Applicability of the API Standard Pile Design Method for Marine and Near-Shore Foundations

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Abstract

The standard API (API, 1991) codified pile design method is frequently used by foundation engineers in preliminary design and tender assessments for near shore structures. It is also frequently specified by Clients in the belief that it's widespread use in the past in offshore construction, will produce a robust design. This paper argues that the limited geographical spread of data on which the code is based, and the stardardised investigation methodology which goes with it, can produce a flawed design tool, which cannot be considered reliable on a world wide basis.

The authors believe that substantial caution should be exercised when designing piled foundations in areas with limited recorded geotechnical precedent. The consequences are discussed in relation to highly over-consolidated clay soils, fused sands and sands with minerals other than quartz, drawing on examples from around the world.

Introduction

The authors have been involved in a number of projects where piling operations have been subject to varying delay and cost overruns associated with unexpected pile performance. In this context it should be noted that such performance problems are not necessarily related to "unexpected" ground conditions, in terms of the presence of materials other than those identified at site investigations. What is unexpected is a particuar element of the overall soil/pile interaction or the unusual interaction between the sampling tool and the soil.

It could be argued that the soil investigations, in some of the problematic projects, were of poor quality or misleading. This may, on hindsight, be the case. However the authors wish to demonstrate that the soil investigation seemed to be sufficient and there also seemed to be a good correlation with the API design rules. Only after the piling work had started and additional testing was done (costing time and money), was it clear that the actual soil conditions and behaviour did not correlate with the API.

There have also been a number of reviews in recent years which have suggested that the commonly used pile design methods, or their mode of application, are unreliable, and in some cases, less than conservative (Briaud & Tucker, 1988 / Wu and Tang, 1989 / Datta et al, 1980 / Bernades & Kormann, 1997).

Nevertheless, many Clients still specify such design methods in their tender documents, and many Contractors still choose to select or offer designs based on them. In the latter case the Contractor's Bid Team is often working under extreme time constraints. The time saving associated with adopting what amounts to an "off the shelf" design package for budgeting and cost planning, is a powerful draw to the bid manager.

Those who are aware of the difficulties inherent with pile design may attempt to shift the risk back to the Client. However, it is our experience that modern Clients are increasingly less likely to respond positively to a Contractor's request to make pile lengths a re-measurable item in the Contract.

This does of course allow Clients to compare bids on a "like for like" basis, and also allows a Client to approach funders and insurers with a fixed cost design that is based on, what appears to be, well established precedent.

The internationally exposed Contractor, working in today's gobal construction market, and in particular in the rapidly developing third world, frequently finds himself working in ground conditions far removed from the "typical" ground in which the standard method was developed. It should be noted that this is not a critism of the API design methodology itself. Those responsible for drafting the API Code were aware of the pitfalls inherent in its use. We are of the opinion that many of those who use or recommend its use do so unaware of, or choose to ignore, the advice contained therin (the bold text is ours):

"The foundation configurations should be based on those that experience has shown can be installed consistently, practically, and economically ***under similar conditions*** *with the pile size and installation equipment being used. Alternatives for possible remedial action in the event design objectives cannot be obtained during installation should also be investigated and defined prior to construction".*

In practice it is the Piling Contactor who generally carries the risk, and cost, of any such remedial measures.

The Standard API Pile Design Procedure

The standard API methodology for calculating Axial Pile Capacity is outlined below:

Formula One

For piles founded in cohesive soils the unit skin friction is:

End bearing is calculated from the simple equation *q= 9c*, with *c* being derived from standard undrained strength tests.

In granular soils the shaft friction is calculated as follows:

End bearing is calculated from the equation:

Table 1, (API Table G4.3-1) is used to ascribe the relevent and *Nq* values based on engineering soil descriptions derived from SPT or CPT data, and to limit skin friction and end bearing capacity.

Highly Over-Consolidated Clays

Problems immediately become apparent when the design engineer is confronted with very stiff over-consolidated clay soils. In these materials the initial driving of the pile produces a hole marginally enlarged with respect to the pile diameter. The hole is initially self supporting, but over periods of hours, days, weeks, or months the relaxation of arching around the pile leads to a partial, or even complete recovery of initial conditions, producing the time related effects known as "set-up".

Whilst the effects of set-up have been discussed by many authors, (notably Tomlinson), it is almost ignored by the API design guidance (The only reference to such effects being the statement in Commentary G6.2.7 relating to Cyclil Loadings:

"These tests should at least implicitly include the effects of pile installation, loading, and time effects".

The problems this can cause for piling contractors are discussed by reference to a project in Venezuela.

The structure being built comprises a 2km long jetty, requiring a combination of 1219mm and 914mm diameter steel pipe piles.

Table 1.

DESIGN PARAMETERS FOR COHESIONLESS SILICEOUS SOIL (after API RP2A)					
Density	Soil Description	Soil-Pile Friction Angle	Limiting Skin Friction (kPa)	Nq	Limiting Unit End Bearing (Mpa)
Very Loose	Sand	15	47.8	8	1.9
Loose	Sand-Silt	15	47.8	8	1.9
Medium	Silt	15	47.8	8	1.9
Loose	Sand	20	67.0	12	2.9
Medium	Sand-Silt	20	67.0	12	2.9
Dense	Silt	20	67.0	12	2.9
Medium	Sand	25	81.3	20	4.8

Dense	Sand-Silt	25	81.3	20	4.8
Dense	Sand	30	95.7	40	9.6
Very Dense	Sand-Silt	30	95.7	40	9.6
Dense	Gravel	35	114.8	50	12.0
Very Dense	Sand	35	114.8	50	12.0

Loads vary throughout the structure, but for the purpose of this paper the required Ultimate Bearing Capacity can be considered to be 6000kN, derived from working loads of the order of 3000kN. The contract requires capacity to be verified by PDA testing, and these results are correlated with static load tests.

Ground conditions comprise a veneer of Holocene marine and littoral sediments overlaying an over-consolidated hard/very stiff alluvial clay, which in turn overlies a sequence of silts, sands and stiff clays. The upper hard/very stiff clay is believed to be highly over-consolidated as a consequence of the desiccation during a period of prolonged Pleistocene emergence, the interested reader is referred to Ladd et. al., 1987. The authors wish to note that similar profiles are common in the coastal waters of the tropics, having recently assessed similar ground conditions at sites as far afield as India, Thailand and Hong Kong.

A generalised soil profile is shown, together with profiles of over-consolidation ratio and shear strength with depth. The extent of the over-consolidation near the top of the sequence, together with the predominantly brownish yellow coloration and occasional secondary deposits of salts (described as "calcareous nodules") suggest that these are *solonchak*, or *white alkai* in origin, (Nortcliff, 1983) typical of arid, semi-arid and of some temperate conditions.

Where such profiles exist it is frequently seen that the near shore strata display high over-consolidated ratios, whilst their off-shore equivalents, and their underlying deposits, neither of which were aerially exposed during Pleistocene retain a normally consolidated profile. This gives rise to higher deposits exhibiting strong (*sensu lato*) shear strength and over-consolidation ratio decreasing with depth.

Fig. 2 Relationship between OCR, Shear strength, effective (vertical) pressure and reduced level

Fig. 3 Generalised soil profile of the site

Piling works commenced at the shoreward part of the structure, with 914mm piles being driven to approximately 24m penetration. At this location the Clients own investigation had shown ground conditions comprise a predominantly sandy profile. Initial PDA tests carried out during driving suggested something of a shortfall in pile capacity, with overall Factors of Safety of approximately 1.3 instead of a design 2.0. Concerns were therefore raised regarding serviceability limits arising from possible excessive settlements, at working load.

A static load test was therefore brought forward in the programme, and carried out at a location 6 bents from the abutment. This location was the nearest which the Contractors Piling Barge (being used for the reaction) could approach shore. The static load test was carried out 5 days after installation of the pile. The load capacity displacement for this test is shown below, from which it can clearly be seen that failure, in soil mechanics terms, occured at about 4300kN load, significantly short of the required margin.

Fig. 4 Load Displacement Curve

There followed an urgent review of driving records for the piles already installed, the boring of a number of additional holes, research into the geotechnical behaviour of nearby piled foundations, a review of the design based on the new MTD design methodology (Jardine & Chow, 1996), and a revision of the API design method used.

The basic conclusions of this extensive review were:

- ground conditions at this location did not conform to the expected predominantly sandy soil profile;

- one month after construction the capacity of a pile is typically 1.7 times the end of driving capacity;
- that these large diameter piles do not plug readily;
- that the as designed pile toe levels would offer the required bearing capacity one month after construction.

A second static load test was then carried out a further 6 bents seaward (bent 12). This temporary pile (914mm diameter but at a location requiring 1219mm permanent piles) was driven to 20m penetration, and was required to display 4300kN capacity (deemed equivalent to the required capacity of a 1219mm pile at this location). The test was carried out appoximately 2 months after installation. The load displacement curve for this test showed failure occuring at approximately 5000kN

From this data it can be seen that there is, for these soils, no discrete ultimate bearing capacity as such. The strength and behaviour of the soil/pile interface is clearly time dependant. The effect in these over consolidated clays is considerable. However, recent work (Chow et. al. , 1997 and 1998) has shown that such time dependant changes can also occur in sands, with two-fold increases of capacity apparent with every log-cycle of time.

Cemented Sands

In this section a case history is described from the island of St. Vincent in the Caribbean, where difficulties were encountered during pile driving operations for a jetty structure.

At the time of tender design (in 1993) the designers had recently been involved in other similar projects in the area which had used standard API design procedures without problem. No particular caution was considered necessary due to the islands volcanic origin. It should be noted that the specific warning in the API commentary first appeared in 1991 Edition (1993 being the most recent).

The investigation carried out yielded a soil description of "SAND" and reasonable SPT N values. On this basis pile design was undertaken and soil parameters fixed according to the API recommendations. Heavy driving was anticipated from the high N values apparent. However during vibrating and driving of 610mm piles virtually no resistance was encountered. No significant difference between open ended [design] and closed [1 test] piles was apparent, and a re-drive test after one week indicated very little on-going set-up.

Subsequent analysis of the material showed that the sand is very fine and 'bonded'. This was attributed to the volcanic origin of the sands. Rather than behaving as the dense/ very dense granular soils assumed at tender stage, the sand was cohesive in nature, due to the presence of point contact welds within the soil skeleton.

The investigation drilling using SPT tools encountered reasonable resistance, the driving energy being insufficient to disrupt the bonds at the tip of the shoe. On passing into the split spoon sampler however the disturbance did break the bonds and the sample when viewed was seen to be essentially granular. The pile when driven with high energy encountered little or no resistance locally, breaking the bonds ahead of the pile tip.

However, in addition to the apparent loss of end bearing the skin friction was also substantially reduced. The welded contacts were not broken down laterally away from the pile. The pile bore remained therefore essentially self supporting. No recovery of conditions occured as the soil arch around the pile remained open, and consequently no friction was generated at the soil/pile interface.

A technically acceptable solution was eventually achieved, at considerable expense, by driving to a deeper stratum. The piles could then achieve the required capacity by acting as end bearing piles, rather than the as designed friction piles.

The only way that this solution could have been avoided, given the standard investigation methods employed, was by the design engineer having, or obtaining, the knowledge that the local sands were volcanic in origin, and would not perform as had the marine sands at nearby sites.

Similar problems are being reported with an increasing frequency from sites around the world (see for example, Yasufuki and Hyde, 1995), in many "meta-stable" soils.

Mica Sands

Recent work (Mundegar, 1998) has drawn attention to the effects of relatively small quantities (1% by mass) of mica flakes on the shear strength of quartz sand.

Mundegar noted the tendency of the mica to suppress dilation of the sand particles and generate a reduction in shear strength by a factor of 3. In relation to pile design, this tendency to suppress dilation will have a significant effect. The normal associated design parameters, whether a limiting skin friction, or a depth dependant derivative of must be selected accordingly.

Recent work reported from Jamuna Bridge, confirmed that these mica sands perform poorly in lateral compression, although there is no specific comment in that literature regarding the performance in axial compression. The need to use the Menck 1700 piling hammer for that project suggests that the driving performance was not considerably eased by the presence of such mica, as might first be thought. The results of using more conservative shear strength parameters where mica sands are present will however produce longer pile designs, requiring driving to greater depths. This depth in itself may lead to the need to use more powerful hammers to achieve target depth, as the driving forces are dissipated by the soil/pile interaction over the whole length of the pile.

Conclusions

In preparing this paper the authors reached a number of conclusions, based on our experience which are outlined below.

Firstly, it is apparent that there is little correlation between the scales of the in-situ testing carried out during site investigation, and the loads and forces imparted during driving and in service.

Any design method that relies on empirical correlation with parameters derived from such SPT or CPT data is introducing an uncertainty into the design process.

By ignoring the presence of pile set-up as a phenomenon a substantial element of soil/pile interaction is ignored. Those planning pile load tests in sandy materials should be aware that sandy soils can exhibit substantial set-up.

There has been little research carried out in relation to establishing any fundamental parameters on which selection of appropriate values can be made. The stress history, mineralogy and pile installation method will all be important. However, without the benefit of local knowledge, and precedent, designs may need to be worked up assuming variations of of 50% of the API calculated value. Pile lengths can then reasonably quickly be modified as piling records and pile testing commence.

The development of alternative pile testing methods, and in particular those based on effective stress methods is to be supported. Whilst pile driving is a total stress dominated action, it is in our view necessary to consider axial pile capacity as a fundamentally effective stress dominated action.

When undertaking pile design the designer should note that ***a conservative selection of pile/soil friction parameters for pile design may lead to the need for much heavier driving equipment being needed, to achieve the target toe level.*** The increased depth offsets any gain in perceived driving performance derived from the lower friction coefficients.

Acknowlegements

The authors gratefully acknowledge the role of their Client Interbeton bv. whose projects we have described.

The paper has also benefited from a critical review by colleagues at Delta Marine Consultants, and Geotechnics Ltd.

The recovery element of the work undertaken in Venezuela was carried out jointly with Geotechnical Consulting Group, particular acknowledgement is due to the contribution of Dr Fiona Chow.

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