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Evaluation of design methods for side-resistance-only rock-socketed piles

Ian W. Johnston

Abstract: The ability of three methods to predict the load–settlement response of rock-socketed piles was examined in a recent paper. The piles considered were complete piles in which both the side and base of the piles contribute to the total resistance. For a range of reasons, it is sometimes necessary to consider rock-socketed piles for which the resistance is provided only by the side of the socket. This paper extends the earlier paper by comparing predictions for side-resistance-only piles with the load–settlement response of a suite of full-scale side-resistance-only pile tests. Only two of the methods of the earlier paper are used for this comparison, as the third method could not decouple the two components of resistance. It is demonstrated that one of the methods produces reasonable predictions of performance, whereas the other appears to underpredict performance.

Key words: side resistance, piles, rock, design methods, load–settlement data.

Résumé : La capacité de trois méthodes à prédire la réponse charge–décompactage des pieux à ancrage rocheux a été examinée dans un article récent. Les pieux considérés étaient des pieux complets dans lesquels le côté et la base des pieux contribuent à la résistance totale. Pour diverses raisons, il est parfois nécessaire d'envisager des pieux ancrés dans le roc, pour lesquels la résistance n'est assurée que par le côté de l'ancrage. Cet article étend la portée de l'article précédent en comparant les prédictions pour les pieux à résistance latérale seulement avec la réponse à l'installation de la charge d'une série d'essais en grandeur réelle de pieux à résistance latérale seulement. Seules deux des méthodes de l'article précédent sont utilisées pour cette comparaison, car la troisième méthode ne pouvait pas découpler les deux composantes de la résistance. Il est démontré que l'une des méthodes produit des prédictions raisonnables des performances alors que l'autre semble sous-estimer les performances. [Traduit par la Rédaction]

Mots-clés : résistance latérale, pieux, roche, méthodes de conception, données sur le règlement des charges.

Introduction

In a recent paper (Johnston 2020), the predictions for load–settlement response made by three methods specifically developed for the design of rock-socketed piles were compared with the results of several pile load tests. The methods were developed by Williams et al (1980), Rowe and Armitage (1987a), and Carter and Kulhawy (1988). This recent paper considered complete piles for which side and base resistance contribute to the total resistance of the pile. For complete piles in reasonably competent rock, as was argued by Williams et al. (1980) and subsequently supported by others (e.g., Horvath 1980; Rowe and Armitage 1987a, 1987b; Carter and Kulhawy 1988; Seol et al. 2008; Haberfield and Lochaden 2019), where the base resistance develops at a much slower rate than the side resistance, these piles rarely display an ultimate load that is constant but increases with increased displacement. Consequently, it is settlement that usually controls the design of complete piles socketed into rock. For this reason, the three design methods considered by Johnston (2020) adopt settlement as the primary input parameter, with subsequent checks conducted to ensure that there is adequate geotechnical strength.

For socketed piles, there are situations where there can be doubt about resistance developing at the base of the pile. This could be for several reasons, including the inability to confidently remove a compressible debris layer that had formed at the

base of the socket during drilling or subsequent to drilling and before casting, or where the base is formed in a much weaker material or indeed where voids are close (or possibly close) to the underside of the base. For piles in tension, side resistance is the only effective component of resistance from the drilled socket. For these situations, it is not uncommon to design the piles as side-resistance-only piles. For this form of socketed pile, since the base resistance no longer contributes to total resistance, the load–settlement response does not display the slow but increasing load development contributed by the base. Instead, as will be discussed, the load–settlement response of side-resistance-only socketed piles typically shows load increasing at a decreasing rate to around 10 to 30 mm (often more, sometimes less) before showing a marked reduction in the rate of increase of load or in many cases displaying a clearly defined ultimate load. For these piles, design can be controlled by either load or settlement.

Unfortunately, the Rowe and Armitage (1987a) method applies only to complete piles and, therefore, for side-resistance-only piles, the evaluation could only be conducted with the other two methods.

There are many other methods that relate to the design of side-resistance-only rock-socketed piles. However, most of these methods (e.g., Horvath and Kenney 1979; Williams and Pells 1981; Seidel and Collingwood 2001; Kulhawy et al. 2005; Asem and Gardoni 2019) are concerned with predicting the ultimate side resistance

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Fig. 1. Principles for normalising side resistance. (© 1980, “The design of socketed piles in weak rock”, A.F. Williams, I.W. Johnston, and I.B. Donald, reprinted with permission from Taylor and Francis.).

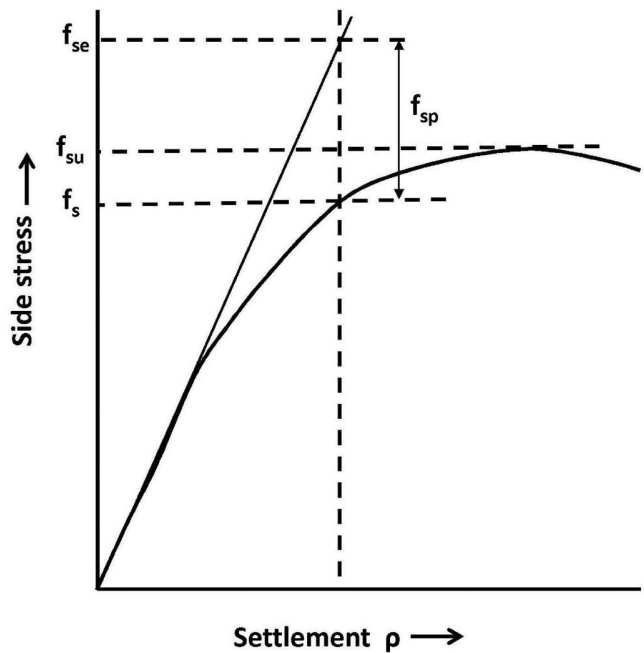


Fig. 2. Normalised design curve for side resistance. (© 1980, “The design of socketed piles in weak rock”, A.F. Williams, I.W. Johnston, and I.B. Donald, reprinted with permission from Taylor and Francis.).

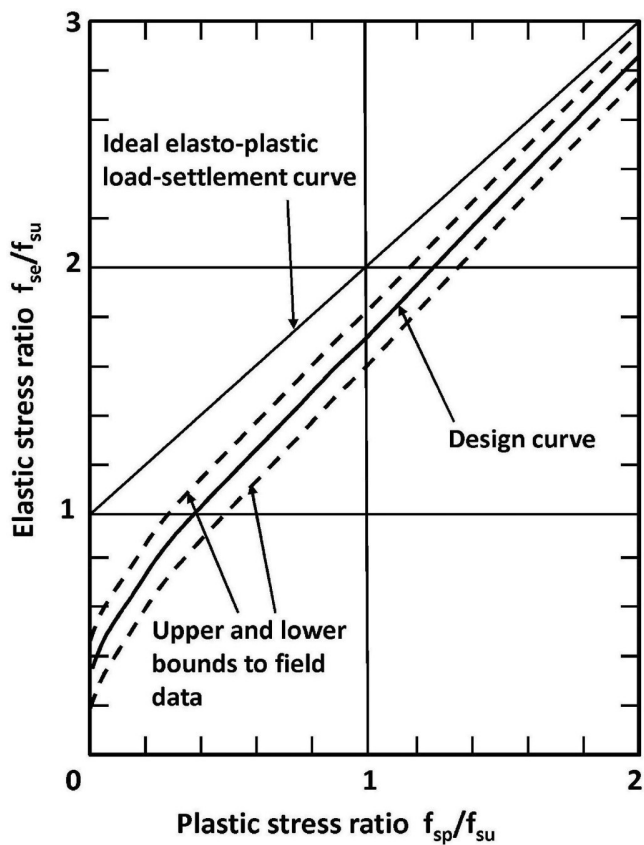


Fig. 3. Side stress reduction factor. (© 1980, “The design of socketed piles in weak rock”, A.F. Williams, I.W. Johnston, and I.B. Donald, reprinted with permission from Taylor and Francis.).

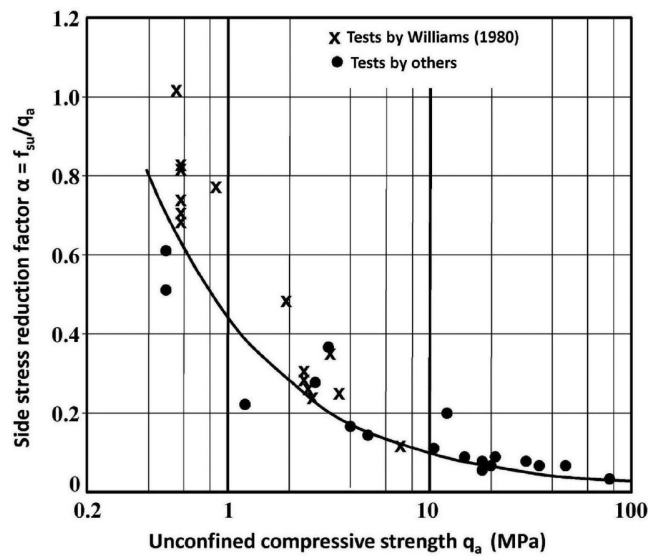
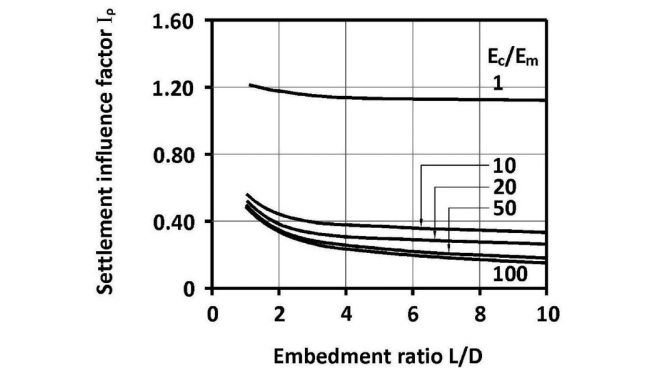


Fig. 4. Elastic settlement influence factor. (© 1980, “Theoretical analyses of rock socketed piles”, I.B. Donald, H.K. Chiu, and S.W. Sloan, reprinted with permission from Taylor and Francis.).



and say little, if anything, about the settlements that might occur. There are methods that do consider settlement but as will be discussed, they involve specialist input knowledge and (or) data that limits their applicability.

Reference is made to Johnston (2020) for a more complete description of the design methods. However, because there are subtle differences between side-resistance-only and complete piles, this paper will provide an overview of the two design methods as applicable to side-resistance-only piles.

Overview of the design methods

Williams, Johnston, and Donald (WJD) method

Full details of the method may be found in Williams (1980) and Williams et al. (1980). The method makes use of the principles of normalisation wherein relationships are expressed in dimensionless terms so that design data becomes independent of dimensions.

The principle for normalising side stress is illustrated in Fig. 1. For any settlement, ρ , the side stress on the pile is given by f_s , which is

Fig. 5. Definition of primary variables for the Carter and Kulhawy (CK) method for a side-resistance-only pile.

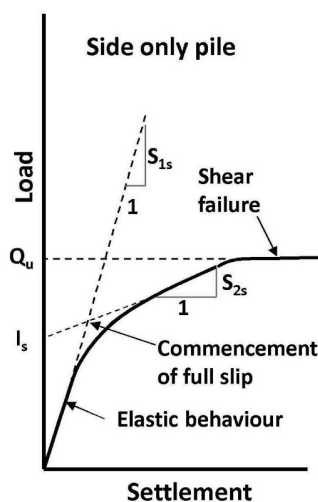


Table 1. Variables used to predict load-settlement response of rigid side-resistance-only socketed piles by Carter and Kulhawy (CK) method.

Slope of elastic behaviour	$S_{1s} = \frac{\pi E_m L}{(1 + \nu) \zeta}$
Slope of slip behaviour	$S_{2s} = \frac{\pi E_m L}{R_1}$
Intercept of slope of slip behaviour with load axis	$I_s = \frac{\pi E_m L D}{2 R_1} R_2$
Ultimate side resistance	$Q_u = f_{su} \pi D L$

$$(1) \quad f_s = f_{se} - f_{sp}$$

where f_{se} is the elastic side stress and f_{sp} is the plastic reduction of side stress as defined in Fig. 1. Equation 1 can be normalised by dividing by the failure or ultimate side stress, f_{su} , to give

$$(2) \quad \frac{f_s}{f_{su}} = \frac{f_{se}}{f_{su}} - \frac{f_{sp}}{f_{su}}$$

This process was applied to all Williams' side resistance tests to produce the normalised design curve shown in Fig. 2.

A key input parameter in the design process is the ultimate side stress, f_{su} , which is determined by

$$(3) \quad f_{su} = \alpha q_a$$

where α is the side stress reduction factor, and q_a is the unconfined compressive strength of the rock. The variation of α with q_a from a number of pile loading tests, as was available at the time, is shown in Fig. 3. The value of f_{su} used in developing Fig. 3 was taken as was measured in each load test with no consideration of the degree of roughness developed on the socket side during drilling.

Overall stability is assessed by the application of Allowable Stress Design (ASD) or Limit State Design (LSD) methods, both of which require an assessment of the ultimate side resistance, Q_{su} . For LSD methods (such as specified by Standards Australia 2009; CSA 2019; and AASHTO 2020), this ultimate resistance is given by the general relationship

Table 2. Factors for use with the variables in Table 1.

Factors	Evaluation of factors
R_1 and R_2	$R_1 = (1 + \nu) \left(\zeta + \frac{1}{2 \tan \phi \tan \psi} \right)$ $R_2 = \frac{(1 + \nu) c}{\tan \phi \tan \psi E_m}$
Other	$\tan \phi \tan \psi = 0.001 \left(\frac{q_a}{p_a} \right)^{2/3}$ $c = 0.1 p_a \left(\frac{q_a}{p_a} \right)^{2/3}$ $\zeta = \ln \left[5(1 - \nu) \frac{L}{D} \right]$

$$(4) \quad Q_{su} = \frac{\gamma_i Q_d}{\phi_i}$$

where Q_d is a design or serviceability load, γ_i is a relevant load factor, and ϕ_i is a relevant resistance factor. For a side-resistance-only pile, for a selected pile diameter, D , the length of socket, L , required to provide this resistance is given by

$$(5) \quad L = \frac{Q_{su}}{\pi D f_{su}}$$

where f_{su} is defined by eq. 3 and Fig. 3.

It now becomes necessary to check that the pile with an elastic modulus (E_c) of selected dimensions (L , D) supporting a design load (Q_d) in rock with strength and elastic mass modulus (q_a , E_m) does not settle more than the maximum allowable design settlement, ρ_{dmax} .

The starting point with this check using the WJD method is estimating the side load, Q_{se} , that would be required to develop a settlement of ρ_{dmax} with the pile responding elastically (referred to as the elastic load). This is given by

$$(6) \quad Q_{se} = \frac{\rho_{dmax} E_m D}{I_\rho}$$

where the settlement influence factor I_ρ is given in Fig. 4 (Donald et al. 1980).

This elastic load can now be relaxed by the corresponding plastic reduction factor defined in Fig. 1. This may be achieved by dividing Q_{se} by the area of the side of the pile to obtain f_{se} , then normalising f_{se} with f_{su} . Using Fig. 2, the corresponding value of f_{sp}/f_{su} can be determined, the difference between the two being the ratio f_s/f_{su} as described in eq. 2. Multiplying f_s/f_{su} by f_{su} then produces f_s , which when multiplied by the side area of the socket gives the relaxed load, Q_s , on the side of the socket for the settlement, ρ_{dmax} . If this relaxed load is greater than the design load (Q_d), then the design is controlled by the load and the design length has been assessed. Alternatively, if this relaxed load is less than the design load (Q_d), then the design is controlled by settlement and the length of the socket needs to be increased until the relaxed load (Q_s) becomes at least equal to the design load. A load-settlement curve for the various input data (L , D , q_a , E_m , E_c) can then be generated for the pile selected. This can be achieved with the same steps described above but starting with a small settlement to obtain a relaxed side load and then incrementally increasing the input settlement for corresponding relaxed loads until the required load-settlement curve is generated. Two examples of the above process are given in Appendix A.

Table 3. Details of side-resistance-only pile tests used in comparisons of performance.

Reference	Socket dimensions (m)				Elastic moduli		Rock unconfined compressive strength, UCS (q_u ; MPa)				Ultimate side stress, f_{su} (MPa)		
	Pile ID	Diameter (D)	Length (L)	Pile (E_c ; GPa)	Rock mass (E_{rm} ; MPa)	Rock type	WJD method (eq. 3)	CK method (eq. 7)	Rigidity criterion ^a (eq. 8)	Rock type	WJD method (eq. 3)	CK method (eq. 7)	Rigidity criterion ^a (eq. 8)
Abu-Hejleh et al. 2003	Broadway, lower sect.	1.37	2.74	28	1020	Siltstone	0.9	0.99	1.72				
Boyd and Ozsoy 2013	Broadway, mid. sect.	1.37	3.05	28	1020	Sandstone	0.71	0.69	1.38				
	TP1	0.75	3	35	962 ^b	Limestone	1.14	1.42	0.57				
Chang et al. 2004	St 1 Sth Tower ^c	1.675	20.7	35	247	Claystone	0.47	0.37	0.23				
	St 2 Sth Tower ^c	1.675	12	35	247	Claystone	0.47	0.37	0.69				
Charif et al. 2010	Pile A3 Layer 1	1	6	38	345	Carbonate	0.7	0.68	0.76				
	Pile G Layer1	0.6	3.5	30	160	Carbonate	0.75	0.75	1.38				
Hassan et al. 1997	Pile G Layer 2	0.6	4.5	30	230	Carbonate	0.74	0.73	0.58				
	Honolulu ^c	1.53	3.35	31	408 ^b	Basalt	0.65	0.6	3.96				
Horvath 1982	5D	0.71	1.37	35	390	Shale	0.92	1.03	6.03				
Horvath et al. 1980	Airport Rd No 3	0.635	1.015	35	1000	Shale	1.04	1.24	3.42				
	Bay St	0.838	0.69	35	1370 ^b	Shale	1.44	2.03	9.53				
Horvath et al. 1983	Whitby	0.76	1.00	35	1130 ^b	Shale	1.27	1.67	4.47				
	P1	0.71	1.37	35	405	Shale	0.74	0.74	5.80				
Johnston et al. 1980	P2	0.71	1.37	35	419	Shale	0.94	1.06	5.61				
	Test 1 ^d	1.212	1	35	600	Mudstone	0.62	0.56	21.42				
Lewis 1993	Test 2 ^d	1.212	1	35	600	Mudstone	0.53	0.44	21.42				
	Centre shaft	0.76	1.2	35	1773 ^b	Shale	1.7	2.62	1.98				
Matich and Kozicki 1967	Caisson No 4	0.61	4.11	35	204 ^b	Shale	0.41	0.3	0.94				
	TS-2, stage 2 ^c	1.17	4	35	372 ^b	Shale	0.61	0.55	2.01				
Miller 2003	TS-2, stage 2/3 ^c	1.17	4.23	35	420 ^b	Shale	0.66	0.62	1.59				
Nam and Vipulanandan 2008	DT site ^c	0.76	2.1	35	212	Shale	0.62	0.56	5.41				
	RC site ^c	0.76	2.6	35	5500	Limestone	0.91	1.02	0.14				
Ng et al. 2001	Pile 3	1.02	2	35	1360 ^b	Tuff	1.43	2.01	1.67				
	Pile 5	1.32	2.1	35	527 ^b	Granitic	0.77	0.78	6.56				
O'Neill et al. 1996	Pile 7	1.2	3.6	35	680 ^b	Granite	0.91	1.01	1.43				
	Pile 9	1	2.5	35	1154 ^b	Granite	1.29	1.71	1.21				
Roberts et al. 2011	Dallas test	0.61	6.09	46	232	Shale	0.38	0.27	0.50				
	TAMU test	0.61	6.76	31	154	Hard clay	0.4	0.29	0.41				
Rosenberg and Journeaux 1976	Owensboro 9-42	1.78	5.16	34.7	2019	Shale-sandstone	0.87	0.94	0.51				
	Tampa test	0.76	2.44	15	264	Limestone	0.56	0.48	1.38				
Seol et al. 2008	Load test ^c	1.07	4.2	35	277 ^b	Shale	0.51	0.41	2.05				
	Load test	0.45	0.56	35	691	Andesite	0.92	1.02	8.18				
Shoib et al. 2017	Nam pile R ^c	0.76	2.9	35	900	Limestone	0.91	1.01	0.67				
	V1-PTP2, F-G	1.2	1.5	35	1360 ^b	Granite	1.43	2.01	4.12				
Spanovich and Garvin 1979	V3-PTP2, F-H	1.35	1.8	35	1117	Granite	1.26	1.65	4.41				
	Caisson 2	0.76	1.5	35	259 ^b	Shale	0.48	0.38	8.67				
Thorne 1980	Caisson 3	0.76	0.9	35	259 ^b	Shale	0.48	0.38	24.09				
	Caisson 4	0.46	0.9	35	259 ^b	Shale	0.48	0.38	8.83				
	Caisson 7	0.61	1.5	35	259 ^b	Shale	0.48	0.38	5.59				
	Caisson 9	0.61	0.9	35	259 ^b	Shale	0.48	0.38	15.52				
	P3	0.45	0.7	40	625	Shale	1.36	1.86	6.61				

Table 3 (concluded).

Reference	Socket dimensions (m)			Elastic moduli		Rock unconfined compressive strength, UCS (q_u ; MPa)			Ultimate side stress, f_{su} (MPa)		
	Pile ID	Diameter (D)	Length (L)	Pile (E_p ; GPa)	Rock mass (E_{rm} ; MPa)	Rock type	WJD method (eq. 3)	CK method (eq. 7)	Rigidity criterion ^a (eq. 8)		
Turner et al. 1993 Walter et al. 1997 Webb and Davies 1980	Denver VI ^c	0.79	3.05	35	197 ^b	0.84	Claystone	0.4	0.29	2.98	
	UTS ^c	0.91	4	35	538 ^b	6.3	Siltstone	0.78	0.8	0.84	
	1A	0.47	1.08	35	100	2.5	Sandstone	0.58	0.5	16.57	
	1B	0.45	1.75	35	100	2.5	Sandstone	0.58	0.5	5.79	
	1C	0.45	2.77	35	100	2.5	Sandstone	0.58	0.5	2.31	
	2B	0.45	0.9	35	100	2.5	Sandstone	0.58	0.5	21.88	
	2C	0.536	1.3	35	100	2.5	Sandstone	0.58	0.5	14.87	
	2D	0.45	0.5	35	100	2.5	Sandstone	0.58	0.5	70.88	
	2F	0.463	1.67	35	100	2.5	Sandstone	0.58	0.5	6.73	
	3A	0.45	0.6	35	800	18	Sandstone	1.1	1.35	6.15	
Williams 1980	3B	0.45	0.8	35	800	18	Sandstone	1.1	1.35	3.46	
	3C	0.45	1.6	35	800	18	Sandstone	1.1	1.35	0.87	
	M1 ^d	1.22	2	35	500	2.5	Mudstone	0.58	0.5	6.51	
	M2 ^d	1.3	2	35	445	2.3	Mudstone	0.56	0.48	8.31	
	M3 ^d	1.23	2	35	445	2.3	Mudstone	0.56	0.48	7.44	
	M4 ^d	1.35	2	35	460	2.34	Mudstone	0.57	0.49	8.67	
	ES	1.09	1.52	35	60	7.2	Mudstone	0.82	0.85	74.99	
	S1	0.66	1.52	35	200	0.83	Mudstone	0.4	0.29	8.25	
	S3	1.17	2.59	35	200	0.57	Mudstone	0.36	0.24	8.93	
	S5	1.17	2.51	35	200	0.59	Mudstone	0.36	0.24	9.51	
Williams and Ervin 1980 Wong and Oliveira 2012	S12	0.335	0.9	35	200	0.58	Mudstone	0.36	0.24	6.06	
	S14	0.395	0.87	35	200	0.58	Mudstone	0.36	0.24	9.02	
	S15	0.395	0.87	35	200	0.58	Mudstone	0.36	0.24	9.02	
	WG303/2 ^d	1.58	2	35	1000	3.49	Mudstone	0.64	0.6	5.46	
	Side test	1.092	1.52	35	70	7.25	Mudstone	0.82	0.86	64.52	
	SC-02 ^c	0.75	5.6	43	2000	30	Sandstone	1.3	1.74	0.10	

^aAccording to Carter and Kulhawy (1988), sockets with this criterion >1 can be considered rigid.^bEstimated from $E_m = 215\sqrt{q_u}$.^cO-Cell test.^dAn early version of the bidirectional O-Cell test (Johnston et al 1980).

Table 4. Summary of qualitative observations of patterns describing the relationship of the predicted responses to the measured responses and to each other.

Pattern observed	Example reference	Fig. No.	Comment
Where WJD and CK predictions reasonably agree with measured load–settlement curves	Boyd and Ozsoy 2013; TP1	7a	Relatively uncommon
Where WJD and CK methods underpredict measured load–settlement curves	Matich and Kozicki 1967; Caisson No. 4	6a	Most common situation with CK predictions below that of WJD
Where both WJD and CK methods overpredict measured load–settlement curves	Webb and Davies 1980; 3A	7b	Relatively uncommon
Where CK predictions reasonably agree with measured load–settlement curves, but WJD method underpredicts measured load–settlement curves	Wong and Oliveira 2012; SC-02	7c	Relatively uncommon
Where CK predictions reasonably agree with measured load–settlement curves, but WJD method overpredicts measured load–settlement curves	Ng et al. 2001; Pile 7	7d	Relatively uncommon
Where WJD predictions reasonably agree with measured load–settlement curves, but CK underpredicts measured load–settlement curves	Williams 1980; M1	6b	This is commonly observed
Where WJD predictions reasonably agree with measured load–settlement curves, but CK overpredicts measured load–settlement curves	—	—	No examples
Where WJD method overpredicts and CK method underpredicts measured load–settlement curves	O'Neill et al. 1996; TAMU test	6c	This is commonly observed
Where CK overpredicts and WJD underpredicts measured load–settlement curves	—	—	No examples

By digitising the various design charts, the author has developed spreadsheets to perform the design and produce load–settlement curves for side-resistance-only piles.

Carter and Kulhawy (CK) method

This method is based on analytical techniques to describe elastic and non-elastic behaviour of a socketed pile. Using the input data with several equations describing elastic, side slip, and side shearing mechanisms, the load–settlement response can be developed.

For a side-resistance-only pile, the response is modelled by the three linear legs shown in Fig. 5. The first leg, with slope S_{1s} , describes the elastic behaviour. The intermediate leg with slope S_{2s} , (and intercept on the load axis of I_s) describes movement with full slip along the side of the socket, and once the ultimate load, $Q_u (=f_{su} \pi DL)$, is generated, the pile settlement will continue to increase but with no increase in load. The parameter f_{su} is estimated from the following relationship:

$$(7) \quad \frac{f_{su}}{p_a} = C \sqrt{\frac{q_a}{p_a}}$$

where p_a is atmospheric pressure in the appropriate units, and C is a variable that reflects the degree of conservatism required. As with the WJD method, the CK method makes no attempt to differentiate between different degrees of roughness on the socket wall.

The original CK method suggests a conservative $C = 0.63$, but according to Kulhawy et al. (2005), a value of 1 reflects a reasonable average of a considerable range of measured data for rock sockets. Therefore, in the application of the CK method, $C = 1$ will be adopted.

As was discussed in Johnston (2020), there can be some difficulty with the CK method when evaluating input parameters for the general solution for a compressible pile relative to the surrounding rock. However, this can be overcome by considering a rigid side-resistance-only pile, which is defined by

$$(8) \quad \frac{E_c}{E_m} \left(\frac{D}{2L} \right)^2 \geq 1$$

For values of E_c , E_m , L , and D typically encountered with rock-socketed piles, eq. 8 often holds, allowing the simpler relationships for rigid piles to be used. A summary of the method's key

relationships for the development of a load–settlement curve for a rigid side-resistance-only rock-socketed pile is provided in Tables 1 and 2. A summary of the CK method can be found in Kulhawy and Carter (1992).

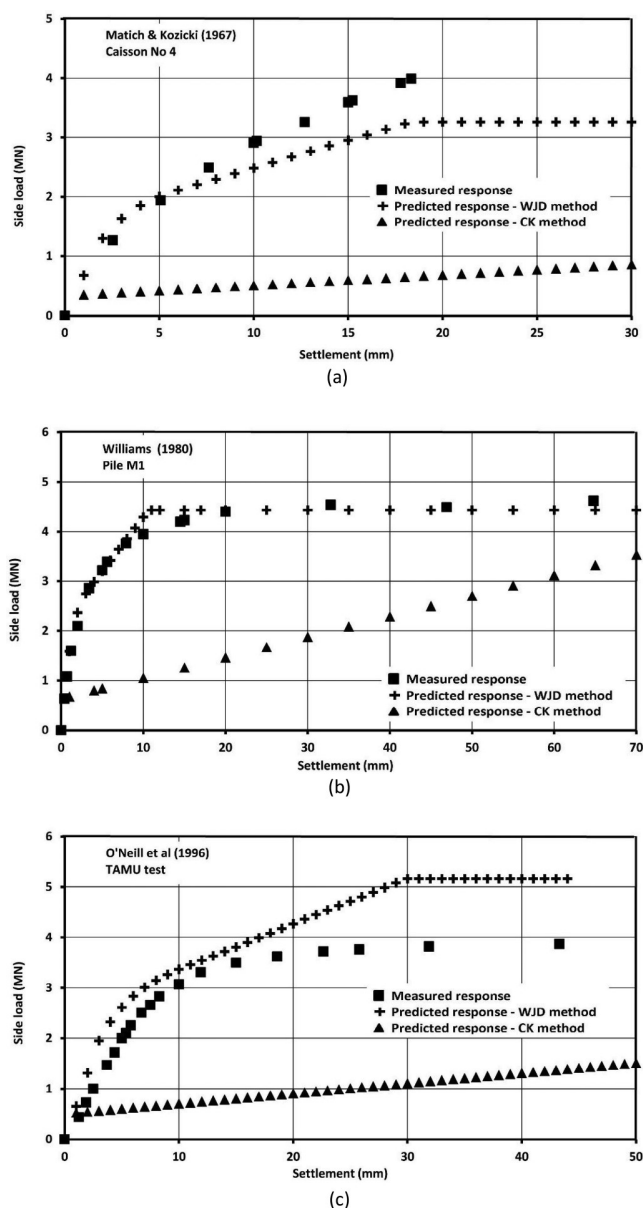
In applying the CK method to design, it would be necessary to establish the ultimate resistance required to be developed on the side of the socket by eq. 4 and the length of the socket required to provide this resistance by eq. 5 with f_{su} defined by eq. 7. It would then be necessary to construct the load–settlement curve with the relevant pile geometry (L , D) and material properties (E_c , E_m , q_a) to check that the settlement criterion is satisfied. An example of the application of the CK method is presented in Appendix A.

Other methods

As was noted above, there are other methods available for the design of rock-socketed piles that can predict performance with load–settlement curves. One of these involves the program ROCKET, which is based on the work of Seidel (1993) and requires input quantifying the roughness of the socket side. Unfortunately, the considerable specialist knowledge and expertise required to compile these data and the difficulty of obtaining data on roughness at some considerable depth in a drilled shaft limits the applicability of this approach. Another method that can produce a load–settlement curve was proposed by O'Neill et al. (1996) and Hassan et al. (1997). This method is based on the results of finite element modelling of the load-transfer mechanisms along the side of the socket, and along with other detailed input data, it must be decided whether the side of the socket is either rough or smooth. The effect of this decision leads to a smooth socket significantly underpredicting the performance of a rough socket. Regrettably, for the many sockets that have a roughness somewhere between these two extremes, even if this could be quantified, there is no apparent means of developing a prediction of response for this roughness.

Another approach to developing load–settlement curves for rock-socketed piles involves the use of load-transfer methods using t - z curves as proposed by Coyle and Reece (1966). However, these methods (e.g., Seol et al. 2008) are dependent on assigning representative t - z curves to the side of a socket, and this generally requires an understanding of the many other factors affecting performance. As noted above, the difficulty with assessing these limits the applicability of this approach.

Fig. 6. Load–settlement patterns most frequently observed with predicted and measured responses.



Another group of methods make use of multipurpose software packages such as PLAXIS and FLAC. However, this software has not been specifically designed for socketed piles and requires considerable expertise and calibration. Therefore, while these methods have undoubted potential, their application will not be considered here.

Comparison of load–settlement predictions with measured data

As discussed above, load–settlement predictions are integral to satisfying load and settlement design criteria. It follows that an assessment of how well the two design methods can predict the load–settlement response of piles is of considerable interest. This section presents a selection of load–settlement results from pile

load tests on side-resistance-only rock-socketed piles. The great majority of these piles were constructed with an effective void at their bases so that all applied vertical load was transferred to side resistance. There were a few tests where instrumentation permitted the side resistance component to be identified and evaluated. Using the reported pile geometry (L , D), the properties of the pile (E_c) and of the rock forming the socket (E_m , q_a), an assessment was made of how well the two socketed pile design methods (i.e., the WJD and CK methods) can predict these results. In all these tests, it is only the settlement of the rock socket that is considered, with no allowance for the compression of the pile shaft that may develop above the socket.

While there are many side-resistance-only pile socket tests reported in the literature, a significant proportion of these results could not be used for the reasons discussed in Johnston (2020). In recent years, mainly for reasons of cost-effectiveness, O-Cell tests (Osterberg 1998) have become popular, particularly for rock-socketed piles where the generally large loads involved make conventional load testing very costly. Where, for reasons discussed in Johnston (2020), these results appear reliable, they have been used. For O-Cell tests, the side resistance component was relatively easy to evaluate because of the separation of the downward loading on the base section and the upward loading of the side section.

Table 3 summarises the tests on side-resistance-only pile sockets (and O-Cell tests) used in this investigation. To be included, in addition to the criteria discussed above, the test data had to include the socket dimensions, (L , D), and reliable estimates of q_a and E_m . There were several reported test results that had all the above except E_m . To expand the available database, these tests were included with a reasonable estimate of E_m based on correlations between q_a and E_m . As discussed in Johnston (2020), the relationship proposed by Rowe and Armitage (1987a) was adopted and is given by

$$(9) \quad E_m = 215\sqrt{q_a}$$

Table 3 also presents the value of the relationship used to assess the rigidity of the rock-socketed pile as defined in the CK method by eq. 8.

As it would be impractical to present all the predicted and measured load–settlement data for the 68 tests results in Table 3, although some are presented herein, these are presented as Figs. S1 to S68 in the online Supplementary Data¹.

Discussion of the measured and predicted load–settlement curves

Before embarking on a detailed statistical assessment of all 68 sets of data, some general qualitative observations will be made. Table 4 gives a range of patterns describing the relationship of the predicted responses to the measured responses along with a qualitative assessment of the frequency of occurrence of these patterns in the data set. Figures 6 and 7 give examples of tests that display these patterns.

As is stated in Table 4, the most common pattern was when both the WJD and CK methods predicted load–settlement curves that underpredicted the measured load–settlement curves, with the WJD predictions a little under the measured load–settlement curves and the CK predictions further under the measured load–settlement curves (Fig. 6a). The other commonly encountered patterns were where the WJD method reasonably predicted or overpredicted the measured load–settlement curves, with the CK data still underpredicting (Figs. 6b and 6c, respectively). Figure 7 shows patterns that were infrequent or unusual.

¹Supplementary data are available with the article at <https://doi.org/10.1139/cgj-2021-0249>.

Fig. 7. Load–settlement patterns infrequently observed with predicted and measured responses.

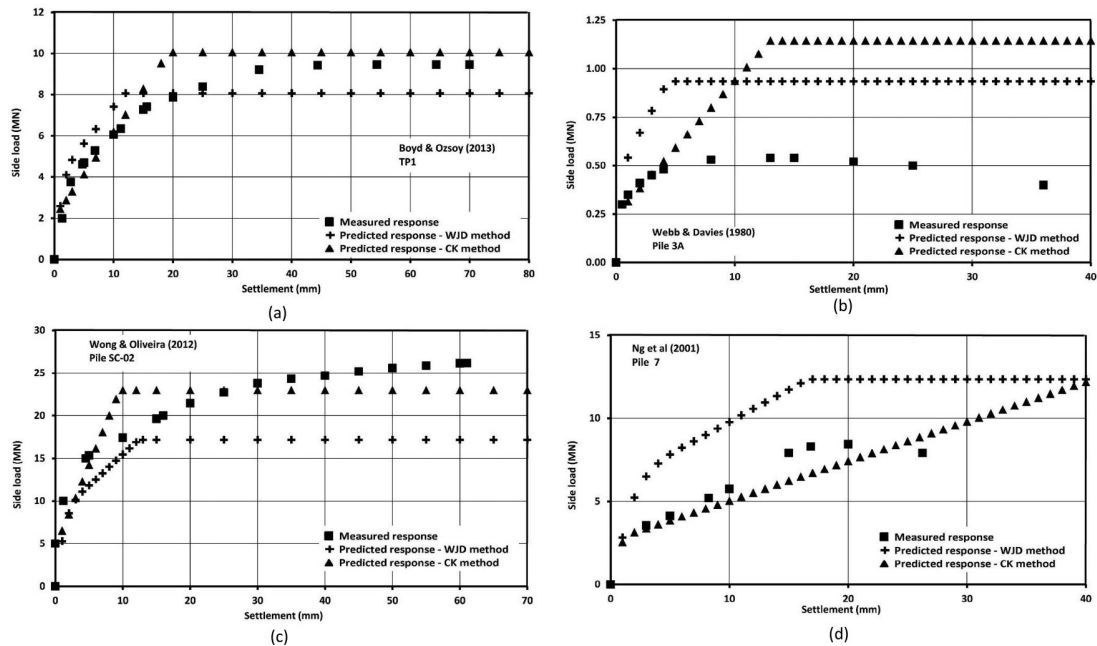


Table 5. Statistical parameters developed from Figs. 8–11.

Statistical parameter	For loads at 5 mm settlement		For loads at 10 mm settlement		For loads at 15 mm settlement		For loads at 20 mm settlement	
	WJD	CK	WJD	CK	WJD	CK	WJD	CK
Mean bias ^a	1.18	3.70	1.14	3.78	1.11	3.43	1.15	3.26
Standard deviation of bias (SDB) ^b	0.45	2.49	0.41	2.65	0.41	2.50	0.43	2.29
Coefficient of variation of bias (CVB) ^c	0.38	0.67	0.36	0.70	0.37	0.73	0.37	0.70
Root mean squared error (RMSE) ^d (MN)	1.89	3.83	1.67	4.22	1.85	4.71	2.09	5.12
Mean absolute error (MAE) ^e (MN)	1.26	2.44	1.23	2.67	1.24	2.78	1.28	2.84

^aMean of individual measured to predicted loads; <1 is overpredicted, >1 is underpredicted.
^bIndicating the standard deviation about the mean bias or trend line of Figs. 8–11.
^cIndicating the variability relative to the mean bias or trend line in Figs. 8–11.
^dIndicating the variability relative to the line of equality in Figs. 8–11, units in MN.
^eAn alternative to RMSE, units also in MN.

It may be worth commenting that in Fig. 6a, the predicted CK load–settlement curve lies significantly below the predicted WJD load–settlement curve, and yet their respective ultimate side stresses, f_{su} , as indicated in Table 3, are much closer at 0.3 and 0.41 MPa, respectively. This is a result of the slow development of side shear resistance according to the CK method, which reaches its full mobilisation of 2.36 MN at a settlement of about 116 mm, well off the scale of the figure shown. The WJD curve reaches full mobilisation of 3.23 MN at a settlement of about 19 mm. The ratio of these two predicted values of fully mobilised side resistances is 1.37, the same as the ratio of their respective values of f_{su} . The same general comment applies to all other figures showing this apparent anomaly, including Fig. 6c, where the fully mobilised side resistance according to the CK method is 3.69 MN at a settlement of 158 mm.

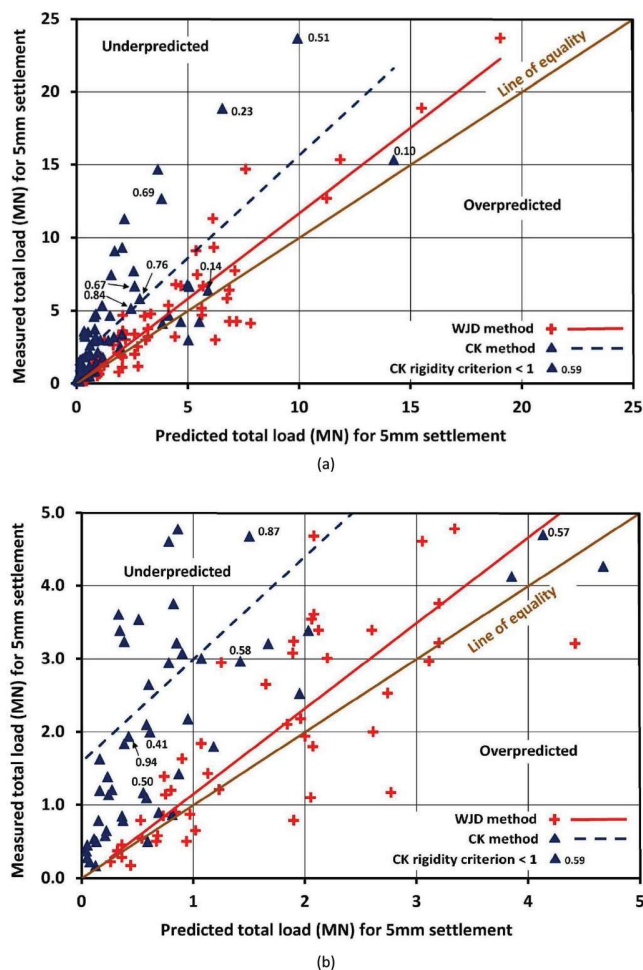
Another general observation is that of the 68 tests considered, 47 showed a clearly defined ultimate load that occurred at settlements of between about 10 and 30 mm settlement. There were only a handful of tests that showed a reduction in load with settlement, and even then, the reduction was relatively minor.

With the considerable number of load test results and predictions of these results, a statistical approach is required to provide a clearer picture of overall performance. As was discussed by

Johnston (2020), settlements of socketed piles are generally required to settle less (and sometimes considerably less) than about 20 mm under serviceability loads. Therefore, comparisons of performance are undertaken for settlements up to 20 mm. Figures 8–11 compare the pile loads measured in the suite of test results with the loads predicted by the WJD and CK methods for settlements of 5, 10, 15, and 20 mm, respectively. These settlements do not include compression of the pile shaft above the socket. Each of Figs. 8–11 have two parts: (a) showing all measured and predicted loads for each settlement and (b) showing the same at the low end of the load range measured to provide greater clarity for these lower range results. Linear trend lines were fitted for each of these data sets. Table 5 presents some statistical parameters related to the data presented in Figs. 8–11.

The data presented in Figs. 8–11 and Table 5 confirm the observations made earlier about the pattern of results. Specifically, the mean bias (i.e., the mean of the ratios of the measured load to the predicted load for each test) for the WJD method is a little more than 1 for all settlements, indicating that the method generally slightly underpredicts the measured load–settlement curves. The CK method, with a mean bias between 3 and 4, indicates that the method generally considerably underpredicts the measured

Fig. 8. Comparison of measured loads with predicted loads for settlements of 5 mm: (a) for the full load range of data and (b) for the low load range. [Colour online.]



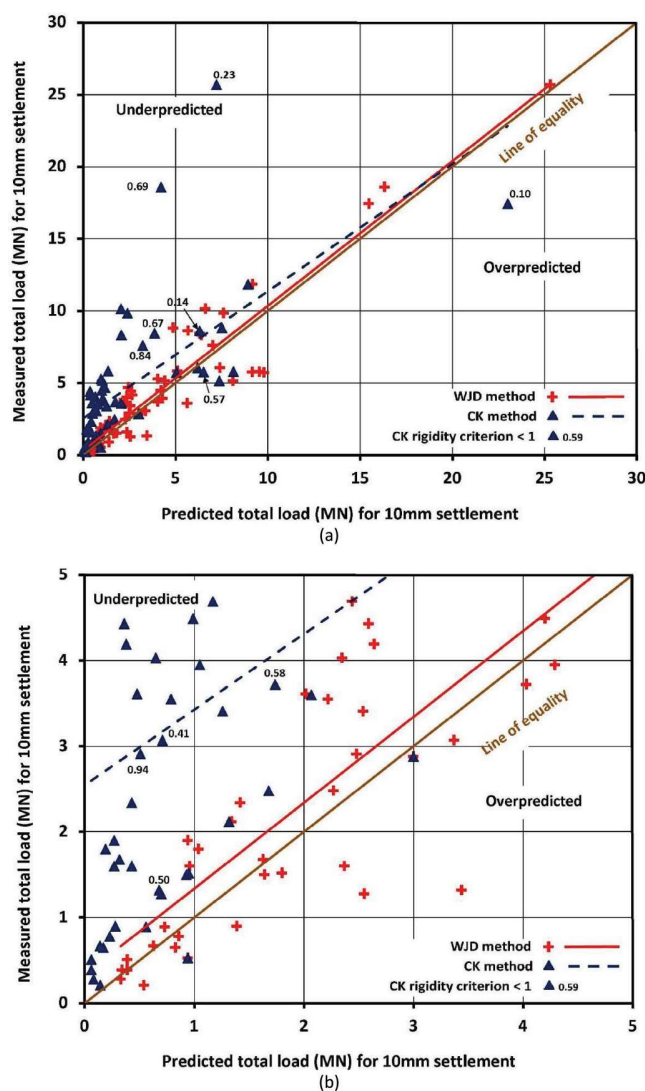
load–settlement curves. The standard deviation about the mean bias (SDB) and the coefficient of variation of the mean bias (CVB) indicate that variability about the mean bias or trend line are relatively small for the WJD method but significantly greater for the CK method. The root mean squared error (RMSE) and mean absolute error (MAE) indicate that the WJD method produces predictions of load relative to the measured values that are similar for all settlements and of relatively low variability. For the CK method, although the predictions are similar for all settlements, their variability is significantly higher.

As shown in Table 3, the condition for rigidity for 54 of the 68 load tests was satisfied, thereby justifying the use of the simpler solutions developed with the CK method. Another 14 tests, however, returned rigidity results below the limit of unity. Figures 8–11 identify these latter tests, with the value of the rigidity criterion presented adjacent to the individual points representing these tests. Apart from 2 to 3 points on the figures representing some of the larger total loads where the measured load is significantly underpredicted, the remaining points seem to be in a similar range to the test piles, which could be considered as rigid.

Assessing ultimate side stress

The ultimate side stress (f_{su}) for the WJD method is determined by eq. 3 (with α given in Fig. 3). The ultimate side stress for the CK method is determined by eq. 7 with $C = 1$.

Fig. 9. Comparison of measured loads with predicted loads for settlements of 10 mm: (a) for the full load range of data and (b) for the low load range. [Colour online.]



As was discussed by Johnston (2020), there have been many studies involving the prediction of f_{su} in reasonably intact rock, and it has been shown that it can be influenced by many factors. Some factors, such as q_a , E_m , E_c , L , and D , can be evaluated relatively easily, but other factors and especially the roughness of the interface between the side of the pile and its socket are not so easy to assess. Even if the roughness could be described, its influence on f_{su} is far from well-understood (Rezazadeh and Eslami 2017; Haberland and Lochaden 2019; Asem and Gardoni 2019; Xu et al. 2021; Gutiérrez-Ch et al. 2020). It follows that when considering the complex factors influencing the development of side resistance, it is perhaps not surprising that f_{su} is difficult to accurately assess, and the scatter of test data are significant.

While it is always possible that a practical relationship involving these many factors may be developed to more accurately and consistently predict f_{su} , as discussed by Johnston (2020), it seems reasonable to suggest that the simple relationship proposed by Kulhawy et al. (2005) for the assessment of f_{su} and given as eq. 7 with $C = 1$ provides an indication of what might be an acceptable practical “average”. This is the relationship used in the CK method. Figure 12 compares

Fig. 10. Comparison of measured loads with predicted loads for settlements of 15 mm: (a) for the full load range of data and (b) for the low load range. [Colour online.]

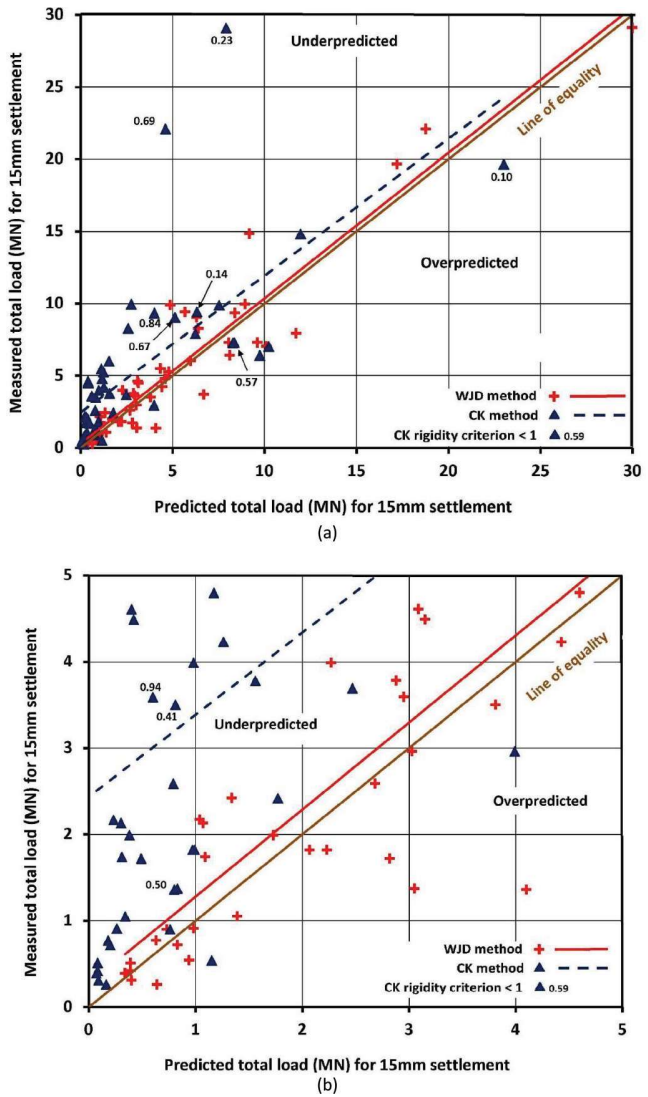


Fig. 11. Comparison of measured loads with predicted loads for settlements of 20 mm: (a) for the full load range of data and (b) for the low load range. [Colour online.]

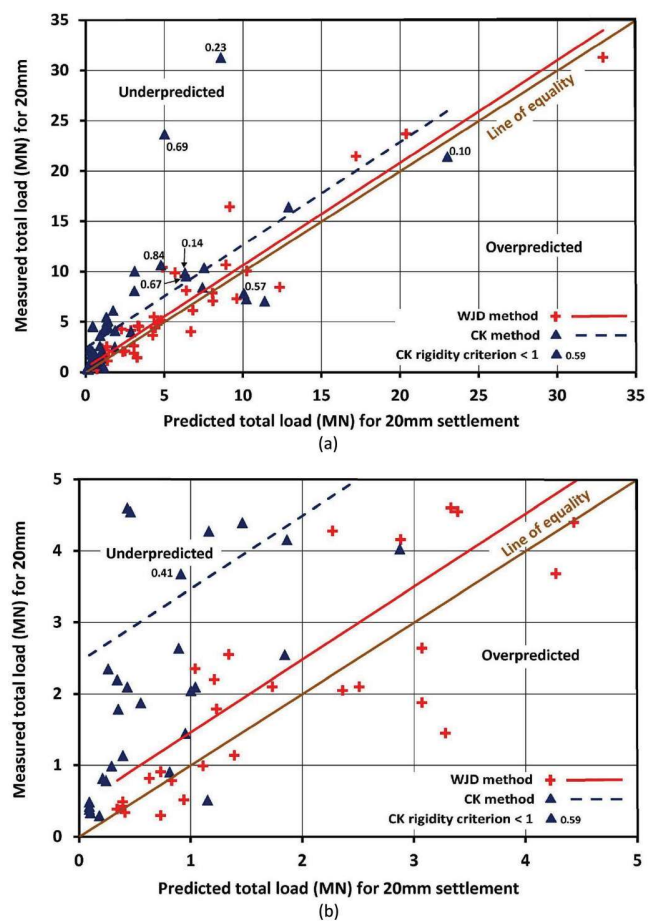
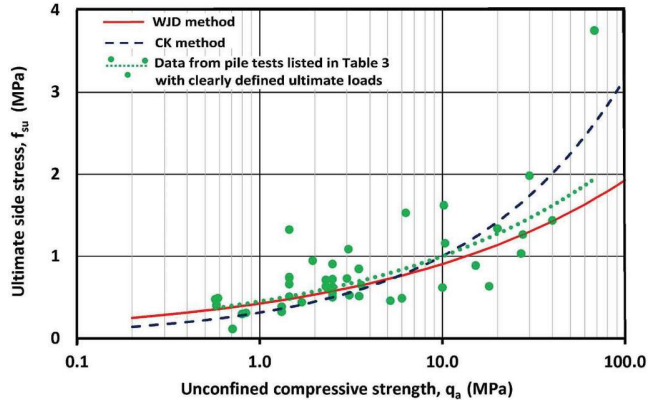


Fig. 12. Variation of f_{su} with q_a according to the Williams, Johnston, and Donald (WJD) and Carter and Kulhawy (CK) methods, along with data from the tests listed in Table 3. [Colour online.]



the WJD and CK methods for assessing f_{su} against q_a . This figure shows that the CK method (with $C = 1$) produces f_{su} values that are less than the WJD method for q_a less than about 6 MPa, with the WJD method producing lower values for q_a more than about 6 MPa. Figure 12 also presents the pairs of f_{su} and q_a extracted from each of the tests listed in Table 3, which clearly defined an ultimate load.

As the CK method with $C = 1$ is the Kulhawy et al. (2005) assessment of f_{su} and was developed many years later than the WJD assessment with more data sets than was possible with the original WJD assessment in 1980, it was thought that if the WJD predictions were repeated but with f_{su} determined by the Kulhawy et al. (2005) relationship with $C = 1$, the WJD prediction might improve. The outcome of this investigation was that there were only small variations in the load-settlement predictions of the 68 pile tests considered, with only minor movements of the data points shown in Figs. 8–11. A re-examination of the data presented in Table 5 indicated a small increase in all the statistical parameters for the range of settlements considered, suggesting that the original WJD method for assessing f_{su} leads to slightly better correlations with the measured loads.

Where design is controlled by load criteria rather than settlement criteria, the choice of f_{su} can have a relatively large impact on side-resistance-only rock-socketed piles. The method by which f_{su} is assessed comes down to a design choice that could be further influenced by the results of representative pile load tests or

of representative constant normal stiffness (CNS) direct shear tests (Lam and Johnston 1982; Johnston et al. 1987) conducted on rock samples from the design site.

Whichever approach is adopted for the assessment of f_{su} , it is unlikely that a justifiably better or more representative practical method will be found to include the influence of roughness (beyond being very smooth or very rough) simply because of the difficulty in establishing the roughness in a pile socket as drilled, how this can be quantified, and what effect this has on the side shear behaviour. Practical methods to assess the strength parameter f_{su} will probably continue to be dominated by the convenient and conventional rock strength parameter q_a , even though q_a itself can be very much harder to establish than might appear (Johnston 1985).

As about 47 of the 68 tests results reported in Table 3 displayed clearly defined ultimate loads, it was possible to develop a data set relating the measured f_{su} with the site-specific q_a . This data set is also shown in Fig. 12, and a best-fit curve has been developed for these data. This curve demonstrates that different data sets will inevitably produce different general relationships between f_{su} and q_a .

Conclusions

In an earlier paper (Johnston 2020), the load–settlement predictions of three design methods, originally proposed in the 1980s for rock-socketed piles, were compared with the measured load–settlement response of full-scale load tests. The piles concerned were complete piles for which both the side and the base resistance contribute to the total load. For complete piles in reasonably competent rock, because base resistance develops at a much slower rate than the side resistance, it is settlement that usually controls design.

For situations where base resistance cannot be relied on, piles can be designed as side-resistance-only sockets. This topic was examined in this paper by considering the methods proposed by Williams et al. (1980) and Carter and Kulhawy (1988). The method proposed by Rowe and Armitage (1987a) could not decouple the base resistance from the side resistance and therefore could not be considered.

Because base resistance is not included, the controlling factor in design can be either load or settlement. It follows that either of these criteria can be the starting point for design, with the other checked subsequently.

As would be expected, and as was observed with complete piles, neither of the methods produced perfect predictions of load–settlement behaviour for the many test pile results examined. Analysis of the WJD predictions indicated that this method generally provided slightly conservative but reasonable predictions for load–settlement response, with a moderate variability in terms of underprediction and overprediction. It appears that the WJD predictions for the load required to develop settlements in the range of 5 to 20 mm was consistent for the tests investigated.

Although as demonstrated by Johnston (2020), the CK method makes reasonable predictions of the performance of complete piles, but it appears to generally underpredict the performance of side-resistance-only socketed piles. It follows that the CK method could be considered to give generally conservative predictions of performance.

A key decision in the design process is the method used to assess the ultimate side stress, f_{su} , particularly for piles where design is controlled by load criteria rather than settlement criteria. This decision has a direct effect on the required socket length. There are several relationships available, including the two used earlier with the two design methods. This comes down to a design choice that may be assisted by conducting representative pile tests and (or) CNS tests in the laboratory. It is argued that while socket roughness must have a major influence on f_{su} , because of the difficulty in defining and measuring roughness and its variability in a drilled socket, how it can be quantified and

how roughness affects performance suggests that a more accurate but practical method is unlikely to be developed.

As with the earlier paper on complete piles (Johnston 2020), there are several factors not considered in the above discussions. These include the influence of overburden on shaft resistance, the compression of the shaft above the socket, any variability of the materials forming the socket, there being other than clean contact between the concrete of the pile shaft and the surrounding rock, softening of the exposed rock in the socket after drilling has been completed, and inadequate roughness of the socket walls.

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Appendix A. Examples of design methods

Three examples of the design methods for side-resistance-only rock-socketed piles are presented. Two of the examples use the WJD method and one the CK method. The input data for the first two examples are provided in [Tables A1](#), and [Table A2](#) shows the parameters derived for a maximum allowable design settlement of 5 mm. Both examples use a load factor $\gamma_f = 1.3$ and a resistance factor $\phi_1 = 0.5$, indicating a factor of safety of 2.6. For Example 1, using a rock mass modulus of 400 MPa, the relaxed load for a socket of length of 4.3 m as determined by the WJD method is 5.83 MN. As this is greater than the design load of 4 MN, the design is controlled by the load criterion and the length of 4.3 m is satisfactory. The resulting load–settlement curve, shown in [Fig. A1](#), indicates that the settlement of this socket under design load is 2.4 mm. For Example 2, using a rock mass modulus of 150 MPa, the relaxed load for a socket of length of 4.3 m as determined by the WJD method is 3.39 MN, which is less than the design load. Further, for this pile, as shown in [Fig. A1](#), the design load of 4 MN would produce about 6 mm settlement. This indicates that the design is controlled by settlement and the length of the socket must be increased until the settlement under design load decreases to 5 mm. The design spreadsheet suggests that the socket length needs to be increased to about 6.1 m.

The same input data as used in Example 1 have been applied to the CK method. The resultant load–settlement curve is shown in [Fig. A1](#) along with the two WJD-based curves. Note that, as defined by [eqs. 3](#) and [7](#), the respective values of f_{su} for both the WJD and CK methods are effectively the same for $q_a = 6$ MPa. The load–settlement curve for the CK method indicates that for a 4.3 m long socket, the settlement corresponding to a design load of 4 MN is about 17.2 mm. This means that settlement controls design and the length of the socket must be increased until the settlement under design load decreases to 5 mm. The design spreadsheet suggests that according to the CK method, the socket length needs to be increased to about 7.0 m.

Table A1. Input data for examples of the WJD method.

Example No.	Q_d (MN)	ρ_{dmax} (mm)	q_a (MPa)	E_m (MPa)	E_c (GPa)	D (m)	Load factor γ_i	Resistance factor φ_i
1	4	5	6	400	35	1	1.3	0.5
2	4	5	6	150	35	1	1.3	0.5

Table A2. Derived parameters for side-resistance-only socket design using WJD method for maximum allowable design settlement of 5 mm.

Parameter	Source	Example 1: $E_m = 400$ MPa	Example 2: $E_m = 150$ MPa
Q_{su} (MN)	Eq. 4	10.40	10.40
α	Fig. 3	0.128	0.128
f_{su} (MPa)	Eq. 3	0.77	0.77
L (m)	Eq. 5	4.3	4.3
L/D	—	4.3	4.3
E_c/E_m	—	88	233
I_ρ	Fig. 4	0.236	0.221
Q_{se} (MN)	Eq. 6	8.47	3.39
f_{se} (MPa)	$= Q_{se}/(\pi DL)$	0.63	0.25
f_{se}/f_{su}	—	0.82	0.33
f_{sp}/f_{su}	Fig. 2	0.26	0
f_s/f_{su}	—	0.56	0.33
f_s (MPa)	$= (f_s/f_{su})f_{su}$	0.43	0.25
Q_s (MN)	$= f_{su}\pi DL$	5.83	3.39
Settlement at Q_d (mm)	Load-settlement curve	2.4	6.0
Controlling criterion	—	Load	Settlement
Design length (m)	—	4.3	6.1

Fig. A1. Load–settlement curves for side-resistance-only socketed pile design examples.

