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Design charts for piles socketed into rock

Graphiques de conception pour pieux encastrés dans la roche

Ian W. Johnston
Department of Infrastructure Engineering, University of Melbourne, Vic 3010, Parkville, Victoria, Australia, ian.w.johnston@outlook.com

ABSTRACT: A method for the design of piles socketed into rock was published over 40 years ago by Williams et al. (1980). The method appears to describe pile response in a range of rock types of varying strength with a reasonable degree of accuracy. Recently, the design curves in the original paper were digitized and included in spreadsheets for rapid socketed pile design. This has led to the development of a series of design charts for use in preliminary design. This paper briefly explains the method and presents charts which can quickly provide the dimensions of socketed piles to suit a wide range of likely conditions.

RÉSUMÉ : Une méthode de conception de pieux encastrés dans la roche a été publiée il y a plus de 40 ans par Williams et al. (1980). La méthode semble décrire la réponse des pieux dans une gamme de types de roches de résistance variable avec un degré de précision raisonnable. Récemment, les courbes de conception du papier original ont été numérisées et incluses dans des tableur pour la conception rapide de pieux encastrés dans la roche. Cela a conduit à l'élaboration d'une série de graphiques de conception à utiliser dans la conception préliminaire. Cet article explique brièvement la méthode et présente des graphiques de conception qui peuvent fournir rapidement les dimensions pieux encastrés dans la roche pour convenir à un large éventail de conditions probables.

KEYWORDS: piled foundations, rock sockets, design

1 INTRODUCTION.

In the 1970s and 1980s, there was considerable international activity involving the design and construction of rock socketed piles. Although there has been a continuing interest since then, particularly with respect to the assessment of ultimate side resistance, there has been little significant development in methods for design for complete piles for which both side and base resistances contribute. The design methods proposed by Williams, Johnston, and Donald (1980), Rowe and Armitage (1987) and Carter and Kulhawy (1988) remain the principal means of designing rock socketed piles. As has been argued in these three design methods, and more recently by Haberfield and Lochaden (2019), for piles in reasonably competent rock utilising side and base resistance, it is axial pile settlement which often controls design. Therefore, methods which consider settlement as well as bearing are required.

In a recent paper, Johnston (2020) reviewed the three methods and compared their predictions for load-settlement response with the results of several pile load tests. It was demonstrated that while all three methods can produce reasonable predictions of performance, the Williams, Johnston, and Donald (or WJD) method appears to have some advantages over the other two.

As the manual use of the several design curves in the original WJD paper is time consuming, the author has digitized them and developed spreadsheets for rapid design purposes. These have been used to construct design charts which provide a convenient means for assessing preliminary design dimensions for a range of rock properties and serviceability criteria.

2 SUMMARY OF THE DESIGN METHOD

The WJD design method involves non-dimensional curves derived from many field tests conducted on piles socketed into the Silurian mudstone of Melbourne (Williams 1980). The following sections give a brief description of the method, but more details can be found in Williams et al. (1980).

2.1 *Side resistance*

The WJD method makes use of a side resistance reduction factor, α, defined in terms of the ultimate side resistance, f_{su}, and the unconfined compressive strength, q_a, as

$$\alpha = \frac{f_{su}}{q_a} \tag{1}$$

The relationship adopted for design is shown in Fig. 1 and was developed from field scale test results as available at the time.

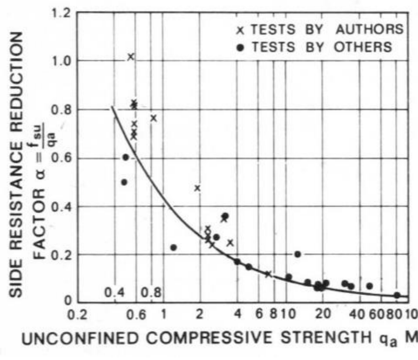

Figure 1. Side resistance reduction factor variations with rock strength (after Williams et al. 1980, with permission from Taylor and Francis).

The principles for normalising side resistance are illustrated in Fig. 2a. For any settlement, ρ, the side resistance on the pile is given by f_s which is

$$f_s = f_{se} - f_{sp} \tag{2}$$

where f_{se} is the elastic side resistance and f_{sp} is the plastic reduction of side resistance as defined in Fig. 2a.

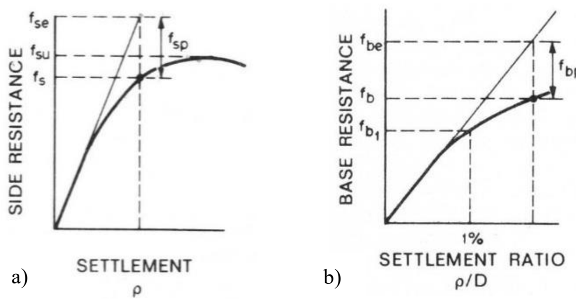

Figure 2. Principles for normalising (a) side resistance and (b) base resistance (after Williams et al, 1980, with permission from Taylor and Francis).

Eq. 1 is normalised by dividing by the ultimate side resistance, f_{su}, which was determined in all the side only tests, to give:

$$\frac{f_s}{f_{su}} = \frac{f_{se}}{f_{su}} - \frac{f_{sp}}{f_{su}} \tag{3}$$

This was applied to all the side resistance tests to produce the normalised design curve for side resistance as shown in Fig. 3.

2.2 *Base resistance*

The principles for normalising base resistance are illustrated in Fig. 2b. For a pile of diameter *D*, for any settlement ratio, *ρ/D*, the base resistance of the pile is given by f_b which is defined by

$$f_b = f_{be} - f_{bp} \tag{4}$$

where f_{be} is the elastic base resistance and f_{bp} is the plastic reduction of base resistance as defined in Fig. 2b.

Unlike the side resistance tests, few of the base resistance tests reached failure and an alternative to ultimate base resistance was required to normalise Eq. 4. After consideration of the effects of embedment on bearing capacity, it was decided to use the estimated base stress at a settlement ratio (*ρ/D*) of 1% or f_{b1} which could be reasonably estimated as

$$f_{b1} = N_s E_m \tag{5}$$

where N_s is a settlement-based bearing capacity factor given in Table 1, and E_m is the elastic modulus of the rock mass. Williams et al. (1980) explain the derivation of N_s in more detail.

Therefore, with the relevant *L/D* ratio, N_s can be selected and f_{b1} estimated from Eq. 5 to give the normalised base resistance as

$$\frac{f_b}{f_{b1}} = \frac{f_{be}}{f_{b1}} - \frac{f_{bp}}{f_{b1}} \tag{6}$$

This equation was applied to all base resistance tests to produce the normalised design curve shown in Fig. 4.

2.3 *Design process for a complete pile*

The WJD method uses the maximum allowable settlement as the principal input parameter for the design process because the relatively large piles considered at the time had low maximum allowable settlements (typically 10mm or less). For complete piles, and particularly where the diameter is large, the often considerable base resistance develops at a much slower rate than the side resistance, and rarely displays an ultimate load but increases with increased displacement. The net result is that

Table 1. Variation of N_s as a function of *L/D* for *ρ/D*=1%

L/D[a]	0	1	3	5	10	15
N_s	0.0065	0.0109	0.0147	0.0169	0.0185	0.0196

[a] N_s for intermediate values of *L/D* can be obtained by linear interpolation.

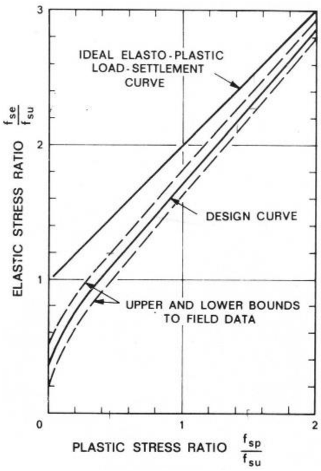

Figure 3. Normalised design curve for side resistance (after Williams et al. 1980, with permission from Taylor and Francis).

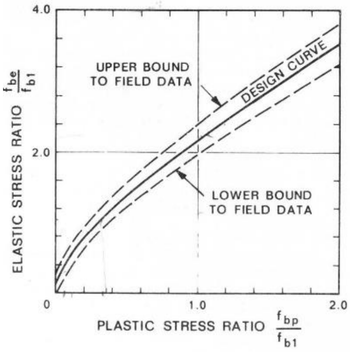

Figure 4. Normalised design curve for base resistance (after Williams et al. 1980, with permission from Taylor and Francis).

settlement often controls design. However, as will be shown, this is not always the case and bearing capacity must be checked.

The iterative WJD design process starts with an initial socket length based on all the design or serviceability load, Q_d, taken by the pile shaft. This gives this initial trial length as

$$L = \frac{Q_d}{f_{su}\pi D} \tag{7}$$

With the maximum allowable settlement of the pile, ρ_{max}, selected, the original WJD method applied a factor of safety, F_ρ, to this settlement and calculated the total elastic load, Q_e, to develop this factored settlement using

$$Q_e = \frac{\rho_{max} E_m D}{F_\rho I_\rho} \tag{8}$$

where I_ρ is a settlement influence factor given by Fig. 5 and E_c in Fig. 5 is the elastic modulus of the concrete in the pile. Then using Fig. 6, the distribution of this total elastic load between the side, Q_{se}, and base, Q_{be}, is established. The method now requires these elastic loads to be relaxed by the respective plastic reduction factors using Eqs 3 and 6, the normalising factor f_{su} from Fig. 1 and Eq. 1, *Ns* for *ρ/D* =1% from Table 1 and f_{b1} from Eq. 5.

This leads to the relaxed loads on the side and base of the socket, and therefore, the total relaxed load for the initial trial length. If this load is less than the design load (which it usually is), then the length of the socket is increased, and the procedure repeated until the total relaxed load equals the design load.

In the original WJD method, the final step is checking that there is a factor of safety of at least 2 against bearing failure as defined by the relationship:

$$FS = \frac{f_{su} A_s + f_{bu} A_b}{Q_d} \geq 2 \qquad (9)$$

where f_{bu} is the ultimate base resistance, and can be assessed as at least 5 times the unconfined compressive strength, q_a, for reasonably competent rock (Williams 1980; Williams et al. 1980; Choi 1984; Haberfield & Lochaden 2019) and A_s and A_b are the areas of the side and base of the socket.

While the use of a factor of safety was acceptable at the time the WJD method was developed, a different approach involving limit state design methods would need to be adopted now. The general relationship that applies is of the form:

$$Q_d \leq \frac{\varphi_{gs} f_{su} A_s + \varphi_{gb} f_{bu} A_b}{\Upsilon} \qquad (10)$$

where φ_{gs} and φ_{gb} are the geotechnical strength reduction or resistance factors for the side and base respectively and Υ is a relevant load factor for the limit state being considered. The allowable stresses in the concrete would also have to be checked.

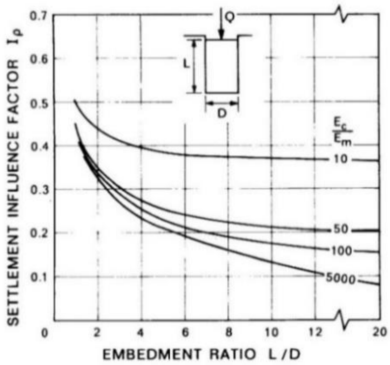

Figure 5. Elastic settlement influence factor against embedment ratio for different modular ratios (after Donald et al. 1980, with permission from Taylor and Francis).

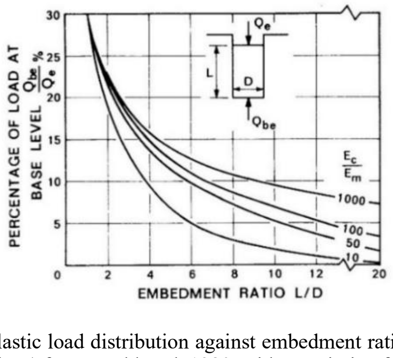

Figure 6. Elastic load distribution against embedment ratio for different modular ratios (after Donald et al. 1980, with permission from Taylor and Francis).

3 DESIGN CHARTS

As noted earlier, the author has developed spreadsheets for complete piles socketed into rock. One type uses the input parameters D, ρ_{max}, q_a, E_m, E_c and Q_d to produce a socket length L. This type of spreadsheet can also check that the design load (Q_d) is acceptable with respect to either a factor of safety or load and resistance factors. The other type uses the input parameters D, L, q_a, E_m, and E_c to produce a load-settlement response.

The first type of spreadsheet has been used to develop design charts for the length of socket required for a complete pile in a reasonably competent rock. The design charts are presented in Figs 7 and 8. Fig. 7 is for pile sockets of diameter 0.5m, 1.0m, 1.5m and 2.0m for maximum allowable settlements (ρ_{max}) of 5mm and 10mm. Fig. 8 is for pile sockets of the same diameters but for maximum allowable settlements of 15mm and 20mm. Each chart is entered from the horizontal axis at the known unconfined compressive strength (q_a) of the rock and tracing vertically to the line representing the design load (Q_d) defined by a full line for ρ_{max}=5mm and 15mm, and a dashed line for ρ_{max}=10mm and 20mm. The socket length complying with the settlement requirement can be read off the vertical axis. Strength requirements with respect to the bearing capacity of the rock and the stress in the concrete must also be checked.

In all cases, f_{su} is the value given by the WJD method (Fig. 1 and Eq. 1). The elastic modulus of the concrete of the pile, E_c, has been set at 35GPa. To limit the number of charts required, the elastic modulus of the rock has been fixed at

$$E_m = 215\sqrt{q_a} \qquad (11)$$

Eq. 11 was proposed by Rowe and Armitage (1987) for rock masses with no open discontinuities. It was seen to give a reasonable correlation based on an extensive survey of relevant data. If a significantly different value of E_m is considered more appropriate, then the charts should not be used and the full WJD method should be employed. For values of D, ρ_{max} and Q_d intermediate to those given in the design charts, linear interpolation will provide a reasonable estimate of length.

The range of unconfined compressive strengths (q_a) covered in the charts is from 0.5MPa to 50MPa. The lower limit was set because weaker materials are more soil-like and less likely to display socket dilation characteristics typically encountered with piles in rock. It also corresponds with the lower strength limit of materials defined as Intermediate Geomaterials by O'Neill et al. (1996). The upper limit was set at 50MPa because this roughly corresponds to the strength of concrete and, therefore, likely to see a somewhat different failure mechanism with both rock and concrete asperities failing.

As noted earlier, it is important that the axial load in the pile does not exceed the allowable stress of the concrete. As this stress is of the order of 25 to 30MPa, Q_d has been limited to 5MN for the 0.5m diameter piles and 20MN for the 1.0m diameter piles.

4 DISCUSSION

As noted earlier, for socketed piles in rock, it is often, but not always, settlement that controls design. Bearing must always be checked. The relative importance of these two criteria will be considered in this section.

Several of the design charts have circular symbols marked on some curves and defined in the legend as "FS = 2, f_{bu}=5q_a". These points represent a design where the settlement and the bearing criteria for that pile are of equal importance with the settlement at ρ_{max} and the bearing capacity based on Eq. 9 with $f_{bu}=5q_a$ giving a factor of safety of 2 against pile ultimate failure. Any design points to the upper left correspond to designs where strength controls and the length of the socket must be increased to satisfy strength criteria.

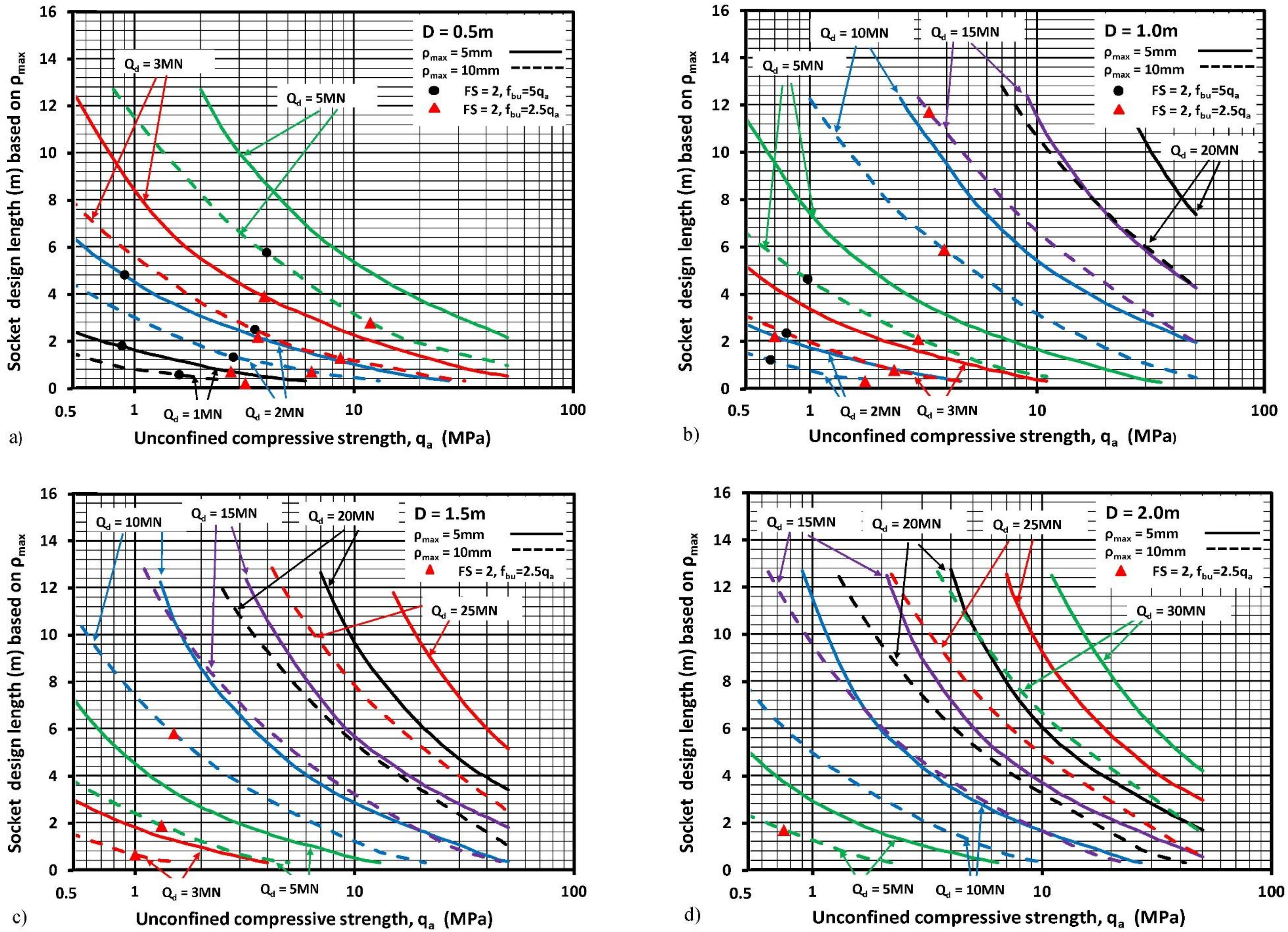

Figure 7. Design charts for ρ_{max}=5mm and 10mm for socket diameters of (a) D=0.5m, (b) D=1.0m, (c) D=1.5m and (d) D=2.0m.

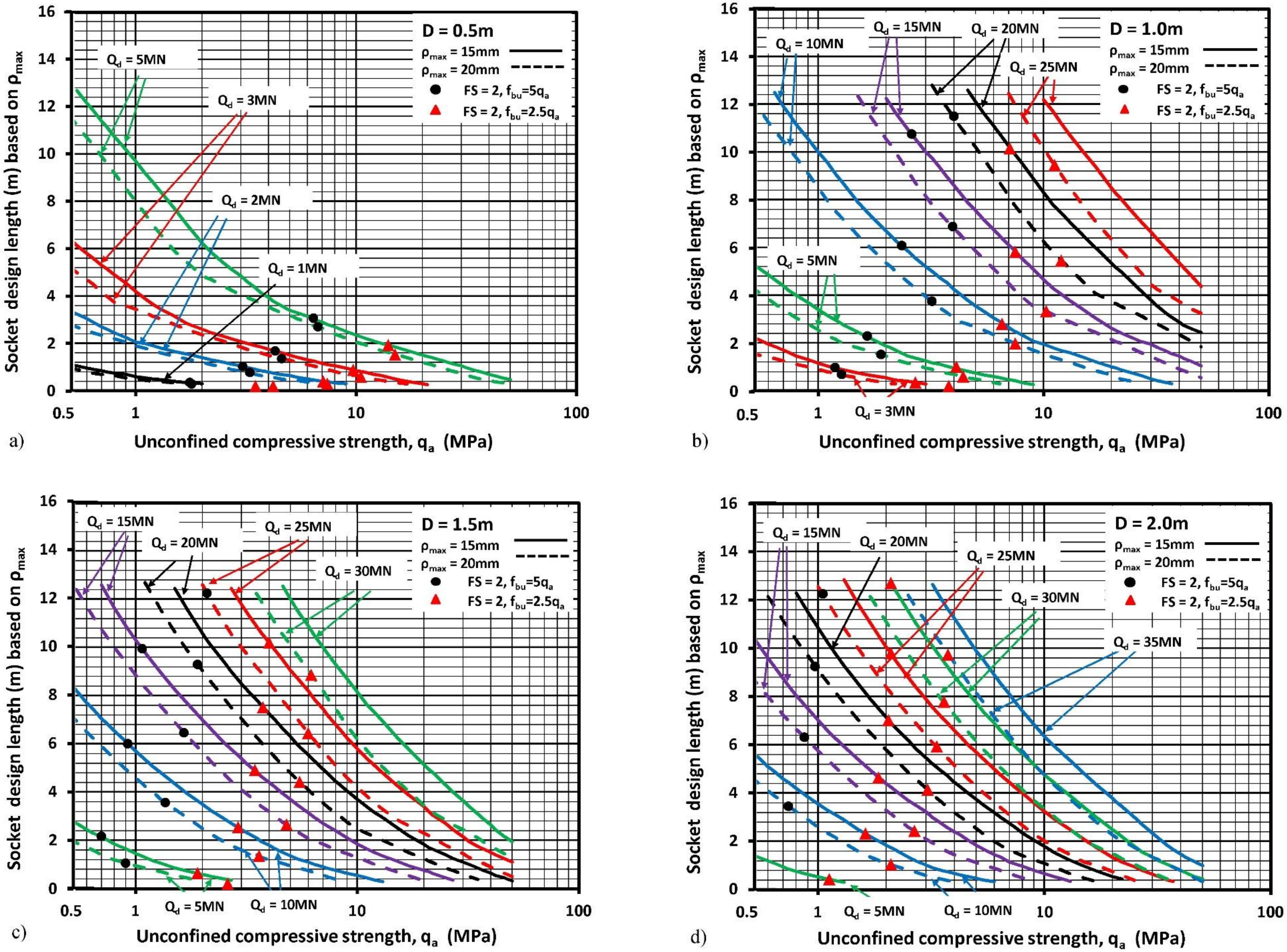

Figure 8. Design charts for ρ_{max}=15mm and 20mm for socket diameters of (a) D=0.5m, (b) D=1.0m, (c) D=1.5m and (d) D=2m.

Design points to the lower right correspond to designs where settlement controls (for the strength criteria given above) and the length of the socket is as given. Where a symbol does not appear on a design curve, settlement is the controlling criterion.

For example, consider three socketed piles, each with D=1m, Q_d=10MN and ρ_{max}=15mm to be drilled into rock with q_a=3.0MPa, 2.35MPa, and 1.7MPa. Table 2 summarises the three designs based on the design charts. Fig. 9 shows how the relevant design chart (Fig. 8b) is used. This figure has been simplified by removing the design curves for loads other than for the Q_d=10MN curves of the example.

For the pile in rock of q_a=3.0MPa, the vertical intercept with the design line is to the lower right of the FS=2 point and gives a socket length of 5.20m with (using Eq. 9) a factor of safety of 2.18, indicating that the design is controlled by settlement. For the pile to be placed in rock of q_a=2.35MPa, the vertical intercept with the design line is exactly on the FS=2 point and gives a socket length of 6.05m with a factor of safety of 2.00. For the pile in rock of q_a=1.70MPa, the vertical intercept with the design line is to the upper left of the FS=2 point and gives a socket length of 7.35m with a factor of safety of 1.85 indicating that the design is now controlled by the strength criterion. Based on Eq. 9, the socket length for this pile would need to be increased to 8.32m so that the factor of safety is increased to 2.00. Based on the second type of spreadsheet discussed above, the extra socket length will reduce the maximum settlement experienced to about 12mm.

Table 2. Data for design examples

q_a (MPa)	L (m)	α	f_{su} (MPa)	f_{bu} (MPa)	Factor of safety
3.00	5.20	0.21	0.61	15.00	2.18
2.35	6.05	0.24	0.57	11.75	2.00
1.70	7.35	0.30	0.51	8.50	1.85

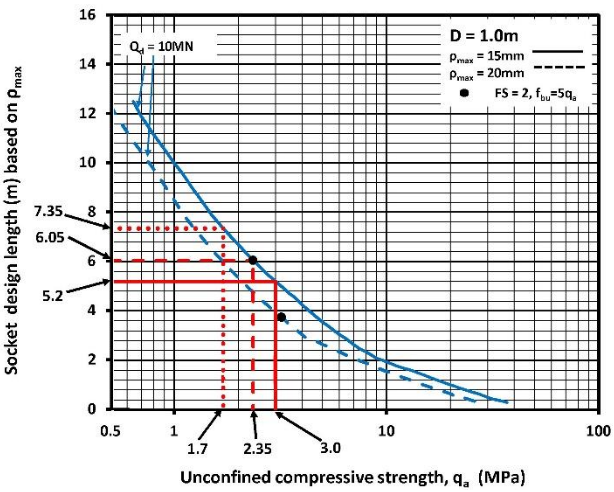

Figure 9 Examples of use of design chart

The above discussion has considered a factor of safety of 2 using the basic relationship of Eq. 9 with $f_{bu}=5q_a$. However, as noted, most modern design procedures now require the use of limit state design procedures which involve the application of load and resistance factors as simply described by Eq. 10. While these procedures may be a little more complex and often quite varied, their application for checking strength limit states can still be achieved relatively simply. For many codes, the points described above for $f_{bu}=5q_a$ and a factor of safety of 2 will not be in vastly different locations from similar balance points defined by the load and resistance factors of these codes.

However, it is emphasised that the factor of safety of 2 is based on the ultimate base resistance of a pile in a reasonably competent rock being given by $f_{bu}=5q_a$. This was the relationship recommended by Williams et al. (1980). There are several codes, standards and design specifications which prescribe a much more conservative approach. One such document is AASHTO (2020) where f_{bu}=2.5q_a. It follows that this leads to a significantly lower assessment of the ultimate resistance of a pile. The overall effect of this is to move the FS=2 points in a downwards right direction to increase the range of pile designs for which strength is the controlling factor. This is particularly true for smaller diameter piles and for larger values of ρ_{max}. The points corresponding to $f_{bu}=2.5q_a$ are represented by triangular symbols and defined in the legend as "FS=2, f_{bu}=2.5q_a".

5 CONCLUSIONS

Spreadsheets derived from the digitized design curves of the original WJD method have allowed the development of a set of design charts providing a convenient means of rapidly assessing dimensions of rock socketed piles.

The charts show that for sockets with diameters greater than about 1m, with maximum allowable settlements of 10mm or less and with the ultimate base resistance defined as 5 times the unconfined compressive strength of the rock, designs are usually controlled by settlement. For smaller diameters, larger maximum allowable settlements and lower estimates of the ultimate base resistance, strength criteria can become critical.

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