# DEEP FOUNDATIONS GENERAL REPORT

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## 1. INTRODUCTION

The papers in the Deep Foundations Session cover a very wide range of subjects, ranging from inclined uplift anchors and laterally loaded piles to deep rock socketed piles. Nevertheless, it has been possible to develop the following grouping:

(a) Driven Piles and Dynamic Analyses - 5 papers.

(b) Laterally Loaded Piles - 3 papers.

(c) Performance of (New) Numerical Models - 2 papers.

(d) Others
- Uplift Anchors
- Piles Socketed into Rock
- Piles Driven to Rock
- Uncertainties in Assessment of Capacity
- Piled Raft Systems

This General Report has attempted to identify the key points in the papers and, when possible, to identify areas where there are common experiences or views.

## 2. DRIVEN PILES AND DYNAMIC ANALYSES

The past 10-15 years has seen the dynamic analysis methods for assessing pile capacity advance from the notoriously unreliable "mechanics of rigid body" methods such as the Hiley and Engineering News formulae, to the methods based on the wave equation such as the CAPWAP analysis. The development of the wave equation methods has been accompanied by:

(a) the availability and development of hardware and software packages,

(b) the emergence of companies which provide this specialized service,

(c) the education of design and construction engineers, including changes to specifications and QA procedures,

(d) the validification of the methods, mainly through static loading tests.

The increase in the use of wave equation methods is reflected in the increase in papers from one in the last ANZ Conference to five papers in this Conference. These five papers tend to concentrate on education and on the validification of the method (essentially by CAPWAP) and in so doing also reflect the general level of familiarity and confidence in the method and the views of the authors which see a need to verify or calibrate CAPWAP analyses with static load tests.

The five papers which consider dynamic analyses with the wave equation are those by:

Ahmad and Peaker
Chandler
Cheng, Ahmad and Peaker
Haustorfer and Plesiotis
Randolph

There is one paper by Douglas which utilizes the Hiley formula.

### 2.1 Pile Capacity and Load Distribution

The case studies presented in these papers cover a wide range of conditions but they all show good agreement (Table 1) between the static pile capacity assessed from dynamic tests with wave equation analyses and that measured directly by static load tests. Given this and the experience world-wide with the reliability of wave equation analyses, one wonders why it was necessary to carry out the static load tests.

There is the view expressed by Ahmad and Peaker, Chandler, Cheng et al, and by Randolph that there is a need to carry out static load tests to "calibrate" the dynamic analysis method, and this means providing a sound basis for the empirical damping factors which are used to convert the dynamic response to the required static pile capacity.

Haustorfer and Plesiotis have reported the results of dynamic and static tests on piles which were instrumented with strain gauges down the full length of the piles. This has allowed them to compare the CAPWAP assessed load distribution with the measured distribution, and the agreement is particularly good.

Douglas reports the results of driving closed steel tube piles into coralline materials at two project sites. He recognized that the designs based on SPT data were likely to be uncertain, and used two load tests at each site to confirm the validity of the Hiley formula to control work on site.

### 2.2 Pile Load - Displacement

The papers by Haustofer and Plesiotis and by Chandler both compare the load-displacement curves assessed using CAPWAP analyses against those measured during static load tests, and for four different pile arrangements the agreement is within 20% to 25%.

## 2.3 Dynamic Testing Procedures

The need to assess capacity using a re-strike after intial pile driving, i.e. after any time dependent effects such as pore pressures generated during driving have dissipated, is recognized by most of the authors. The papers by Chandler and by Cheng et al, also raise the question of the required movement to mobilize the full capacity of a pile: this is often considered to be a set of at least 3 mm/blow but Chandler found good results for sets of 0.5 to 1.0 mm/blow and Cheng et al found good results for less than 2 mm/blow. It is interesting that Randolph questions the validity of measurements made when the driving is hard and the assumption that the pile continues to penetrate is violated: sets of 0.5 to 1 mm/blow with an appropriate hammer would be considered hard driving and significant rebound would be expected.

## 3. LATERALLY LOADED PILES

Although there are three papers which concern laterally loaded piles, they all deal with very different sets of conditions which requires separate comment on each paper.

**Pender and Matuschka** have sought to apply simple concepts to the solution for laterally loaded rigid poles in sand, and in so doing have rejected for example linear and non-linear p-y analyses and preferred the method of Broms or similar. However, they found difficulty in implementing such methods in the back analysis of load tests, mainly because test piles are rarely loaded to failure. This prompted the development of a hyperbolic curve fitting to test results, which then permitted failure loads to be determined and assessed against the two models considered. Although the simplicity of Broms' model is recognized, it was found to coincide with the results of field tests on short rigid piles (L/D = 2.6 to 5.7) cast into prebored holes in sand.

**Goh and Lam** have reported the results of four lateral load tests on 1 m dia bored cast-in-place concrete piles. The tests were carried out because it was considered that there was inadequate experience with the application of non-linear p-y analyses in layered soils of the types at their site. They conclude that the non-linear p-y methods of Reese et al has given a good prediction (generally within a few percent) of the observed performance. However, a reader may think that this agreement is fortuitous, considering that

(a) the p-y curves were synthesized from SPT data, particularly considering the very low N values of 1 to 2 in the critical upper levels,

(b) the recognized importance of the upper 10 to 15 m (10 D to 15 D) and the effect that the construction may have had on the performance of bored piles in such weak surface materials.

**Moore and Webb** have tackled the difficult problem of lateral impact loads on fender piles. Laboratory tests using 13 mm hollow square section piles, 1800 mm long and pushed into dry sand were tested. The effect of loading rate on $n_h$ (the rate of increase of the modulus of subgrade reaction) was measured and found to be negligible for the dry sand for deflection rates of 2.8 to 24 mm/sec but values of $n_h$ ranged from 14 to 21 MN/m$^3$. The laboratory measurements of pile response were compared with the response predicted by the p-y methods of Davisson and Robinson, the equivalent stiffness method of Webb and the elasto-plastic model, where agreement was found to be within 30%. This order of discrepancy has been attributed largely to experimental procedure and variations in soil properties, rather than inadequacies in the models.

## 4. NEW NUMERICAL MODELS

The papers by Lee and Poulos and by Hewitt and Poulos both present numerical models and the laboratory verification of these models.

The paper by **Lee and Poulos** addresses the side resistance of a pile in calcareous sand and develops a non-linear model of the pile-soil interface which they implement in a boundary element analysis. The model includes two parameters ($\alpha$ and $R$) which need to be determined experimentally from pile loading tests, and the modulus and limiting skin friction, which may be determined from model pile tests or other laboratory tests. These, and the other required parameters were back-figured from the authors' model tests and then used to predict the performance of the model piles. This somewhat circular approach results in close agreement between the measured and calculated load-displacement curves for three tests with different overburden pressures. The model is also used to predict the performance of field tests at three sites in Kuwait, with the numerical model using $\alpha$ values back-figured from load tests not taken to failure and other parameters derived from the authors' model tests: the comparison is good with the exception of the initial slopes, differences which the authors attribute to doubtful pile test measurements. The prediction of peak load is very good. A similarly good agreement has been obtained in the analysis of a field test in France. The model clearly provides a good representation of field conditions, but model pile tests need to be carried out to obtain the input parameters.

The paper by **Hewitt and Poulos** covers the degradation of axially loaded pile groups in clay, and was designed to assess the reliability of a simple degradation model with a boundary element analysis to predict the effect of cycling on capacity and accumulated displacements. They use the degradation model of Matlock and Foo, which uses the minimum degradation factor $D_{min}$ and the rate of degradation $\lambda$, and thus show firstly that the model can be matched to the behaviour of the pile groups and secondly that pile groups suffer degradation at lower values of $P_c/P_u$ than apply to single piles, implying that interaction has an effect on degradation. Load distributions are measured down the length of the model piles and it was found that cyclic loading caused little degradation of side resistance during cycling but that cycling had greatly reduced the side resistance over the lower half of the pile when the piles were then taken to failure. Unfortunately the authors offer no explanation for this mechanism except to note that it matches their theoretical degradation.

## 5. FROM INCLINED UPLIFT ANCHORS TO STATISTICS

The remaining five papers do not readily fall into a common group, and they are reported separately.

**Philips and Young** report experiences with the performance of Frankipiles and segmented Hercules-type piles on a project involving about 3000 piles. Both pile types were driven through fill and alluvium to sandstone. The project adopted an approach which saw site investigation drilling at about 50% of all pile locations (to accommodate the risks of founding on boulders or over-hangs). Four loading tests were conducted on trial piles to 2.5 times the design load to verify the adopted design end-bearing stresses and a further 21 piles were proof loaded to 1.5 times the design load to resolve uncertainties raised by higher than expected founding levels. In all cases, failure did not occur and settlement was close to or less than the predicted settlement based on an assumed rock modulus of 1 GPa and zero side resistance in the alluvium or fill. However, the authors have analysed the test results in more detail and have used a technique to separate the side and end-bearing components. Their back-analyses have shown

(a) that the design assumption of neglecting side resistance was more conservative than anticipated,

(b) that the moduli of the sandstone varied widely but with means of 0.7 to 0.9 GPa which is similar to the assumed value of 1.0 GPa.

**Buttling and Lam** report the results of load tests on 10 instrumented rock socketed piles (9 were 1 m dia and 1 was 0.6 m dia). The piles were constructed through sands and clays to socket into sandstone, siltstone and mudstone. The piles were test loaded to 2.5 times their design load i.e. 750 t to 875 t for the 1 m $\varphi$ piles and 282 t for the 0.6 m $\varphi$ piles to provide the basis for accepting a more weathered base criterion. This program of testing has been fortunate to have instrumented the test piles; however, it is unfortunate that the soil and rock data are based only on SPT's when other current methods for designing socketed piles would consider the use of pressuremeter tests, particularly for modulus measurement, with either UCS or triaxial tests. The importance of base cleanliness for piles which are designed to include base resistance was appreciated.

**Chu, Lo, Lee and Zhao's** paper considers the sources and importance of the various uncertainties in estimating the axial capacity of a long friction pile on the basis of cone penetrometer data. They identify the three areas of uncertainty as

(a) measurements of sleeve resistance,

(b) spatial variability of soil properties,

(c) the coefficient m which relates CPT sleeve resistance to pile side resistance.

The errors in measuring sleeve resistance were ignored because it is assumed that a good quality penetrometer is used and the question of 3-D variability of soil properties was simplified to a 1-D case for the long single pile and CPT data. The statistical analysis then shows that the main source of uncertainty lies in the values of m. However, one hardly needs a complex statistical analysis to appreciate this conclusion, which suggests that the main point of the paper concerns the statistical method rather than the example chosen. It would be interesting to extend the analysis to include the radial variability of soil properties, so that the uncertainties in the capacity of several piles could be considered.

**Gerrard and Cameron** assess the performance of various ground anchors of the stake and buried plate types which are designed to be recoverable and to withstand inclined uplift loads. For the stake anchors, their theoretical considerations are based on a strong rigid stake anchor and a simple statics analysis using the pressure distributions of Broms but considering the interaction of the vertical and horizontal components of load. The analysis is applied for anchors in sand and clay. The deep buried plate solutions are used for their duck-bill (plate) anchors. Inclined uplift tests were conducted in sands and clays with single and multiple anchors. These series of tests have allowed conclusions to be drawn concerning the effects of inclination of load on capacity (e.g. for the simple stake anchor in sand, there was negligible loss of capacity until $\alpha$ exceeded 60°). However, the agreement with the theory is generally poor. The interpretation of the multi-pin anchors was made difficult by uncertainty of the failure mechanism: for example it is suggested by the authors that for multi-pin anchors in sand, some pins may fail horizontally while other may fail vertically.

**Goad, Morgan and Tchepak** have presented the basis of their design for foundations consisting of piled rafts, where the piles are founded in compressible material and the design tailors the pile lengths to control the differential settlement between adjacent slabs. Their approach to the capacity of the Frankipiles was simple: base capacity = $A_b.q_c$ and side capacity = $A_s.\alpha.q_c$ and tolerable agreement was found with load tests for base resistance. The greatest difficulty was experienced in determining settlement, i.e. the value of the soil equivalent modulus to be used in their elastic method of analysis. An equivalent modulus was required to accommodate the expected creep in the soil. Preconstruction pile tests were used to confirm the assessments of capacity but particularly to provide the basis for fixing the equivalent soil modulus. The tests indicated values which were stiffer than those adopted for the design, but they also indicated significant set-up effects. The settlement of the project is being monitored.

TABLE 1

COMPARISON OF DYNAMIC AND STATIC PILE CAPACITIES

| Authors | Pile Type | Pile Length | Soils | PDA Capacity | Load Test | PDA/static Capacity | Comments |
|---|---|---|---|---|---|---|---|
| Haustorfer & Plesiotis | 450×450 PSC | 14.5 | Through sand to limestone | (a) 3700<br>(b) 4118 | 3900<br>4200 | 0.95<br>0.98 | Load distri-butions also agree |
| Haustorfer & Plesiotis | 355×355 RC | 12.0 | Through sand to clayey sand | 1200 | 1270 | 0.95 | |
| Chandler | 550 dia CIP | 18.0 | Through silts and clay to dense sand | 3400 | >3000 | <1.13 | 3000 kN at 10 mm |
| Chandler | 760 dia open steel tube | 20.0 | Through clay and silt to v stiff clay | 4800 | 4800–4900 | 1.0–0.98 | Sets <3 mm |
| Ahmad et al | 325 dia closed tube | 7.0 | Through fine sands to cemented calcareous sands | 2197–2688 | >2943 (at 11.4 mm) | <0.8–0.9 | |
| Ahmad et al | 400×400 PSC | 20.0 | Through loose coral sand to dense coral | (a) 1158<br>(b) 2325 | 1226<br>>1226 | 0.94<br><1.9 | |
| Cheng et al | 194–298? dia closed tube | (a) 20.0 | Fill over dense sand | 2180 | 2170 | 1.0 | Static capacity by off-set limit criterion |
| | 244 dia closed tube | (b) 9.0 | Fly ash over shale | 880 | 1020 | 0.9 | " |
| | " " | (c) 47.0 | Fill over clayey to sandy silt | 2375 | 2400 | 1.0 | " |
| | " " | (d) 33.5 | | 1525 | 1630 | 0.9 | " |
| Randolph et al | 12–13 dia solid bar | 0.2 | Sand | 4.6 | 6.4 | 0.72 | Dynamic analysis by IMPACT |