Shear Strength Analysis for Rock Slope Stability in the Brisbane City Area

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ABSTRACT: Slope failure of road rock cuttings in the Brisbane City area occur infrequently and to ensure safety to the public the Brisbane City Council launched a slope stability assessment of failed slopes. The present study was to determine the shear strength parameters of the underlying rocks. The main rock types present are phyllites, and meta-sedimentary units. Two approaches were taken for shear strength estimation and they were, a) by back analysing existing failures and b) by predicting using an empirical shear strength model for analysing the failure. These values so obtained were compared with laboratory shear strength values for the rock types obtained during past site investigations. The paper highlights the consistent underestimation of the shear strength parameters obtained from direct shear testing compared with the strengths estimated during failure. It also highlights the variability of parameters due to the anisotropy and special variability of the failure plane strength properties.

1 INTRODUCTION

In populated urban environments such as the greater Brisbane city, safety of road rock cuttings from failure is the responsibility of the municipal authorities. The Brisbane City Council initiated a slope stability investigation in the early 1990’s and as a part of the investigation, discontinuity shear strength of the suite of rocks present was studied and the results are presented in this paper. Brisbane is the capital city of the state of Queensland, situated in the mid-eastern coast of Australia. It has a population close to 2 million and covers an area of about 1220 km^2. The road network in Brisbane is extensive and considered generally to be of high standard. Typically a shared path (footpath or bikeway) is located on the left hand side of the road. Where the road passes through a cutting, the shared path is therefore located at the toe of the rock cutting. Consequently if a slope failure occurs, it could potentially cause injury, damage to property, disruption to the city and at worse cause even fatality. Rock slope failures are frequent, and occur at the rate of about two or three per year in Brisbane, and a few more in wetter years. If a section of a road cutting fails, while remedial measures are carried out promptly, the failure is also analysed, to gather information to assess the stability of the remainder of the slope. To successfully transfer the knowledge from the failure to its surrounds it is necessary to obtain knowledge of applicable joint shear strength properties.

The two main meta-sedimentary rock types that underlie the Brisbane area, i.e., Neranleigh - Fernvale Beds and Bunya Phyllite are shown on Fig. 1. These rocks have been subjected to three major deformations and a minor kinking episode. The middle section comprises Bunya Phyllite and the structures of these rocks are dominated by a second-generation transposition layering. Neranleigh Fernvale beds lie on either side of the Bunya Phyllite, and they are dominated by first-generation transposition structures. The rocks are typically present in a northwest to southeast trending band crossing the central business district and the Brisbane River. The mountainous parts of the city are underlain by these rocks where a majority of road cuttings are found.

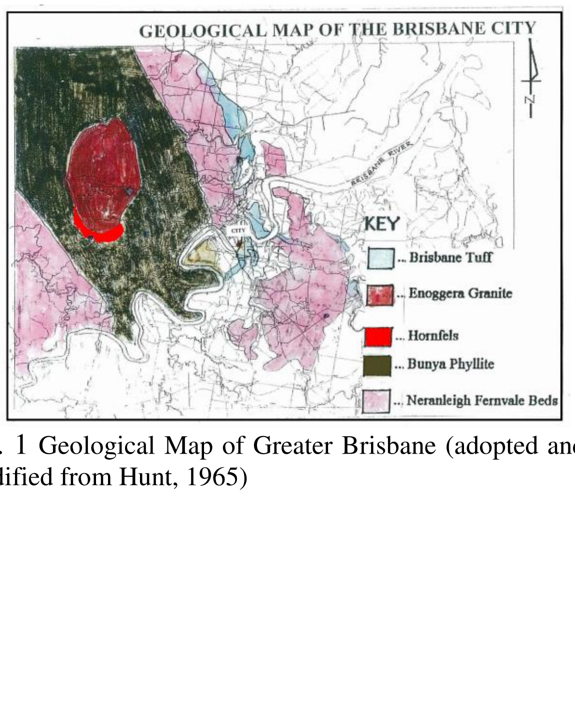

Fig. 1 Geological Map of Greater Brisbane (adopted and modified from Hunt, 1965)

2 METHOD OF RESEARCH

The following methods were used to source or derive and compare rock discontinuity shear strengths in Brisbane:

a) Gathering shear strength results from reports of past site investigations done for construction work in the city.

b) Studying existing slope failures and back-analysing to determine shear strength.

c) Measuring and testing above (back-analysed) failure scars to make shear strength predictions using an empirical shear strength model.

2.1 Back Analysis

By a method of back-analysis of failures it was possible to narrow down the range of discontinuity strength parameters likely to have been present just prior to failure, i.e. when the factor of safety was approaching unity. The best suite of parameters was obtained after many trial computations. For the present research the method of analysis proposed by Sarma (1979) was used. A few of the back-analyses were cross-checked by using the stability analysis program 'Galena' coded by BHP in 1994.

Data for analysis form existing rock slope failures was obtained by means of geological mapping, measuring the face and scar geometry, together with measuring the properties of minor and major structural defects. Further, in order to eliminate any bias when recording discontinuity properties during mapping, a discontinuity survey was undertaken. In slopes where there were scars of failures they were mapped in detail using the same procedures adopted for face mapping. Further, following the methods for rock characterisation and monitoring proposed by the International Society for Rock Mechanics (1981), roughness of the failure planes were recorded with the aim of deriving a Joint Roughness Coefficient (JRC).

The unconfined compressive strength of a material is known to influence the magnitude of its shear strength. Likewise the shear strength of a rock joint is directly proportional to the compressive strength of the joint wall. Due to preferential weathering of a joint however the joint wall compressive strength is not normally the same as the unconfined compressive strength of the surrounding rock, often 3 to 4 orders of magnitude less (Barton, 1976). Contrary to the foregoing, there are instances where a thin crust of leached material deposited on the joint surface yield a higher compressive strength than the failure slope rock. An example of the above is the thin limonite encrustation that is often found on the Brisbane Tuff joints; has a higher compressive strength than the tuff, in turn providing a higher than normal joint shear strength.

To estimate the joint compressive strengths (JCS) a Schmidt hammer (L type) with an accompanying conversion table and graph together with the respective rock density values were used.

2.2 Shear Strength Prediction

All shear strength prediction models in use are based on empirical methods. The present study is based on Nicholas Barton's Shear Strength Criterion (1974); it has been simple in its development and application. For field applications, Bandis (1994) has shown that large scale undulations on the joint needs to be added as an angular value and the following modified form of the equation is used:

$$\tau = \sigma_n \tan\left[JRC \log_{10}\left(\frac{JCS}{-}\right) + \emptyset_r + i_u\right]$$

where τ = peak shear strength;
σ_n = effective normal stress;
JRC = Joint Roughness Coefficient;
JCS = Joint (wall) Compressive Strength;
ϕ_r = residual (ultimate) friction angle.
i_u = waviness angle

Barton's criterion is based on three index parameters, i.e. JCS, JRC and ϕ_r. During the formulation of the failure criterion, Barton observed that the typical shear strength envelope produced by plotting shear strength as a function of normal stress was curved. He, however observed that the curvature was particularly apparent for shear strengths at low normal stress ranges, ie 0.1 to 2.0 MPa. The range corresponds to stress levels produced on a failure plane by a vertical height of rock ranging from 5 to 100 meters (assuming the density of rock to be 20 kN/m^3, without any water pressure) above the plane. The criterion, therefore, was applicable to the rock cut slopes of Brisbane City area as the maximum height of rock cut slopes are of the order of 25 meters.

3 SHEAR STRENGTH PROPERTIES OF MAIN ROCK TYPES

3.1 Bunya Phyllite low grade metamorphic rock

The Bunya Phyllite has not been tested as frequently as the Neranleigh Fernvale rock found within the Brisbane central business district and test results are scarce. James (1993), in a study of case histories, has reported of a large block slide in the Bunya Phyllite at Chapel Hill in the early

1970's. The failure surface was planar dipping between 35° and 40°. Direct shear testing of this failure plane has indicated a joint friction angle of

The group comprises Argillite; a dark grey to black, very fine grained and foliated rock, Metagreywacke; a slightly metamorphosed, recrystallised fine to coarse grained sediment, usually grey to dark grey, and of high strength when fresh, and Quartzite and Chert.

34° with a small value obtained for cohesion.

A plane failure located on Samford Rd, north of Brisbane central business district measuring 18 meters wide and 10 meters high was back-analysed. The material texture and roughness of the failure plane was homogeneous for the most part. Analyses were carried out for both, a drained slope and one with a phreatic surface. The inferred strengths at failure were ϕ of 43 ° with a corresponding 0 kPa cohesion. A groundwater table starting from a filled tension crack was considered for analysis for the undrained condition giving a ϕ of 43° for a corresponding c of 0 kPa at failure.

By measuring the failure plane roughness and the compressive strength; refer to Table 1, and

Table 1. Samford Road slope failure discontinuity parameters (Amarasekera 1996)

Av Schmidt rebound number	Joint compression strength (JCS) kPa	Asperity angle i	Joint Roughness coefficient (JRC)	ϕr	Waviness angle i_u
60.8	48,000	9.6°	4.1	28°	4°

applying an estimated maximum effective normal stress on the plane of 70 kPa, Barton's non-linear criterion provided predicted shearing angle ϕ of 46 ° for an apparent ***c*** of 12 kPa.

Shear strength results obtained from the failure criterion are a slightly lower compared with values obtained by back analysis of slides. The higher shear strength values obtained by back analysis could be attributed to the scale effects of sampling. The actual large failure surface influences the back analysed result compared to the smaller section of the surface sampled for joint roughness measurements used in the criterion.

Measurements input into the Barton criterion are surface roughness and joint compressive strength measured at the points of sampling. The criterion does not account for factors such as the small increases in strength due to the micro shearing through weathered intact rock bridging the adjacent foliation laminae transecting the failure plane or a decrease in strength due to soil infilling on the sliding plane.

3.2 Neranleigh Fernvale group of low grade metamorphic rocks

A wide range of joint friction angles are evident from the results of geotechnical investigations in the centre of Brisbane. Factors influencing the angle may include joint roughness, joint compressive strengths, and infilling material. From these tests, φ values of between 25º and 30º for the general clean discontinuities, and an φ of 15º - 18º for residual conditions were obtained.

A plane failure in Metagreywacke in a cutting on the Mt. Glorious road in the northern western part of the research area was considered for this study. The c - φ plots obtained by back-analysis of failure showed that for drained conditions, the maximum φ obtained was 58º corresponding with 0 kPa cohesion while for a small apparent (hypothetical) cohesion say of 10 kPa the corresponding φ value was 53º. These values are well in excess of the laboratory shear strength values obtained for joint planes mentioned earlier. The measured asperity angle for the failure plane lies in the order of 13º - 14º. Following Patton's (1966) model and incorporating waviness to the laboratory shearing angle, the net result obtained was φ ranging from 48º - 53º. The modified laboratory values are lower than the back-analysed values of 53º - 58º for drained conditions. However for wet condition the analysis gave a value of 60º - 63º, which is an increase of between 5 to 10% above drained conditions.

The effective normal stress level at the failure plane was estimated to be in the order of 40 kPa. A tangent constructed to the logarithmic strength curve derived from Barton’s shear strength criterion for the aforementioned normal stress gives a friction angle of 50° and a corresponding instantaneous cohesion intercept 5 kPa. Further, a tangent constructed at a normal stress level of say 100 kPa results in a ϕ value of 47° and a ***c*** of 15 kPa.

The ϕ value predicted is lower than the back analysed value by about 9 °. From a tangent constructed to the curve in Fig. 2, it can be seen that a ϕ value of 49° will correspond with cohesion of 12 kPa, values which do appear reasonable for the failure analysed. As discussed earlier the cohesive resistance could arise from the micro-rupture of weathered rock material bridging the adjacent thin foliation laminae and suction from the smear of clay present on the failure surface.

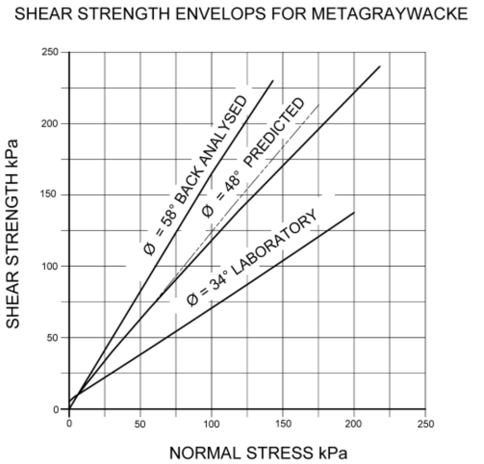

Fig. 2 Shear strength envelopes by three methods of analysis for Metagreywake

4 CONCLUSIONS

Results indicate the variability of the shear strength parameter values derived from the three different methods of analysis and measurement. Joint Roughness of the entire failure plane must influence the back analysed result compared with the measured roughness of a small sample of the plane to apply in the failure criterion. . Further rock masses are anisotropic and heterogeneous in nature because of the inherent spatial variability in composition texture weathering and stress conditions this variability influences the engineering properties of discontinuities. In conclusion no two-failure planes of the same rock type would bear the same shear strength properties. It would appear that successful prediction will rely more on the predictor's experience and ability to choose parameters, and less on the complexity of the analysis used.

ACKNOWLEDGEMENT

The research was supported by the Brisbane City Council and the author would like to thank Dr Peter James and Mr David Stewart, formerly of City Design, for their guidance and assistance in conducting this research.

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