AN ENGINEERING ASSESSMENT OF THE STRENGTH AND DEFORMATION PROPERTIES OF BRISBANE ROCKS

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ABSTRACT
The use of standard relationships between point load index (I₅₀), Uniaxial Compressive Strength (UCS) and elastic modulus values is commonly used within the geotechnical engineering community. The validity of these relationships is examined using data from several major projects carried out in Brisbane recently. Additionally, the methods of Point Load Index testing is examined, particularly the impact of the direction of testing. It is established that in extremely low to low strength rocks, axial tests are likely to produce a higher point load index value. Based on our data set, the use of the standard multiplier of 24 (Broch and Franklin, 1972) to obtain UCS from I₅₀ values is shown to be unconservative for several rock types. In terms of modulus values, in comparison with published recommendations (e.g. Deere, 1968), the established ratios are generally within the 100-500 times UCS ratio quoted, however the value for the DW metasediments is considerably higher.

1 INTRODUCTION
The Geology of Brisbane and its surroundings are varied and each rock type exhibits distinct engineering properties. The economics of site investigation results in the quick and cheap tests being predominantly used. Correlations with more fundamental properties are subsequently made using typical correlation values. The errors in using such typical values for the Brisbane rocks are shown to be on the unsafe side for strength analysis, and overly conservative for deformation analysis. The state of practice in the region is briefly discussed.

This paper presents a summary of the results of various route investigations carried out in the Brisbane region and surrounds. Data from the South East Transit Project (SETP), Sections 1 and 2 – a 4 km length roadway; the Inner Northern Busway (INB) - a 4.5 km length roadway and the Sewer Tunnel (S1) – a 5 km length sewer are used to show the wide variation of correlations that apply for the various types of rock in various cross-sections of the Brisbane region. These projects provided a significant amount of Uniaxial Compressive Strength (UCS) and modulus testing. This represents the exception rather than the rule, as typical geotechnical investigations rely on the point load tests to derive any numerical data for the rock properties, even in the very low strength ranges where such tests are not supposed to be applicable. Even then, many geotechnical consultants rely on their "local" experience in their engineering assessment of parameters irrespective of test results.

Laboratory test results are used to show the relationships between the point load tests and the UCS tests on rock and the elastic modulus values. The aim was to provide guidance on design parameters for various tunnels and bridge foundations, and has implications for the strengths adopted in the rock mass rating system. The sensitivity of the strength parameters to methods of testing and the statistical variation of various Brisbane rock types are discussed.

2 DESCRIPTION OF PROJECTS
The South East Transit Project (SETP) is a dedicated busway from Brisbane City to Logan City. As part of the geotechnical investigations performed on Sections 1 and 2 of the SETP, we specified that one UCS test be carried out for every five I₅₀ tests. Sections 1 and 2 are within 5 km from the Brisbane GPO.

The S1 Sewer is a major sewer in the S1 Sewer Catchment within Brisbane City. The route traverses the area from North Quay in the city and proceeds eastwards to the Breakfast Creek area, and is described in Stewart and Waters (1996). The site is also generally within 5 km from the Brisbane GPO.

The Inner Northern Busway (INB) is a dedicated busway consisting of a series of bus stations positioned between the existing Queens Street Station and the Royal Brisbane Complex (RBH) at Herston. Table 1 provides a summary of the various suburbs and the geological units that the routes traverse.
The rocks intersected in these projects are volcanics of the Brisbane Tuff formation and metasediments of Bunya Phyllite and Neranleigh Fernvale Stratigraphic Units. The rock strength and deformation data was obtained from rock cores that varied from distinctly weathered (DW) to fresh (Fr).

### 3 TESTING PROCEDURES

The testing for the three major projects was carried out by various government agencies, three different Universities (two outside Queensland) and a geotechnical testing laboratory. The reporting and information detail varied considerably and a brief overview is presented herein on the “standards” adopted. These six agencies are referred to as either University or Non-University.

Typically the $I_{s}(50)$ is related to the Uniaxial Compressive Strength (UCS) of the rock by a multiplying factor of 24 (Broch and Franklin, 1972). The use of this relationship with respect to rocks in the Brisbane area was examined using the test data available from these major projects described, with testing by different laboratories.

Bieniawski (1989) in his rock mass rating system prefers the UCS test where $I_{s}(50)$ tests results are less than 1 MPa, that is, in the very low to medium strength range. Geoguide 3 (1988) has the $I_{s}(50)$ generally not applicable for values below 0.2 MPa for granitic and volcanic rocks. The general practice for geotechnical consultants in Brisbane is for the UCS tests not to be carried out unless the test is specifically requested. The validity of such a practice is investigated herein as the UCS data from the above projects represents the exception rather the general rule of testing.

#### Strength Test

The point load index test ($I_{s}(50)$) is one of the most common tests in rock mechanics. Australian Standard (AS1726 – 1993) uses the $I_{s}(50)$ scale as the basis for assessment of the strength of the rock material. The strength of the rock mass would be considerably weaker due to the effect of rock defects.

The $I_{s}(50)$ test data from the University laboratories generally covered both diametral and axial testing of the rocks. This is generally consistent with the procedures outlined in Australian Standard 4133.4.1, which states that rock that is bedded, schistose or otherwise shows observable anisotropy should be tested in both weakest and strongest directions. Two of the Non-University laboratories adopt the “commercial” approach which involves testing by either axial or diametral directions depending on the foliation, but not both tests. One was even more commercial and (sometimes) tested only the axial direction irrespective of foliation.

The test should be rejected if the fracture intersects a bedding plane or if excessive crushing occurs. It is assumed that this had been carried out in the testing such that the reported test results do not reflect the above bias. One agency described the failure mode, i.e. whether failure occurred along a pre-existing defect or failure through the intact rock.

For the UCS test, two of the Non-University and one of the University laboratories did not indicate the mode of failure. The others indicated various modes of failures, but varied in the detail of the descriptions and were not always comparable with each other e.g. one laboratory refers to axial cleavage while the other refers to vertical failure. One of the Non-University laboratories (sometimes) indicated the angle of failure to the vertical, but was descriptive in other reports. It is readily apparent that the reporting standards varied for the testing agencies. The rational is simple – the test is not performed to a fixed standard and the test is subject to the technician’s interpretation of useful information.

Assessment of design capacities would be based on converting the laboratory results to appropriate field values based on weathering/defect spacing or RQD. Bowles, (1996) and NCHRP (1991) for example outline such methods for assessment of the base resistance and side resistance of rock sockets.
Deformation Tests
The determination of the modulus of elasticity (Young’s modulus) for rock is an important part of many analyses. This is especially relevant in settlement sensitive structures and finite element analysis. This parameter is even more relevant with present limit codes where serviceability criteria forms an explicit consideration, rather than previously when movement considerations were implicit only by having large factors of safety.

Despite the significance of this property, the predominant method of assignment has been to use “typical” values reported in the literature in the absence of reported relationships and case studies for the south-east Queensland region. As a result, the potential dangers appear to be either the use of a highly conservative value, or an inaccuracy resulting in a potentially deficit structure. This paper presents preliminary research into the determination of the Young’s modulus for Brisbane rocks and attempts to establish a better substantiated correlation between a more prevalent test, the Unconfined Compressive Strength test and Young’s modulus.

Figure 1 shows the Deere-Millar graph for modulus ratios for quartzite, gneiss, marble and schist (here from Pells, 1993). The modulus ratio value relate to the tangent modulus at 50 % of the unconfined compressive strength.

Figure 1: Deere – Miller Graph

It is generally assumed that laboratory testing is carried out on fresh, hard rock fragments which exhibit elastic behaviour. As a result, an intact Young’s modulus (E_i) and Poisson’s ratio are obtained. In order to produce a design value ie to account for cracking, faulting, shearing and bedding, then a reduction factor is required to reduce the elastic constants to an insitu value (E_r). This correlation factor may be either based on the RQD, fracture frequency or velocity index of the rock (Hobbs, 1974, NCHRP, 1991). Modulus values may also be obtained from relationships with rock mass rating (RMR) or rock quality (Q) values.

As an alternative, field testing such as pressure plate or pressuremeter testing may be carried out at the location of the structure, this obviously provides the most accurate value that could be obtained. Unfortunately, budget limitations on a project often rules out these preferable test methods for the majority of engineering projects.

4 POINT LOAD TESTING IN THE AXIAL & DIAMETRAL DIRECTIONS
This section assesses the possible effect of reported test results and the design implications. From the onset, any questionable practice issues raised should be viewed as deficiencies in standards/budgetary constraints/the system of selection of testing agencies/geotechnical consultants and/or the education system that needs to provide a more rational approach.

A case study on the effect of diametral and axial directions for the point load index test was examined. During one of the projects, the test results of one of the Geotechnical laboratories (Lab A) (for the detailed design) was noted to be markedly different from another laboratory (Lab B) test results who had tested nearby (during the planning preliminary design phase 1 year earlier). However, this was accepted “as is “ and the usual geotechnical explanation applied i.e.
ground variation occurring locally. During the tabulation of data and write up for this paper, the cause for the variation was evident.

Table 2 shows the spread of results for the low $I_0(50)$ values. The Lab B carried out 80% of the $I_0(50)$ testing in the diametral directions while Lab A carried out all tests in the axial direction. The result was that Lab A thought that the rock was weaker than earlier indicated. The question of which is “correct” needed to be examined further i.e. how important is the test direction for the various Brisbane rock types.

Table 2: Summary of Point Load Index Tests results with weathering grade (SETP)

<table>
<thead>
<tr>
<th>SETP</th>
<th>% of tests for DW/SW Phyllite</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$I_0(50) &lt; 1.0MPa$</td>
</tr>
<tr>
<td>Lab A – 2 BHs</td>
<td>97%</td>
</tr>
<tr>
<td>Lab B - 2 BHs</td>
<td>70%</td>
</tr>
<tr>
<td></td>
<td>$I_0(50) &lt; 0.2MPa$</td>
</tr>
<tr>
<td>Lab A – 2 BHs</td>
<td>40%</td>
</tr>
<tr>
<td>Lab B - 2 BHs</td>
<td>5%</td>
</tr>
</tbody>
</table>

Figures 2 to 5 show the evaluation of the axial/diametral effect for various Brisbane rocks. This tabulation was for values within 100 mm of each other. Statistical analysis was then carried out for the groupings of argillite / metagreywacke, phyllite / arenite and tuff. Figure 2 shows the results for the argillite and metagreywacke. The high regression coefficient is indicative of the effect of the direction of testing and shows that values in the low to extremely low strength range can expect to have an axial/diametral ratio of 2 or greater. Figure 3 shows the result for the arenite and phyllite and indicates that the axial is expected to be 10 times the diametral value in the low strength range. However, the correlation is not as strong although a larger sample was obtained. Similar trends as Figure 2 is noted with the tendency for the high strength values to be less sensitive to test direction.

Figure 2: Comparison of Axial/Diametral ratio with strength for argillite and metagreywacke
Figure 3: Comparison of Axial/Diametral ratio with strength for phyllite and arenite.

Figure 4 graphs the tuff results and indicates that this material is similar to the argillite and metagreywacke in that the axial value can be expected to be a factor of two times the diametral value in the low strength.

Figure 5 provides a normal distribution analysis of the samples evaluated. The tuff has the least coefficient of variation for the axial/diametral ratio while the phyllite/arenite grouping shows considerable spread. This highlights the errors and variation inherent in carrying out point load index tests in one direction only. This is especially critical for the phyllites, where the mean axial strength can be a factor of 4.4 times higher than the weaker diametral direction.

The analysis is indicative only as it uses a normal distribution for comparison purposes, although the distribution has varying skews for the tabulated data. A Weibul Distribution applies rather than a normal distribution but the latter is used here. Hence the negative values of the distribution are not applicable. A more rigorous analysis would be required.
to take into account the above effects and to shift the distribution to values above 0.0 only. In all cases the median was
less than the mean indicating a positive skew.

Figure 5: Axial / diametral point load index ratios for Brisbane Rocks

Figure 5 also shows the effect of removing all values less than 0.2 MPa. The tuff presents negligible change, however
some change occurs with the argillite/metagreywacke (though this represents only a sample of 7 in this case and must
be treated with caution). The phyllite represents the greatest change with a significant reduction in the ratio and its
coefficient of variation.

Thus, the discrepancy highlighted is a reflection of the phyllite at this site and the different methods of testing adopted.
The inference is that for the phyllite, where a large proportion of the I₅₀ tests values are likely to be below 1.0 MPa
even for DW/SW rock, then the use of UCS testing would be more appropriate and/or the use of testing in both
directions. Table 3 provides a summary of the implication of these tests for the test results analysed, and again
highlights the above-mentioned point. Interestingly, this Table is the reverse of the initial problem that was highlighted
in Table 2.

Table 3: Summary of axial / diametral Point Load Index Test ratios for various rock types

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Point Load Index Strength Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Extremely Low</td>
</tr>
<tr>
<td>Argillite / Metagreywacke</td>
<td>Axial &gt; Diametral ?? (1)</td>
</tr>
<tr>
<td>Phyllite / Arenite</td>
<td>Axial &gt;&gt; Diametral</td>
</tr>
<tr>
<td>Tuff</td>
<td>Axial &gt; Diametral ?? (2)</td>
</tr>
</tbody>
</table>

Notes:  (1) - Limited test data and inferred from trend lines only – only one data point was actually in this region.
(2) - From trend line only as this rock tested in the high strength ranges.

Table 3 provides a summary of the implication of these tests for the test results analysed, and again
highlights the above-mentioned point. Interestingly, this Table is the reverse of the initial problem that was highlighted
in Table 2.

In addition these results suggests that the axial values are likely to produce a higher point load index value in the
extremely low to low strength ranges. In the high to extremely high strength ranges the axial and diametral test results
will be approximately similar. While the higher value may be indicative of the problems on excavation characteristics,
the lesser value would be more applicable for the assessment of load bearing capacity.

Having noted that the axial test is likely to be greater than the diametral in the low strength ranges then the unresolved
question is which one is likely to be more appropriate in assessment of the strength. This is discussed in Section 5.
5 STRENGTH TEST DATA

Figure 6 shows the 167 points tabulated for the three projects for all the rock types investigated. It is readily apparent from the spread of results that no one UCS / $L_1(50)$ relationship is evident for all the various rock types, and certainly a multiplying factor of 24 is not valid.

![Graph showing UCS vs. PLI for all 3 projects](attachment:Graph.png)

The mode of failure was observed to have an effect on the test results. The different modes of failure can be categorised as follows:
- no information provided by the testing laboratory
- failure on a shear plane
- failure on a bedding plane or parallel to a bedding plane
- failure in an axial or vertical mode

However, insufficient data points were available with the descriptions and this analysis was not pursued to its natural conclusion. Instead the rock data was analysed in terms of its test direction, effect of the outliers and low points, and the degree of weathering (Distinctly Weathered (DW); Slightly Weathered (SW); and Fresh (Fr)).

The rock type groupings were based on the similarity of origin and to some extent on the available data to ensure sufficient quantity was available for analysis. Conglomerate and basalt rock was also tested, but not enough data was available to pursue any analysis of those rock types.

**Phyllites and Arenites**

Figure 7 shows the phyllites and arenites (combined data) and clearly shows the 24 ratio would be inappropriate in this instance. The point load index tests below 1 MPa show considerable variability.

The phyllites and arenites show a regression coefficient of 0.1 and 0.5 for the SW/Fr and DW rock weathering grades, respectively (Figure 9). One would have expected the less weathered rock to have a better correlation, but this proved not to be the case. Considering the effect of testing in the axial and diametral direction the former showed a better correlation coefficient, but this relationship was still considered weak (Figure 8).
The scatter of points were evident and the low point load values (< 0.2 MPa) and the self evident outlier points were removed (Figure 9). The strength of the relationship was markedly improved as can be expected, with the axial and diametral both showing similar trend relationships. However, the axial tests produced the stronger relationship. Overall therefore these trends suggests that a UCS/\textsubscript{I}_\textsubscript{50} ratio of 5 would be more appropriate for these Brisbane rocks.

A few general observations are also made from other graphs plotted (but not shown herein) although this was from the limited pool of data in those categories. Data described by Lab B show the points failing on a shear plane generally have a lower value. The points from Lab A, seem to trend in the shear plane mode of failure where the lower UCS/\textsubscript{I}_\textsubscript{50} ratio applies although Lab A did not describe the mode of failure. The data also suggests that for the quartz arenite the 24 multiplier may be appropriate for non bedded material but 5 for the bedded material. The distinction in the relationship for "bedded" and non bedded failure was evident.
Figure 9: Phyllites and Arenites rock strength relationships for test direction with low points and outliers removed

**Metagreywacke and Argillites**

Figures 10 to 12 show the argillite and metagreywacke data and similar comments as the phyllites also apply. Figure 10 shows that no meaningful relationship exists for the SW and Fr weathered rock for the data tabulated. No data was available for the DW weathering grade. The quantity of data was very small and no linear relationship was evident for the test direction. However once the low values and outliers were removed the axial test direction provided a good correlation. The UCS/\(I_t(50)\) ratio is slightly higher than the phyllites, but the 24 ratio again does not apply for these Brisbane rocks.
Argillite / Metagreywacke rock strength relationships for various weathering

Figure 10: Argillite and Metagreywacke rock strength relationships for various weathering

Argillite / Metagreywacke rock strength relationships for test direction with outlier and low values removed

Figure 11: Argillite and Metagreywacke rock strength relationships for test direction

Argillite / Metagreywacke rock strength relationships for test direction with outlier and low values removed

Figure 12: Argillite and Metagreywacke rock strength relationships for test direction with outlier and low values removed

Tuff

Figure 13 shows the tuff data tending towards the “normal” ratio of 24 but that ratio is still a high value for this material with actual values of 16 and 18 applying for the DW and SW/Fr weathering grades respectively. More importantly there seems to be a good correlation as compared with the other Brisbane rocks described previously. The effect of test direction in the axial and diametral direction is also not as pronounced as for the other rock types. Removal of any outliers had little effect on the relationship or the regression coefficient, and suggests that the point load index values for tuff is likely to be more reliable than the other rock types examined herein.
Figure 13: Brisbane Tuff Rock Strength Relationships for various degrees of weathering

Figure 14: Brisbane Tuff Rock Strength Relationships for axial and diametral effects

Figure 15: Brisbane Tuff Rock Strength Relationships for axial and diametral effects neglecting outliers and low values

On a project specific basis a multiplier of 24 and 18 was obtained for the SETP1 and S1 projects, respectively.
6 DEFORMATION PROPERTIES

Laboratory tests were carried out to determine the modulus for the three projects. This data set was considerably less than the strength data. Other tests such as plate loading and pressuremeter tests were also carried out. However the field tests would be indicative of the *in-situ* rock while the lab test would indicate the intact rock modulus. The field test results have therefore not been included in this analysis.

Test Data

Figures 16 to 18 presents the results of the Youngs Modulus (Ei) versus Unconfined Compressive Strength Testing.

![Graph](#)

**Figure 16: Distinctly Weathered Neranleigh Fernvale Rocks**

**Figure 17: Slightly Weathered Neranleigh Fernvale Rocks**
7 CONCLUSION

The point load test is a quick and cheap laboratory and field indicator strength test. It was developed at a time when it was recognised that strength should be a normal part of all rock core descriptions. Unfortunately, it has increasingly become the most common test to provide rock strength, and in many cases the only rock test undertaken for a project. As can be expected, the variances inherent in the multiplier can make this test impracticable in design terms. However, provided that certain precautions are used in the correlation, it can still be a valuable source of information. These precautions include supplementary UCS testing in addition to the point load tests to provide a practical correlation, also to carry out a statistically significant number of tests, and to use good geological/engineering judgement in its use. This judgement should include consideration of the type of test (axial and/or diametral) in relation to any strength anisotropies. Description of the mode of failure in the laboratory testing is essential.

The point load index test is more an indicator of relative tensile strength than a means of accurately determining the compressive strength of the rock. However, its attractiveness of relative simplicity and cost means that while a large number of such tests may be carried out, it must be supplemented with UCS testing for a more accurate determination of strength.

Use of a 24 multiplier should not be applied unless confirmed by calibrating with actual UCS values. The relationship while approximately valid for the Brisbane Tuff, has been shown to be not applicable for the Brisbane Phyllites, bedded Arenites, Argillites and Greywacke. Recommended strength relationships for Brisbane rocks are provided in Table 4 below.

For comparison, a multiplier of 8 (6 tests) in DW/SW Argillite/Metagreywacke was evaluated for a Gold Coast (Tugun) site. At another site in Springwood a multiplier of 8 was also evaluated for DW Siltstone and Sandstone (12 tests).

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>UCS/Is (50) Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>General (Brisbane Area)</td>
<td>11</td>
</tr>
<tr>
<td>Tuff</td>
<td>18</td>
</tr>
<tr>
<td>Phyllite</td>
<td>5</td>
</tr>
<tr>
<td>Greywacke / Argillite</td>
<td>8</td>
</tr>
<tr>
<td>Quartz Arenite (bedded)</td>
<td>6</td>
</tr>
</tbody>
</table>

Table 4: Summary of Strength Relationships for Brisbane Rock
Based on the data presented, analysis was carried out to determine applicable Ei / UCS ratios. These are presented in Table 5. In comparison with published recommendations (eg Deere, 1968), the ratios are generally within the 100-500 ratio quoted, however the DW metasediments value is considerably higher.

<table>
<thead>
<tr>
<th>Rock Unit</th>
<th>Weathering Grade</th>
<th>Ei/UCS ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Neranleigh Fernvale -</td>
<td>Distinctly</td>
<td>760</td>
</tr>
<tr>
<td>Metasediments</td>
<td>Weathered</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Slightly</td>
<td>330</td>
</tr>
<tr>
<td></td>
<td>Weathered</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fresh</td>
<td>390</td>
</tr>
</tbody>
</table>

The relationship shows that there is some considerable variation in the determination of the modulus ratio.

In terms of engineering design processes, the strength and deformation parameters for rock can play a significant role. Mis-assignation can have severe cost or safety implications. The present method of grabbing a conservative (hopefully) parameter from a text is inappropriate considering the effort and application that is inherent to the remainder of the design process. It is hoped that additional research into the rocks in South East Queensland will address the situation.

Any attempt herein to provide relationships does not diminish the need for appropriate site specific testing. This paper illustrates the trends with simple linear relationships. A more rigorous regression analysis and additional data is required to provide definitive relationships for all the Brisbane rocks.

8 REFERENCES

Rock classification systems for Engineering purposes, ASTM STP 984, Louis Kirkdaal, Ed, ASTM, pp 17-34.
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Structures, pp 579 – 609.