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CURRENT PILE DESIGN METHODS

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ABSTRACT: This paper discusses the use of current pile design methods and an alternative method for driven piles, Decourt/Quaresma empirical method, with respect to a major project undertaken for the Queensland Railways. The results of the case study discussed, supports the use of this empirical method as a useful design tool.

1. INTRODUCTION

Geotechnical investigations and foundation design was undertaken as part of a major project for the Queensland Railways between Townsville and Mackay in North Queensland. The scope of work covered the replacement or upgrading of 285 structures, including 187 reinforced box culverts, 83 concrete bridges and 15 steel bridges, and 21.7 km of deviations. This paper focuses on the design of foundations associated with the replacement bridges.

The investigations involved field drilling and insitu-testing. The in-situ testing generally consisted of standard penetration tests (SPT) with hand penetrometer testing undertaken on undisturbed samples where applicable. Piezocone testing was undertaken in one low-lying swamp area.

At the commencement of the project, the client's usual requirement was to drill one hole at each pier location. However, with so many structures to cover, the most cost effective method of investigation was to drill two holes per structure to gather preliminary design information. In cases where specific problems or complex subsurface conditions were encountered, supplementary investigations were undertaken.

The paper will present some experiences encountered during the project, views on current pile design procedures and thoughts on areas which may see further development.

2. DISCUSSION OF PILE DESIGN METHODS

2.1 Bored Piles

Due to the restraints of "on-line" construction, bored piles were the client's preferred foundation option. The two primary restraints were (i) no machinery can operate within 2 m of the track centre line except during track closures, and (ii) piles cannot project above track level. The main advantage bored piles have in this situation, is that they do not extend above track level at any stage and piling rigs can be moved quickly away when trains are passing.

"On-line" construction method also requires the construction of the piles, columns and headstocks beneath the existing timber structure whilst still under traffic. The existing timber structure is then demolished, and the new precast bridge deck units are then placed during one single track closure.

At least forty-nine bridge structures in the project have been designed to be founded on bored piles, with most of these being socketed into rock. Based on previous experience with similar projects, the design of bored piles is relatively straight forward, with the most difficult aspect being the determination of shaft skin friction and end bearing pressure. Methods of determining both of these parameters are well researched and documented.

Socket lengths were determined using an

elastic design method which proportions the shaft load according to the socket length and estimated modulus of the rock. Assessment of shaft adhesion was of primary importance, hence emphasis was placed on the logging of defects during the field work. The rock strength was estimated by the engineer/geologist when logging the core on site.

Assessment of skin friction and end bearing capacity in sedimentary strata was then made, based on work by Pells et al.(1978). Where other rock types were encountered, design parameters were determined based on work by Pells and Rowe(1992).

2.2 Driven Piles

Driven prestressed concrete piles have been designed for use with 20 of the concrete bridges, with 14 bridges utilising enlarged base cast-in-situ piles (or Frankipiles). This foundation option was utilised where the construction of bored piles is uneconomic due to the depth of bedrock, or where construction is undertaken on deviation and standard procedures for pile selection apply.

Where piles have been driven to found on, or just into, bedrock or a very dense/hard layer, the design was again relatively straight forward. However, where the alluvium at a site is particularly deep (>20 m), design and specification of estimated pile toe levels has required greater effort.

The major problem associated with these deep alluvial sites was the difficulty in selecting a founding depth based solely on static capacity.

Contractually, final pile founding levels are based on pile set. The set is usually determined by the contractor based on calculations such as the Hiley Formula, and then approved by the superintending officer. New quality assurance contracts require the use of a dynamic pile analyser (PDA) in combination with calculated pile set to determine final toe levels on site.

Complications can arise on site should the pile achieve design set above the level specified by static capacity. In this case, the pile must either be cut-off if it protrudes above track level or redriven if the PDA indicates insufficient capacity.

Contracts generally also state that the contractor should make a minimum allowance for overdriving. This allows some flexibility should the pile not achieve design set at the specified toe level. If the set (or required load from the PDA) is not achieved when the top of the pile reaches base of the pile cap, the pile is driven on and then extended using reinforced concrete construction. Splicing is not preferred due to the relatively high lateral loads that are applied to the piles.

Experience has indicated that current methods (utilising the suggested design method from AS2159 -1978) generally overestimate the required depth of driven piles to achieve the required design load. The Decourt/Quaresma empirical method has been used extensively on the project for the determination of pile static capacity based solely on standard penetration test data.

3.0 ALTERNATIVE METHOD FOR DRIVEN PILE DESIGN

3.1 Decourt/Quaresma Empirical Method

The Decourt/Quaresma empirical method (Decourt & Quaresma, 1982) is a very simple yet effective method of determining static pile capacity using standard penetration test data only. The formula was established based on detailed comparisons between load test results and information on the subsoil conditions. As the method is based on pile load tests, "set-up" of the pile is implicitly included in the analysis.

The average ultimate unit skin friction is given by:

$$f_s = (\overline{N}/3 + 1)\ 9.81 \quad \text{(kPa)}$$

where
$\overline{N}$ is the average value of N along the shaft

length of the pile
with $N < 3$ taken as 3
$N > 50$ taken as 50

The ultimate end bearing capacity is given by:

$f_b = 9.81\ K_N\ N_b$ (Kpa)

where
N_b = SPT value in vicinity of the pile base
K_N= 12 t.m^{-2} (clays)

= 20 t.m^{-2} (clayey silts*)

= 25 t.m^{-2} (sandy silts*)

= 40 t.m^{-2} (sands)

* denotes residual soils.

The N values are not corrected for overburden effective stress or phreatic surface level unless the pile is to be driven for markedly different overburden stresses than for the SPT test.

It has been highlighted that such a formulae should be used with caution (Weisner, 1982). This is due to the significant differences in "N" values that can occur due to differences in equipment and procedure.

Difficulties also arise in the selection of the value of K_N for materials which do not come under the classification given by Decourt/Quaresma. This is of particular importance for materials such as clayey sands or sandy clays.

3.2 Comparison with AS2159 - 1978

Driven piles are often designed using the suggested method given in Appendix A of the current Australian Standard, namely AS2159 - 1978. The method provides estimated unit skin friction and end-bearing capacity based on laboratory data or in-situ testing.

The current code imposes limits on skin friction and end bearing at a relative depth/diameter (z/d) for non-cohesive soils. This has been shown to be inconsistent with measured values as discussed by Randolph(1993). The draft piling code proposes to impose absolute limits irrespective of pile diameter.

For cohesive soils the design parameters are determined by applying a reduction factor (alpha) to determine skin friction and a multiplier (N_c) for end bearing.

This method also relies upon the SPT "N" (or cone end resistance, qc, from the Cone Penetrometer Test)to determine relative density in sands. In the case of SPT "N" values, the same shortcomings apply to using the "N" value directly. However, AS2159 can be conservative if the "N" value falls near the maximum of the range. This is particularly the case for medium dense sands.

Difficulties arise in the selection of parameters for materials such as clayey sands and sandy clays which may behave as clays or sands. These differences are more pronounced compared to the Decourt/Quaresma method, which allows a sliding scale of intermediate values to be used.

3.3 Case Study - Sandy Creek

A section of the railway line follows the route of the Bruce Highway with average distances between the railway and road bridges in this vicinity of 25 to 50 m. Construction of new road bridges along this section of highway was undertaken by the Queensland Department of Transport (QDOT) in the late 1980's and early 1990's. The site investigation reports, construction drawings and good pile driving records were available from the QDOT. These were used to assess the applicability of the Decourt/Quaresma method.

The piles used for the road bridge were 450 mm octagonal prestressed concrete. The design load for these piles was 650 kN. A typical summary drillhole log, design and final pile toe levels and pile driving records are given in Figure 1.

Estimates of pile capacity using the two methods discussed in the sections above were made using an overall factor of safety of 2.5. A plot of allowable load against penetration is given in Figure 2. The code method indicates that the design load will be achieved at a depth of around 7.0 to 7.5 m while the Decourt/ Quaresma method indicates a depth of between 6.0 to 6.5 m depth.

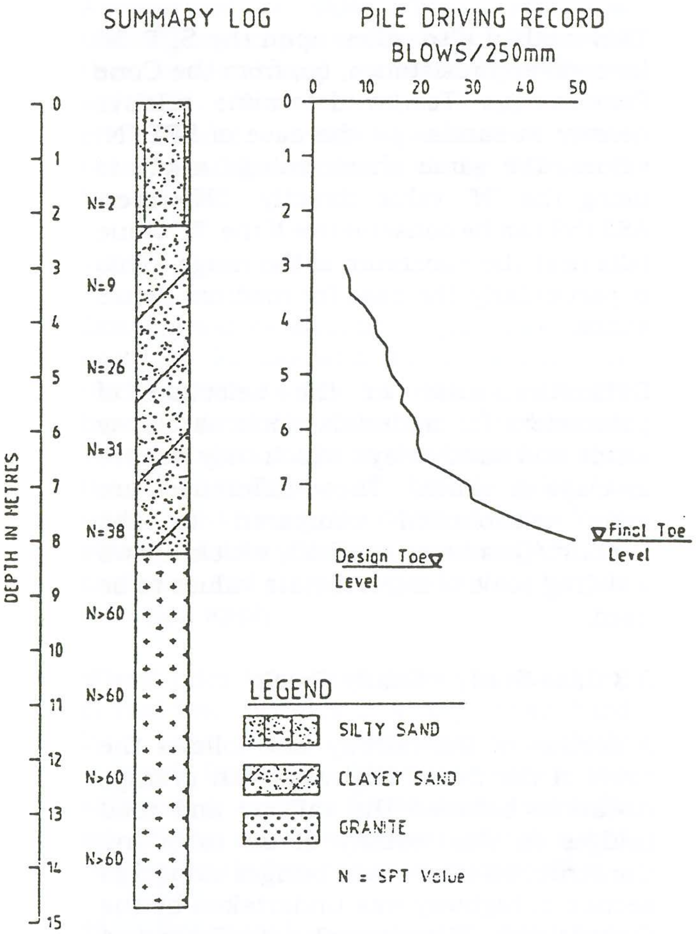

FIGURE 1

The estimated safe load using the Hiley Pile Driving Formula was also determined based on the driving records. A Mitsubishi M23 diesel hammer was assumed to have a rated energy of 58.34 kN.m. Pile weight was taken as 35 kN. The load determined using this method is also provided in Figure 3.

The results obtained indicate a reasonably close correlation between the Hiley formula and the Decourt/ Quaresma empirical method, particularly in the medium dense sands between 3 to 7 m. Pile capacity based on AS2159 appears more sensitive to the material classification. The Decourt/Quaresma method gives a better indication of the increase in pile capacity with penetration. However, comparisons using the Hiley formula must be viewed with some degree of caution. This is because the Hiley formula is subject to a number of shortcomings, primarily due to the number of variables involved.

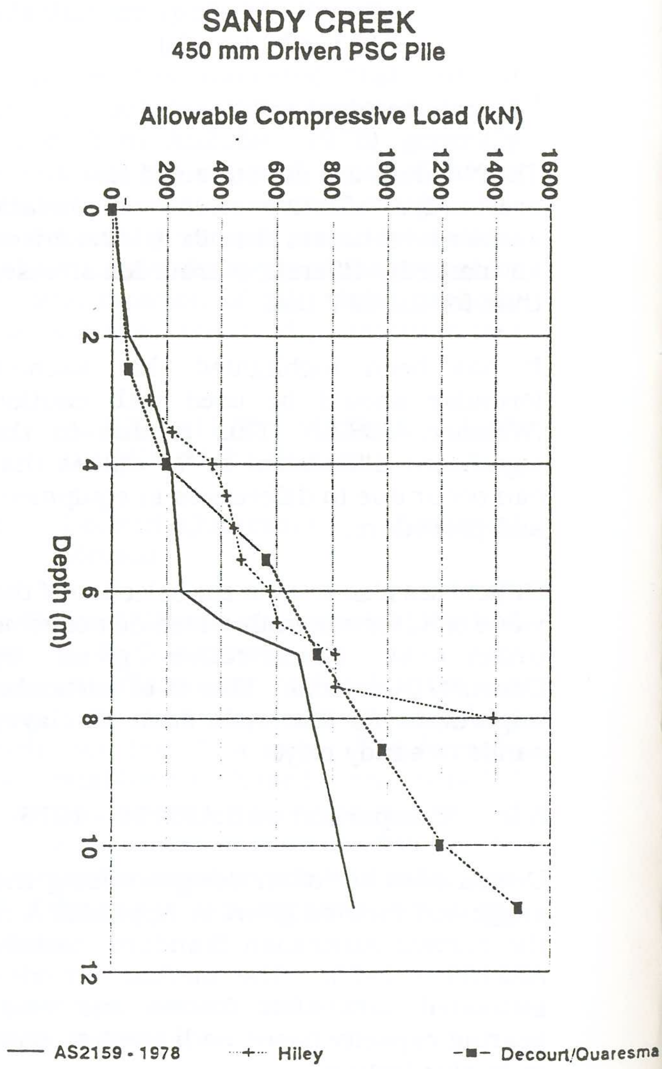

FIGURE 2

4.0 CONCLUSIONS

Based on the case study above, and comparisons at other bridge sites, it would appear that the Decourt/ Quaresma empirical method provides a rapid and reliable tool for the determination of static pile capacity of driven piles. The method easily accommodates materials with properties intermediate between clays and sands.

Our experience with other bridges indicates that the Decourt/Quaresma method generally results in higher pile toe levels compared to those obtained using AS2159. This in turn results in cost savings due to shorter piles.

It is appreciated that the Decourt/Quaresma empirical method has been developed for conditions in Brazil and in particular for residual soils. Although the method achieves results similar or less conservative than AS2159, it will be necessary to undertake additional comparisons with load/PDA tests to verify the method for local conditions.

During the course of this project for Queensland Railways, pile driving records will be closely monitored for a wide range of subsurface conditions. Non-destructive pile testing (PDA - Dynamic Pile Analyser) is scheduled and comparisons will be made with the predicted values as they become available.

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