Characteristic modulus values for rock socket design

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ABSTRACT

Various rock socket design procedures rely on the adoption of 'characteristic' modulus parameters to estimate pile settlement. However, significantly different settlement magnitudes can be calculated from the same modulus value applied within various common pile design procedures. Although all common design procedures are derived from comprehensive data by their authors, limited opportunity exists to confirm which procedure is most relevant to any specific region. The Gateway Bridge duplication was the largest bridge project in the history of Queensland, Australia. Two (2) test piles embedded with Osterberg Cells were constructed to investigate the rock socket behaviour under high loads prior to construction of the two (2) river piers. It is the only known local calibration site for the various "universal" rock socket analysis models typically adopted in Queensland. Rather than develop a site specific approach, this paper used the pile load test data to back-calculate the required modulus values to be input into common pile design procedures in order to replicate the observed settlements. It assumed four (4) different pile design procedures were equally valid, but the 'characteristic' modulus required to be adopted by the various approaches may differ. Thus the selection of significantly different 'characteristic' values is shown to be required by each design procedure in order to produce a convergent estimate of pile settlement. In addition, as modulus values are frequently estimated from rock strength information, this study also assessed the 'percentile' of the rock strength dataset that would provide the required 'characteristic' modulus for each considered design procedure.

Keywords: Characteristic design value, modulus, rock socket, Osterberg pile tests, sedimentary rocks

1 INTRODUCTION

Look and Lacey (2013) back-analysed data from two (2) large-scale pile load tests fitted with Osterberg Cells (O-Cell®) completed for the Gateway Upgrade Project (GUP) in Queensland, Australia, and assessed the "characteristic" values of rock strength that were required to be input in a variety of rock socket design methods in order to reproduce the ultimate shaft capacity exhibited by the load tests. However, this 2013 study only considered the rock strength required to be input into established rock socket design methods, and did not consider pile settlement or moduli parameters.

Due to the type of large scale testing completed from which the back analyses have been completed, additional data relating to the rockmass into which the rock socket was installed can also be directly back-calculated. This paper extends the analysis completed by Look and Lacey (2013) and assesses the modulus values for the material comprising the rock socket of the tested piles and, via a number of design methods, evaluates the effect that selection of various "characteristic" rock strength and deformation values has on the resultant calculated pile capacities. Specifically, this paper investigates the range of moduli that can be derived from the available data (e.g. Young's modulus, E; intact rock modulus, E_i or E_R; and rock socket rockmass, E_m) and an assessment of which derived modulus value would be most appropriate for use in conjunction with various pile rock socket design methodologies.

2 STUDY METHODOLOGY

The methodology adopted was to initially estimate E_m parameters based upon available information relating to the condition of the material within the pile rock socket (e.g. rock strength test results, engineering descriptions of rock materials included in borehole logs), as would be typically completed during a project's design phase. Independently, *in situ* rockmass modulus values were back-calculated from the test piles' observed load / deformation curves and, via a number of common rock socket design methods, E_m values were determined. If adopted for design, these E_m values would result in the predicted pile settlements replicating the deformations observed during field tests. These two (2) calculated E_m parameters were then compared to assess how well the range of calculated moduli parameters aligned. Figure 1 conceptually shows the methodology adopted.

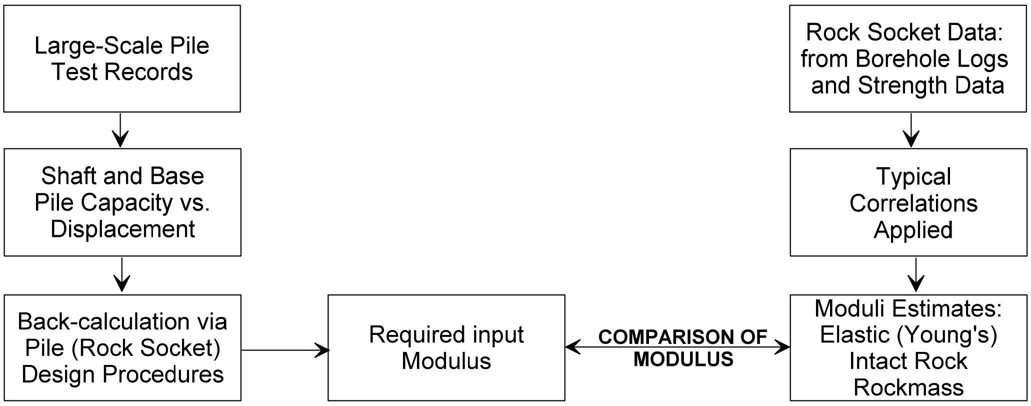


Figure 1. Flowchart showing primary steps adopted for study methodology

3 SITE DETAILS AND GEOLOGICAL SETTING

The GUP site from which the data analysed by this study was obtained was the largest road and bridge infrastructure project ever undertaken in Queensland, Australia. The six (6) lane bridge structure spans 1.6 km between abutments with a main river span structure of 520 metres. The two (2) pile load tests were completed on sacrificial land-based piles constructed to investigate the rock socket behaviour under high loads and identify any constructability issues prior to construction of the two (2) river bridge piers. The key geological features of the founding materials were:

- Basement rock consisted of interbedded layers of Triassic aged (220 to 180mya) sandstone, siltstone, mudstone and low grade coal. The material was not observed to have any significant folding, but is known to have faulting as a consequence of crustal tension in the Tertiary period.
- Deposition of Quaternary Alluvium occurred in the recent past. This site is located close to the mouth of the Brisbane River and generally has Holocene (young) overlying the Pleistocene (older) Alluvium.

Further details regarding the GUP project, foundation design and subsurface material parameters can be found in Look and Wijeyakulasuriya (2009) and Look and Lacey (2013).

4 ROCK SOCKET DATA AND ESTIMATION OF ROCK MODULUS

4.1 Relevant Geotechnical Investigation Data

The geotechnical site investigation completed at the location of each test pile involved the drilling of a borehole to below the depth of the toe of each test pile (denoted TP-01 and TP-02). Both rock sockets were comprised of interbedded mudstone, siltstone and sandstone; moderately to slightly weathered, medium to high strength. Look and Wijeyakulasuriya (2009) and Look and Lacey (2013) further analysed and discussed the rock strength data relevant to each test pile rock socket, and the basic statistical data of the inferred Uniaxial Compressive Strength (UCS) rock strength results has been reproduced in Table 1. Note that these statistical values have been derived from non-normal distribution function fitting as per the recommendations of the identified historical studies (2009, 2013).

Logged material units, descriptors of the *in situ* state of the rockmass (weathering, fracture spacing) and Rock Quality Designation (RQD) values for each rock socket (the length of drilled socket between load cell location and pile toe, refer Section 5) have also been extracted, with relevant data presented in Table 2. Data included in Tables 1 and 2 was considered representative of the overall rockmass of the rock socket, and formed the basis of moduli estimation via generic correlations.

Table 1: Basic non-normal statistical data of rock strength within test pile rock sockets

Test Pile ID	Percentiles of Uniaxial Compressive Strength (UCS) data distribution (MPa)					
	5th	25th	50th (Median)	Average	75th	95th
TP-01	3.3	7.2	14.4	32.1	35.2	201.8
TP-02	14.0	21.3	34.3	58.2	63.2	175.8

Table 2: Selected details regarding rock condition of identified lengths of test pile rock sockets

Test Pile ID	**Socket Geological Makeup**	**Logged Weathering State**	**RQD (%)**	**Average Fracture Spacing (m)**
TP-01	Sandstone (27%) / Mudstone (65%) / Siltstone (8%)	Slightly Weathered (100%)	70	0.25
TP-02	Sandstone (99%) / Mudstone (1%)	Highly Weathered (6%) / Slightly Weathered (94%)	57	0.20

4.2 Direct estimation of *in situ* rock modulus (E_m) values

As identified by previous authors (e.g. Prakoso, 2002) the cost of obtaining *in situ* rockmass moduli is generally prohibitive, and thus E_m values are typically inferred from results of UCS testing. It has also been demonstrated (e.g. Hobbs, 1974; Bieniawski, 1984; O'Neill and Reese, 1999) that the E_m is a reduced value of the intact rock modulus (E_i), with the magnitude of reduction based on site-specific rockmass properties; such as discontinuity spacing, confining stress and rock structure. The use of E_m values for rock socket pile design is considered preferential over the simple adoption of E_i values, which would likely overestimate the stiffness of the deformation response to pile loading.

Various studies suggest adopting a linear transformation, via a modular ratio (*MR*), to estimate E_i values from known rock strength (UCS), and then applying a reduction factor (*j*) to transform E_i to E_m values. The reduction factor can be estimated based on rock quality, as reflected in the RQD or fracture spacing of the rockmass. In such studies, *MR* values are commonly nominated by rock type or material origin. For sedimentary rocks similar to those encountered at the GUP site, previous *MR* estimates range from 150 for weak (mudstone) materials and 275 to 300 for higher strength (sandstone) materials (Hobbs, 1974). Weighting such *MR* values by the rock composition detailed in Table 2 would result in UCS:E_i of 200 and 295 for TP-01 and TP-02's rock sockets respectively.

Previously published reduction factors based on RQD assessments can be used to convert E_i to E_m. With the identification from borehole logs that only tight joints were present within the GUP test pile's sockets, the range of applicable rockmass factors (*j*) for TP-01 (RQD = 70%) ranged between 0.30 and 0.70. For TP-02 (RQD = 57%), *j* values varied between 0.24 and 0.35. In both cases the minimum *j* values were allocated by Bieniawski (1984) correlations, with the maximum value being assigned by O'Neill and Reese (1999) relationship. Resultant ranges of linear q_u:E_m relationships – 60 to 140 for TP-01 and 60 to 105 for TP-02 – were thus produced to provide upper and lower bounds for E_m estimation from rock strength (UCS) data.

Other studies have derived non-linear relationships for the same UCS:E_m transformation. Rowe and Armitage (1987) suggested the adoption of Equation 1, whilst Prakoso (2002) completed a review of 88 case studies and determined the *in situ* rockmass modulus could be best estimated via Equation 2.

$$E_m = 215 \times (q_u)^{0.5} \qquad (1)$$

$$\text{Log}_{10}(E_m / q_u) = 2.73 - 0.49 \times \text{Log}_{10}(q_u / p_a) \qquad (R^2 = 0.48) \qquad (2)$$

Additional material parameters to enhance correlation between rock strength data and E_m values can also be incorporated. This could include the use of the Geological Strength Index (GSI), RQD or Rock Mass Rating (RMR) parameters. *In situ* modulus values derived after an assessment of GSI from the existing geotechnical information was completed, as per Equation 3 (Hoek and Brown, 1997). GSI values applicable to the subsurface conditions of each test pile were determined as 50 for TP-01 and 42 for TP-02, based on the methodology and descriptions provided by Hoek and Brown (1997).

$$E_m\ (\text{GPa}) = (q_u / 100)^{0.5} \times 10^{[(GSI-10)/40]} \qquad (\text{for } q_u \leq 100\ \text{MPa}) \qquad (3)$$

Table 3 summarises the resultant E_m values by adoption of the identified q_u:E_m relationships and the characteristic rock socket data relevant to each of the test piles (from Tables 1 and 2). From this data it is observed that for the same rock strength (q_u) input values, the Prakoso (2002) relationship consistently produced the lowest E_m values, followed by the Rowe and Armitage (1987) relationship. Significantly higher E_m estimates were produced when the UCS values were submitted to the 'upper bound' linear relationship or combined with the test pile's GSI, as per Hoek and Brown (1997).

Table 3: In situ modulus (E_m) inferred by rock strength (UCS) data and generic relationships

UCS:E_m Reference	Pile ID	TP-01				TP-02			
	Distribution Percentile	5th	25th	50th	Ave.	5th	25th	50th	Ave.
	UCS (MPa)	3.3	7.2	14.4	32.1	14.0	21.3	34.3	58.2
Linear q_u:E_m Relationships	Lower Bound	196	429	866	1926	839	1278	2059	3251
	Upper Bound	457	1001	2022	4494	1469	2236	3603	5689
Rowe and Armitage (1987)	(Eq. 1)	388	575	817	1218	804	992	1260	1583
Prakoso (2002)	(Eq. 2)	322	480	688	1034	677	839	1070	1401
Hoek and Brown (1997)	(Eq. 3)	1806	2674	3800	5666	2360	2912	3696	4644

5 RESULTS OF PILE TESTING

5.1 Pile Load Tests

Two (2) full scale test piles (1.5m diameter) with drilled rock sockets were constructed at the GUP site, each fitted with a single O-Cell® to allow controlled pile loading. Each test pile was fitted with encased tell-tale rods and strain sensors, and information regarding the total compressive load applied by the O-Cell® and the associated observed pile displacements was recorded during each loading stage contained within three (3) loading–unloading cycles. Accordingly, as per the methodology detailed by Osterberg (1998), the load components carried by the instrumented pile shaft and pile base could be calculated and correlated with displacement observations (refer Figure 2).

This study has isolated the load components carried by the pile base and the length of pile shaft between the location of the O-Cell® and pile toe. Table 4 details the geometric characteristics relating to this section of rock socket within the drilled shafts of TP-01 and TP-02.

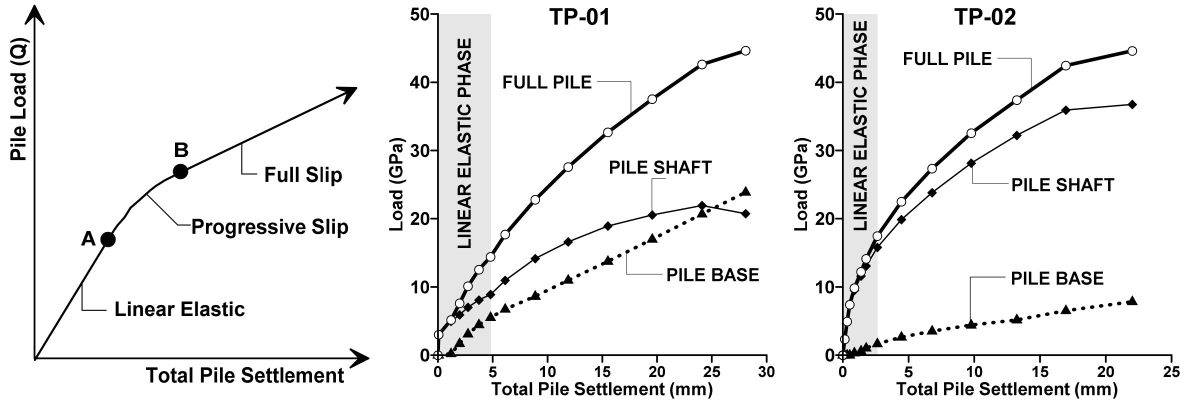
Table 4: Geometric characteristics of isolated section of test pile rock sockets

Pile ID (both 1.5m diameter, D)	TP-01	TP-02
Length of Rock Socket between O-Cell® and pile toe (L, m)	2.66	5.24
Ratio of Rock Socket Length / Pile Diameter (L/D, m)	1.77	3.49

5.2 Results of Pile Load Testing

As per Carter and Kulhawy (1988), and as shown in Figure 2(a), a pile's displacement response to axial load application can be generalised into three components; 'linear elastic', 'progressive slip' and 'full slip' components. The 'linear elastic' phase occurs upon initial loading and continues whilst the pile demonstrates behaviour as if it is fully contained within an elastic half space. Non-linear behaviour is considered to be any load response beyond 'linear-elastic' loading, as defined by Point 'A' in Figure 2(a) (i.e. 'progressive slip' and 'full slip' zones). Figure 2(b) and (c) show the load versus displacement curves for TP-01 and TP-02 respectively, overlaid with interpretations of their 'linear-elastic' and non-linear response phases. By observation, both test piles followed the expected load-deformation response, and both tests were concluded immediately after the pile entered the 'full slip' phase.

The point at which the 'linear-elastic' phase of the pile was exceeded was identified to occur once total pile displacements of approximately 0.32% and 0.17% of the pile diameter (1.5m) were observed for TP-01 and TP-02 respectively. Maximum unit side shear resistance (and 'full slip' of the pile) was observed once total pile displacements were 1.5% (TP-01) and 1.6% (TP-02) of the pile diameter. These displacement values correspond well to those previously published for large scale pile tests (Zhang and Einstein, 1998). For each step of load testing of both test piles, further analysis of the recorded data also allowed the calculation of the pile load distribution within the isolated pile shaft section and pile base, as shown in Figure 3 for TP-01.



(a) *(b)* *(c)*

Figure 2. (a) Idealised pile loading versus displacement behaviour (after Carter and Kulhawy, 1988); (b) load versus deformation curve for TP-01; and load versus deformation curve for TP-02

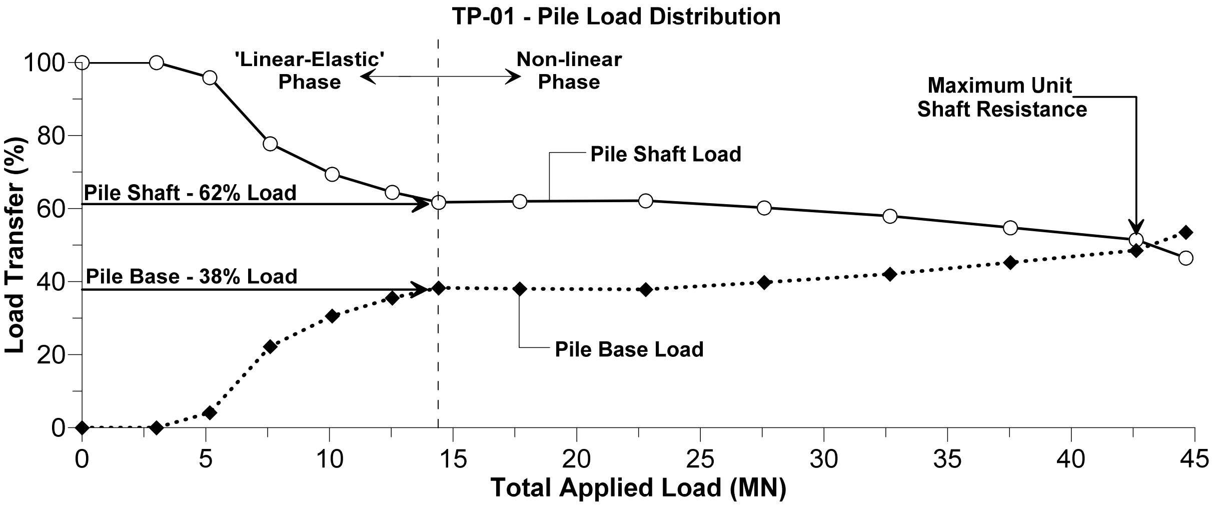


Figure 3. Pile load distribution for TP-01, highlighting area of "elastic" and "full slip" displacement

5.3 Separation of 'linear-elastic' and 'non-linear' phase response data

As identified by Zhang (1997), pile design procedures can be considered to fall into two (2) distinct groups; (a) procedures that provide estimates of pile displacements within the 'linear elastic' phase (i.e. considers the displacement of pile acts fully within an elastic half space); and (b) procedures that provide estimates of pile displacements that incorporate the non-linear movement of the rock socket (i.e. account for material yielding). Carter and Kulhawy (1988) identify that whilst non-linear design methods are only technically applicable to the 'full slip' phase, most practical cases can consider pile behaviour to be bilinear with only 'linear elastic' and 'full slip' linear relationships being definable.

To allow the use of the pile design procedures that assume only an elastic half space, the test pile results were reviewed and limited to the phase of observed 'linear-elastic' behaviour. By inspection of the pile load distribution curves (Figure 2 and 3) the maximum shear resistance and base load associated with the 'linear-elastic' loading phase was isolated from the full test record.

In addition, as the maximum applied load to each test pile was observed to have fully mobilised the shaft capacity of the isolated section of the drilled rock socket, the ultimate side shear resistance (τ_{PEAK}) of this identifiable section of the rock socket could be calculated. As described by Zhang (1997), this value corresponds to Point 'B' as defined by Figure 2(a).

Table 5 details the maximum side shear resistance (τ) and associated displacement values associated with the 'linear-elastic' loading phase of each test pile, along with the τ_{PEAK} and corresponding displacement values observed during the non-linear loading phase. The parameters presented in Table 5 became the basis for the *in situ* rockmass modulus back-calculations completed by this study.

Table 5: Pile Loading Tests – Details of for both 'Linear-Elastic' and Non-linear Phases

Load-Displacement Type	TP-01		TP-02	
	Linear-Elastic	Non-Linear	Linear-Elastic	Non-Linear
Maximum / Peak Unit Shaft Load (τ_{PEAK}, kPa)	710	1,640	640	1,490
Shaft displacement at τ_{PEAK} (δ_{SHAFT}, mm)	2.53	12.34	1.67	11.74
Base Load carried at τ_{PEAK} (Q_{BASE}, kN)	5,520	20,690	1,665	7,850
Displacement of Base at τ_{PEAK} (δ_{BASE}, mm)	2.28	11.77	0.96	10.28
Total Applied Load at τ_{PEAK} (Q_{TOTAL}, kN)	14,420	42,625	17,470	44,605

6 BACK-CALCULATION OF *IN SITU* MODULUS

A number of commonly used rock socket design procedures allow the estimation of pile shaft or full rock socket (shaft and pile base) displacement in response to an applied axial load. Based on the observed results from the completed pile load tests four (4) common rock socket pile design methodologies were used to back-calculate the input rock modulus value required to replicate the observed pile displacements. Table 6 lists the rock socket design procedures that have been assessed as part of this study, and identifies the grouping to which they apply. Note that although the Carter and Kulhawy (1988) design method is able to consider the pile during 'non-linear' deformation, this study did not consider this aspect of design as estimation of additional parameters would have been required (e.g. Poisson's ratio and strength parameters that were not directly tested).

Table 6: Details of rock socket bored pile design methodologies considered

Rock Socket Design Method	Linear-Elastic Phase	Full Slip (Non-Linear) Phase
Pells and Turner (1979)	✓	×
Williams, Johnson and Donald (1980)	✓	×
Rowe and Armitage (1987)	✓	✓
Carter and Kulhawy (1988)	✓	✓ (but not analysed by this study)

For all rock socket design procedures considered by this study, the generalised form of equation used to estimate deflection is presented in Equation 4.

$$\text{Displacement } (\delta) = \frac{\text{Applied Load (P) x Settlement Influence Factor } (I_p)}{\text{Pile Diameter (D) x } \textit{In situ} \text{ Rock Modulus } (E_m)} \qquad (4)$$

The "Settlement Influence Factor" (I_p or I_d) is applied by all design procedures to the pile load. The applicable I_p value is selected via graphical solutions in which the I_p is influenced by the pile geometry (rock socket length versus pile width) and ratio between the *in situ* pile modulus (E_P) and *in situ* rock modulus (E_m). For this analysis E_P was approximated by the adoption of the design concrete modulus (E_C), as calculated the American Concrete Institute's (ACI) formula (1995) presented in Equation 5.

$$E_C = 4700(f'_c)^{0.5} \qquad (5)$$

As the characteristic strength of the test pile's concrete was known (50MPa), E_P was estimated to be in the order of 33 GPa. Thus, by combining the observed load distribution (base versus shaft) for the end of the conclusion of the 'linear-elastic' phase of loading and design charts included in Pells and Turner (1979) and Rowe and Armitage (1987), an initial estimation of E_P / E_M of 250 and 10 was made for TP-01 and TP-02 respectively. Such an E_P / E_M result was also found to be consistent when the observed shaft / base load distributions shown for the 'linear-elastic' interface when plotted on Rowe and Armitage (1987) design charts for complete socketed piers.

Using the relevant design charts for each pile design method, corresponding influence factors (I_p or I_d) were determined. Table 7 details the I_p considered applicable to each rock socket design method, based on known rock socket geometry, and presents the E_m values required for each design methodology to reproduce the displacement values observed under the considered loading scenarios ('linear-elastic' and ultimate shear resistance). The E_m values resultant from the back-calculation as shown in Table 7 can be directly compared with those derived by generic correlations (refer Table 3).

Table 7: Back-calculated in situ rockmass modulus (E_m) to replicate pile deformations

Design Procedure	**Influence Factor (I_p or I_d)**			
	TP-01 (E_P / E_M = 250)		**TP-02** (E_P / E_M = 10)	
	Linear-Elastic	**Non-Linear**	**Linear-Elastic**	**Non-Linear**
Pells and Turner (1979)	E_M = 760 MPa (I_p = 0.19)	–	E_M = 1,640 MPa (I_p = 0.19)	–
Williams, Johnson and Donald (1980)	E_M = 780 MPa (I_p = 0.39)	–	E_M = 1,775 MPa (I_p = 0.40)	–
Rowe and Armitage (1987)*	E_M = 1,000 MPa (I_p = 0.35)	E_M = 860 MPa (I_p = 0.51)	E_M = 2,535 MPa (I_p = 0.40)	E_M = 885 MPa (I_p = 0.46)
Carter and Kulhawy (1988)	E_M = 680 MPa (I_p = 0.17)	Not Analysed	E_M = 1,685 MPa (I_p = 0.19)	Not Analysed

*Partial factor of 0.7 applied, as per design methodology author's recommendation.

From inspection of Table 7, three (3) of the four (4) design procedures required input E_m values that were within 10% of their average. The exception, the Rowe and Armitage (1987) methodology, produced significantly higher input E_m values due to the recommendation by the authors that a partial factor be applied to E_m values based on their own statistical analysis of available data. A reduction factor of 0.7 was recommended by Rowe and Armitage (1987), and was adopted by this study. This analysis also demonstrated that for the non-linear phase of loading significantly lower E_m values (compared to those back-calculated for the 'linear-elastic' phase) were required such that the test piles' load-deformation behaviour could be replicated through the considered design methodologies.

7 REQUIRED 'CHARACTERISTIC' ROCK STRENGTHS FOR DESIGN

'Characteristic' rock strength values that would be appropriate for use to produce the back-calculated E_m values were determined. Table 8 and 9 present the required UCS value to produce the back-calculated E_m value for each considered design methodology for TP-01 and TP-02 respectively, based on each of the relationships between rock strength and E_m (presented in Section 4). This analysis also identifies the closest 5th percentile of the non-normal distributions fitted by Look and Lacey (2013).

Table 8: 'Characteristic' UCS Value (MPa) to be adopted to result in applicable E_m value – TP-01

Pile Design Method / **UCS to E_m Relationship**		**Required UCS Value (MPa) to reproduce back-calculated E_m**				
		Pells and Turner	Williams, Johnson and Donald	Rowe and Armitage		Carter and Kulhawy
				Elastic	Non-Linear	
Linear q_u:E_m Relationships	*MR = 60*	12.7 (45%)	13.0 (45%)	16.7 (55%)	14.3 (50%)	11.5 (40%)
	MR = 105	5.4 (15%)	5.5 (15%)	7.1 (25%)	6.1 (20%)	4.9 (15%)
Rowe and Armitage (1987)		12.5 (45%)	13.2 (45%)	21.6 (65%)	16.0 (55%)	10.3 (40%)
Prakoso (2002)		17.6 (55%)	18.5 (60%)	30.1 (75%)	22.4 (65%)	14.1 (50%)
Hoek and Brown (1997)		0.6 (<5%)	0.6 (<5%)	1.0 (<5%)	0.7 (<5%)	0.5 (<5%)

Table 9: 'Characteristic' UCS Value (MPa) to be adopted to result in applicable E_m value – TP-02

Pile Design Method / **UCS to E_m Relationship**		**Required UCS Value (MPa) to reproduce back-calculated E_m**				
		Pells and Turner	Williams, Johnson and Donald	Rowe and Armitage		Carter and Kulhawy
				Elastic	Non-Linear	
Linear q_u:E_m Relationships	*MR = 60*	27.3 (40%)	29.6 (40%)	42.3 (60%)	29.6 (40%)	28.1 (40%)
	MR = 105	15.6 (10%)	16.9 (15%)	24.1 (30%)	8.4 (5%)	16.0 (10%)
Rowe and Armitage (1987)		58.2 (70%)	68.2 (75%)	139 (90%)	16.9 (15%)	61.4 (75%)
Prakoso (2002)		79.2 (80%)	92.3 (85%)	186 (95%)	23.7 (30%)	83.6 (85%)
Hoek and Brown (1997)		6.8 (<5%)	7.9 (<5%)	16.1 (<5%)	2.0 (<5%)	7.1 (<5%)

The results of the 'characteristic' rock strength back-calculation indicate that for the site-specific fitted non-normal distributions (Look and Lacey, 2013), the use of the Hoek and Brown (1997) correlation to determine E_m required the adoption of very low input UCS values (<5th percentile). Contrastingly, both the Rowe and Armitage (1987) and Prakoso (2002) methods suggest values at, or above, the median (50th percentile) UCS value should be used. Adoption of the highest *MR* for a linear q_u:E_m relationship results in a requirement for UCS values of between the 10th and 25th percentile (i.e lower quartile or below), whilst the lowest *MR* suggests UCS values within 10% of the median are most appropriate.

Resolving the required UCS values back to the equivalent normal distributions, the results suggest that for all linear q_u:E_m relationships a UCS value with within the 20th to 30th range would provide appropriate 'characteristic' rock strength. A slightly lower, 15th to 20th percentile, value would be appropriate if the Hoek and Brown (1997) procedure was used (although this would also be affected by the GSI adopted), whilst higher (30th to 50th percentile) value would be appropriate for both Rowe and Armitage (1987) and Prakoso (2002) methodologies. For the non-linear pile behaviour considered, all q_u:E_m correlation methodologies suggest values of between the 20th and 30th percentile (normal distribution) would best calculate an E_m that would produce the observed pile deformation.

8 CONCLUSIONS

An estimate of the magnitude of *in situ* rockmass modulus required for input into four (4) methods of rock socket design was made from data of two (2) large-scale instrumented bored piles. This back-calculation suggests all design methods required input E_m values of similar magnitudes to reproduce the load-deformation response observed in field testing. A single design method (Rowe and Armitage, 1987) – recommended higher E_m values be adopted to account for inherent rockmass variability, and also required comparatively lower E_m values to be input once non-linear pile behaviour was observed.

The selection of the q_u:E_m correlation method resulted in greater variability than the subsequent choice of pile design methodology. Varied relationships between UCS and E_m were assessed and the required 'characteristic' UCS value found to vary between very low (<5th percentile) and high (above median) values. Thus, the accuracy of any rock socket design method is dependent on its characteristic (design) E_m value, which in turn requires both appropriate q_u:E_m correlations and rock strength distribution functions to be adopted.

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