Stability Analysis in Stiff Fissured Clay at Raby Bay, Queensland

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SUMMARY. The Raby Bay Project in southeast Queensland consists of a coastal residential and recreational complex based around a system of canals. Construction failures and the presence of fissured clay had caused concern about canal slope stability and the adoption of remedial works. Most of the stiff clays is of volcanic origin but sedimentary clay occurs in parts of the site. Defects (fissures) occur in layers and trends were identified in the occurrence and character of defects across the site. Stability charts have been developed based on a generalised model which takes into account the depth and thickness of the layer containing defects (weak layer) and the dip and continuity of the defects. The concept of a Strength Reduction Factor has been developed to characterise the strength of the weak layer.

1. INTRODUCTION

1.1 Background And Approach

Raby Bay is situated about 25km east of Brisbane in southeast Queensland. The Raby Bay Project consists of a coastal residential and recreational complex based around a system of tidal canals. Construction of the canals started in 1983 and design slopes of 3 to 1 (horizontal to vertical) were adopted early in the project. The canals are excavated to a depth of RL -3.8m and fill is placed between the canals to construct road and residential areas to about RL +3m.

The presence of closely spaced slickensided and/or polished defects in the stiff soils ("fissured clay") was recognised in 1983 following a slope failure next to an overdeepened borrow area. During the period 1983 to 1988 the presence of fissured clay was observed in several areas and appeared to contribute to several construction slope failures. Back analyses appeared to indicate that the mass strength parameters at the time of failure were between residual and fully softened although there was uncertainty about the pore water pressure assumptions.

Remedial measures in the form of excavation and replacement of the slopes were adopted where fissured clay occurred. Flatter slopes were not considered. The soil profile was described as extremely variable and decisions on replacement were made on a lot by lot basis. During 1988 construction was underway in a part of the site where, in places, fissured clay occurred at depths greater than RL -7m. The scale (and therefore the cost) of earthworks replacement in this area was a major concern to the developer.

This paper presents some of the results of the work carried out by the writer on the project during 1989. The initial engagement was to carry out a review of the site geology. Subsequent engagements during that year were to carry out investigations and provide advice on slope design for various parts of the project. Some construction monitoring was also carried out and the results of previous studies were reviewed and used where relevant to the development and quantification of the geotechnical model.

Excavations or large test pits were the preferred method of field investigation as they provide the most complete picture of subsurface conditions. Where depth limitations occurred, or there were problems with pit stability or access, large diameter cored boreholes (100mm diameter core) were used. The work was carried out as a number of individual studies of different parts of the project area. However, emphasis was placed on obtaining an overall understanding of the site geology so that an overall approach to stability analysis and slope design could be developed for the entire project area.

2. GEOTECHNICAL MODEL

2.1 Geological Setting

According to published geological maps the Raby Bay Project area consists of estuarine deposits underlain by volcanics of the Tertiary age Lamington Group. The estuarine deposits are described as consisting of mud, silt, sand, clay, gravel, and minor peat and coral debris. While the Lamington Group consists mainly of basalt, some pyroclastic material (ash or tuff) and sediments also occur.

2.2 Site Geology

2.2.1 Soil and rock materials

Figure 1 is a diagrammatic cross section showing the site geology. A brief description of the materials is given in the legend. Seven materials were identified with subdivision of each material based on both geological origin and engineering properties. From the engineering point of view the information on materials may be summarised as follows:-

- Most of the material consists of soil. Very little rock was encountered.
- Most of Materials 3, 4 and 5 consist of stiff to very stiff clay. Parts of Material 2 were observed to be soft to firm.
- The upper parts of Materials 3 and 4 are lateritised. They are mottled red and include ironstone gravels.

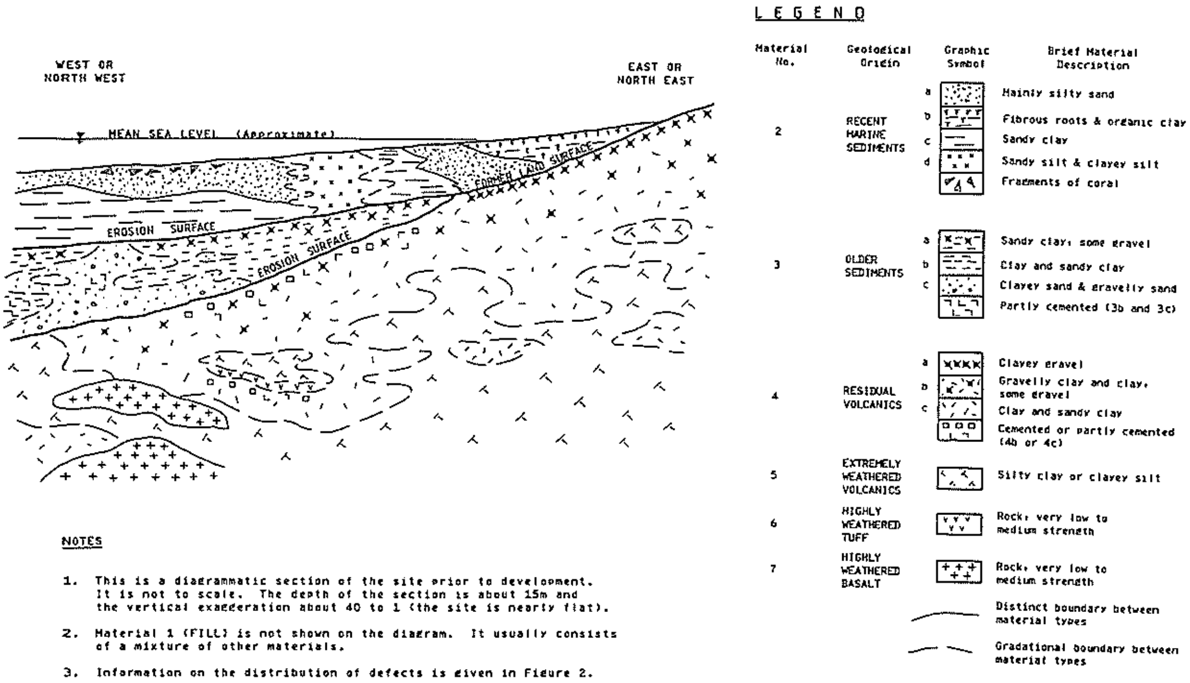

Figure 1 Diagrammatic cross section showing site geology

2.2.2 Defects

In geotechnical engineering, defects may be defined as discontinuities or breaks in the continuity of a material. All rock masses and many soil masses contain defects. Stiff cohesive soils such as occur at Raby Bay almost invariably contain defects. Defects in cohesive soils are sometimes referred to as fissures although there is no consistent definition of fissures in the literature. Sometimes the term is applied specifically to short closely spaced defects with dull surfaces, sometimes to all defects.

Several different types of defects as defined by Stapledon (1970) occur at Raby Bay, but emphasis has been placed on the systematic collection of information on defects with slickensided and/or polished surfaces. As is discussed in Section 3 these defects are the most significant to slope design. Unless stated otherwise, subsequent references to defects will refer to those defects with slickensided and/or polished surfaces.

Information was collected on the following aspects of defects:-

- Occurrence.
- Thickness of layers containing defects.
- Dip and preferred orientation.
- Shape.
- Spacing.
- Intersections.
- Length and effective continuity.

Defects do not occur in all materials and where they do occur they tend to be confined to layers which are typically 1 to 2m thick. Most defects dip between 30° and 50° and do not show a preferred orientation. Flatter defects occur at greater depths. Most dips are curved or undulose although some planar defects occur. Defect spacing varies but is typically 0.1 to 1m. In most areas few defect intersections were observed but in some local areas where defects were closely spaced there are many intersections. Defects are generally less than 1m in length.

The concept of the Effective Continuity Assessment (ECA) was developed after observations in test pits and construction excavations indicated that defects are not continuous and that flatter defects are often shorter than the steeper defects. The ECA is defined as an assessment of the continuity of defects at the lower angle in the dip range of a particular defect set. It is expressed as a percentage of the maximum length possible at this angle across the thickness of the layer in which the defect set occurs.

A schematic section showing the distribution of defects is given in Figure 2. The defects were not uniformly distributed across the site. Further from the original shore defects are less common, occur in deeper and thinner layers, and are more widely spaced.

2.2.3 Origin of defects

The pattern of defects observed at Raby Bay is very similar to that reported in South Africa by Williams and Jennings (1977). High horizontal stresses associated with wetting and drying in a semi arid climate was considered to be the most likely origin of the South African defects. Similar conditions would have applied to the Raby Bay area during periods of Pleistocene lower sea levels and probably during the Tertiary when the lateritisation is likely to have occurred. The pattern of defects at Raby Bay appears to differ from the pattern reported in some of the London Clay in Britain (Skempton, Schuster, and Petley, 1969).

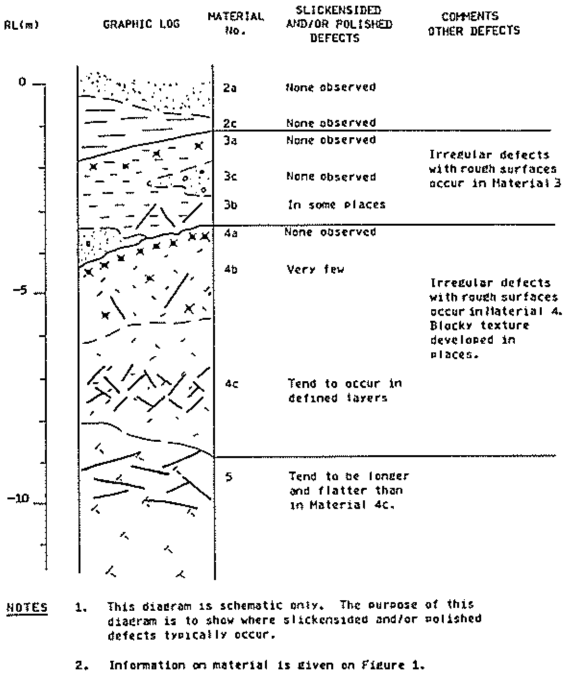

Figure 2 Schematic section showing the distribution of defects

2.3 Material Properties

2.3.1 Introduction

Skempton (1977), Moon (1984) and Burland (1990) discuss the effective strength parameters applicable to the mass strength of stiff fissured clay. They describe the different approaches to the assessment of the strength parameters applicable to the first time failure of largely intact material or material that does not contain extensive slickensided and/or polished defects. Terms which have been used to describe those strength parameters include fully softened, remoulded, post peak and post rupture. At low stress levels the different approaches appear to result in strengths similar to those derived from back analysis. The term fully softened has been used in this paper. If movement occurs on pre-existing shear failure surfaces it is the residual strength which is applicable.

The approach to the assessment of the design effective strength parameters was as follows:-

- To define materials according to their geological origin and engineering properties (Figure 1).
- To use index tests to categorise each material type, assess its variability, and make preliminary predictions of strength parameters based on published and unpublished work on similar soils.
- To carry out effective shear strength testing using methods discussed by Moon (1984) to assess the fully softened strength of the soils and the residual strength of all materials in which slickensided and/or polished defects occur. Test results from previous studies at Raby Bay were used wherever possible.

Back analysis of past slope failures is also a potential method of assessing strength parameters. The usefulness of back analysis at Raby Bay is limited by the fact that all the past failures are "construction failures" in the sense that they occurred after excavation. In most cases fill placement had also occurred. Pore pressure measurements were not available and there is uncertainty about the rate of pore pressure equilibration following unloading (excavation) and loading (fill placement). No major slope failures occurred in parts of the project investigated by the writer but previous back analyses appeared to indicate that mass strengths were less than the fully softened strength in some places.

2.3.2 Test results and strength parameters

The results of the Atterberg Limits tests given in Figure 3 illustrates how the geological origin and material classification may be related to the engineering properties. The test results for Material 5 fall below the A line (diagonal line) which is typical of weathered volcanics. Further weathering produces residual volcanic soil (Material 4c). Such weathering has produced a greater proportion of clay minerals which is reflected by the higher liquid limit and plasticity index. Lateritisation, on the other hand, involves oxidation and a reduction in plasticity (Materials 4a and 4b). Material 3 is a sedimentary cohesive soil and the test results occupy the typical position of just above the A line for such materials. The Recent Marine Sediments (Material 2) are lower plasticity materials because the fine fraction tends to consist of silt rather than clay.

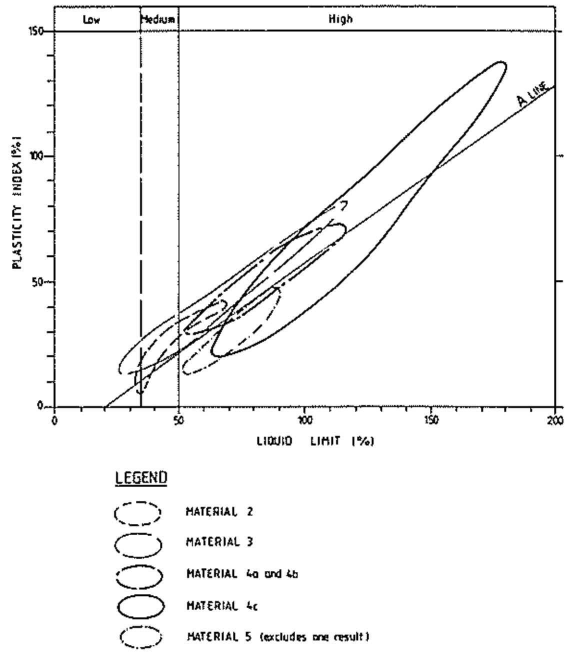

Figure 3 Summary of Atterberg Limits results

The design parameters adopted for stability analysis of the relevant material types are given in Table I.

TABLE I

STRENGTH PARAMETERS ADOPTED FOR STABILITY ANALYSIS

Material No. [1]	Density t/m^3	Fully softened cohesion kPa	Fully softened friction angle	Residual cohesion kPa	Residual friction angle
1 (Fill)	1.9	5	27		
2a	1.8	0	36		
2b (Not assessed, organic material removed)					
2c	1.8	2	29		
2d	1.8	0	33		
3a	2	5	24	2	12
3b	2	5	24	2	12
3c	2	2	30		
4a	2	2	36		
4b	1.7	5	29	2	11
4c	1.7	5	27	2	11
5	1.7	10	26	2	11
6,7 (not assessed, rock)					
G[2]	1.9	5	27	2	11

NOTES:
[1] Brief descriptions of materials are given in Figure 1.
[2] Generalised strength parameters for the entire soil profile.

3. APPROACH TO THE STABILITY ANALYSIS

3.1 Present Practice In Fissured Clay

At Raby Bay the past topography appears to have been subdued and pre-existing slope failures prior to construction are not likely to have occurred. However, on a smaller scale, slickensided and/or polished defects are failure surfaces for which residual strength parameters will apply. Parts of potential large scale failure surfaces may occur along such defects. Present practice in assessing the effective strength parameters for slope design in stiff soils with non-continuous defects appears to include the following approaches:-

1. assume fully softened parameters on the basis of observations that defects are not continuous;

2. assume residual strength parameters in situations where extensive closely spaced defects occur;

3. assume intermediate values between fully softened and residual (or in some cases peak and residual).

The first approach developed for the London Clay (Skempton, 1977) appears to be widely used (Chandler, 1984). Descriptions of defects in London Clay in Skempton, Schuster, and Petley (1969) indicate most defects are short (less than 100mm) are not slickensided and polished, and are flat lying or steeply dipping. For soils where longer, slickensided and/or polished, inclined defects occur the use of fully softened strength may lead to an over-estimate of shear strength.

The second approach is appropriate if slope movement, or any form of shear movement on an extensive scale has taken place and continuous pre-existing failure surfaces exist. This approach is likely to result in an underestimate of shear strength if applied to slopes where continuous, unfavourably oriented, slickensided and/or polished defects do not occur.

The third approach has been used by Thorne (1984) and Macgregor, Olds, and Fell (1990). In both cases intermediate strength values were based on laboratory testing and observations in materials where closely spaced slickensided and polished defects occurred. Intermediate strength values had also been applied at Raby Bay prior to this study because mass strengths lower than fully softened had been indicated by back analysis. In all of these cases a single value had been adopted to apply to the "highly fissured" material.

In this study a generalised model has been developed which provides a method of quantifying a range of strengths intermediate between fully softened and residual which is based on observations of the defects occurring at particular test locations.

3.2 Development Of The Generalised Slope Model

The starting point of the model is the generalised density and fully softened strength given in Table I. These parameters were adopted as being representative of the soil profile where no defects occur. If there are defects, the layer in which the defects occur is referred to as the weak layer. A density of 1.7 t/m^3 was adopted for the weak layer. The strength of the weak layer is assumed to vary between the fully softened value and the residual value according to the proportion of potential failure surfaces which may occur along pre-existing defects. In assessing this proportion the following variables were taken into account:-

- the thickness of the weak layer;
- the dip of the defects;
- the continuity of the defects.

The influence of the thickness and dip were taken into account in a single factor referred to as the Thickness Dip Factor or TDF. The TDF is a measure of the proportion of failure surface that can occur along pre-existing defects within the weak layer assuming the defects are continuous. Geometrical constraints mean that the feasible failure surfaces cannot entirely occur along pre-existing defects unless the defects are near horizontal or the layer containing defects is sufficiently thick. A mathematical expression of this geometrical constraint is given in Figure 4.

Stability analyses indicate that the length of critical failure surfaces in the weak layer is reasonably constant even when the depth and strength of the weak layer is varied. On the assumption that L is constant general TDF charts have been developed for the Raby Bay Project. Figure 5 is a TDF chart for weak layers below the canal floor with defects which do not show a preferred orientation.

The TDF assumes that defects are continuous and thus overestimates the proportion of potential failure surfaces on pre-existing defects. This lack of continuity may be taken into account by multiplying the TDF by the Effective Continuity Assessment (ECA). The product of the TDF and the ECA has been defined as the Strength Reduction Factor (SRF) of the weak layer. It has been assumed that the reduction in effective strength of the weak layer from fully softened to residual is linearly proportional to the SRF. Examples of the SRF are given in Table II.

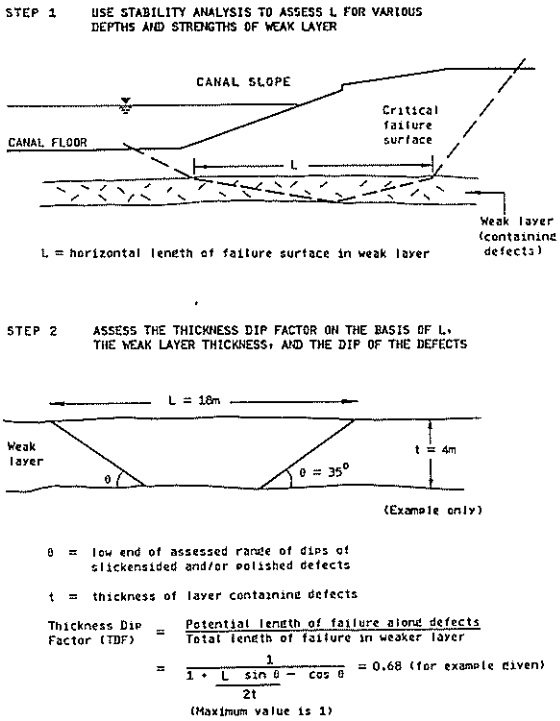

Figure 4 Development and definition of the Thickness Dip Factor

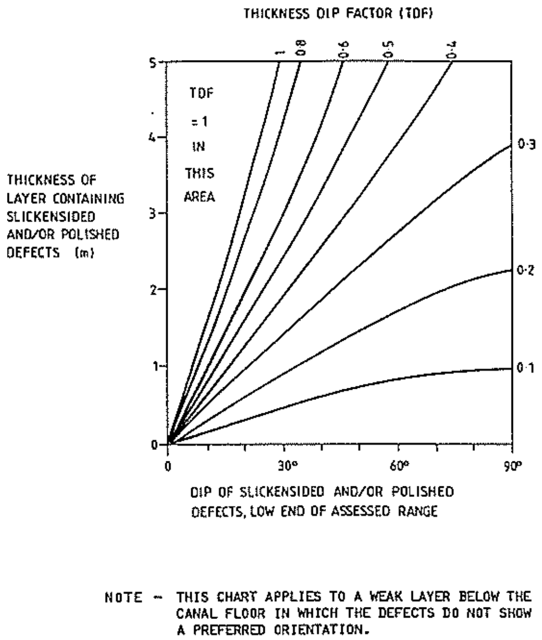

Figure 5 Example of a Thickness Dip Factor chart

TABLE II

EXAMPLES OF THE WEAK LAYER STRENGTH REDUCTION FACTOR

Strength Reduction Factor %	Adopted strength for weak layer cohesion kPa	friction angle[1]	Comment
0	5	27	Fully softened strength
25	4.3	23.3	
50	3.5	19.4	
75	2.8	15.3	
100	2	11	Residual strength

[1] based on a linear relationship between the Strength Reduction Factor and the tangent of the friction angle.

Figure 6 is a stability chart which shows the variation in factor of safety with SRF and depth of the top of the weak layer. Rapid assessment of factors of safety for any combination of weak layer depth and thickness and defect dip and continuity can be made by using the TDF and stability charts.

Although the thickness of the weak layer is taken into account in the TDF (and hence the SRF) the actual stability analyses used to develop the stability chart are based on an assumed defect layer thickness of 1.5m. Observations indicate that this is a typical thickness and sensitivity analyses indicated that factors of safety were not particularly sensitive to increased thicknesses of the weak layer. Increasing the weak layer thickness to 4m reduced the factor of safety by less than 5%.

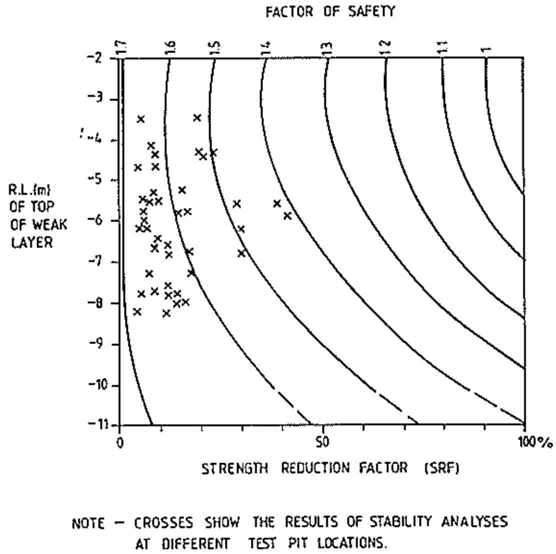

Figure 6 Stability chart showing results for part of the project area

3.3 Method Of Analysis And Design Factor Of Safety

Janbu's Simplified Method was used for the stability analyses. For particular analyses comparisons have been made with other non-circular methods of stability analysis such as Spencer's Method and Morgenstern and Price's Method. Where such comparisons have been made factors of safety have been within 5% of the values obtained by Janbu's Simplified Method.

The stability chart based on the generalised model allows rapid assessment of the approximate factor of safety to be made for particular slope models. The slope models are based on information on the depth and thickness of the layer containing defects and the dip and continuity of the defects from the area concerned. In order to make decisions based on these stability assessments it is necessary to adopt a design factor of safety below which further action is taken. Such further action might consist of one or more of the following activities:-

- carry out individual stability analyses of more complex slope models representative of the area concerned;
- carry out further site investigation in order to provide more information and confirm or revise the geotechnical model;
- assess remedial measures to improve the stability.

A design factor of safety of 1.5 was adopted for the Raby Bay Project based on sensitivity and probabilistic analyses.

4. RESULTS OF STABILITY ANALYSES

The results of the stability assessments based on the generalised model carried out for part of the project area are presented on Figure 6. In this area the comprehensive logging of the sides of the excavations and test pits in the base of the canal provided a more complete picture of the geotechnical model than in the other areas investigated. The crosses on the chart indicate the test pit locations.

The assessment using the generalised model stability chart indicated that most factors of safety were greater than the adopted design value of 1.5 but several marginal or lower values occurred. Individual stability analyses based on the detailed profile at the particular test pit location were carried out for all slopes where the preliminary assessment indicated a factor of safety of less than or close to 1.5. The individual analyses confirmed the marginal values and can be regarded as a check on the validity of the generalised model. The two sets of results compared very well with differences of less than 5% in all cases.

The marginal factors of safety occur at the inland end of the area investigated. The principal reason for this is that flatter defects occur higher in the profile in this area. Some overdeepening of the canal had also contributed to a lower factor of safety. Trial excavations were carried out in order to confirm or revise the geotechnical model and, in particular, reassess the continuity of defects in larger exposures of the weak layer. This resulted in a reduction in the Effective Continuity Assessment (ECA) and no remedial measures were required. If remedial measures had been required, restoring the canal depth to RL -3.8m and limited replacement of the weak layer in critical areas with granular material were the options considered.

5. DISCUSSION AND CONCLUSIONS

This paper presents a quantitative approach to the assessment of the effect of slickensided and/or polished defects on the effective shear strength of soil for the Raby Bay Project. Decision-making using this approach is based on careful observation of the distribution and character of defects (the structural geology) of the soil. Generalised descriptions such as "fissured" are of limited use in slope design for soil masses in the same way as the description of a rock mass as "jointed" is inadequate for rock slope design. In both cases the starting point for slope design should be an adequate understanding and description of the structural geology using clearly defined terms.

As far as geotechnical risk is concerned it is considered that the risk of overconservatism in the Raby Bay slope design has been reduced by the improved understanding of the site geology. The risk of unexpectedly adverse conditions is also likely to be lessened by an approach which is dependent on detailed observations and construction monitoring.

6. ACKNOWLEDGMENTS

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