Design of Grouted Offshore Piles in Calcareous Soils

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SUMMARY A method is presented for analysing and designing grouted piles in weakly cemented calcareous sediments. The method is based upon correlation with observed field tests and utilises a brittle load transfer curve to calculate pile capacity.

1. INTRODUCTION

It is well known that conventional driven steel pipe piles in calcareous soils may show very low unit skin friction values and as a result drilled and grouted insert piles have generally been used for support of offshore platforms in such soils.

This paper reviews the current state of practice in design of grouted piles in calcareous soils, and proposes a new alternative to the existing methods using cone penetration test cone resistance values to determine peak skin friction for design. The method proposed also uses load transfer analysis and brittle load transfer curves to calculate axial pile capacity.

2. HISTORICAL BACKGROUND

The design for drilled and grouted insert piles in calcareous soils for many years was based upon the work by Angemeer *et al* (1), (2), where field tests were conducted in Bass Strait and on the North West Shelf of Western Australia. These tests showed that grouted piles offered superior load carrying capacity to driven piles in calcareous soils, and the results of the field tests allowed for subsequent installation of grouted piles in many locations.

In his state of the art review Murff(3) concluded that analysis of drilled and grouted piles in calcareous soils was rather "simplistic". A uniform value of unit skin friction of 100kPa was often adopted, loosely based upon the rather crude original test work by Angemeer. This simplistic approach existed until the publication and adoption of the two methods as discussed below.

3. CURRENT PRACTICE

Both methods in current use load transfer analysis to calculate pile capacity in recognition of the brittle load deflection curves typical of grouted piles in these soils. Therefore it is necessary to review not only the method of assessment of pile skin friction but also the form of the load transfer relationship.

3.1 Peak Skin Friction

In the mid to late 1980's, a significant amount of research was performed on this subject, most notably by Esso Australia and Woodside Offshore Petroleum. These programmes of research was driven by the need to resolve real engineering issues in connection with existing offshore structures, and included field tests on drilled and grouted pile sections, (4), (5), followed by analytical work (6), (7), (8). These latter two papers concentrated on back analysis of the grouted pile section tests at the North Rankin A platform and developed innovative techniques for analysis of cyclic loading. However they offer little guidance as how to develop a pile design away from the site specific data of the North Rankin A platform on the North West Shelf of Western Australia.

It is worth noting, however that peak skin friction values of about 300kPa to 680kPa were back analysed from the tests at North Rankin A, significantly higher than the previously used value of 100kPa.

The work reported by Hyden *et al.* (7) did however propose a new design technique. The approach used was to model the soil as frictional in nature, with the peak skin friction given by:

$$\tau_p = F \, \sigma_{rg} \tan \phi_p$$

where:

σ_{rg} = effective grout pressure

ϕ_p = peak friction angle from triaxial tests

F = design correlation factors

The method uses this basic approach, including various empirical factors to account for correlation with the field data and design and construction uncertainties.

Inspection of the method shows that it is conservative with respect to the observed field tests. Typically the method yields design peak skin friction values of up to about 400kPa compared to the field data which gave values in the range 350 to 950kPa, again significantly higher than the previously often adopted 100kPa.

The Hyden *et al.* method is attractively simple and empirical, but it does not recognise the cementation inevitably present in calcareous soils. However it provides a robust rational method to design such piles and has been used in Bass Strait. To the authors knowledge it has also been used on at least one project on the NW Shelf where it was employed as a design check.

In well cemented soils, such as may be found in shallower waters in the NW Shelf of Western Australia, a different approach is often used based upon Abbs & Needham (9).

This method was developed in the early 1980's by the authors during a series of projects in the Arabian Gulf. This approach treats the cemented carbonate soil as a weak rock and adopts a "rock socket" approach to the development of peak skin friction. For this method to be used, the soil must be sufficiently cemented to be successfully cored, normally using wireline triple tube techniques, and as such is really applicable to cemented soils or weak rocks showing an unconfined compressive strength of greater than about 500kPa - 750kPa.

3.2 Load Transfer Curves

Both the methods currently in use employ load transfer analysis to calculate the ultimate axial pile capacity. This has been adopted because of the observation of brittle, strain softening behaviour in pile tests as shown in Figure 1. Different load transfers curves (t-z) have been developed by the two design methods outlined above, but they are broadly similar in shape to each other and the typical field test curve on Figure 1.

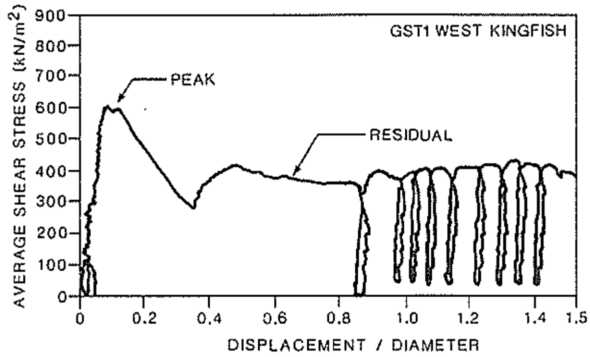


Figure 1 Typical load transfer curve from grouted section test (ref 6)

4. PROPOSED METHOD

4.1 Need for Alternative Method

As discussed above, the two methods in use are different and suffer from some limitations, Abbs & Needham is applicable for well cemented, weak rock materials, whereas Hyden *et al.* assumes no cementation, relying instead upon good quality triaxial tests to measure effective friction angles. Accordingly, there is a need for a method that may be used in the case of weakly cemented soils that are too weak to core successfully, but still with significant cementation. It is also desirable for the method to employ conventional field or laboratory techniques as input. The method selected developed during the foundation design of major offshore structure, opted for cone penetration test data as the basic input for correlation with peak skin friction values.

In cohesive soils it has been common practice to estimate the undrained shear strength of clays and pile capacities using cone resistance data. Therefore it seemed logical that cone resistance values, which provide a repeatable index of a soil's in-situ strength and compressibility, should provide a basis for a method in cemented calcareous soils.

4.2 Correlation of Cone Resistance with Peak Skin Friction

To develop a correlation between cone resistance and unit pile skin friction, it was necessary to assemble a data base of grouted pile tests and cone resistance data in calcareous soils. Figure 2 shows the data assembled during this exercise. The data contains results for piles of a variety of sizes, some of which have failed, some not failed and some failed by cycling.

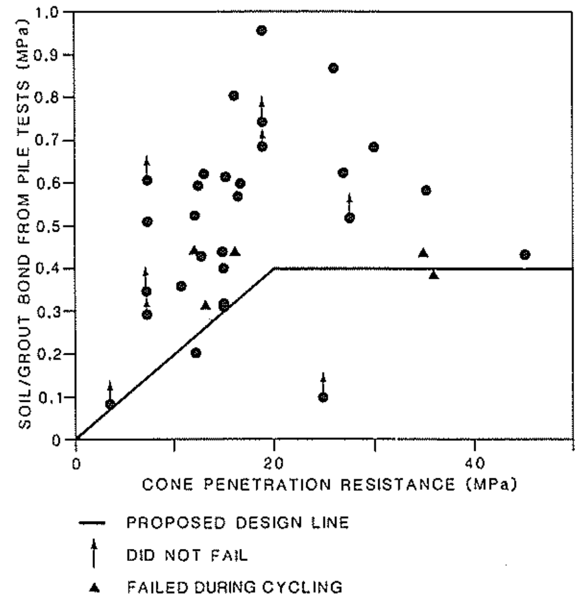


Figure 2 Calcareous Soil Grouted Pile test result data base

A design line was picked as a conservative lower bound to the available test data. This selection of the lower bound accounts for many unknown factors including the effects of pile diameter. It has been found that unit skin friction decreases with increasing pile size Khorshid (10), and reasoning for this has been proposed by Randolph (11).

The proposed correlation is given by:

$$\tau_p = 0.02q_c$$

where: q_c = cone resistance in MPa

τ_p = peak skin friction in MPa

This ratio may be compared to values quoted in the State of the Art Address to the Carbonate Sediments Conference (11) where it was noted that grouted pile peak skin friction ranged from 0.02 to 0.15 q_c and 0.03 to 0.05 q_c for Bass Strait data.

It was considered that the results reported by Khorshid (10) should be giving the greatest weighting in the selection of a design correlation as these represent a consistent set of high quality test data from one site. Figure 3 presents this sub set of data from one test site, together with the cone resistance profile converted to peak skin friction by the proposed correlation.

The agreement between the proposed correlation and the test data for the full size pile sections is considered encouraging. The figure also illustrates the scatter of laboratory test data from the same site.

The data set indicates that the linear relationship should not continue, and a cut off at τ = 400kPa has been selected. This also has the prudent affect of preventing the pile design relying on a few thin particularly strong layers. In addition, for τ values of greater than about 400kPa the pile design may become governed by allowable stresses between the steel insert pile and the grout.

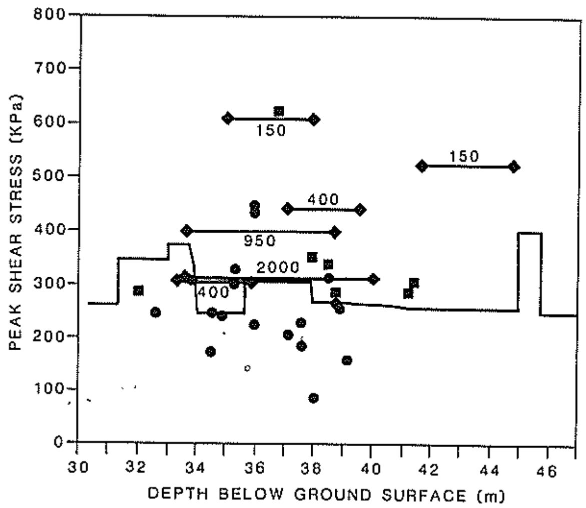


Figure 3 Field and laboratory test data from a calcareous soil test site

For continuous profiles of stronger materials, above say q_c = 40MPa, which corresponds to an unconfined compressive strength of 1 to 2MPa, it would be more appropriate to use the Abbs & Needham method for weak rocks, as the cone penetration becomes difficult in these stronger materials.

4.3 Comparison of Design Line with Laboratory Direct Shear Data

As a check, the design line has been compared to laboratory test data. In view of the questions concerning the effects of pile diameter on unit skin friction, it was decided not to use laboratory scale model grouted piles, although these tests do provide valuable insight into the mechanisms (12). Instead, the test selected was the large scale constant normal stiffness direct shear test (CNS tests) (13). This test, which allows the use of large samples tested longitudinally, is considered to be more realistic representation of rough pile/soil interface behaviour during monotonic loading in that it permits dilation or contraction during the shearing process depending upon the radial stresses and stiffness at the soil/grout interface.

Figure 4 shows the data from CNS shear tests which have been performed on samples where CPT data were available from adjacent boreholes. This scatter is due in part to this spatial difference in the data source, but also is due to a variety of initial normal stresses and normal stiffness in the test. These boundary conditions may result in differing peak skin friction values being recorded (13). In spite of the scatter it is reassuring to see that the design line lies conservatively with respect to the data points.

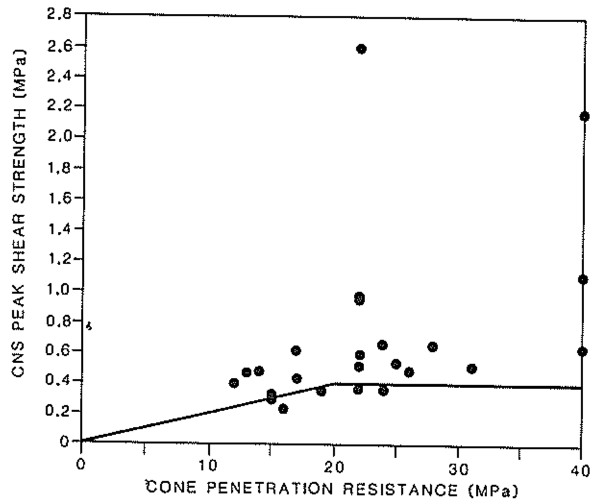


Figure 4 Constant normal stiffness direct shear test data

4.4 Shape of Load Transfer Curve

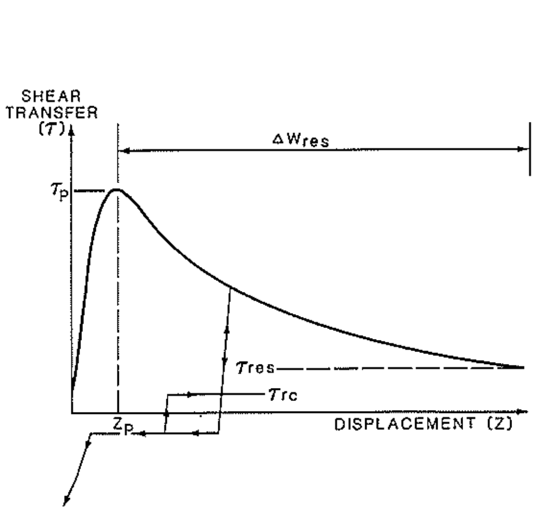
In conjunction with the peak skin friction relationship, a load transfer curve (t-z curve) is needed to permit calculation of the pile axial capacity and response. The type of curve proposed is that developed by Randolph (7) and Randolph and Jewell (14). The form of the t-z curve is shown on Figure 5. Various parameters need to be selected to define the curve, including:

τ_p	-	peak skin friction
G	-	Soil shear modulus
ξ	-	yield parameter (proportion of linear portion of initial slope of t-z curve)
Δw_{res}	-	displacement to residual skin friction
τ_{res}	-	monotonic residual skin friction
τ_{rc}	-	cyclic residual

Selection of τ_r and Δw_{res} is difficult without site specific tests, but from a review of the data available, τ_{res} appears to range from 10% to 60% of τ_p and Δw_{res} from 5mm (small model piles) to 1000mm (large diameter pile sections).

Generally speaking conservative values for τ_{res} and ΔW_{res} should be selected (say 20% and 50mm-100mm would be typical).

A full discussion of the curve and the source of the somewhat novel cyclic residual is given in reference 14.



1. $\tau_p = 0.02q_c \ngtr 400$kPa
2. $\tau_{res} = 0.2\tau_p$ (TYPICALLY)
3. $\tau_{rc} = 0.01\tau_p$ (TYPICALLY)
4. $Z_p \propto \frac{\tau_p D}{G}$, D= PILE DIAMETER
   G= SHEAR MODULUS
5. YIELD PARAMETER = 0

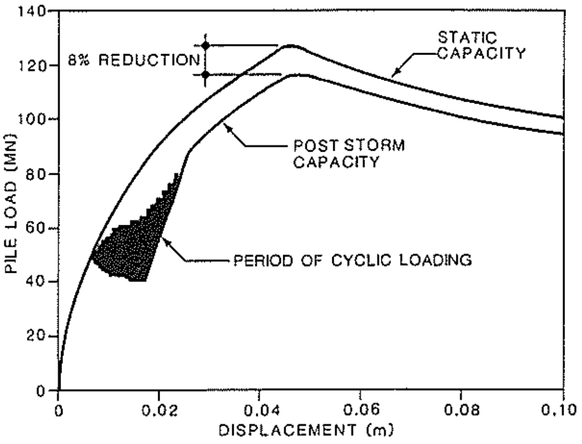
Figure 5 Proposed load transfer curve

4.5 Cyclic Loading

The discussion thus far has been centred around static axial capacity. In calcareous soils, cyclic loading is important, as failure of a pile element may result from repeated cyclic loading at shear stresses below the local peak skin friction. With a strain softening t-z curve this may lead to the pile "unzipping". Cyclic loading may be readily analysed using the Randolph type t-z curve described above and the program RATZ (15). The results of many studies have shown that for properly designed piles with normal safety factors (16), the effects of cyclic loading may often be accommodated within the static safety factor. This would not necessarily be true in every case, however. (In the two methods described above in Section 3 'Current Practice', cyclic loading is implicitly included in this way).

4.6 Example

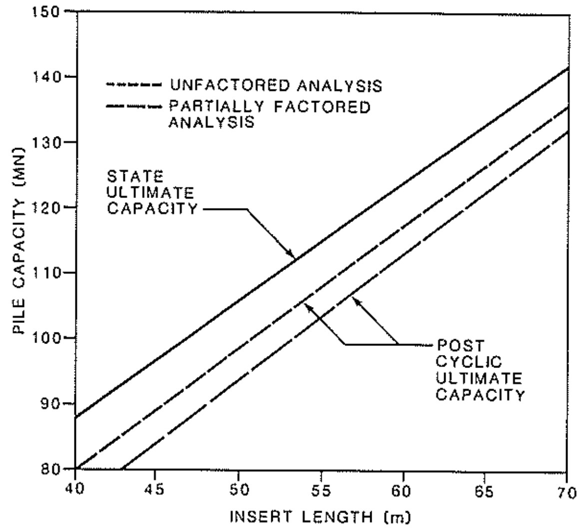
Figure 6 shows the results of an actual load transfer analysis for a typical offshore pile. Both monotonic and cyclic analyses are shown on this figure. The cyclic analysis shows the reduction in capacity after a period of cyclic loading.



1. 62m LONG x 2mO.D. x 0.05m w.t. TUBULAR IN 2.3m DIA. HOLE

Figure 6 Result of typical load transfer analysis

Figure 7 shows the results of a series of analyses of a 2m diameter insert pile plotted as peak pile capacity versus insert length. The effects of cyclic loading are clearly shown albeit fairly modest in this case. The analyses were run for the static case as well as factored and unfactored cyclic cases.



1. 2m O.D. x 0.05m w.t. TUBULAR IN 2.3m DIA. HOLE

Figure 7 Typical pile capacity length relationship

5. CONCLUSION

This paper has presented a new method for analysing and designing grouted insert piles in calcareous soils. The method uses an empirical approach to calculate peak skin friction based upon cone resistance data from cone penetration tests and then uses a load transfer analysis with strain softening t-z wires to calculate pile capacity. While the method is based upon empirical correlation with field data, it should be used with caution in soil types which differ from those on the NW Shelf of Australia. The method compliments the existing approaches described by Abbs & Needham (9) and Hyden *et al* (6), and therefore allows for the use of more than one method if the field and laboratory data are sufficient.

6. REFERENCES

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