# Case Studies of Cost-effective Foundation Design in Rock

## Études de cas sur la conception de la Fondation rentable dans Rock

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ABSTRACT: In the Sydney region of Australia, the design of rock socketed piles in medium to strong rock is generally governed by settlement criteria, and designs are typically carried out using presumptive "serviceability" values quoted in the literature. The benefits of using a load-settlement performance rock socketed pile design method, rather than using "presumptive" values, are presented via two case studies in the Sydney region. On one site underlain by medium to high strength shale, dynamic pile load testing was carried out, and on another site underlain by high strength sandstone, Osterberg Cell (O-Cell) testing was carried out to validate the designs. The case studies presented clearly demonstrated that better understanding of load-deformation characteristics of pile foundations will lead to more cost-effective designs. Project owners have a tendency to resist spending money on testing, but the savings that can be achieved by adopting shorter rock socket lengths on medium to large projects far exceed the cost of pile load testing. The adoption of shorter rock socket lengths is generally the objective of project owners and contractors, but at the same time, has the benefit of preserving natural resources and reduction in CO2 emissions.

RÉSUMÉ : Dans la région de Sydney, de l'Australie, la conception des haldes encastrés dans un milieu à forte rock est généralement régie par critères de règlement, et conceptions sont généralement réalisées en utilisant les valeurs présumées « fonctionnalité » cités dans la littérature. Les avantages de l'utilisation d'un rocher de rendement de charge-tassement encastrés conception de pieux méthode, plutôt que d'utiliser les valeurs « présumées », est présentée par l'intermédiaire de deux études de cas dans la région de Sydney. Sur un même site sur des moyennes et schiste de haute résistance, essai de chargement de pieux dynamique a été réalisée, et sur un autre site sur des grès de haute résistance, Osterberg cellules (O-) essais a été réalisée pour valider les modèles. Les études de cas présentées a clairement démontré que meilleure compréhension des caractéristiques de contrainte-déformation des fondations sur pieux aboutira à des conceptions plus rentables. Maîtrise d'ouvrage ont tendance à résister à dépenser de l'argent sur les essais, mais les économies qui peuvent être obtenus en adoptant plus rock socket courtes sur moyens et grands projets bien dépassent le coût des tests de charge de pile. L'adoption de plus courtes longueurs de douille de roche est généralement l'objectif de la maîtrise d'ouvrage et les entrepreneurs, mais en même temps, a l'avantage de préserver les ressources naturelles et la réduction des émissions de CO2.

KEYWORDS: Rock sockets, bored piles, dynamic load test, Osterberg cell test, Sandstone, Shale, Serviceability

## 1 INTRODUCTION

The design of pile foundations in Sydney Sandstone and Shale in Australia has been carried out largely on the basis of "recipe book" approach, using the well-known references of Pells et al (1978) in Working Stress format, and Pells et al (1998) in Limit State format. Virtually in all cases, piles founded in rock are governed by serviceability limit (i.e. settlement criteria) rather than strength limit. Yet, there is little information on the deformation characteristics of piles founded in Sydney Sandstone and Shale. The use of presumptive design values quoted in references often leads to conservative designs.

Table 1. Design Values for Shale based on Pells et al (1998)

| Shale Class(1) | Typical UCS (MPa) | Serv. Base, $f_{ba}$ (MPa) | Ult. Base, $f_{bu}$ (MPa)(2) | Ult. Shaft, $f_{sa}$ (MPa) | Typical Field Modulus, E (GPa) |
|---|---|---|---|---|---|
| I | > 16 | 8 | > 120 | 1 | > 2 |
| II | > 7 | 6 | 30 - 120 | 0.6 - 1 | 0.7 - 2 |
| III | > 2 | 3.5 | 6 - 30 | 0.35 - 0.6 | 0.2 - 1.2 |
| IV | > 1 | 1 | > 3 | 0.15 | 0.1 - 0.5 |
| V | > 1 | 0.7 | > 3 | 0.05 - 0.1 | 0.05- 0.3 |

Table 2. Design Values for Sandstone based on Pells et al (1998)

| Sandstone Class(1) | Typical UCS (MPa) | Serv. Base, $f_{ba}$ (MPa) | Ult. Base, $f_{bu}$ (MPa)(2) | Ult. Shaft, $f_{sa}$ (MPa) | Typical Field Modulus, E (GPa) |
|---|---|---|---|---|---|
| I | > 24 | 12 | > 120 | 3 | > 2 |
| II | > 12 | 12 | 60 - 120 | 1.5 - 3 | 0.9 - 2 |
| III | > 7 | 6 | 20 - 40 | 0.8 - 1.5 | 0.35 - 1.2 |
| IV | > 2 | 3.5 | 4 - 15 | 0.25 - 0.8 | 0 .1 - 0.7 |
| V | > 1 | 1 | > 3 | 0.15 | 0.05 - 0.1(3) |

(1) Rock Classification also depends on defect spacing and amount of seams
(2) Not more than 0.5 x UCS for Classes I, II and III Rock
(3) Not sure why this is less than modulus values for shale

Based on Pells et al (1998), the typical values adopted for design of rock socketed piles in Sydney Shale and Sandstone are tabulated in Tables 1 and 2 respectively. Pells et al (1998) suggested that the "serviceability" end bearing values given are for settlement < 1% of the minimum footing dimension, and that "ultimate" values occur at settlement > 5% of the minimum footing dimension. No "serviceability" values are given for shaft friction, because under serviceability loads, the pile shaft may take a majority of the load and the mobilized shaft resistance, particularly towards the top of the pile shaft, may reach close to the "ultimate" values.

It can be seen from Tables 1 and 2 that in conventional working stress terms, the ratio of ultimate end bearing value to the serviceability value would give rise to equivalent factors of safety of about 3 for the poorer quality rock, to 10 or more for Class I Shale and Sandstone. While it may be "safe" to adopt the presumptive "serviceability" values based on the notion that settlement will be less than 1% of the minimum footing size, there is no assessment on "how much less than 1%". Also, 1% of a relatively small diameter pile (say 0.6m dia.) would be very different to 1% of a 2m square footing (i.e. < 6mm compared to < 20mm settlement).

The difference between conducting a design based simply on presumptive "serviceability" values and a more detailed assessment of load-deformation response of a 1.8m diameter pile socketed 6m into rock (1m in Class V Sandstone, 2m in Class IV Sandstone, and 3m in Class III Sandstone) is illustrated in Figure 1 below. In both cases, the ultimate load capacity of the pile was assessed using the same values ($f_{su}$ of 0.1MPa, 0.5MPa and 0.8MPa in Class V, IV and III Sandstone respectively, and $f_{bu}$ of 20MPa for the Class III Sandstone). Using these parameters, the ultimate load for this pile was assessed to be 70MN. However, the load-deformation curves were in one case assessed using the method described by Poulos (1979), while in the other case as an extrapolation of a linear line between zero and the assessed ultimate load, with the line intersecting an assumed settlement of 1% at the pile load computed using the presumptive "serviceability" design values given by Pells et al (1998).

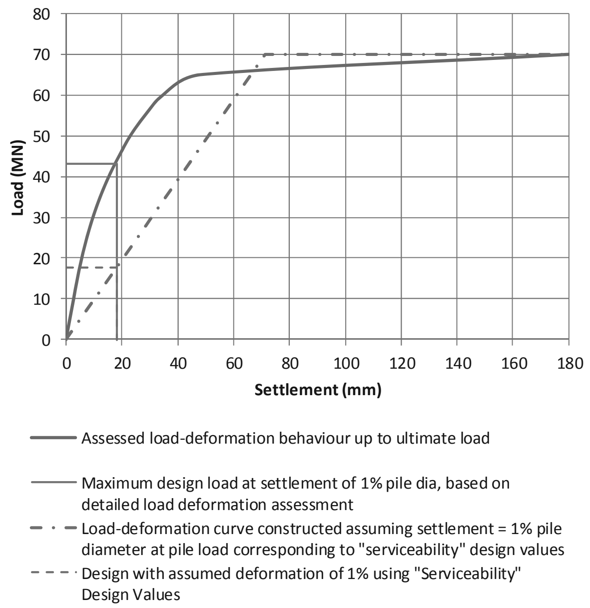


Figure 1. Load-deformation curves for Illustrative Example

Figure 1 shows that for the case corresponding to a presumptive settlement of 1%, the computed "serviceability" capacity would be limited to 18MN which corresponds to a relatively high factor of safety of 3.9. However, based on the more detailed load-deformation assessment, the maximum load to cause the same settlement of 18mm could be as high as 43MN. It should be pointed out that Pells et al (1998) acknowledges that the "elastic" design method is conservative, and supports that design be based on non-linear sidewall slip methods. It is therefore not surprising that the use of more sophisticated, non-linear load-deformation assessment methods would result in more economic foundation designs than adopting presumptive "serviceability" design values.

However, the accurate prediction of pile settlement relies heavily on knowledge of the foundation material stiffness in addition to adopting appropriate evaluation methods. Therefore, the author is of the opinion that using a performance based design, with pile load testing to validate the load-deformation response assessed, is more likely to achieve cost-effective designs, and increase confidence of meeting design objectives. This performance based design approach is illustrated in two case studies described below.

## 2 CASE STUDY 1

The first case study involves the testing of a 600mm diameter continuous flight augered pile socketed into weathered Ashfield Shale in Campbelltown, an outer south-western suburb of Sydney. The testing was carried out using dynamic technique with wave matching using the CAPWAP method.

The subsurface stratigraphy at this site comprised 7.3m of stiff to very stiff compacted clay fill and residual soil, underlain by a thin veneer (0.3m) of very low to low strength, highly to moderately weathered shale (Class IV Shale), followed by medium to high strength shale with Point Load Strength Index typically between 0.5MPa and 1.5MPa. Based on a typical correlation factor of 20 for Sydney Shale and Sandstone (although the range may be between 10 and 30), the approximate unconfined compressive strength of the medium to high strength shale is 10MPa to 30MPa, and the rock was classified as Class II Shale based on Pells et al (1998). The test pile was socketed 0.3m through the very low to low strength shale and penetrated only 0.1m into the medium to high strength shale so that its end bearing pressure can be readily assessed.

The dynamic load testing was carried out using an 11 tonne drop hammer and a purpose built testing frame as shown in Figure 2.

Figure 2. Dynamic pile load test set up in Case Study 1

The CAPWAP analysis results provided an estimated mobilized total capacity of 12.39MN, with a mobilized shaft resistance of 1.25MN and a mobilized pile toe resistance of 11.14MN. The mobilized end bearing resistance therefore corresponded to 39.4MPa. The mobilized pile toe settlement during the test blow was less than 6mm and the inferred static load-displacement response was relatively stiff with no suggestion that the ultimate end bearing resistance was reached.

Based on the test results, a "serviceability" design capacity of 3.4MN (i.e. 12 MPa end bearing pressure) was adopted. If the piles had been designed using a presumptive "serviceability" end bearing pressure of 6MPa, the design serviceability load would have been limited to 1.7MN (i.e. 50% less). The benefit of the dynamic load test in providing design confidence and economic design was clearly demonstrated in this example.

## 3 CASE STUDY 2

This is a recent case study associated with the Barangaroo South Stage 1A Development located on the western fringe of the Sydney CBD, on the eastern shoreline of Darling Harbour.

The site is situated over reclaimed land in an eroded and infilled paleovalley, with rock ranging in depth from 0m to 30m as shown in Figure 3. As such, most of the weathered rock has been removed during the erosional process, and replaced with overlying Holocene alluvial sand, silt, clay, and manmade fill.

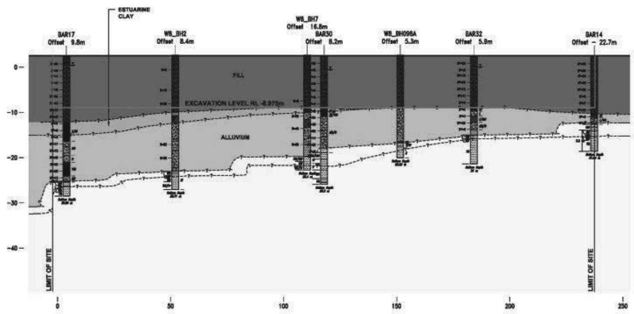


Figure 3. Stratigraphic profile at the site of Case Study 2

The rocks at the site comprise Hawkesbury Sandstone, with average Point Load Strength Index ranging from 0.05MPa to 1.5MPa between 1m to 3m below rock surface, thereafter having an average Point Load Strength of about 1.5MPa as shown in Figure 4. These Point Load Strength index tests suggested the unconfined compressive strength of the rock to be on average about 30MPa below 3m depth, but layers having strengths as high as 60MPa to 80MPa are likely to exist.

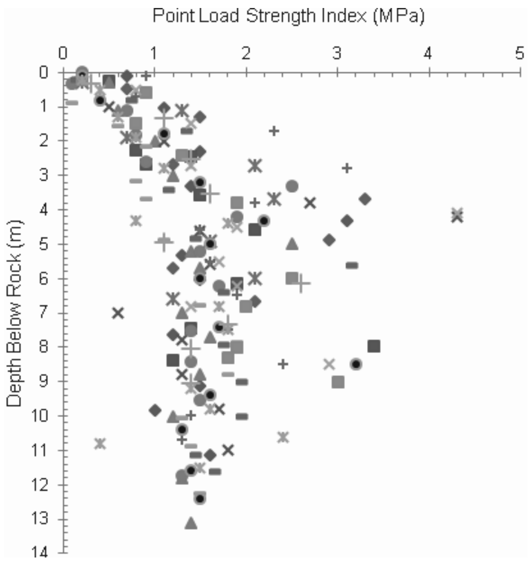


Figure 4. Point Load Strength Test Results (Case Study 2)

The sandstone rock at the site was classified in accordance with Pells et al (1998), and preliminary design parameters for pile design were assigned as summarized in Table 3. The project required nearly 1000 piles with diameters ranging from 1m to 2.4m to support 3 towers of up to 34 storeys in height and a number of low rise buildings over a 4ha site. The design serviceability pile loads ranged from 7MN to 14MN for 1m diameter piles, to 72MN to 81MN for the 2.4m diameter piles.

The structural engineer for the project was particularly concerned about long-term differential settlement effects on the tower structure, and therefore specified tight pile settlement criteria. A pile toe settlement limit of 0.3% of the pile diameter was stipulated at the pile toe. This was an unusual specification but it was adopted by the structural engineer so that equivalent structural "springs" could be adopted for the piles in his structural model of the superstructure, to include modeling of the piles due to the varying pile lengths on the project.

Table 3. Design Values Adopted for Design (Case Study 2)

| Sandstone Rock Class | Elastic Modulus* (GPa) | Poisson's Ratio | Ultimate End Bearing Pressure (MPa) | Ultimate Shaft Friction (MPa) |
|---|---|---|---|---|
| IV | 0.5 | 0.3 | 10 | 0.5 |
| III | 1.0 | 0.3 | 20 | 0.8 |
| II or better | 2.0 | 0.2 | 80 | 2 |

* These represented the initial tangent modulus values.

A non-linear analysis using the following equation was adopted to describe the secant modulus:

$E_{sec} = E_t [1 - R_f(p/p_f)]$

where:

$E_t$ = initial tangent elastic modulus (values given in the table)

$R_f$ = hyperbolic curve-fitting constant ($R_{fs}$ = 0.25 adopted for shaft and $R_{fb}$ = 0.7 adopted for base)

p = mobilized pile-soil stress

$p_f$ = limiting value of pile-soil stress (values of fb and fs given in table)

Because of concerns that dynamic testing would not be able to provide sufficient test load and would not capture the potential creep effect of the rock at high loads, pile load testing was conducted on two 750mm diameter prototype piles fitted with Osterberg Cells (O-Cell). Load-settlement prediction, pile load testing, and back-analyses of the pile load testing results for the project has been described in Wong and Oliveira (2012). In brief, the two test piles had rock socket lengths of 7.85m (SC-01) and 6.38m (SC-02); both founded with the pile toe socketed more than 3.5m in Class II Sandstone. The O-Cell was located at the toe of the test piles and the maximum O-Cell load reached was 22.6MN for SC-01 and 26MN for SC-02.

The test results for SC-02, together with results of back-analyses using the embedded pile element method in the commercial finite element analysis program FLAC (3D), are presented in Figure 5.

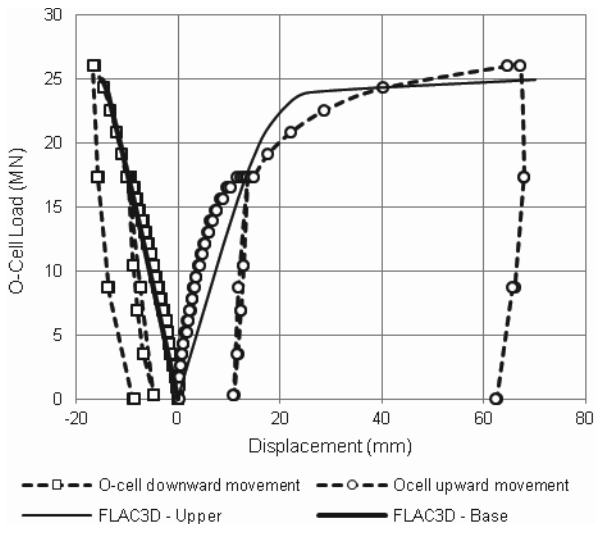


Figure 5. O-Cell Test and Back-analysis Results (Test Pile SC-02)

The key findings of the O-Cell testing are summarized below:

**End Bearing Resistance**

- Maximum mobilised resistance = 59MPa (ultimate end bearing pressure not reached)
- Little or no creep at an end bearing pressure of 38MPa (this pressure was held for 30 minutes)

**Shaft Resistance**

- Creep started at an average shaft resistance (over 5.5m length) of 1.06MPa and was significant at 1.3MPa
- The shaft response became "plastic" at a movement of about 30mm, with a corresponding average shaft resistance of 1.74MPa

The O-Cell test results confirmed the ultimate design values adopted for design, and as in Case Study 1, demonstrated that significantly higher serviceability end bearing pressure could be considered in the design of rock socketed piles in Sydney rock. If the presumptive end bearing pressure given in Table 2 for Class I and II Sandstone was adopted, the serviceability end bearing resistance would have been limited to 12MPa. The O-Cell test clearly demonstrated that significantly higher serviceability end bearing could be adopted, provided the base of the rock socket is adequately cleaned. The pile construction aspect of this case study to ensure adequate rock socket roughness and base cleanliness is described in Sethi et al (2012). However, it should also be stressed that under serviceability loading, a large proportion of the applied load may be carried by the pile shaft depending on the length to diameter ratio of the rock socket. Therefore, the use of excessively high serviceability end bearing pressure may not be warranted. A detailed assessment of the rock-socket load-deformation response is necessary for each specific case.

In the above case study, the non-linear load-deformation behavior observed from the O-Cell test is of particular interest. Using the back-analyzed test results, and by close inspection of the load-deformation behavior of both the shaft and base, it was possible to deduce the operating secant modulus of the rock socket material at various mobilized base and shaft resistance as shown in Figures 6 and 7.

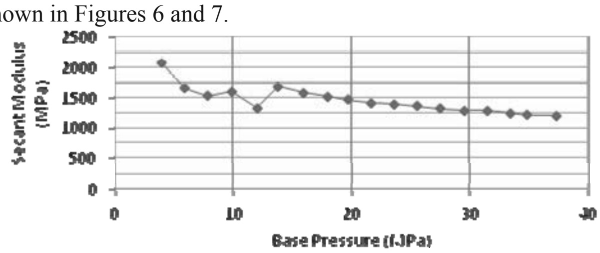


Figure 6. Deduced Secant Modulus of Rock below Pile Base

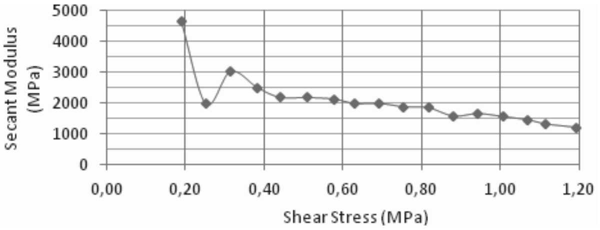


Figure 7. Deduced Secant Modulus of Rock around Pile Shaft

It can be seen from Figure 6 that there was a rapid drop in the inferred secant modulus of the rock below the pile base when a base pressure of 5MPa was reached, and remained at approximately 1.6GPa to 1.7GPa until a base pressure of 14MPa was reached. Above this pressure, the inferred secant modulus continued to drop steadily and reached a value of 1.3GPa at a base pressure of 30MPa. The initial drop in secant modulus at a base pressure of 5MPa to 14MPa could be attributed to compression of disturbed material or residual debris at the base of the socket, and the gradual drop of secant modulus beyond a base pressure of 14MPa is considered to be representative of the actual rock mass behavior.

From Figure 7, it can be seen that the inferred secant modulus of the rock socket material was initially very high (over 5GPa), then dropped rapidly to 2.5GPa at an average shaft resistance of 0.4MPa, then continued to drop steadily to 1.2GPa at a mobilized shaft resistance of 1.2MPa. Comparing these results with the non-linear function to describe the secant modulus adopted for design as shown in Table 3, it may be concluded that different initial rock modulus should be applied to describe the base and shaft response. However, for simplicity of design, and considering the operating stresses at the serviceability loads for the piles on this project, it was concluded that an initial tangent modulus value of 2GPa would still be appropriate for the Class II Sandstone if the hyperbolic pile base and shaft factors, $R_{fb}$ and $R_{fs}$ (see Table 3), were modified to 0.55 and 0.8 respectively. These values correspond to secant modulus values of approximately:

- Pile Base Response – 1.7GPa and 1.2GPa for the rock below the pile base, for end bearing pressures of 14MPa and 30MPa respectively, and
- Pile Shaft Response – 1.8GPa and 1.2GPa for the rock around the pile shaft for shaft resistance of 0.4MPa and 1.2MPa respectively.

However, these changes would only make very small changes ($\leq$ 3mm) to settlement prediction values at serviceability loading. Therefore, the original design parameters were adopted without changes for subsequent designs.

Supported by the O-Cell pile load testing, significant reduction in pile lengths and cost savings were achieved for this project as a result of the load-deformation analyses and performance based design carried out.

## 4 CONCLUSIONS

Other than very weak to weak rock, socketed pile design is generally governed by serviceability requirements rather than ultimate capacity. In such circumstances, economy pile designs can be achieved if accurate predictions of load-deformation behavior of the piles are made, rather than adopting recipe style presumptive values. Pile load testing should be carried out for such performance based design method.

Two case studies of rock socketed pile design and pile load testing have been presented in this paper, both of which clearly illustrated the advantages of this performance based design approach, with significant cost savings in foundation works. The use of the O-Cell testing method in Case Study 2 demonstrated the non-linear nature of high strength rock commonly encountered in the Sydney area of Australia.

## 5 REFERENCES

Pells P.J.N., Douglas D.J., Rodway, B., Thorne, C.P. and McMahon, B.R. (1978) Design Loading for Foundations on Shale and Sandstone in the Sydney Region, *Australian Geomechnics*. Jnl. Vol. 8, 31-39.

Pells P.J.N., Mostyn G., and Walker, B.F. (1998) Foundations on Sandstone and Shale in the Sydney Region, *Australian Geomechnics*. Jnl. No. 33 Part 3, 17-29.

Poulos, H.G. (1979) Settlement of Single Piles in Non-homogenous Soil, Jnl. Geot. Eng. Div., ASCE, Vol. 105, No. GT5, 627-641.

Wong, P.K. and Oliveira, D. (2012) Class A Prediction versus Performance of O-Cell Pile Load Tests in Sydney Sandstone, *Australian Geomechanics* Jnl,. Vol 47, No. 3, 89-96.

Sethi, P.L., Geng, I. and Wong, P.K. (2012) Construction of Rock Socketed Piles in Sydney Sandstone to Meet Performance Requirements, *Australian Geomechan.* Jnl., Vol 47, No. 3, 97-102.