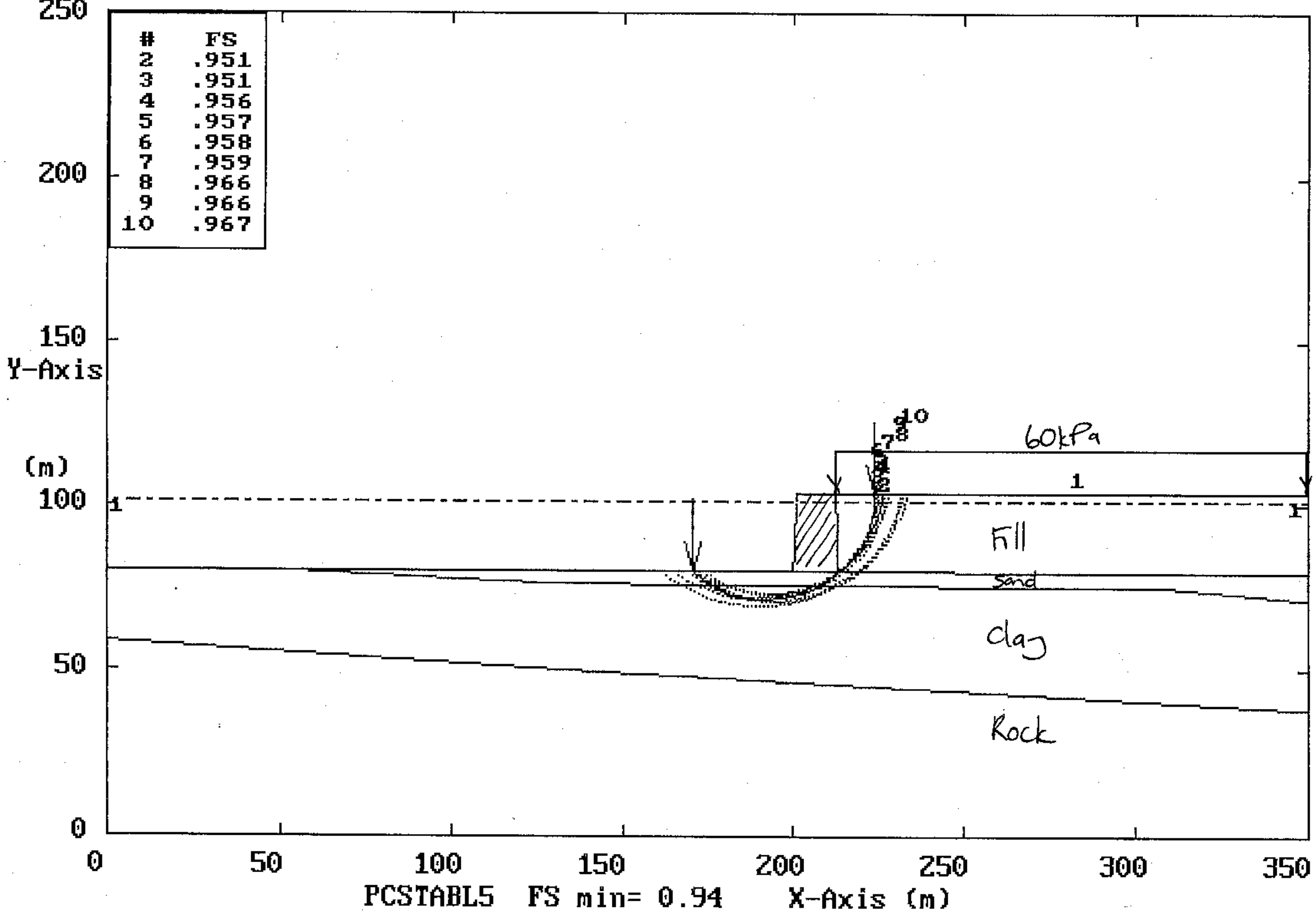
Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB42.PLT By: 07-05-02 12:25 pm



PCSTABL5 FS min= 1.143 X-Axis (m)

Dredged to RL-20

Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB42.PLT By: 07-05-02 12:26 pm

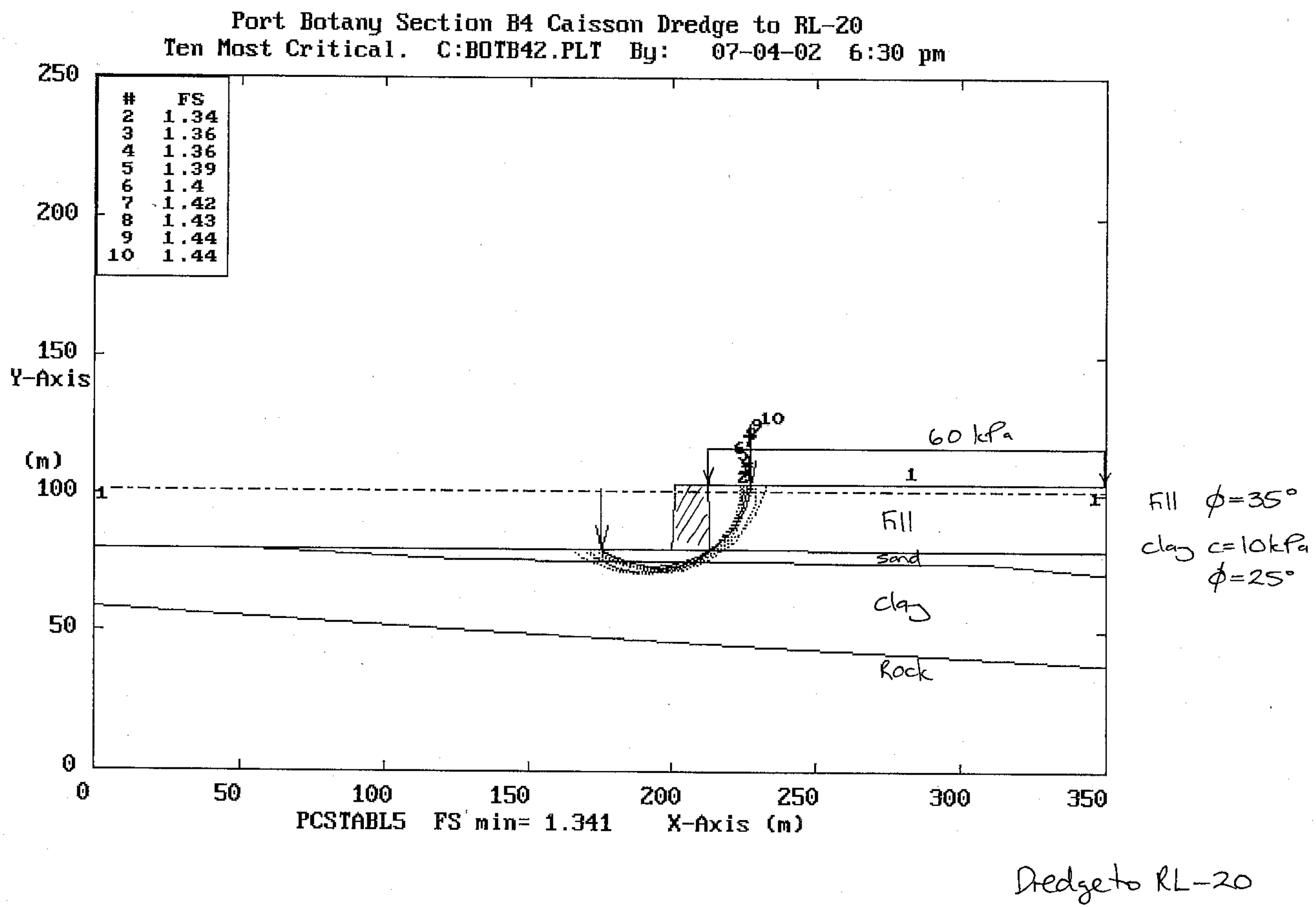


Fill $\phi = 35°$

clay $c = 5kPa$
$\phi = 18°$

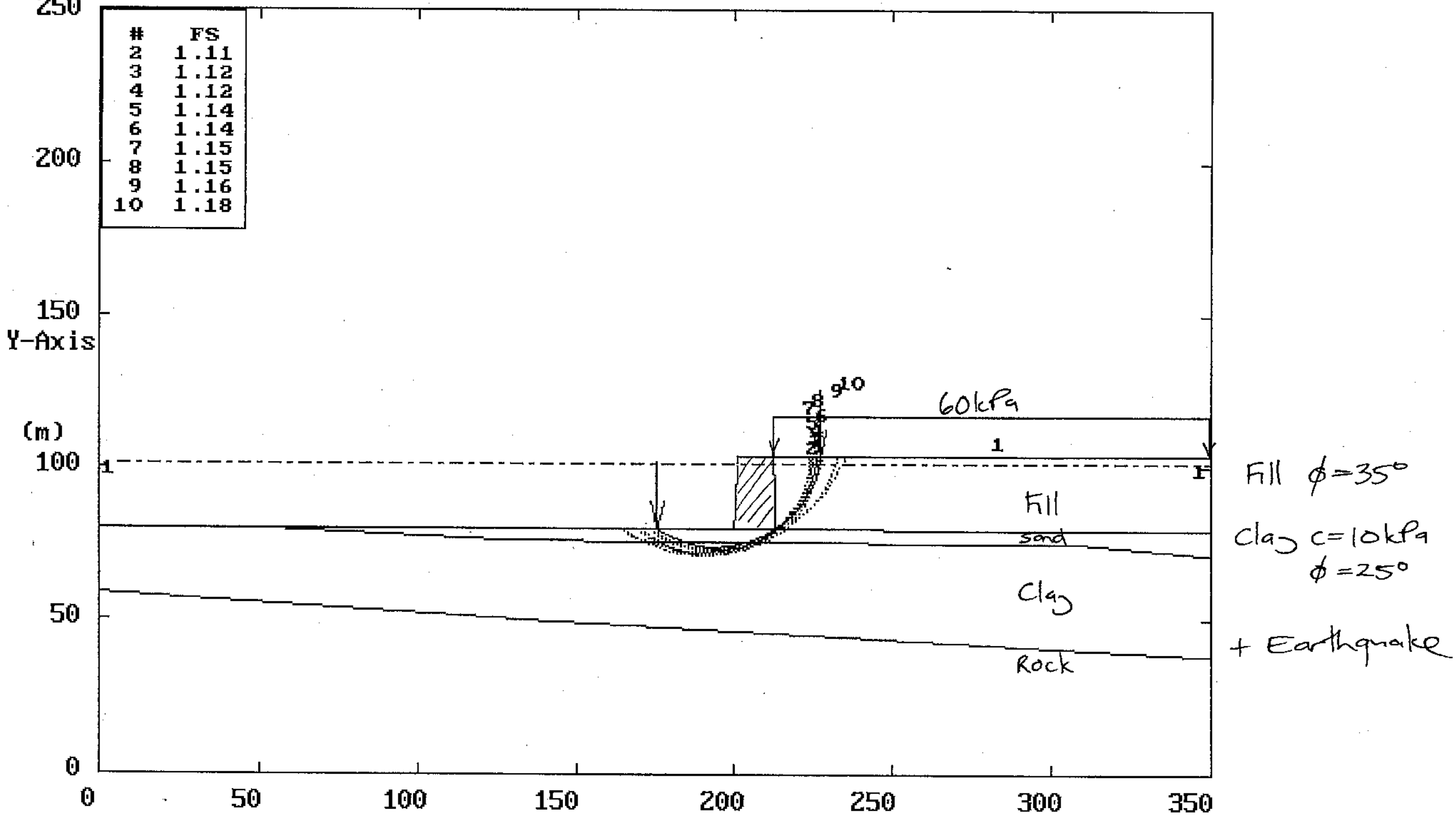
Plus Earthquake.

Dredged to RL-20



Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB42.PLT By: 07-04-02 6:30 pm
FS
2 1.34
3 1.36
4 1.36
5 1.39
6 1.4
7 1.42
8 1.43
9 1.44
10 1.44
Y-Axis (m)
60 kPa
Fill
sand
clay
Rock
Fill $\phi=35°$
clay c=10kPa $\phi=25°$
PCSTABL5 FS min= 1.341 X-Axis (m)
Dredge to RL-20

Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB42.PLT By: 07-04-02 6:31 pm



PCSTABL5 FS min= 1.104 X-Axis (m)

Dredge to RL-20

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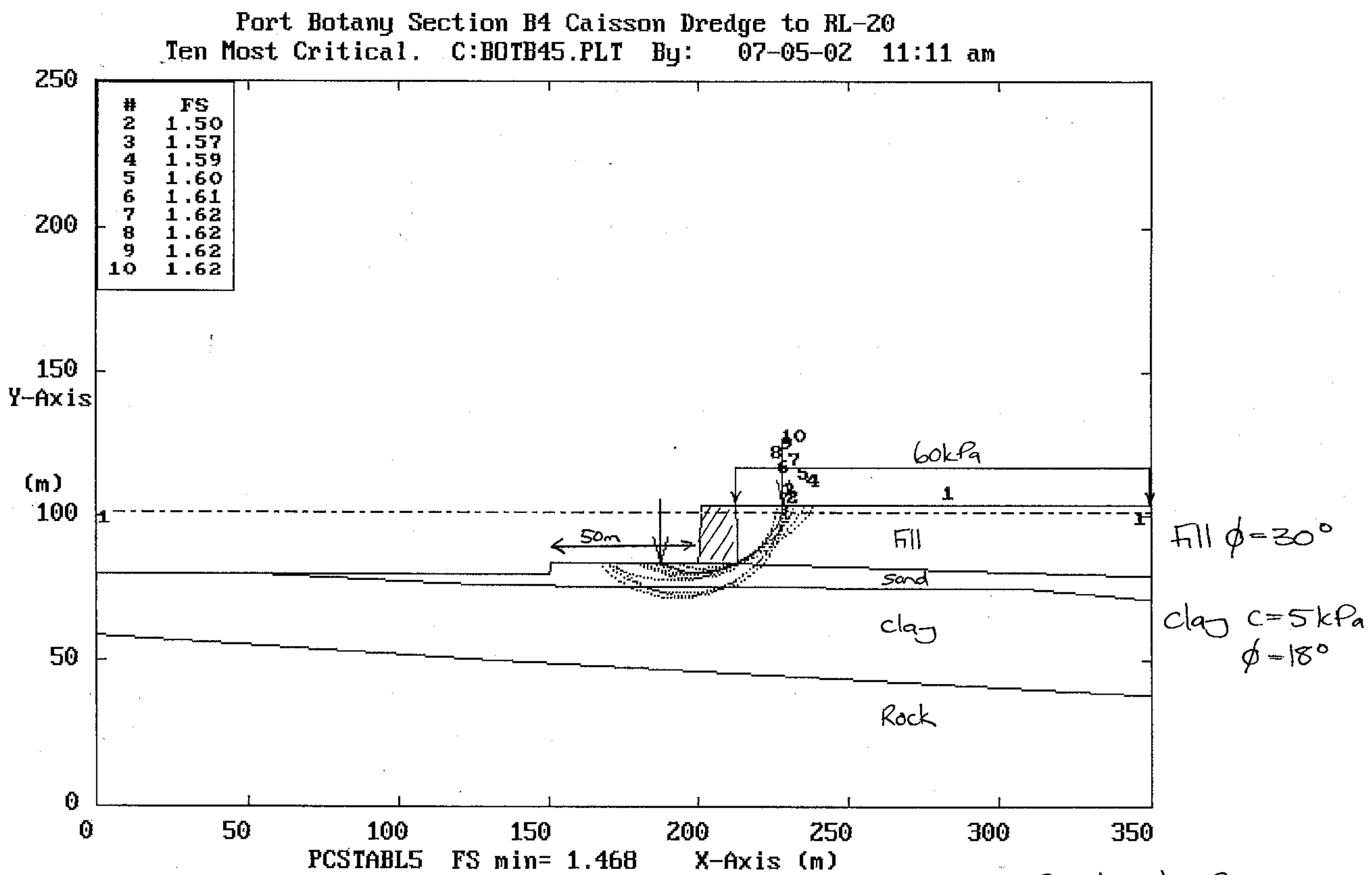
CASE: SECTION B4

CAISSON

DREDGE TO RL-20m – 50m from Caisson

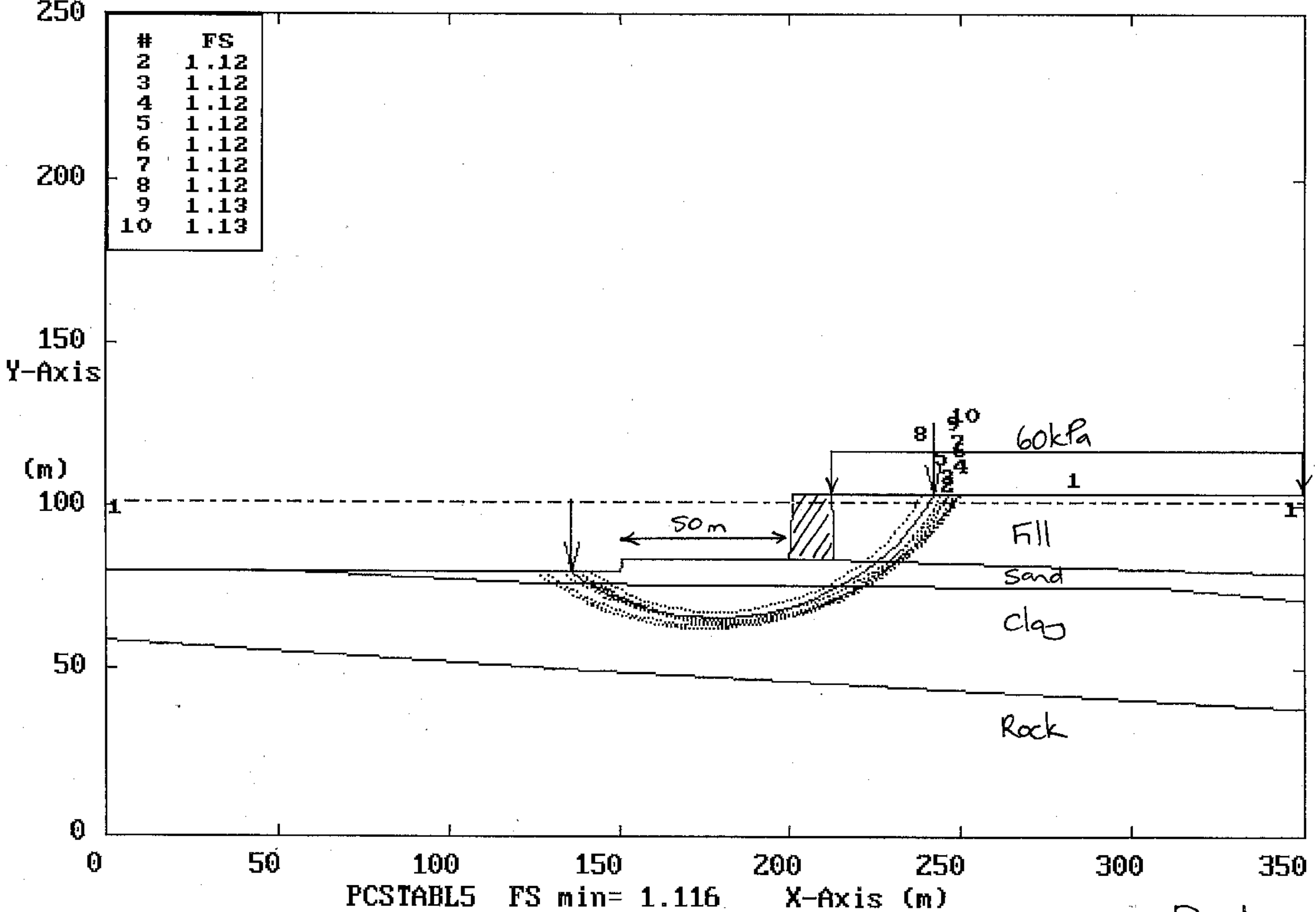
Materials.		Minimum Factors of Safety		
Clays	Sand Fill	Longterm 60kPa	60kPa + Earthquake	
c=5kPa ϕ=18°	ϕ=30°	1.47	1.12	
	ϕ=35°	1.58	1.14	
c=10kPa ϕ=25°	ϕ=30°	1.47	1.20	
	ϕ=35°	1.58	1.28	

Comps By / / Checked By / / Approved By / /



Dredge to RL-20
50 m from caisson

Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB45.PLT By: 07-05-02 11:12 am



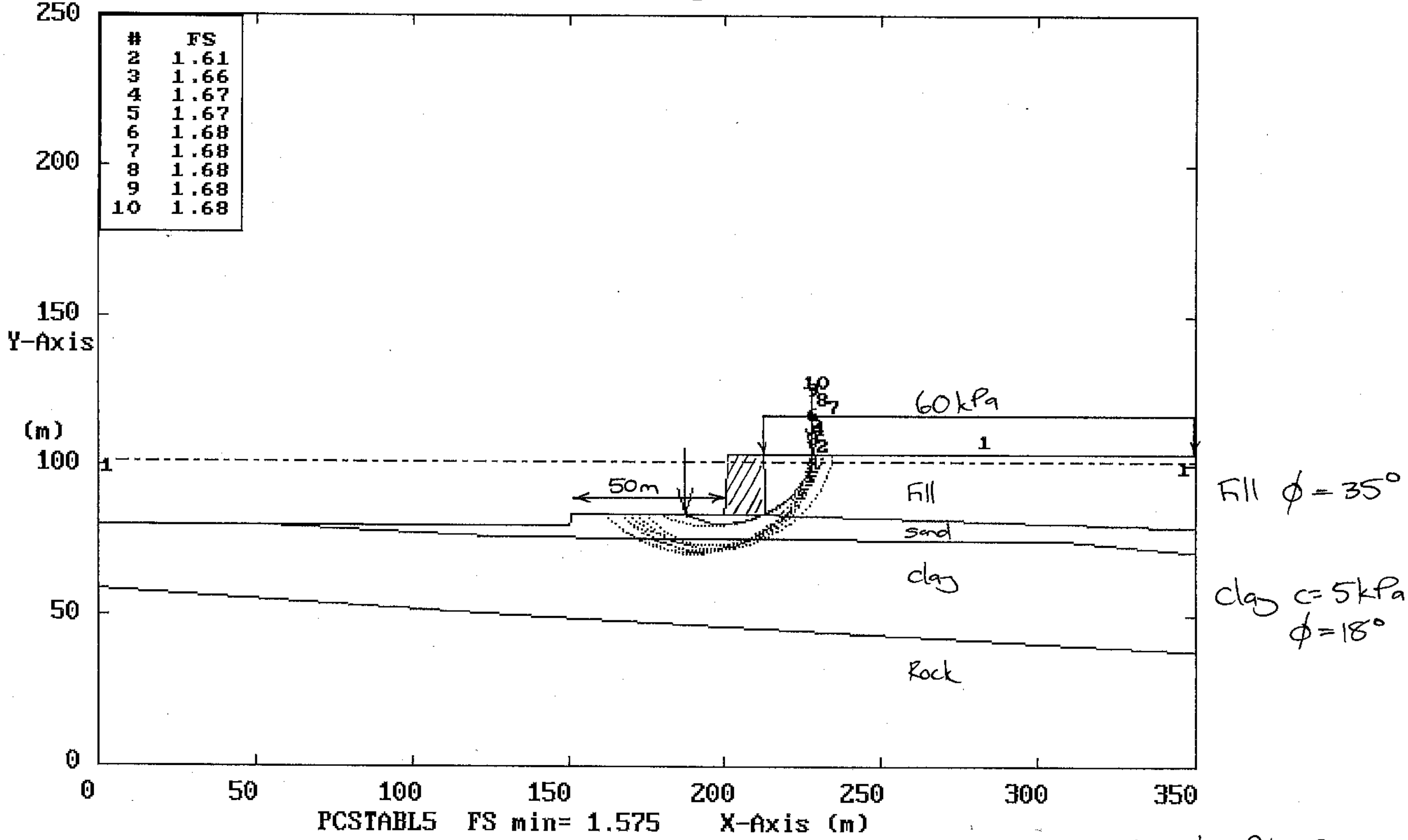
Fill $\phi = 30°$

clay $c = 5kPa$
$\phi = 18°$

Plus Earthquake

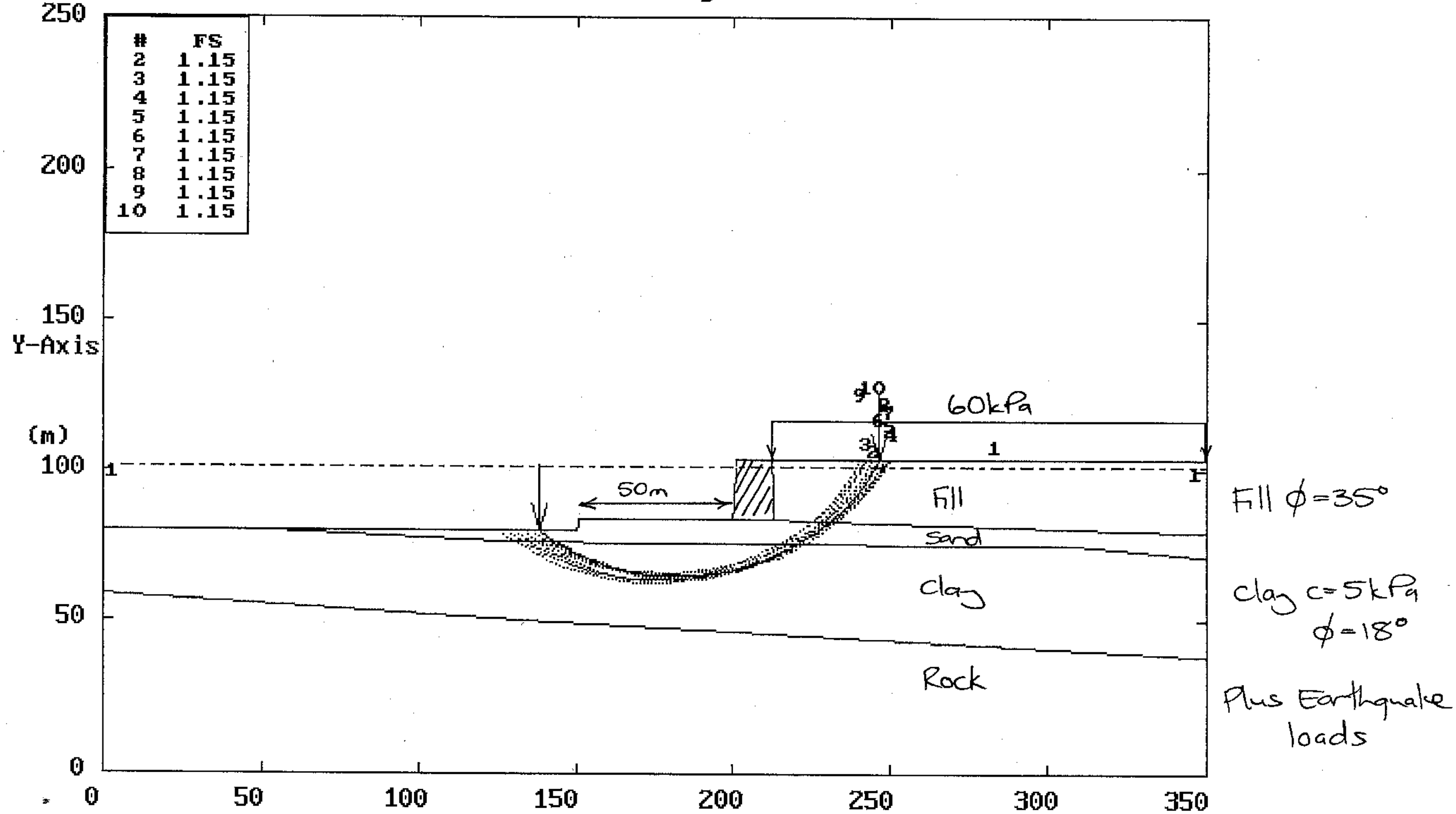
Dredge to RL-20
50m from caisson

Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB45.PLT By: 07-05-02 11:13 am



Dredge to RL-20
50m from caisson

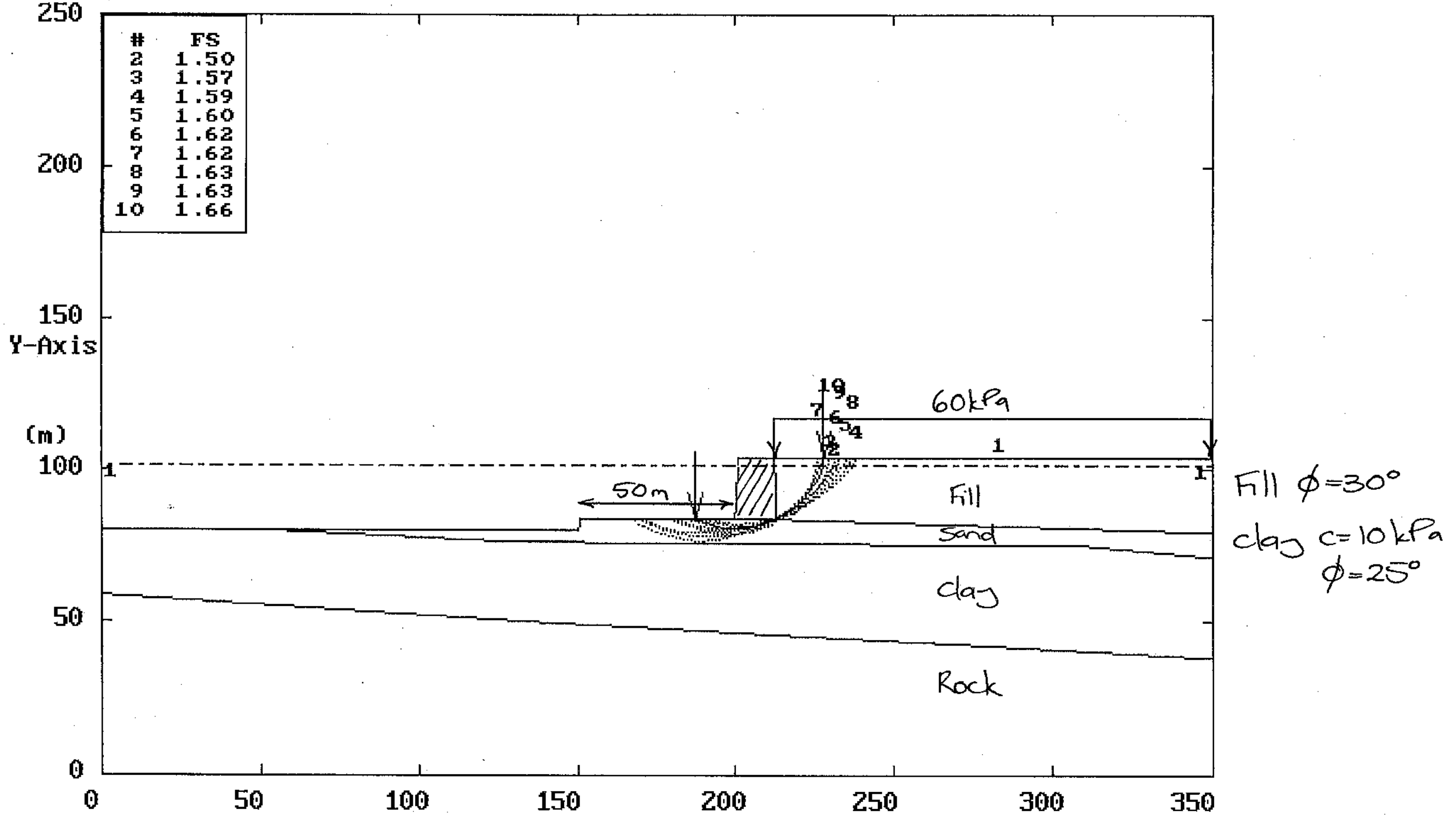
Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB45.PLT By: 07-05-02 11:13 am



PCSTABL5 FS min= 1.144 X-Axis (m)

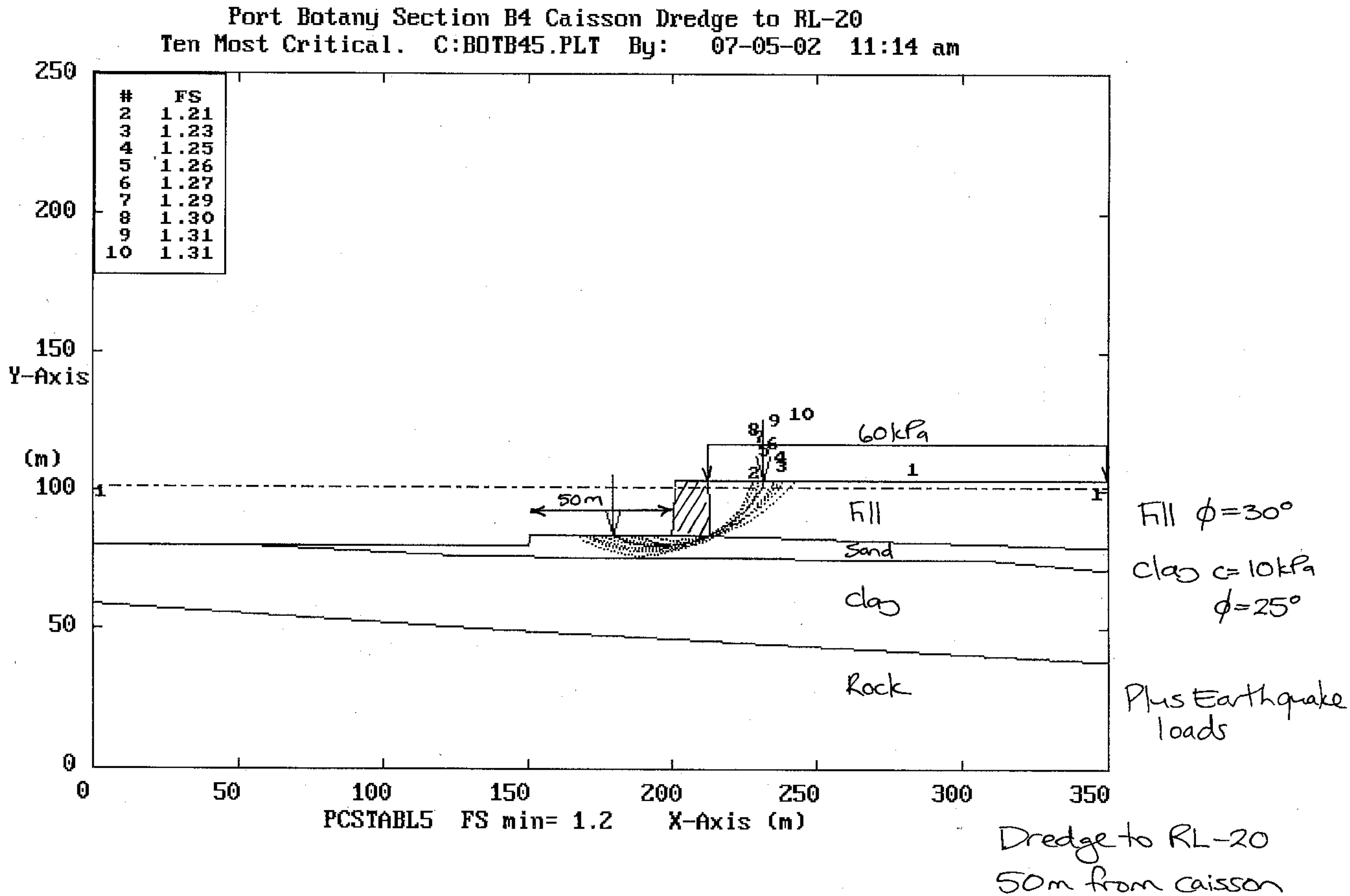
Dredge to RL-20
50m from caisson

Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB45.PLT By: 07-05-02 11:14 am

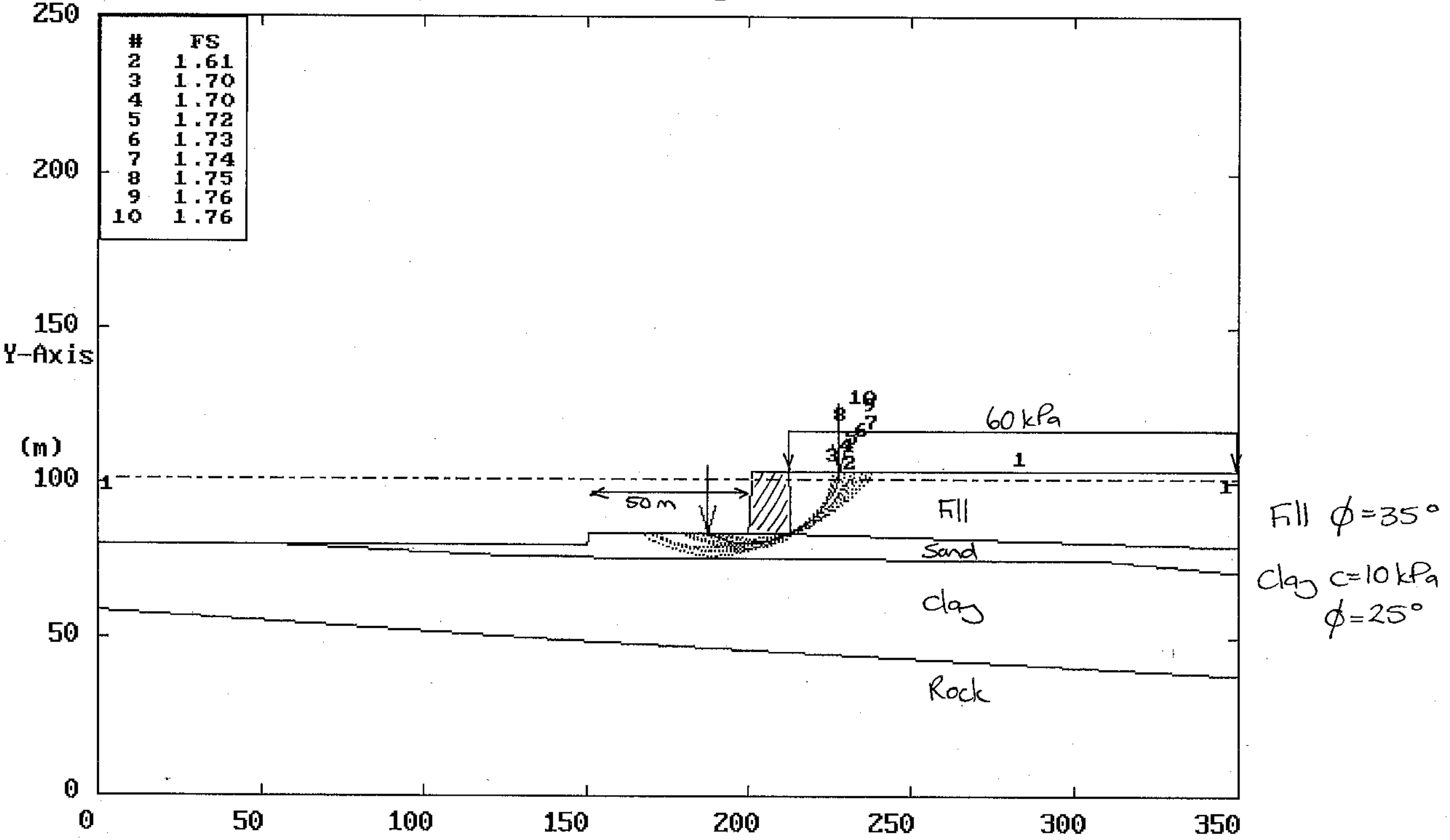


PCSTABL5 FS min= 1.468 X-Axis (m)

Dredge to RL-20
50m from caisson


Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB45.PLT By: 07-05-02 11:14 am
FS
2 1.21
3 1.23
4 1.25
5 1.26
6 1.27
7 1.29
8 1.30
9 1.31
10 1.31
Y-Axis (m)
X-Axis (m)
PCSTABL5 FS min= 1.2
60kPa
50m
Fill
Sand
clay
Rock
Fill $\phi=30°$
clay c= 10kPa
$\phi=25°$
Plus Earthquake loads
Dredge to RL-20
50m from caisson

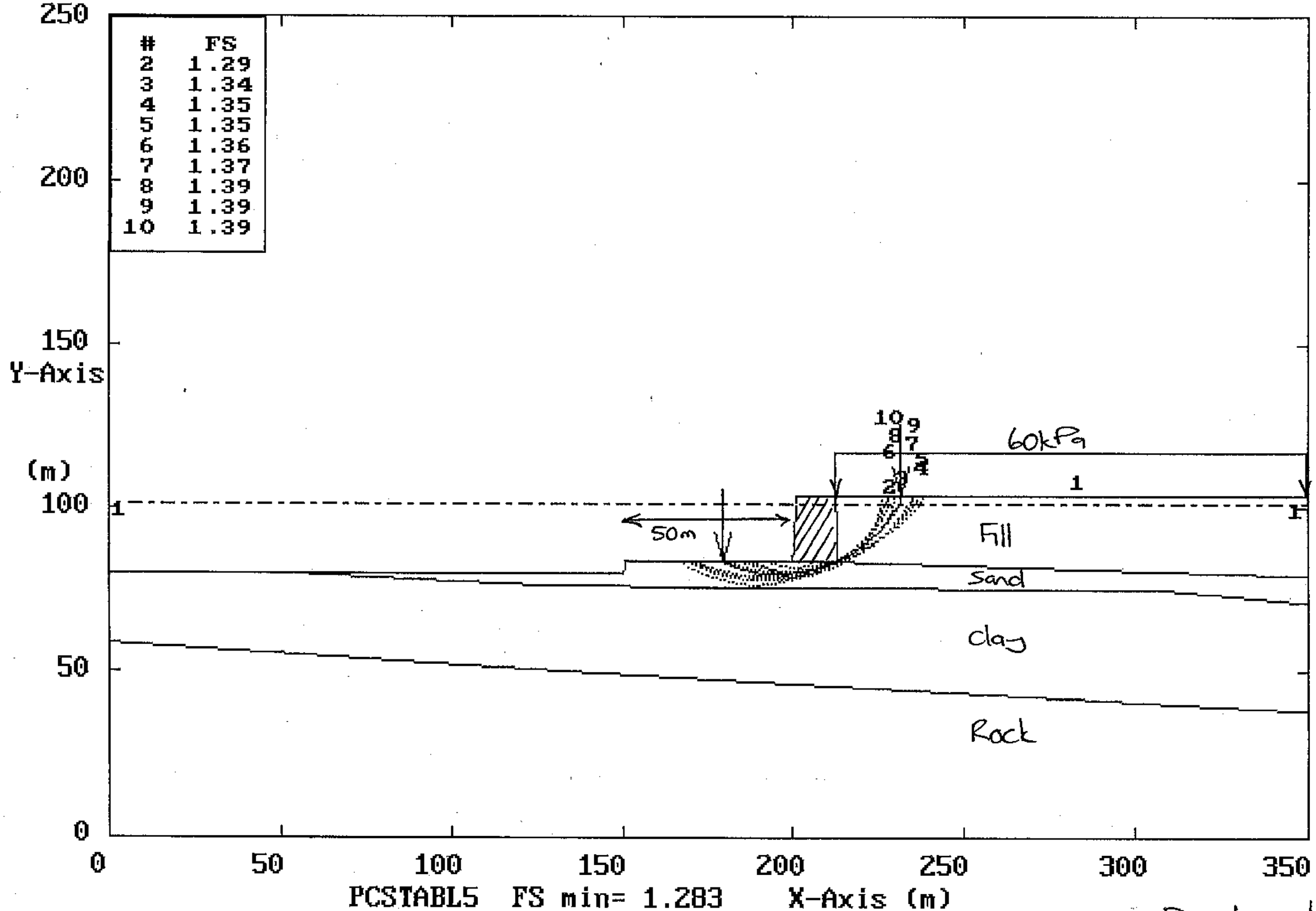
Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB45.PLT By: 07-05-02 11:16 am



PCSTABL5 FS min= 1.575 X-Axis (m)

Dredge to RL-20
50 m from caisson

Port Botany Section B4 Caisson Dredge to RL-20
Ten Most Critical. C:BOTB45.PLT By: 07-05-02 11:15 am



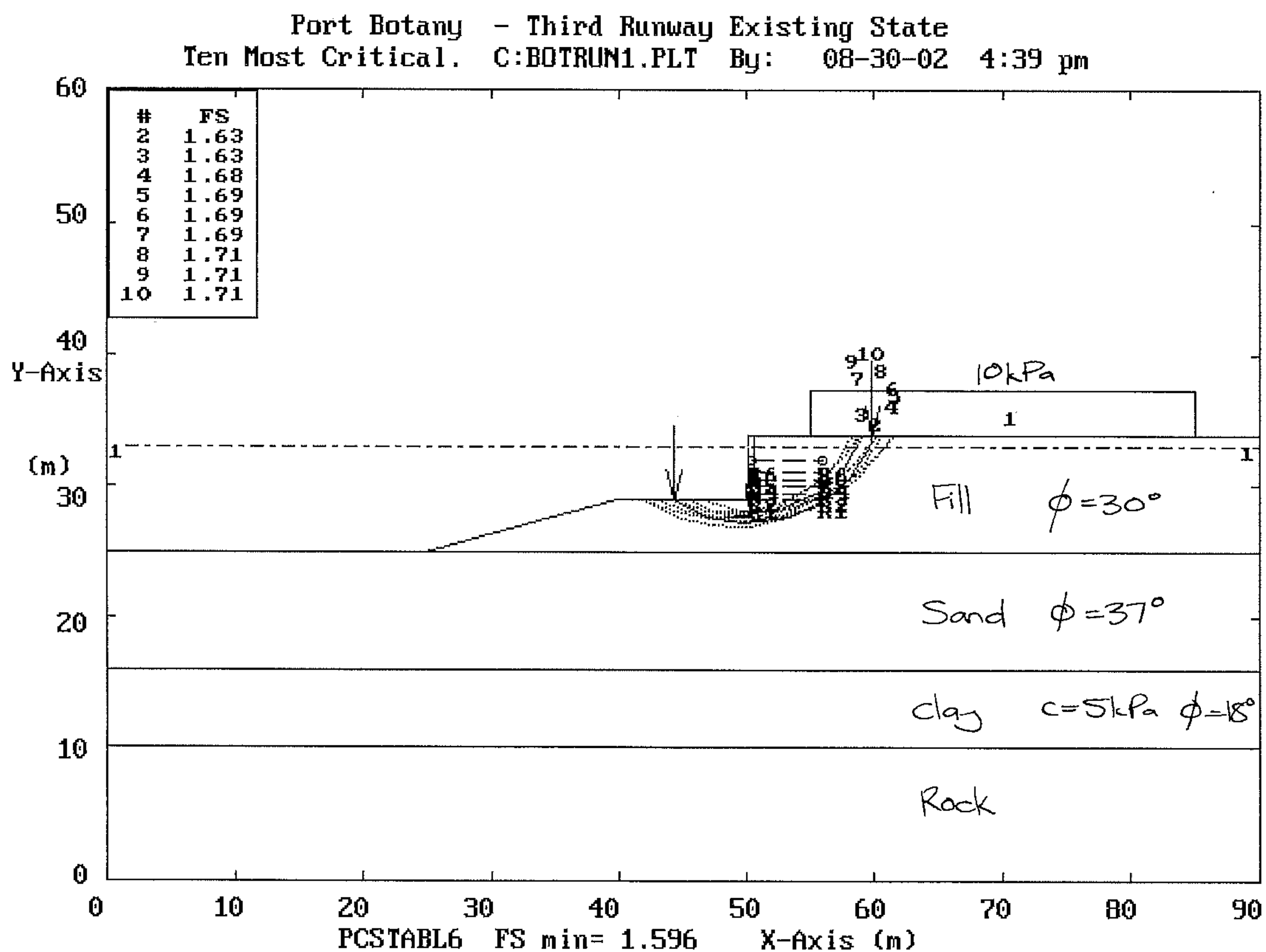
Fill $\phi = 35^\circ$

Clay $c = 10kPa$
$\phi = 25^\circ$

Plus Earthquake loads

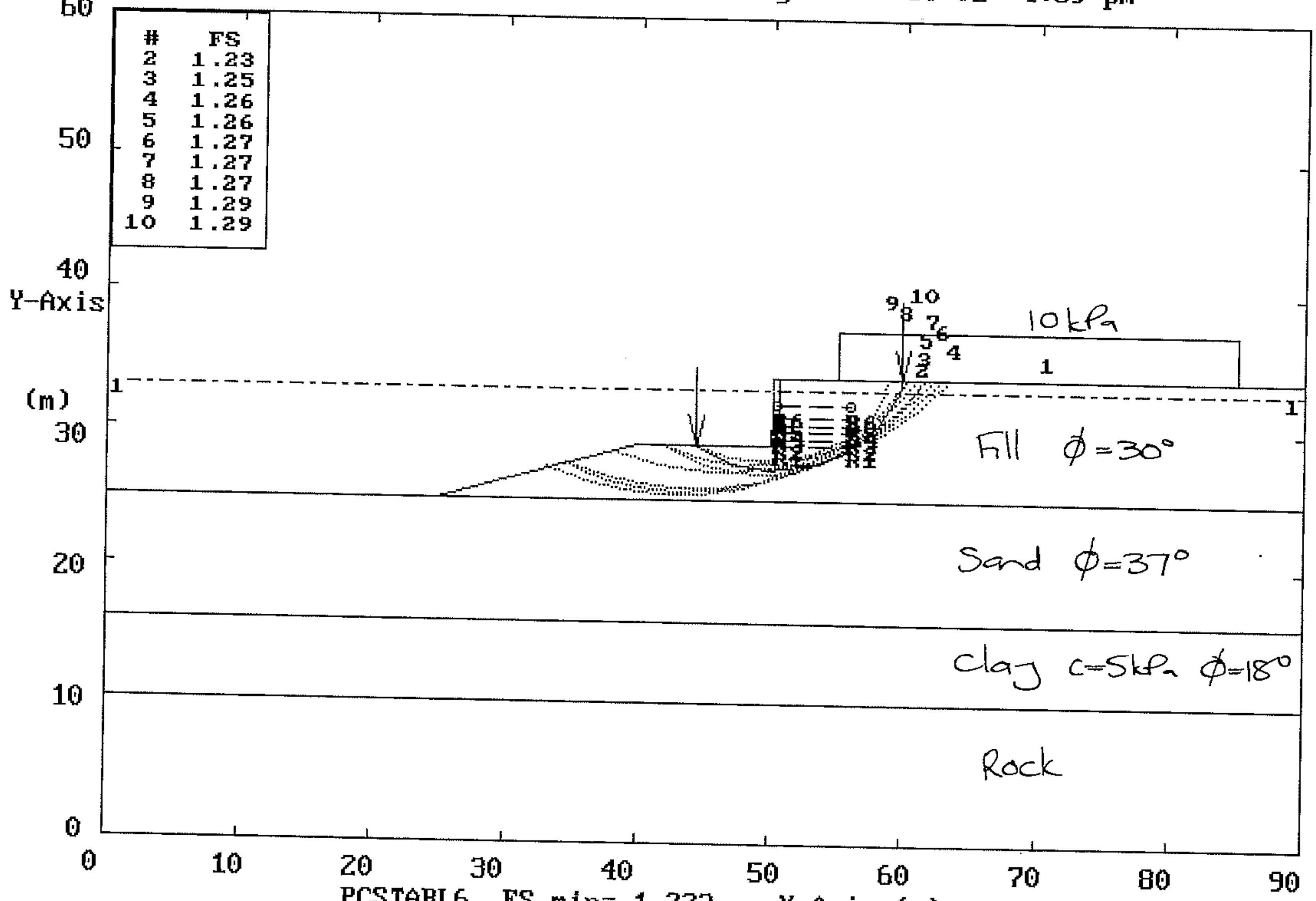
Dredge to RL-20
50 m from caisson

APPENDIX D4
Third Runway



Existing State

Port Botany - Third Runway Existing State
Ten Most Critical. C:BOTRUM1.PLT By: 08-30-02 4:39 pm

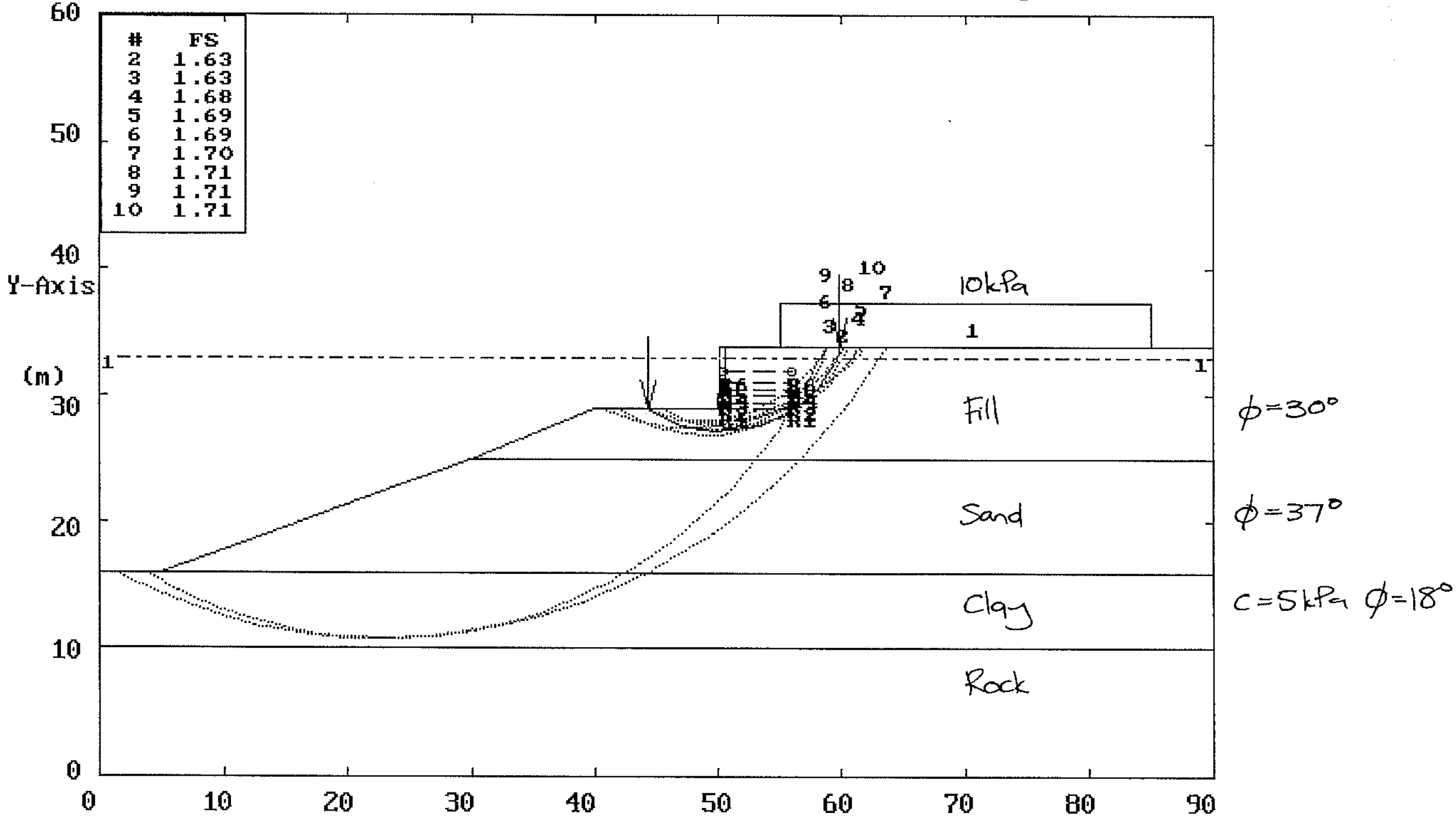


PCSTABL6 FS min= 1.223

Plus Earthquake

Existing State

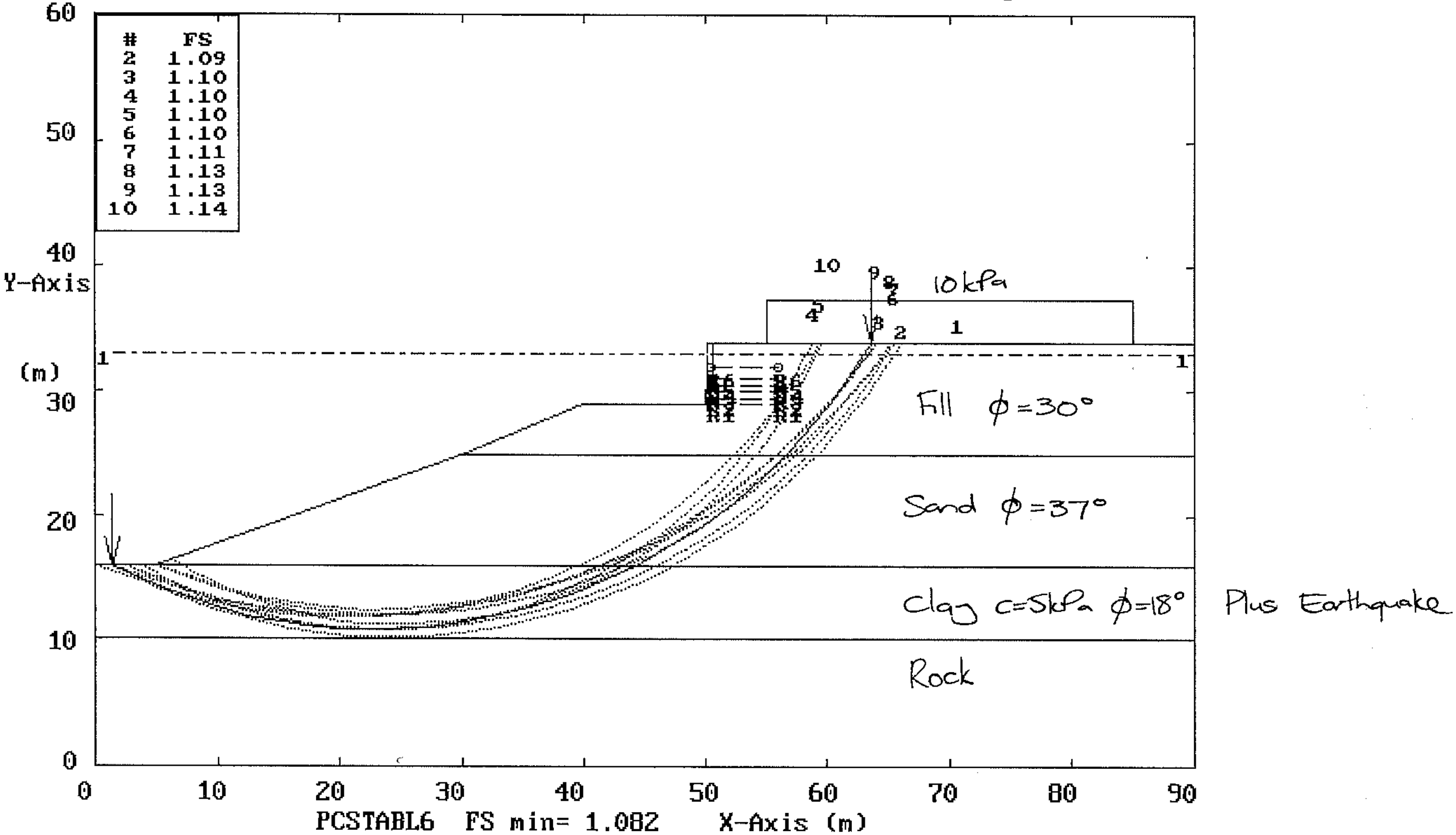
Port Botany - Third Runway Dredge to base of sand
Ten Most Critical. C:BOTRUN2.PLT By: 08-30-02 4:37 pm



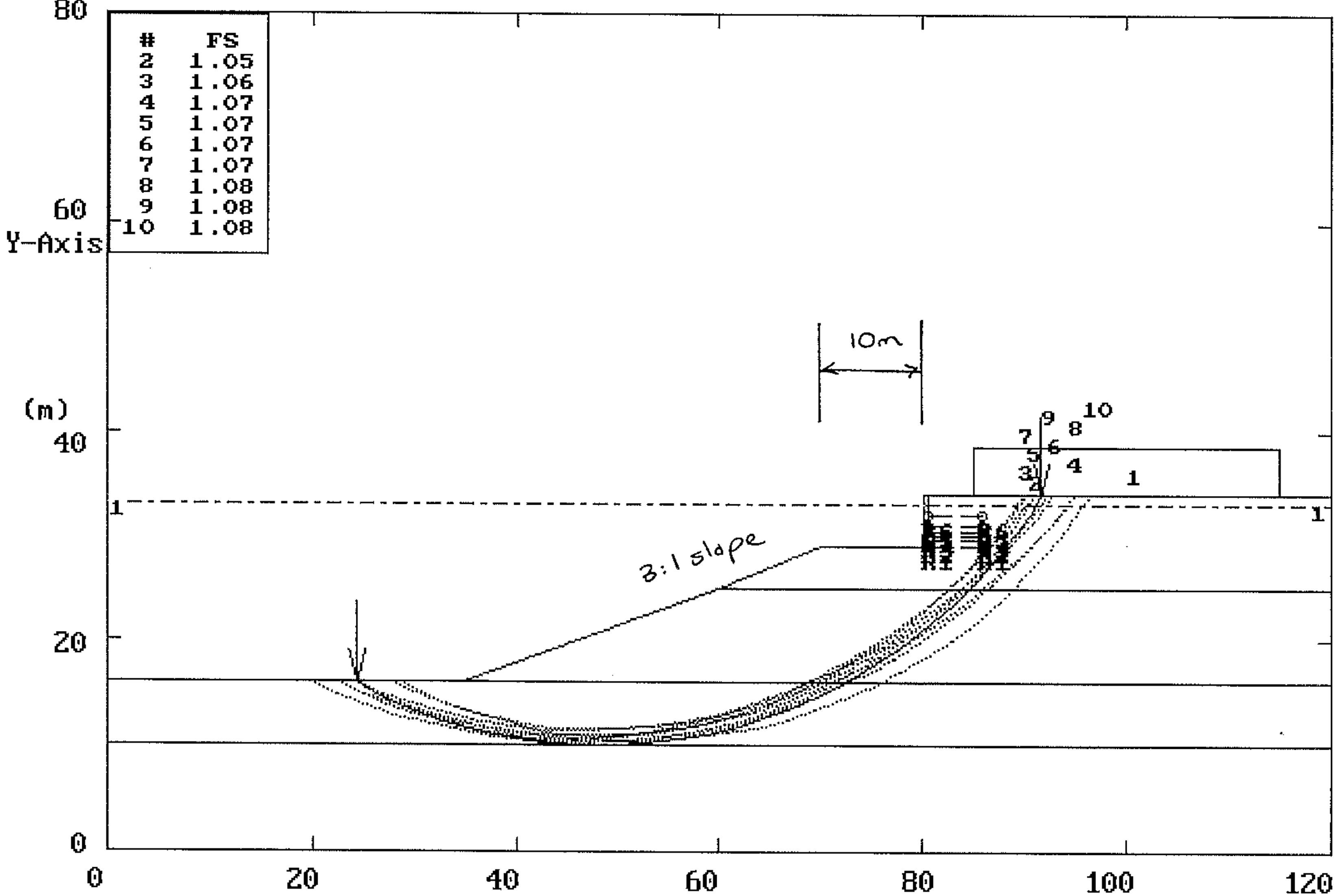
PCSTABL6 FS min= 1.596 X-Axis (m)

Dredged

Port Botany - Third Runway Dredge to base of sand
Ten Most Critical. C:BOTRUN2.PLT By: 08-30-02 4:38 pm



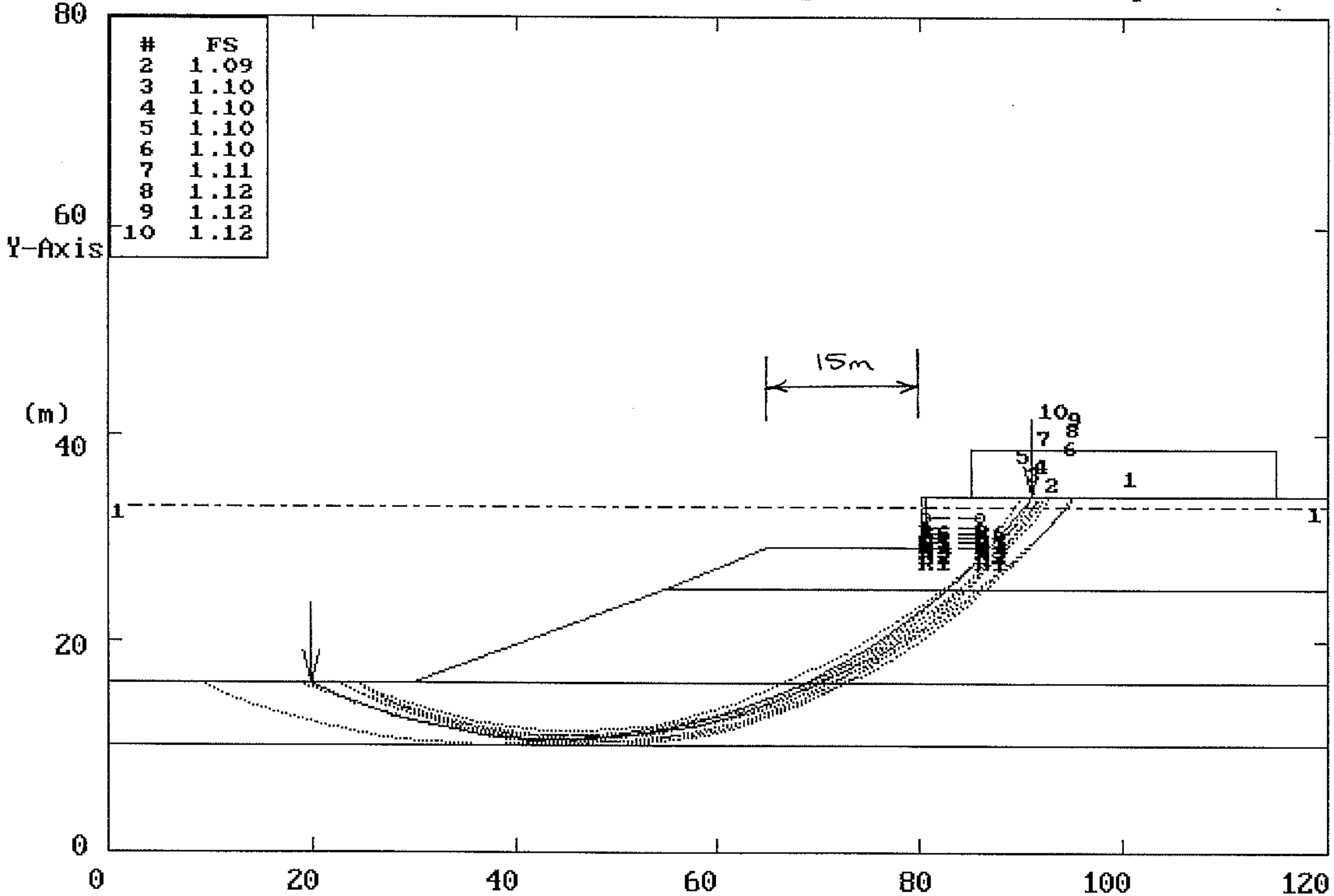
Port Botany - Third Runway Dredge to 10 m of wall
Ten Most Critical. C:BOTRUN3.PLT By: 10-15-02 6:49 pm



PCSTABL6 FS min= 1.051

Dredge to 10m
With Earthquake

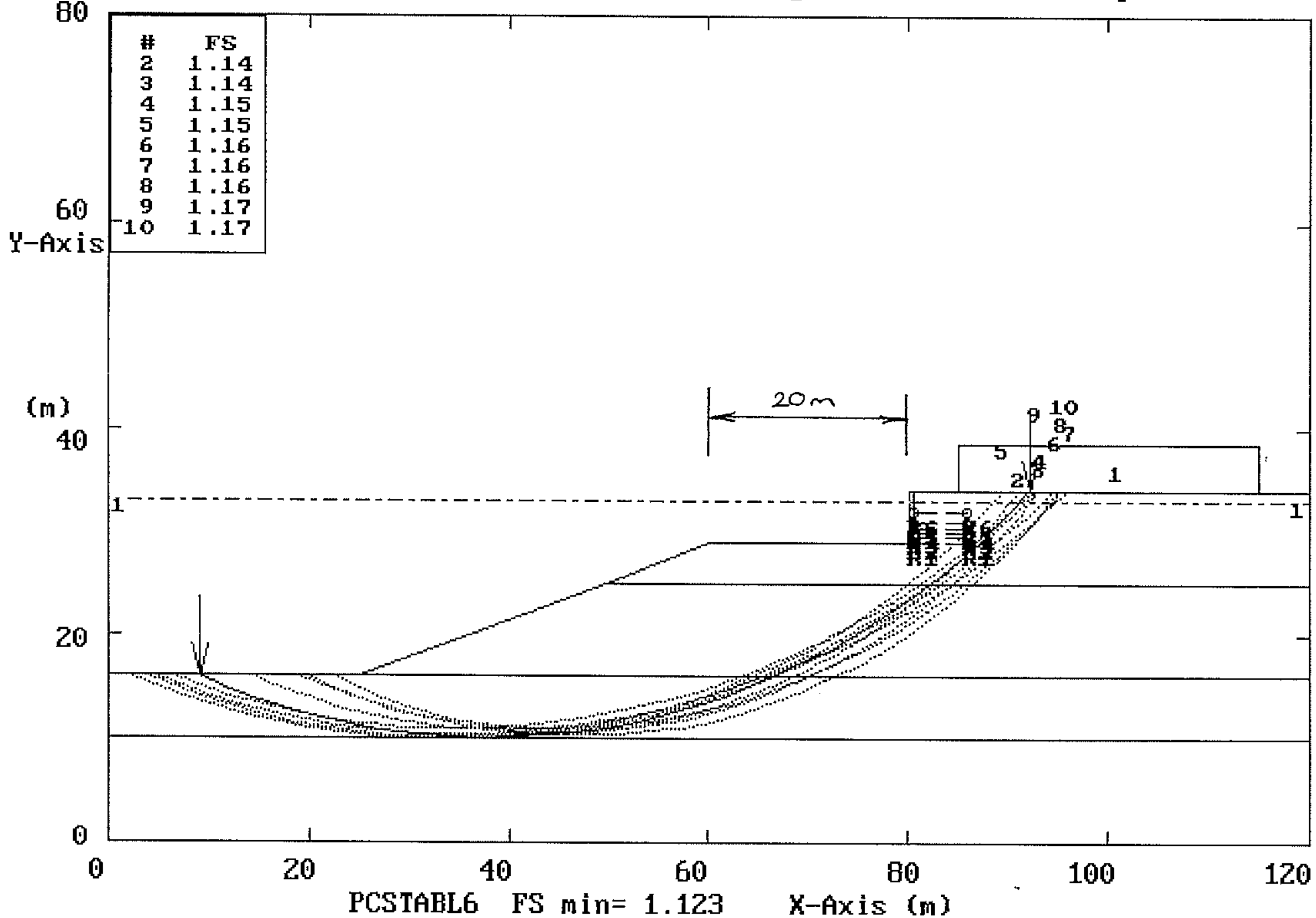
Port Botany - Third Runway Dredge to 15m from wall
Ten Most Critical. C:BOTRUN3B.PLT By: 10-15-02 6:49 pm



PCSTABL6 FS min= 1.09 X-Axis (m)

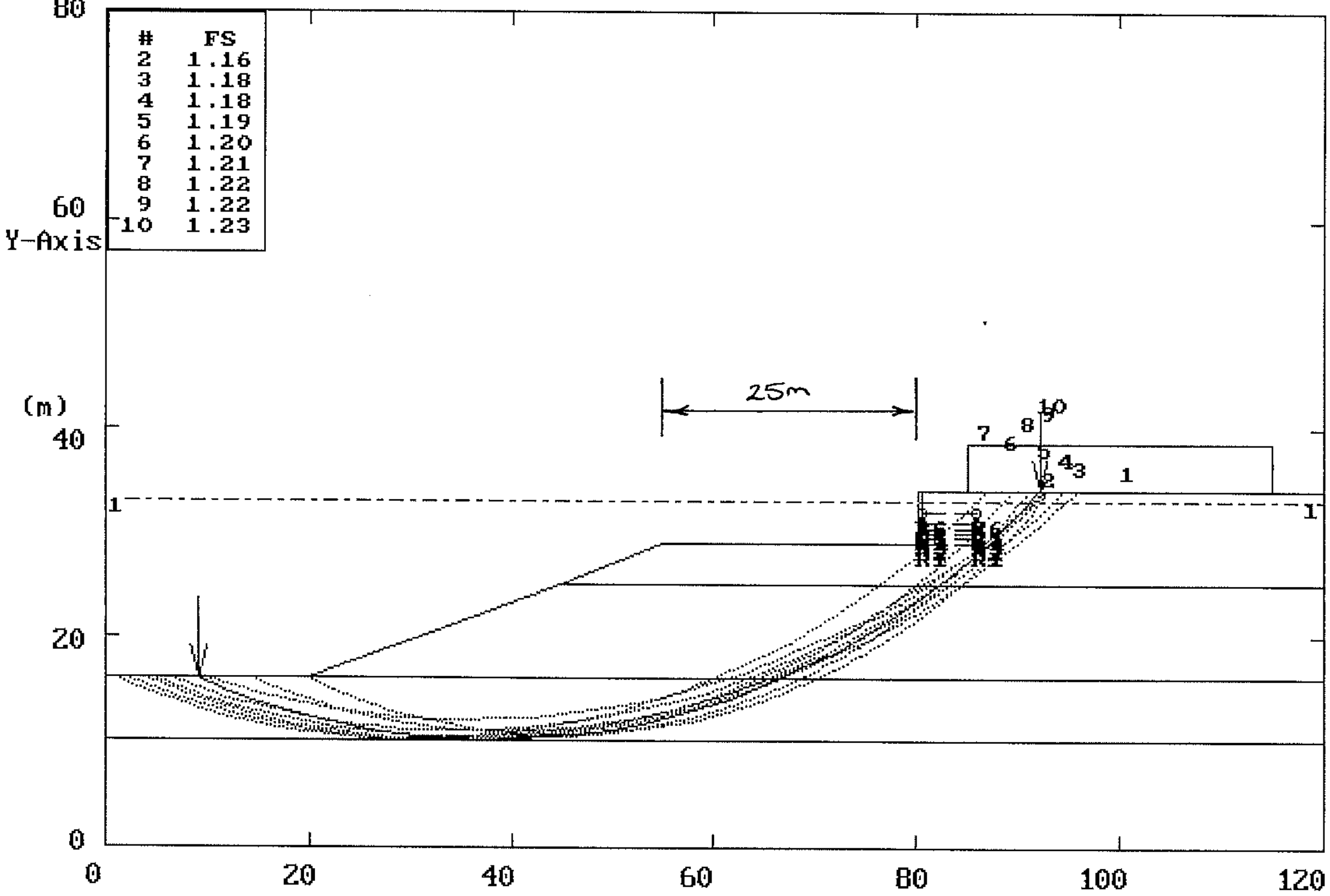
Dredge to 15m
with earthquake

Port Botany - Third Runway Dredge to 20m from wall
Ten Most Critical. C:BOTRUN3A.PLT By: 10-15-02 6:50 pm



Dredge to 20m
With Earthquake

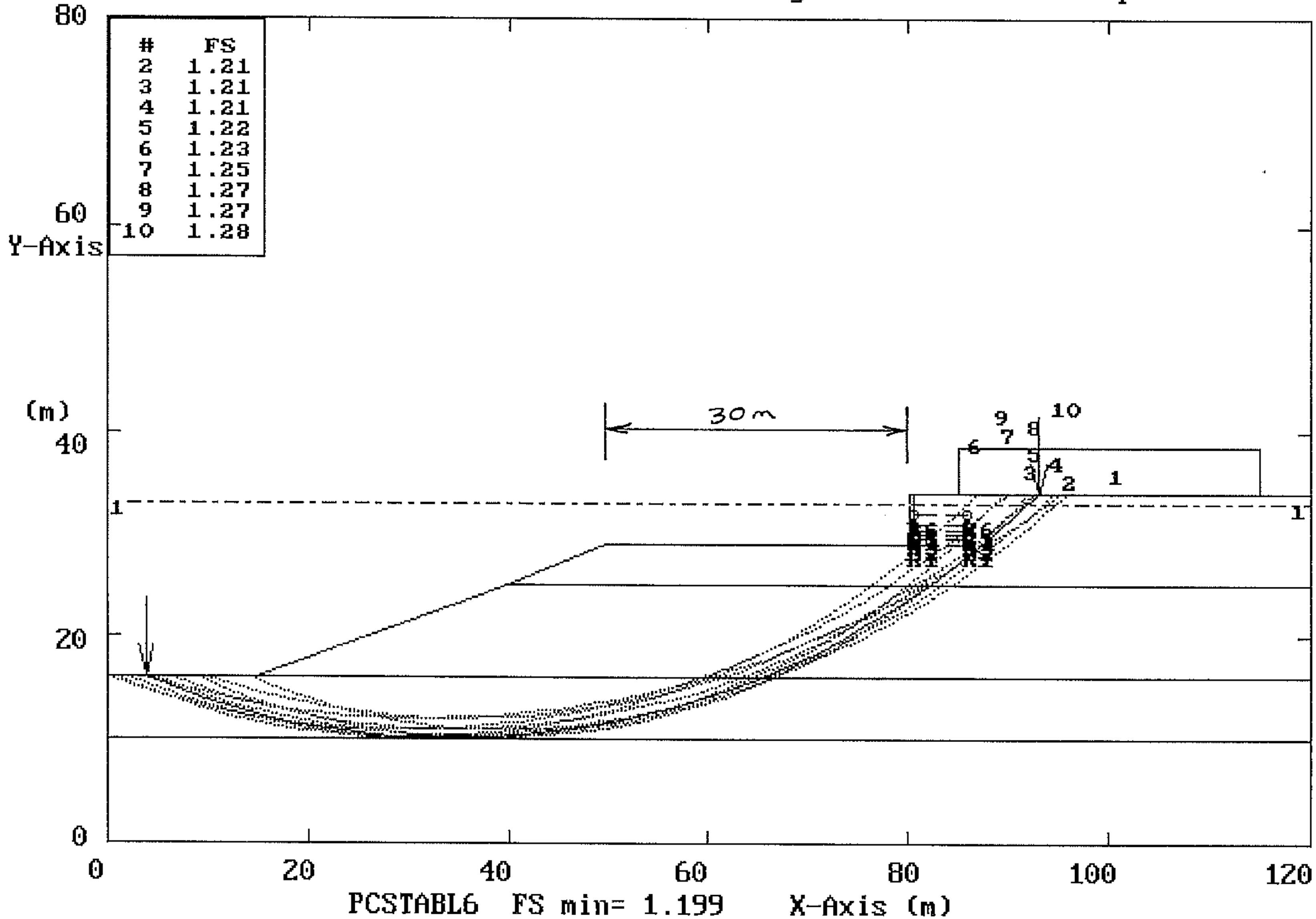
Port Botany - Third Runway Dredge to 25m from wall
Ten Most Critical. C:BOTRUN3D.PLT By: 10-15-02 6:50 pm



PCSTABL6 FS min= 1.159

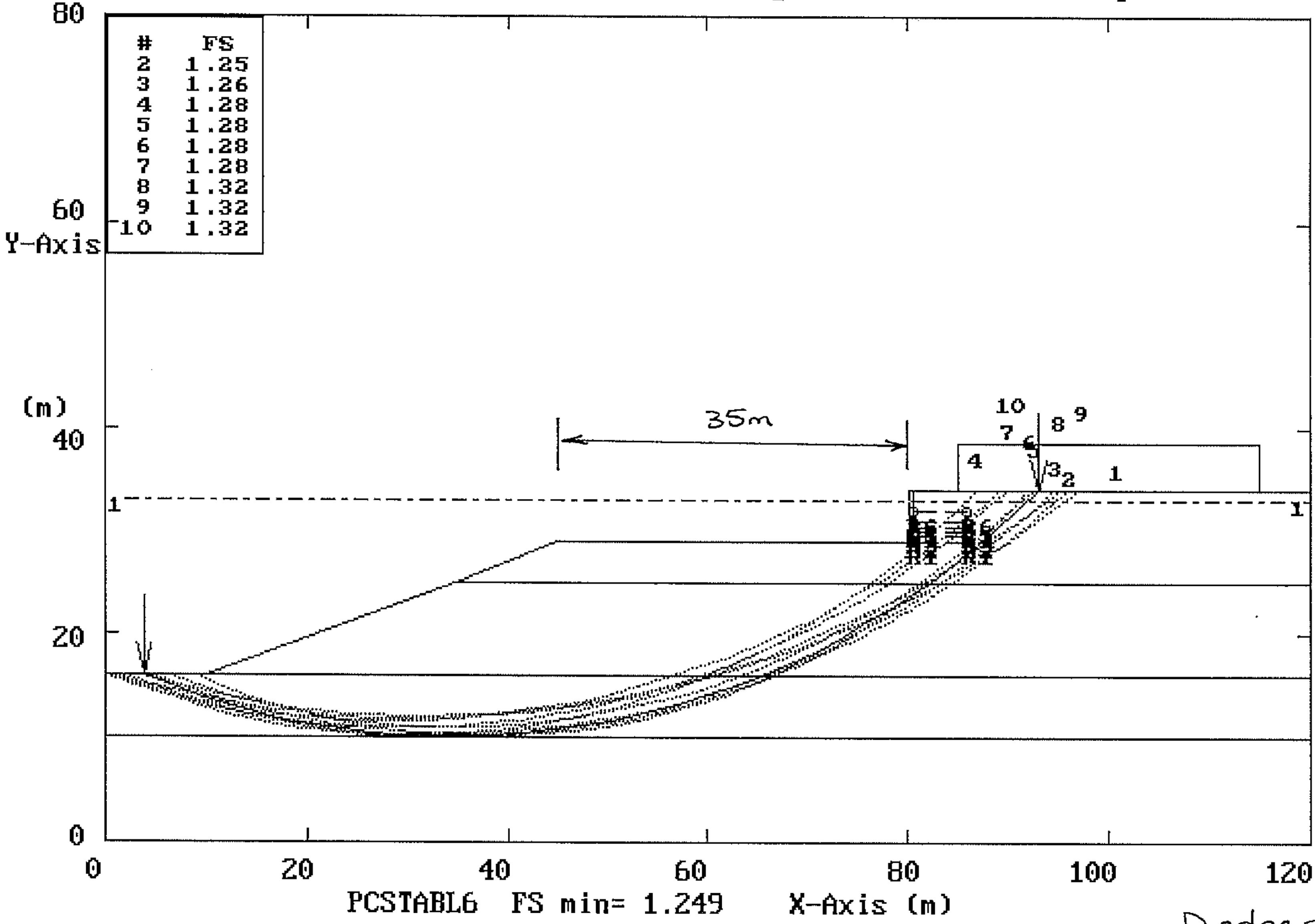
Dredge to 25m
with Earthquake

Port Botany - Third Runway Dredge to 30m from wall
Ten Most Critical. C:BOTRUN3E.PLT By: 10-15-02 6:51 pm



Dredge to 30m
with Earthquake

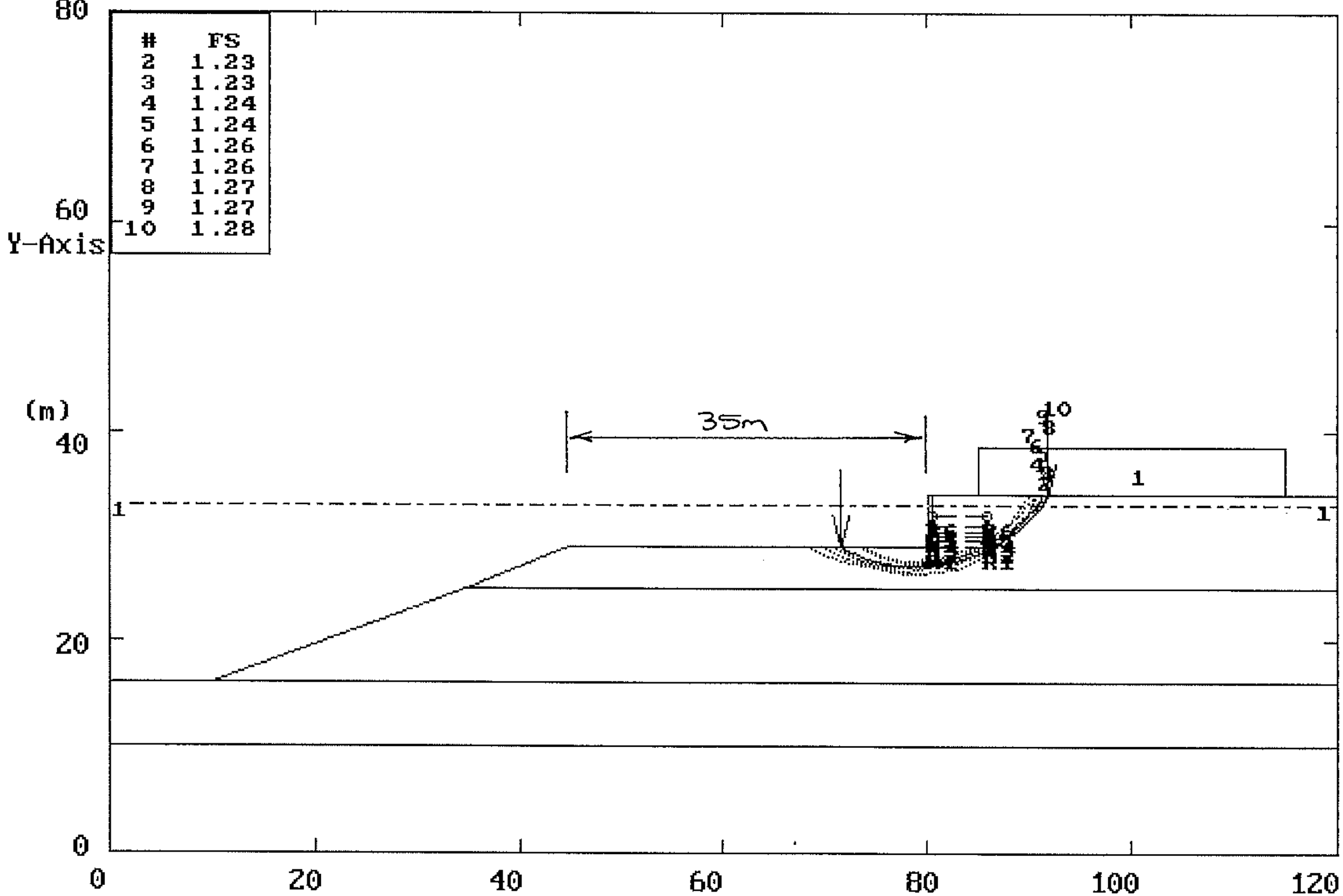
Port Botany - Third Runway Dredge to 35m from wall
Ten Most Critical. C:BOTRUN3F.PLT By: 10-15-02 12:02 pm



Dredge to 35m
With Earthquake

Port Botany - Third Runway Dredge to 35m from wall

Ten Most Critical. C:BOTRUN3F.PLT By: 10-15-02 12:03 pm



PCSTABL6 FS min= 1.221 X-Axis (m)

Dredge to 35m
With Earthquake

APPENDIX E
Settlement Analysis

Proposed Third Terminal - Port Botany
Client : Sydney Ports Corporation Limited

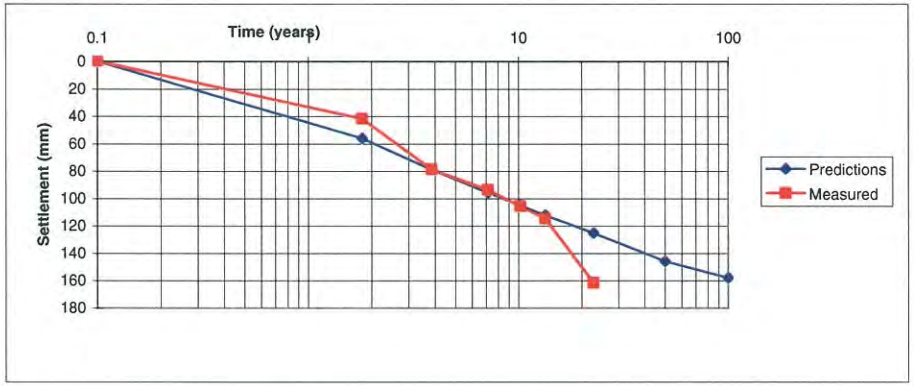
Existing Patrick Terminal - Ch 210
Settlement of foundations and filling up to water level due to 4m of fill plus 2m surcharge replaced by 40 kPa uniform loading

Project : 35224

Bore : CW7 & CPT5

Applied surface load (kPa) **40**
Height of fill above water (m): **4**
Depth to Seabed (m): **0**

Layer	Material	Description	Depth to base	Thickness	Consolidation parameters			Primary Consolidation		Creep per log cycle	Settlement at			
					mv	ca	cv	Settlement	Time		1 year	10 years	50 years	100 years
			(m)	(m)	(m2/MN)		(m2/yr)	(mm)	(yrs)	(mm)	(mm)	(mm)	(mm)	(mm)
1	Sand	Fill - loose	1.3	1.3	0.0500	0.0000	100	7	0	0	7	7	7	7
2	Sand	Fill - medium dense	14.2	12.9	0.0050	0.0000	100	7	1	0	6	7	7	7
3	Clay	Fill - firm	15	0.8	0.2000	0.0300	0.2	18	3	24	11	42	59	66
4	Sand	Unit 2 - very dense	39	24	0.0050	0.0000	100	13	5	0	6	13	13	13
5	Clay	Unit 3B - hard	43	4	0.0600	0.0030	0.5	27	27	12	5	29	47	51
6	Sand	Unit 3C - med to very dense	46	3	0.0050	0.0000	50	2	0	0	2	2	2	2
7	Clay	Unit 4A - hard	49	3	0.0400	0.0010	0.5	2	15	3	2	5	7	8
8	Sand	Unit 4B - med to very dense	56	7	0.0050	0.0000	50	4	1	0	4	4	4	4
9		Sandstone bedrock												
10														
11														
12														
13														
14														
15														
							Totals	**80**			**43**	**109**	**146**	**158**



Assumptions
1: settlement under fill with unit weight 18 kN/m3
2: settlement = mv x stress change x thickness
3: one way drainage for consolidation
4: one dimensional consolidation

Date: 26/08/2002
Calculated: FM
Checked:

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Ch210CONS

Proposed Third Terminal - Port Botany
Client : Sydney Ports Corporation Limited

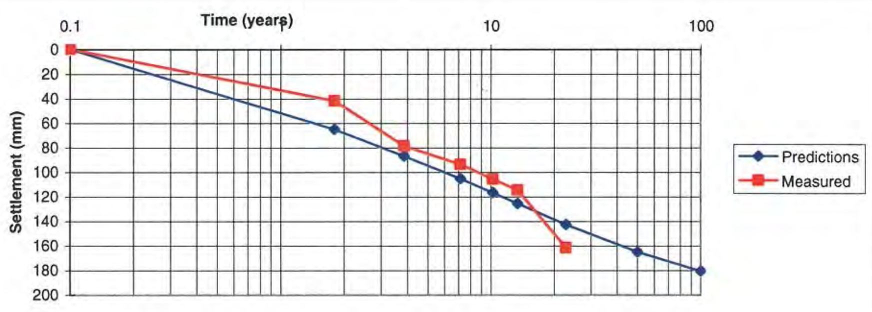
Existing Patrick Terminal - Ch 430
Settlement of foundations and filling up to water level due to 4m of fill plus 2m surcharge replaced by 40 kPa uniform loading

Project : 35224

Bore : CW8 & CPT2

Applied surface load (kPa) 40
Height of fill above water (m): 4
Depth to Seabed (m): 0

Layer	Material	Description	Depth to base (m)	Thickness (m)	Consolidation parameters mv (m2/MN)	ca	cv (m2/yr)	Primary Consolidation Settlement (mm)	Time (yrs)	Creep per log cycle (mm)	Settlement at 1 year (mm)	10 years (mm)	50 years (mm)	100 years (mm)
1	Sand	Fill - loose	3	3	0.0500	0.0000	100	17	0	0	17	17	17	17
2	Clay	Fill - firm	3.5	0.5	0.2000	0.0300	0.2	11	1	15	10	26	37	41
3	Sand	Fill - medium dense	15	11.5	0.0050	0.0000	100	6	1	0	6	6	6	6
4	Clay	Fill - firm to stiff	17.3	2.3	0.1000	0.0100	0.2	26	22	23	6	41	65	72
5	Sand	Unit 2 - very dense	34	16.7	0.0050	0.0000	100	9	2	0	6	9	9	9
6	Clay	Unit 4A - hard	39.5	5.5	0.0400	0.0010	0.5	25	51	6	4	17	31	35
7		Sandstone bedrock												
8														
9														
10														
11														
12														
13														
14														
15														
							Totals	94			48	116	165	181



Assumptions
1: settlement under fill with unit weight 18 kN/m3
2: settlement = mv x stress change x thickness
3: one way drainage for consolidation
4: one dimensional consolidation

Date: 26/08/2002
Calculated: FM
Checked:

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Ch430CONS

Proposed Third Terminal - Port Botany
Client : Sydney Ports Corporation Limited

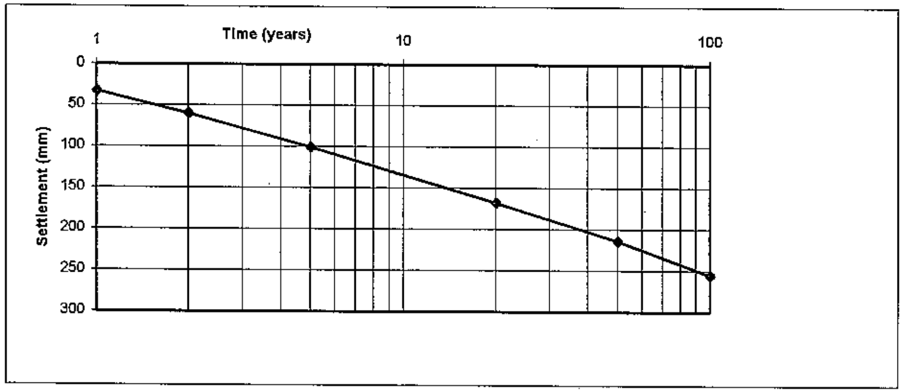
Project : 35224

Bore : CP1

Applied surface load (kPa) 60
Height of fill above water (m): 3.7
Depth to Seabed (m): 1.65

Layer	Material	Description	Depth to base (m)	Thickness (m)	Consolidation parameters mv (m2/MN)	ca	cv (m2/yr)	Primary Consolidation Settlement (mm)	Time (yrs)	Creep per log cycle (mm)	Settlement at 1 year (mm)	10 years (mm)	50 years (mm)	100 years (mm)
1	Sand	1C - very loose	2	2	0.0500	0.0000	100	14	0	0	14	14	14	14
2	Sand	2 - very dense	6	4	0.0050	0.0000	100	3	0	0	3	3	3	3
3	Clay	3B - very stiff	27.8	21.8	0.0600	0.0030	0.5	183	806	65	7	87	158	198
4	Clay	4A - very stiff	30.8	3	0.0400	0.0010	0.5	17	15	3	4	16	22	23
5	Sand	4B - very dense	31.4	0.6	0.0050	0.0000	50	0	0	0	0	0	0	0
6	Clay	4A - hard	33.7	2.3	0.0400	0.0010	0.5	13	9	2	4	14	17	17
7														
8														
9														
10														
11														
12														
13														
14														
15														
							Totals	230			33	134	214	255

25% of 10yrs



Assumptions
1: settlement under fill with unit weight 18 kN/m3
2: settlement = mv x stress change x thickness
3: one way drainage for consolidation
4: one dimensional consolidation

Date: 18/06/2002
Calculated: FM
Checked:

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Proposed Third Terminal - Port Botany

Client : Sydney Ports Corporation Limited

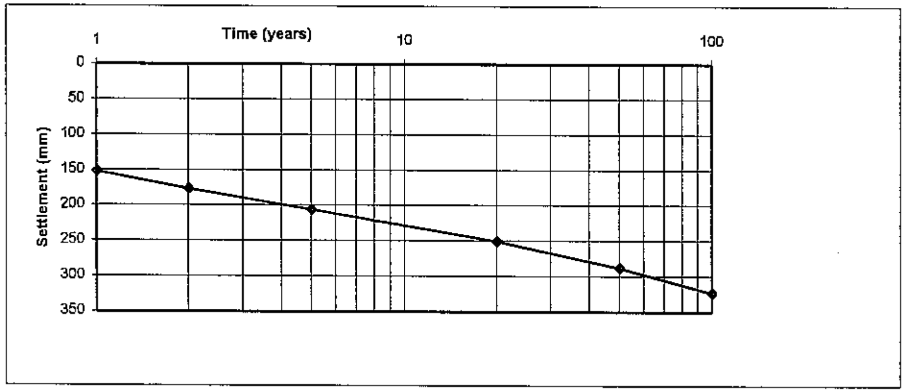
Project : 35224

Bore : CP3

Applied surface load (kPa) **60**
Height of fill above water (m): **3.7**
Depth to Seabed (m): **15.1**

Layer	Material	Description	Depth to base (m)	Thickness (m)	Consolidation parameters mv (m2/MN)	ca	cv (m2/yr)	Primary Consolidation Settlement (mm)	Time (yrs)	Creep per log cycle (mm)	Settlement at 1 year (mm)	10 years (mm)	50 years (mm)	100 years (mm)
1	Sand	1B - very loose	1.5	1.5	0.0500	0.0000	100	19	0	0	19	19	19	19
2	Sand	2 - very dense	7.8	6.3	0.0050	0.0000	100	8	0	0	8	8	8	8
3	Clay	3A - hard	8.8	1	0.0800	0.0050	10	20	0	5	20	25	28	30
4	Clay	3B - very stiff	9.7	0.9	0.0600	0.0030	0.5	13	1	3	11	16	18	19
5	Clay	3A - hard	10.5	0.8	0.0800	0.0050	10	16	0	4	16	20	23	24
6	Clay	3B - very stiff	12	1.5	0.0600	0.0030	0.5	22	4	5	12	27	30	31
7	Clay	3A - hard	14.9	2.9	0.0800	0.0050	10	57	1	15	55	72	82	86
8	Sand	3C - very dense	16	1.1	0.0050	0.0000	50	1	0	0	1	1	1	1
9	Clay	4A - very stiff to hard	28.5	12.5	0.0400	0.0010	0.5	124	265	13	8	37	77	103
10	Sand	4B - medium dense	30.8	2.3	0.0050	0.0000	50	3	0	0	3	3	3	3
11														
12														
13														
14														
15														
							Totals	283			152	227	288	324

67% of 10 yrs



Assumptions
1: settlement under fill with unit weight 18 kN/m3
2: settlement = mv x stress change x thickness
3: one way drainage for consolidation
4: one dimensional consolidation

Date: 18/06/2002
Calculated: FM
Checked:

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Proposed Third Terminal - Port Botany
Client : Sydney Ports Corporation Limited

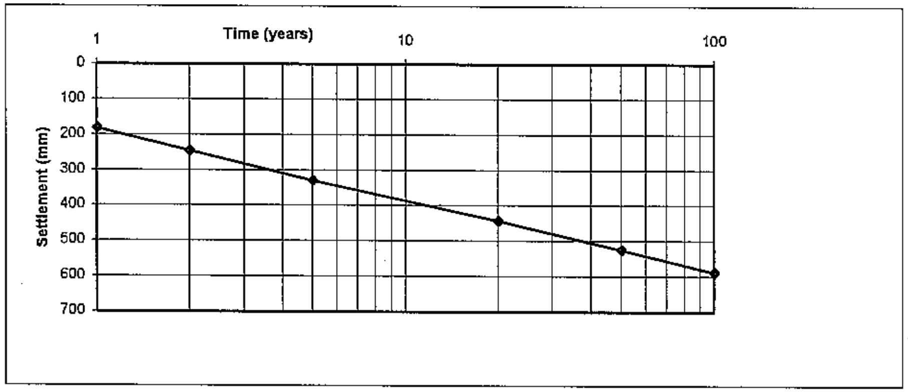
Project : 35224

Applied surface load (kPa) 60
Height of fill above water (m): 3.7
Depth to Seabed (m): 18.4

Bore : CP5

Layer	Material	Description	Depth to base (m)	Thickness (m)	Consolidation parameters mv (m2/MN)	ca	cv (m2/yr)	Primary Consolidation Settlement (mm)	Time (yrs)	Creep per log cycle (mm)	Settlement at 1 year (mm)	10 years (mm)	50 years (mm)	100 years (mm)
1	Sand	1A - loose	3.2	3.2	0.0500	0.0000	100	44	0	0	44	44	44	44
2	Sand	2 - very dense	4	0.8	0.0050	0.0000	100	1	0	0	1	1	1	1
3	Sand	3A - hard sandy peat	5.1	1.1	0.0800	0.0050	10	24	0	6	24	30	33	35
4	Clay	3B - very stiff to hard	9	3.9	0.0600	0.0030	0.5	64	26	12	13	53	83	87
5	Clay	3A - hard clayey peat	15.5	6.5	0.0800	0.0050	10	142	4	33	78	175	198	207
6	Clay	4A - very stiff	23.5	8	0.0400	0.0010	0.5	88	109	8	9	36	74	93
7	Sand	4B - very dense	25.1	1.6	0.0050	0.0000	50	2	0	0	2	2	2	2
8	Clay	4A - very stiff to hard	39.5	14.4	0.0400	0.0010	0.5	158	352	14	9	42	86	116
9	Clay	5 - very dense clayey sand	41.8	2.3	0.0050	0.0000	20	3	0	0	3	3	3	3
10														
11														
12														
13														
14														
15														
							Totals	526			183	385	525	589

48% of 10yr



Assumptions
1: settlement under fill with unit weight 18 kN/m3
2: settlement = mv x stress change x thickness
3: one way drainage for consolidation
4: one dimensional consolidation

Date: 18/06/2002
Calculated: FM
Checked:

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CP5CONS.XLS

Proposed Third Terminal - Port Botany

Client : Sydney Ports Corporation Limited

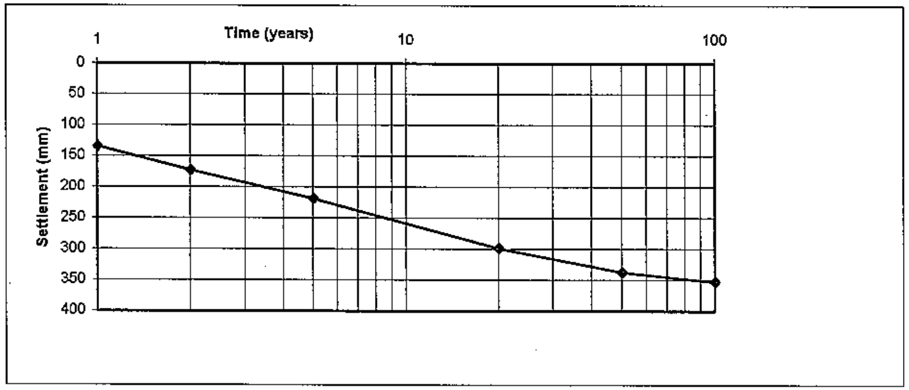
Project : 35224

Bore : CP8

Applied surface load (kPa): 60
Height of fill above water (m): 3.7
Depth to Seabed (m): 21.95

Layer	Material	Description	Depth to base (m)	Thickness (m)	Consolidation parameters mv (m2/MN)	ca	cv (m2/yr)	Primary Consolidation Settlement (mm)	Time (yrs)	Creep per log cycle (mm)	Settlement at 1 year (mm)	10 years (mm)	50 years (mm)	100 years (mm)	
1	Sand	1B - very loose	0.7	0.7	0.0500	0.0000	100	11	0	0	11	11	11	11	0
2	Sand	2 - very dense	2	1.3	0.0050	0.0000	100	2	0	0	2	2	2	2	0
3	Clay	3B - stiff to very stiff	5	3	0.0600	0.0030	0.5	54	15	9	14	52	70	72	58
4	Clay	3A - hard	9	4	0.0800	0.0050	10	97	1	20	80	117	131	137	57
5	Clay	4A - hard	12.8	3.8	0.0400	0.0010	0.5	46	24	4	10	34	52	54	44
6	Sand	4B - very dense	15.6	2.8	0.0050	0.0000	50	4	0	0	4	4	4	4	0
7	Clay	4A - hard	20.6	5	0.0400	0.0010	0.5	60	42	5	10	35	65	70	60
8	Sand	4B - very dense	22.2	1.6	0.0050	0.0000	50	2	0	0	2	2	2	2	
9	Sand	5 - very dense	23	0.8	0.0050	0.0000	20	1	0	0	1	1	1	1	
10															
11															
12															
13															
14															
15															
							Totals	278			134	259	337	353	219

52% of 100yr



Assumptions
1: settlement under fill with unit weight 18 kN/m3
2: settlement = mv x stress change x thickness
3: one way drainage for consolidation
4: one dimensional consolidation

Date: 18/06/2002
Calculated: FM
Checked:

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CP8CONS.XLS

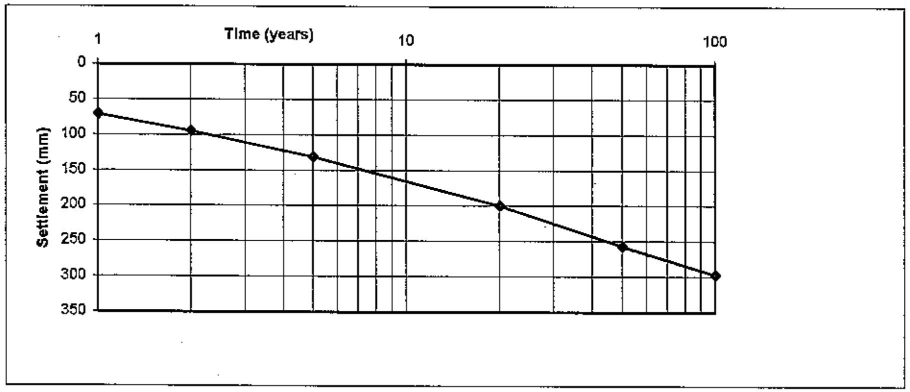
Proposed Third Terminal - Port Botany
Client : Sydney Ports Corporation Limited

Project : 35224

Bore : CP10

Applied surface load (kPa) **60**
Height of fill above water (m): **3.7**
Depth to Seabed (m): **15.95**

Layer	Material	Description	Depth to base (m)	Thickness (m)	Consolidation parameters mv (m2/MN)	ca	cv (m2/yr)	Primary Consolidation Settlement (mm)	Time (yrs)	Creep per log cycle (mm)	Settlement at 1 year (mm)	10 years (mm)	50 years (mm)	100 years (mm)
1	Sand	1B - very loose	0.8	0.8	0.0500	0.0000	100	10	0	0	10	10	10	10
2	Sand	2 - very dense	4	3.2	0.0050	0.0000	100	4	0	0	4	4	4	4
3	Sand	3C - medium dense	5.2	1.2	0.0050	0.0000	50	2	0	0	2	2	2	2
4	Clay	3B - very stiff	6.7	1.5	0.0600	0.0030	0.5	23	4	5	12	27	31	32
5	Clay	3A - hard	7.8	1.1	0.0800	0.0050	10	22	0	6	22	28	32	33
6	Clay	3B - hard	13.5	5.7	0.0600	0.0030	0.5	87	55	17	12	56	105	120
7	Clay	4A - hard	23	9.5	0.0400	0.0010	0.5	97	153	10	8	35	73	96
8	Sand	5 - very dense	23.5	0.5	0.0050	0.0000	20	1	0	0	1	1	1	1
9														
10														
11														
12														
13														
14														
15														
							Totals	245			71	162	257	297



44°/6/0vr

Assumptions
1: settlement under fill with unit weight 18 kN/m3
2: settlement = mv x stress change x thickness
3: one way drainage for consolidation
4: one dimensional consolidation

Date: 18/06/2002
Calculated: FM
Checked:

Douglas Partners
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CP10CONS.XLS

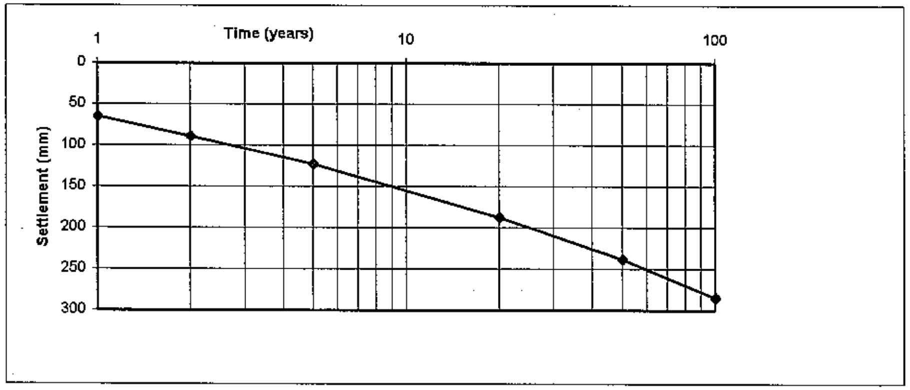
Proposed Third Terminal - Port Botany
Client : Sydney Ports Corporation Limited

Project : 35224

Bore : CP12

Applied surface load (kPa) **60**
Height of fill above water (m): **3.7**
Depth to Seabed (m): **11.7**

Layer	Material	Description	Depth to base (m)	Thickness (m)	Consolidation parameters mv (m2/MN)	ca	cv (m2/yr)	Primary Consolidation Settlement (mm)	Time (yrs)	Creep per log cycle (mm)	Settlement at 1 year (mm)	10 years (mm)	50 years (mm)	100 years (mm)
1	Sand	1B - very loose	0.6	0.6	0.0500	0.0000	100	7	0	0	7	7	7	7
2	Sand	2 - medium dense	11.5	10.9	0.0050	0.0000	100	12	1	0	11	12	12	12
3	Sand	3C - very dense	22.5	11	0.0050	0.0000	50	12	2	0	9	12	12	12
4	Clay	3B - hard	25.5	3	0.0600	0.0030	0.5	40	15	9	11	40	55	58
5	Clay	4A - hard	35	9.5	0.0400	0.0010	0.5	84	153	10	7	32	66	85
6	Sand	4B - very dense	40.9	5.9	0.0050	0.0000	50	6	1	0	6	6	6	6
7	Clay	4A - hard	54	13.1	0.0400	0.0010	0.5	115	291	13	7	35	72	96
8	Sand	4B - very dense	61.8	7.8	0.0050	0.0000	50	9	1	0	8	9	9	9
9	Sand	5 - very dense	62	0.2	0.0050	0.0000	20	0	0	0	0	0	0	0
10														
11														
12														
13														
14														
15														
							Totals	285			65	154	238	285



Assumptions
1: settlement under fill with unit weight 18 kN/m3
2: settlement = mv x stress change x thickness
3: one way drainage for consolidation
4: one dimensional consolidation

Date: 18/06/2002
Calculated: FM
Checked:

Douglas Partners
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CP12CONS.XLS