Characteristics Values in Rock Socket Design

Valeurs caractéristiques d'ancrage sur roche

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ABSTRACT: The substructure of the Gateway Bridge comprises 1.5 metre diameter bored piers socketed into sedimentary rock. Characterisation of the rock strength properties, through goodness-of-fit tests, showed the use of non-normal distributions produced realistic characteristic strengths, while comparable predictions based on a Normal distribution showed unrealistically low values existed below the 20th percentile reliability. Since limit state codes imply characteristic design strengths should be derived from conservative (low) percentile values, erroneous characteristic strength values may be produced due to an assumption of a Normal distribution. Two land based test piles fitted with Osterberg cells tested the sedimentary bedrock for shaft capacity at the bridge site, and "Characteristic" rock strengths required by various rock socket design methods to replicate observed pile shaft capacity have been back-calculated. This paper assumes that all considered design methods are equally "correct", and compares the required design values (the selection of which are often subjective) to their relative location within the applied strength profile distributions.

RÉSUMÉ : L'infrastructure du pont "Gateway Bridge" a des piliers forés de 1,5 mètre de diamètre ancrés dans de la roche sédimentaire. La caractérisation des propriétés de résistance de la roche, faîte par le biais de test d'ajustement, a montré que l'utilisation de loi non-normale a produit des caractéristiques de résistance plausibles alors que des prédictions comparables basées sur une loi normale pouvaient devenir artificiellement basses pour un degré de probabilité au 20ième centile. Certaines normes d' états limites suggèrent que les caractéristiques de résistance devraient être dérivées à partir de valeurs (au bas mot) de bas centile, des valeurs de caractéristiques de résistance erronées pouvant être calculées en assumant une loi normale. Deux piliers tests installés sur terre-sèche avec des cellules d'Osterberg ont été utilisés pour tester la capacité de la roche sédimentaire par rapport au pilier mais les valeurs de résistance caractéristique de roche nécessitées par les diverses conceptions d'ancrage sur roche ont été rétro-calculées afin de reproduire la capacité observée des piliers. Cet article assume que toutes les méthodes de conception considérées sont tout aussi "correctes" et dérive les valeurs caractéristiques de conception requises (la sélection desquels est souvent subjective) afin de reproduire les capacités observées des piliers durant des tests à grande échelle.

KEYWORDS: Rock socket, bored piles, characteristic design value, statistical distribution, Osterberg pile tests, sedimentary rocks

1 INTRODUCTION

The Gateway Upgrade Project (GUP) was the largest road and bridge infrastructure project ever undertaken in Queensland, Australia. The six lane bridge structure spans 1.6 km between abutments with a main river span structure of 520 metres. This paper focuses on rock socket design procedures applied to two large-scale, land-based pile load tests conducted for this project.

The rock founding conditions varied across the bridge footprint as summarised in Table 1. Characterisation of rock strength properties included the derivation of site-specific correlation of Point Load Index ($I_{s(50)}$) data with Uniaxial Compressive Strength (UCS) test results, and a statistical analysis of resulting datasets.

The two test piles (TP1 and TP2) installed with Osterberg Cells were constructed to investigate the rock socket behaviour under high loads and identify any constructability issues prior to construction of the two river piers. This paper considers various accepted methods of pile rock socket design and compares their applicability to the load tests completed at this site.

Rock socket design methods typically have similar formulations for the estimation of side shear capacity, but produce varying results due to their method of derivation, and the available data or tested rock types used for formulation. While the rock type may be a governing factor, this paper assumes that all the methods produce "correct" pile designs, but require varying "characteristic" design input values to produce equivalent results. Reliability theory implies a moderately conservative or cautious estimate should be used as the characteristic design value, yet without a statistical basis the selection of appropriate characteristic values remains subjective.

Table 1. Background Data

Location / Pier	No. of Bored Piers	Key Geological Issue within Rock Founding Layers
Land 5 – Southern	10 (+ TP1)	Dipping Coal seam layer within zone of influence of pier.
River 6	24	Random Shear zones with varying length of piles
River 7	24	
Land 8 – Northern	10 (+ TP2)	Uncertain and inconsistent data with possible weak zones

1.1 *Background*

Whilst driven piles were used extensively across the GUP site, the river span of the Gateway Bridge is founded upon 1.5 metre diameter bored piers socketed into sedimentary rock. Piers 5 and 8 are located on the riverbank while Piers 6 and 7 are located within the river. River piers consisted of 24 piles that extended to a depth of over 50m below the river level, and each of the land based piers consisted of only 10 bored piers. Day et. al. (2009) provides further GUP foundation and project details.

For the bridge foundation the key geological features were:

- The basement rock consisted of Triassic aged material. This includes layers of sandstone, siltstone, mudstone and low grade coal formed about 220 to 180 million years ago. This material does not 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.

2 STATISTICAL ANALYSIS OF DATA

Look and Wijeyakulasuriya (2009) carried out a statistical review of the intact rock strength data at Piers 6 and 7 for the sub-horizontally interbedded sedimentary layers at the GUP site, and defined Point Load Index ($I_{s(50)}$):UCS ratios of 40 and 25 for diametral and axial orientated $I_{s(50)}$ tests respectively. This highlighted the need to account for strength anisotropy in the rock socket design, due to the radial normal stresses on the socket wall. Via use of goodness-of-fit tests, Look et. al. (2004, 2009) has demonstrated that the use of non-normal distribution functions for describing rock strength datasets produces more realistic characteristic strength values than comparable values based on assumption of a Normal distribution. Use of a Normal distribution was reported to produce unrealistically low, or even negative, values at low percentile values of the rock strength.

Figure 1 compares the best fit distribution (Log-logistic) with the better known Log-normal and Normal distributions for all Pier 6 diametrally orientated $I_{s(50)}$ values completed in the interbedded sandstone layer (n = 330). The Log-normal distribution, while not the best fit, is observed to provide a much closer fit to the dataset than the Normal distribution does.

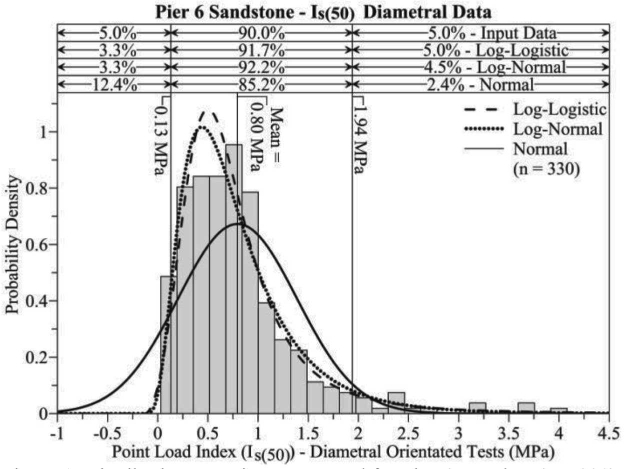

Figure 1. Distribution Functions compared for Pier 6 $I_{s(50)}$ data (n = 330)

2.1 *Characterisation of rock strength*

The test piles, TP1 and TP2, were completed upon the south and north riverbank respectively (~600m apart). TP1 was located approximately 160m from the location of Pier 6. Strength data was compiled by conversion of $I_{s(50)}$ values of tests completed along of the length of the instrumented pile to equivalent UCS values via use of the site calibrated conversion ratios of 25 and 40. Figure 2 compares statistical distribution functions fitted to the equivalent UCS strength results applicable TP2. Similar distribution fitting was also completed independently for TP1.

Tests related to TP2's rock socket indicated the presence of higher strength sandstone layers than encountered in TP1, which illustrated the localised material variation within the geological sequences that existed below the bridge footprint.

Both non-normal and Normal distribution functions were fitted to each test pile's strength dataset through application of the Anderson-Darling goodness-of-fit test. The resultant UCS values for selected fractiles of the fitted non-normal and Normal distributions are detailed in Table 2 for each test pile. At low percentile values (TP1 ≤ 15th percentile; TP2 ≤ 10th percentile), the use of a Normal distribution function would output a negative "characteristic" design value. This supports the assertion that a non-normal distribution is most appropriate for use in characterising rock strength data for this site.

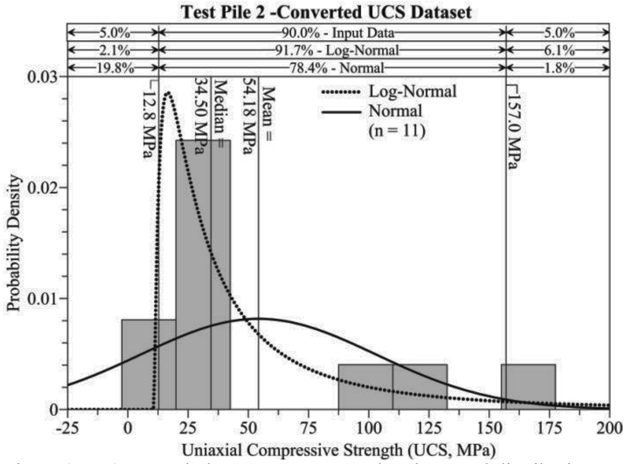

Figure 2. TP2 strength dataset – Log-normal and Normal distributions

Table 2 also shows the localized variation in rock strength for the interbedded sandstone layer. Numerical similarity is not apparent between TP1 and TP2, and the comparatively low number of strength tests available for each test pile should also be noted. Pier 6 can be considered geologically similar to TP1, in both strength data and as both were logged as having deep alluvium overlying rock. If a single characteristic rock strength value was selected to represent the entire GUP site, the location of the value upon the derived strength profiles would vary. If, arbitrarily, 10MPa was selected as the design characteristic rock strength, this could represent either the 40th or 30th percentile, depending on the distribution applied to the TP1 dataset, or either the 5th or 20th percentile for the TP2 rock strength data.

The data presented herein demonstrates that the choice of distribution function used to define such fractiles plays a critical role in the calculated design value, especially within the lower percentiles (below the 20th percentile). As the shear capacity of a rock socket is largely defined by the design rock strength, the selection of the distribution function applied to calculate the characteristic strength value can thus potentially have a large impact upon the resulting pile design and the required length of rock socket to withstand the design load.

Table 2. Test Piles UCS distribution

Distribution	Percentile	UCS (MPa)		
		TP1 $n = 8^{+}$	Pier 6 $n = 330^{*}$	TP2 $n = 11^{+}$
Normal	10%	-11.5	0.9	-8.4
	25%	9.2	9.9	21.2
	Median (50%)	32.1	19.9	54.2
	75%	55	29.9	87.1
Non-normal	10%	4.3	5.9	15.8
	25%	7.2	10.0	21.3
	Median (50%)	14.4	16.4	34.3
	75%	35.2	25.8	63.2

+Over length of Pile Shaft; *in equivalent interbedded sandstone layers

Approximate "equivalency" between the non-normal and Normal distributions occurs at the 25th percentile. Thus, if the inconsistencies associated with use of inappropriate distribution functions are to be minimised then the adopted design UCS value should be close to, or at, this fractile.

2.2 *Full Scale Load Tests*

Osterberg Cell (O-Cell™) data was recorded during cyclical loading / unloading of the two 1.2m diameter test piles. The rock socket of both test piles consisted of slightly weathered, medium to high strength, Triassic aged sedimentary rock (interbedded layers of mudstone, sandstone and siltstone).

In both TP1 and TP2 the shaft resistance of the section of rock socket existing between the level of the installed Osterberg Cell and pile tip was observed to have become fully mobilised during the application of a peak load (up to 56.6MN, approximately 3.1 times the expected SLS load).

The maximum shaft capacity of the 2.66m length of shaft that existed below the installed Osterberg Cell of TP1 was calculated to be 17.55MN, with a residual value of 16.4MN. The residual value represented a 7% decrease from the maximum observed value. Similarly, the peak shaft capacity of the 5.24m length of TP2 between the Osterberg Cell and pile tip was determined to be 29.2MN.

3 ROCK SOCKET DESIGN PROCEDURES

Early work for rock socket design occurred in Australia by Williams, Johnston and Donald (1980) who examined non-linear pile design in Melbourne Mudstones, and Rowe and Pells (1980) who calibrated elastic pile design with Sydney Sandstones and Shales. Horvath and Kenny (1979) undertook similar field and laboratory testing on mudstones in Canada while Meigh and Wolshi (1979) conducted comparable work in Europe. Side slip design was subsequently detailed by Rowe and Armitage (1987).

Kulhawy and Phoon (1993) showed the discontinuity in shaft friction between clays and various soft rocks (shales, mudstones and limestone). Seidel and Haberfield (1995) extended that work to demonstrate that rock socket performance is highly dependent on shaft roughness and socket diameter; whereby pile shaft friction reduces as the pile diameter increases.

Generally, rock socket design is governed by serviceability conditions rather than ultimate load conditions, and the load – deformation behaviour of the rock sockets are determined largely by the rockmass deformation properties. The rockmass modulus (E_m) value can be estimated from the modulus of intact rock (E_i) reduced for the frequency of rock defects. Relevant theory is discussed by Zhang (2004).

Various pile rock socket design procedures are now available which frequently calculate the design shaft capacity based on correlation with a "characteristic" compressive rock strength (q_{uc}) value. A good summary of the shaft shear capacity equations derived by design method researcher is provided in Kulhawy et al. (2005). Gannon et al. (1999) described four of these methods and showed, even when adopting consistent rock properties for design, the resulting design pile socket shear capacities ranged widely. The longest pile socket lengths for the example provided were predicted by the Carter and Kulhawy (1988) design method, while the Rowe and Armitage (1987) and Williams et al. (1980) procedures reduced the socket lengths by 40-60%.

This paper aimed to provide guidance on two key questions:

- Which rock socket design method should be used?
- What characteristic rock strength value should be selected (and does the selected method alter the required value)?

Ng et al. (2001) showed that the correlations presented by Rowe and Armitage (1987) and Hovarth et al. (1983) are applicable for sedimentary and volcanic rocks respectively.

Table 3. Unit side resistance formulas for considered rock socket pile design methodologies, normalised with atmospheric pressure (p_a)

Design Method	Normalised Unit Side Resistance Equation	
Hovarth and Kenny (1979)*	$\frac{f_{su}}{p_a} = 0.65\sqrt{\frac{q_{uc}}{p_a}}$	(1)
Meigh and Wolski (1979)	$\frac{f_{su}}{p_a} = 0.55(q_{uc})^{0.6}$	(2)
Williams, Johnson and Donald (1980)	$f_{su} = \alpha\beta(q_{uc})$	(3)
Rowe and Armitage (1987)	$\frac{f_{su}}{p_a} = 1.42\sqrt{\frac{q_{uc}}{p_a}}$	(4)
Carter and Kulhawy (1988)	Lower Bound: $\frac{f_{su}}{p_a} = 0.63\sqrt{\frac{q_{uc}}{p_a}}$	(5)
	Upper Bound: $\frac{f_{su}}{p_a} = 1.42\sqrt{\frac{q_{uc}}{p_a}}$	(6)
Kulhawy and Phoon (1993)	$\frac{f_{su}}{p_a} = C\sqrt{\frac{q_{uc}}{2p_a}}$	(7)
Prakoso (2002)	$\frac{f_{su}}{p_a} = \sqrt{\frac{q_{uc}}{p_a}}$	(8)

*Also confirmed by Zhang (1999) and Reese and O'Neill (1988)

3.1 *Back-analysis of rock socket design methodologies*

By using the measured ultimate shaft frictional capacity as the basis for back-analysis, "characteristic" q_{uc} input values could be determined for each considered rock socket design method.

Table 3 details the rock socket pile design methodologies considered and the published formulae used in each to calculate dimensionless unit side resistance values (f_{su}). These values are transformed to rock socket design capacities via multiplication of the calculated f_{su} value by the surface area of the segment of the rock socket that was loaded to capacity. In this study it has been assumed that the pile socket is effectively smooth and that concrete strength does not limit the shear capacity of the pile. No factors of safety have been applied as field data is being fitted back to design equations.

Notes relevant to the formulae presented in Table 3 include:

- Eq. (3) calculates shear capacity based on both the rock strength value and a mass factor (j) which is defined as the ratio of rock mass modulus to intact rock modulus. Based on the average logged RQD values (TP1 = 70%; TP2 = 55%), a mass factor (j) of 0.33 would be appropriate for TP1 (j = 0.20 for TP2). Also, in Eq. (3) α is directly related to the adopted q_{uc}, whilst β is estimated from the j value.
- Shaft capacity values for Eq. (4) are recommended to be multiplied by a partial factor of 0.7 to ensure the probability of exceeding design settlements is lower than 30%.
- The coefficient C in Eq. (7) is based on conservatism and rock socket roughness; C = 1 provides a lower bound estimate, C = 2 for mean pile behaviour and C = 3 for upper bound estimates or for rough rock sockets.
- The approach used to derive Eq. (8) was cited by Kulhawy et al. (2005) as providing the most consistent approach in evaluation of the constructed pile load dataset.

3.2 *Back-calculation Results*

Table 4 provides a summary of the various input UCS values required to achieve the ultimate shaft capacity values observed in each test pile. These values have been back-calulated via use of the equations detailed in Table 3. The 5th percentile closest to the required UCS value has been determined for both the normal and non-normal distribution functions fitted to each test pile's strength data (refer Table 2).

Table 4. Typical Correlations between UCS and shaft friction

Design Method Equation	TP1 – f_{su} = 1.64 MPa		TP2 – f_{su} = 1.48 MPa	
	UCS (MPa)	Percentile (Pearson5 / Normal)	UCS (MPa)	Percentile (Log-Normal / Normal)
Hovarth and Kenny (1979)	62.6	80% / 85%	51.2	70% / 50%
Meigh and Wolski (1979)	28.3	70% / 45%	24.0	35% / 30%
Williams, Johnson and Donald (1980)	20.5 (α = 0.1) (β = 0.8)	60% / 35%	22.8 (α = 0.1) (β = 0.65)	30% / 25%
Rowe and Armitage (1987)	13.1	60% / 30%	10.8	< 5% / 20%
Carter and Kulhawy (1988)	66.6	85% / 85%	54.5	70% / 50%
	(Lower Bound Equation)			
	13.1	50% / 30%	10.7	< 5% / 20%
	(Upper Bound Equation)			
Kulhawy and Phoon (1993)	52.9 (C = 1)	85% / 75%	43.3 (C = 1)	60% / 40%
	13.2 (C = 2)	50% / 30%	10.8 (C = 2)	< 5% / 20%
Prakoso (2002)	26.5	70% / 45%	21.6	25% / 25%

The results indicate that various researchers appear to have assumed a Normal distribution in developing shear capacity formulae, with a lower quartile to mean / median value generally adopted (20th to 50th percentiles). Higher (≥50th) percentiles were required to replicate the observed ultimate capacity values for lower bound (conservative) pile capacity formulas. As the adopted design UCS value is commonly above the point of equivalency between the Normal and non-normal distribution ($q_{uc} \cong$ 25th percentile, refer Table 2), the comparable back-calculated design strength percentiles are generally higher for the non-normal distributions.

However, the more accurate distribution function has been shown to be non-normal. Using the best fitting distribution, the derived UCS values required to replicate the shaft capacity observed in TP1 were consistently at, or above, the median value. This suggests that all considered design methodologies would, if the non-normal 50th percentile value was adopted, provide overly conservative shear capacity values for this site.

To avoid the inconsistencies associated with use of incorrect distribution functions a characteristic q_{uc} value about the 20th to 30th percentile range was previously recommended. Using this percentile range of the Normal and non-normal TP1 rock strength (UCS) datasets, or the larger Pier 6 datasets (also assumed representative of TP1), the formula provided by Rowe and Armitage (1987), and upper bound equations from Carter and Kulhawy (1988) and Kulhawy and Phoon (1993) calculate pile shaft capacities closest to those observed. In the higher strength rock profile of TP2, the capacity equations provided by Meigh and Wolski (1979) and Prakoso (2002) displayed the closest match to the observed shaft capacity when the 20th, 25th or 30th percentiles of the UCS datasets were adopted.

4 CONCLUSIONS

Statistical analysis of the available GUP rock strength data shows that if a Normal distribution is assumed for characteristic value determination, then errors may result. To minimise inconsistencies associated with the use of ill-fitting distribution functions to describe strength data, then the selection of values near the lower quartile of the UCS dataset is recommended.

Two large-scale instrumented test piles were loaded beyond shaft capacity at the GUP site. Based on this test data, the required input UCS value has been back calculated for a number of pile design methods, and the indicative strength percentile reliability of the UCS value has been determined. Five of the examined methods have produced results that match the observed shaft capacities via the adoption of a design UCS value close to the UCS lower quartile "characteristic" value.

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6 REFERENCES

Carter, J.P., and Kulhawy, F.H., 1988. Analysis and design of drilled shaft foundations socketed into rock. *Report EL-5918*. Palo Alto: Electric Power Research Institute, 190p.

Day, R., Johnston, I. and Yang, D. 2007. Design of Foundations to Second Gateway Bridge – Brisbane. *7th Austroads Bridge Conference*, Auckland, N.Z., 12p.

Gannon, J.A., Masterton, G.G., Wallace, W.A. & Wood, D.M. 1999. *Piled Foundation in Weak Rock (CIRIA Report No.181).* Construction Industry Research & Information Assoc., UK, 80p.

Horvath, R.G. and Kenney, T.C. 1979. Shaft Resistance of Rock Socketed Drilled Piers. *Proc. Symp. on Deep Foundations*, ASCE, New York, N.Y., pp. 182–214

Horvath, R.G., Kenney, T.C. and Koziki, P. 1979. Methods of improving the performance of drilled piers in weak rock. *Canadian Geotech. Journal*, 20(4), pp.758-772

Kulhawy, F.H. and Phoon, K-K. 1993. Drilled Shaft Side Resistance in Clay Soil to Rock. *Geotechnical Special Publication No. 38: Design and Performance of Deep Foundations*, pp. 172–183

Kulhawy, F.H., Prakoso, W.A., & Akbas, S.O. 2005. Evaluation of Capacity of Rock Foundation Sockets. *Proc. 40th U.S. Symp. Rock Mechanics*, G.Chen et al. (Eds.), Anchorage, Alaska, paper 05-767

Look, B.G. and Griffiths, S. 2004. Characterization of rock strengths in South East Queensland. *Proc. of the 9th Australia New Zealand Conf. in Geomechanics*, Auckland, N.Z., Vol. 1, pp.187–194

Look, B.G. and Wijeyakulasuriya, V. 2009. The statistical modelling of rock strength for reliability assessment. *17th Int. Conf. on Soil Mechanics and Foundation Eng.*, Alexandria, Vol. 1, pp.60–63

Meigh, A.C. and Wolski. W. 1979. Design parameters for weak rock. *Proc., 7th European Conf. on Soil Mechanics and Foundation Eng.,* Brighton, UK, Vol. 5, pp.59-79

Ng., C.W.W., Yau, T.L. Li, J.H. and Tang, W.H. 2001. Side resistance of large diameter bored piles socketed into decomposed rocks. *Journal of Geotech. & Geoenvir. Eng.,* ASCE 127(8), pp.642-657

Prakoso, W.A. 2002. Reliability-Based Design of Foundations on Rock for Transmission Line & Similar Structures. *PhD Thesis.* Cornell University.

Reese, L.C. and O'Neill, M.W. 1988. Drilled shafts: construction procedures and design methods. *Report FHWA-HI-88-042*. McLean: Federal Highway Administration.

Rowe, R. K. and Armitage, H. H. 1987. A design method for drilled piers in soft rock. *Canadian Geotech. Journal*, 24(1), pp.126-142.

Rowe, R.K. and Pells, P.J.N. 1980. A Theoretical Study of Pile-Rock Socket Behaviour. *Proc., Int. Conf. on Structural Foundations on Rock*, Vol. 1, Sydney, Australia, pp.253-264

Seidel J.P. and Haberfield C.M. 1995. The axial capacity of pile sockets in rocks and hard soils. *Ground Eng.,* 28(2), pp.33 – 38

Williams, A.F., Johnston, I.W. and Donald, I.B. 1980. The Design of Sockets in Weak Rock. *Proc., Int. Conf. on Structural Foundations on Rock*, Vol. 1, Sydney, Australia, pp. 327–347

Zhang, L. 1999. Analysis and design of drilled shafts in rock. *PhD Thesis*, Massachusetts Institute of Technology, Cambridge, Mass.

Zhang, L. 2004. *Drilled Shaft in Rock: Analysis and Design.* A.A. Balkema Publishers