The paper was published in the proceedings of the 7th Australia New Zealand Conference on Geomechanics and was edited by M.B. Jaksa, W.S. Kaggwa and D.A. Cameron. The conference was held in Adelaide, Australia, 1-5 July 1996.

The Performance of Driven Steel Piles Founding in Coral

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Summary The Reserve Bank of Tonga, at five stories, is the tallest building in Tonga. Traditionally buildings in Nuku'alofa, the capital of Tonga, have been building on a thin calcareous layer of cemented coral located close to natural surface. This layer is underlain by loose coral sands, which required this taller building to be supported on piles. Steel H piles were selected, designed to found in a layer of very dense coral (SPT refusal) underlying these loose sands. Initial pile driving was monitored using the Pile Driving Analyser (PDA) equipment, which revealed that the required pile capacities could not be achieved. Sourcing of additional piles was impractical. Using offcuts from the available pile sections, the piles were modified to increase their projected base area, adopting the ultimate end-bearing capacity indicated by the PDA testing as a basis for sizing the modifications. These modified piles were driven with little delay to the project, and now successfully support the completed structure.

1. INTRODUCTION

Most buildings in Nuku'alofa, capital of the Pacific Island nation of Tonga, are of one and two levels, with only a few three level structures.

These buildings are typically supported on spread footings or raft slabs and in general have performed satisfactorily. However, one four level building constructed in this way underwent significant differential settlement and has never been fully occupied. Only limited, if any, geotechnical investigations have traditionally been performed for new structures.

In 1992 the government of the Kingdom of Tonga approved the construction of a new Reserve Bank Building in Nuku'alofa. This was, at that time, to be a four storey structure, about 30 m square. Fortunately it was recognised that a geotechnical investigation was required for this, the largest building ever to be constructed in the country.

Following the geotechnical investigation, driven piles founding in coraline limestone were recommended for support of the building. However, prior to construction the building height was increased to five stories, with individual pile loads of up to 2500 kN specified on 310UC piles. These loads were not able to be achieved for the ground conditions at the site, and modifications to the piles were necessary.

2. SITE INVESTIGATION

At the time of the geotechnical investigation (1992), no published geological maps could be located for the island of Tongatapu, on which Nuku'alofa is situated. The expectation was that coraline deposits overlying basalt or other volcanic rock would be present, but the depth to this basement rock was unknown.

Four boreholes were drilled at the site, to depths of 8.2 m to 10.6 m. The drilling was performed using a drilling rig and crew provided by the local Public Works Department. Although Standard Penetration Testing (SPT) and diamond core (NMLC) equipment was available, it soon became apparent that little site investigation drilling had been performed by this equipment. No facility initially existed to run casing, and suitable drilling mud for support of the boreholes was not available.

The first two boreholes were therefore drilled with difficulty and with only limited testing due to collapsing of the borehole walls. However, a presumed suitable founding layer was found and proved for a few metres by a combination of wash boring and NMLC coring. By the third borehole, a casing adaptor had been manufactured, and the remaining two boreholes were able to be drilled with frequent and reliable testing. However, drilling beyond about 10 m depth also proved to be difficult because inadequate pumping capacity was available to satisfactorily flush the drill cuttings from the borehole.

Such are the frustrations of site investigation in remote and less developed countries.

3. SUBSURFACE CONDITIONS

The boreholes, and some backhoe excavated test pits, indicated relatively consistent ground conditions at the site, comprising:-

Topsoil and Fill, to depths ranging from 0.2 m to 0.7 m, overlying

Cemented Coral/Limestone, ranging in thickness from about 1.0 m to 2.0 m. Able to be excavated with difficulty with the rubber tyred backhoe, SPT N values about 10.

Silty Sand, very loose to loose, up to 12% fines, with some bands of sandy silt, soft to firm. Varies in thickness from 3.5 m to 4.5 m.

Limestone/Cemented Coral, encountered below about 5.5 metres depth and continued to maximum depth investigated of 10.6 m. Core recovery about 50%, with recovered core comprising high strength moderately strongly cemented coral and limestone pieces in cemented silty matrix. SPT refusal consistently obtained, with spoon penetrations in the range 80 mm to 200 mm.

The results of the Standard Penetration testing performed are shown on Figure 1.

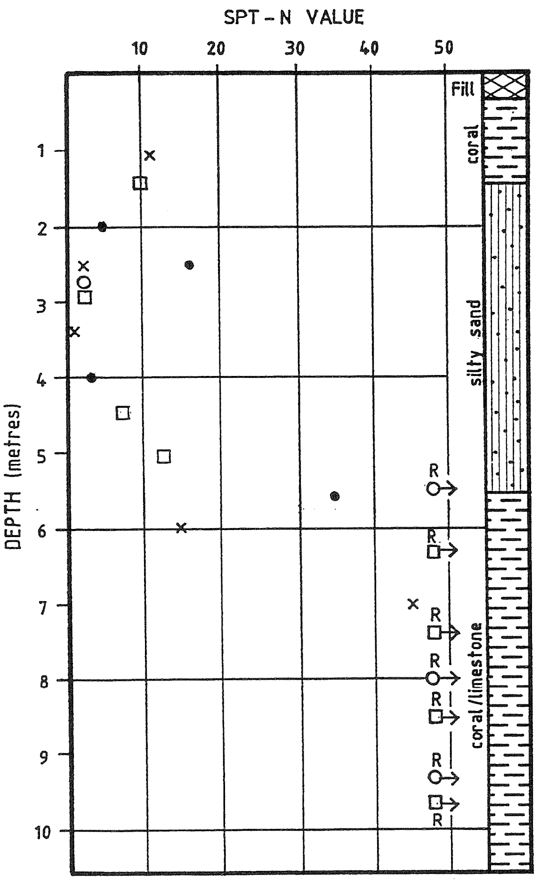

Figure 1. Standard penetration test results.

Based on this investigation it was recommended the proposed building be supported on driven piles, founding in the limestone layer encountered about six metres below original ground surface level. The use of driven steel piles was preferred, to allow for the possible hard driving that might be required to penetrate the upper cemented layer. Maximum pile working loads of 1500 kN were indicated as likely to be practical for these piles.

Seemingly consistent with these recommendations, the designers of the building elected to use steel H-piles to support the building. Safe working loads on the piles ranging from 500 kN to 2400 kN were proposed by the designers. For pile loads up to 1300 kN, 250UC pile sections were specified. For higher loads, 310UC sections were specified.

At the time of tender, the feasibility of achieving these loads on the specified pile sections was questioned by the successful tenderer, and the writer was commissioned to be present during the initial stages of pile driving, to evaluate the performance of the proposed piling system.

To compound the concern already existing regarding the adopted pile loads, an error in the quantities of each pile type ordered, and delivered to site, resulted in a need for re-design of the piled footing system to allow the pile sections actually delivered to site to be utilised. This resulted in changes to the required pile loads at some locations, and modifications to some pile sections. The highest working stresses on the piles were due to a design load of 1700 kN, proposed on a 250UC90 pile (plated on one flange to achieve the required structural capacity). That is permissible structural stresses rather than geotechnical capacities now governed even more than before. This re-design also meant that there was negligible surplus steel available to allow splicing of piles if additional driving was required to achieved the specified load. Furthermore, no new steel and negligible scrap steel could be sourced in Tonga. To import additional steel would have resulted in unacceptable delays to the project

4. INITIAL PILING

Piles were driven with a 7 tonne cable operated drop hammer, striking a hardwood timber cushion. Soon after driving commenced it became apparent through the use of the Hiley pile driving formula that the required pile capacities were unlikely to be achieved with the pile sections proposed. The necessary equipment and operator to allow analysis of pile capacities using the Pile Driving Analyser (PDA) equipment were then mobilised to site.

The PDA testing performed confirmed the initial conclusion that it was unlikely that other than the most lightly loaded piles would achieve their required capacity. It was apparent that either additional piles would be required or modifications made to the existing piles to allow them to mobilise higher capacity. It was decided to modify the piles.

5. MODIFIED PILE DESIGN

Based on the results of the pile driving, and the PDA testing, it was apparent the piles were predominantly end bearing in the lower limestone layer. No appreciable gain in resistance was observed from further driving into this layer, as would be expected if significant skin friction was being developed.

This initial driving also indicated the ultimate base resistance available for piles founding in the lower limestone was about 15 MPa to 20 MPa. It therefore was decided to modify the piles by adding "wings" to the flanges of the UC sections, thus increasing the available end bearing area and then designing them to accommodation the design load solely in end bearing. The wings were initially located one metre behind the toe of the pile, to allow some toe fixity. After driving of some winged piles, it was decided the distance from the toe could be reduced to 0.5 m.

The wings were manufactured using off-cuts from the already driven piles, and using spare steel plate available after the modifications to the initial pile sections were complete. Several wing sizes were developed, based on the required pile capacity. A maximum ultimate base resistance of 15 MPa was adopted when specifying a particular wing design. Figure 2 indicates a typical wing detail.

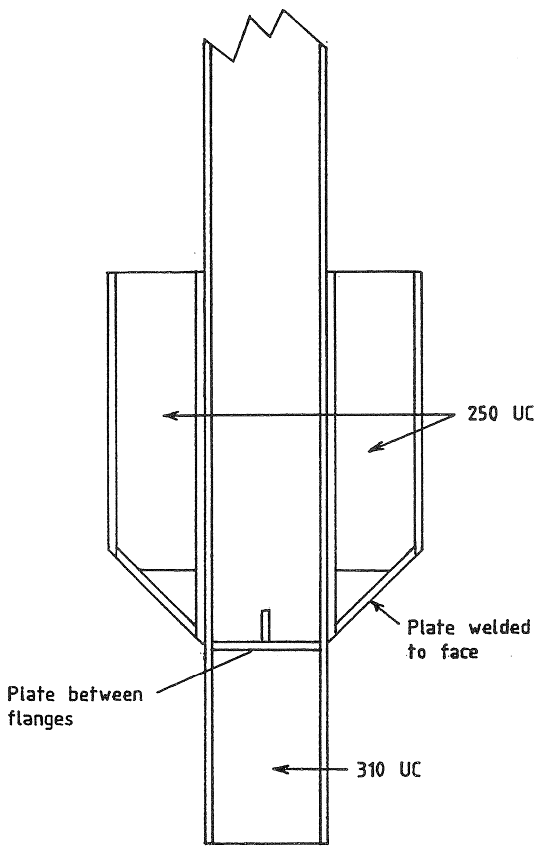

Figure 2. Typical winged pile construction.

Based on the use of PDA testing to establish the ultimate pile capacity, a factor of safety of 1.5 was adopted where earthquake loading governed, and 1.8 for dead load plus live load,. The PDA testing was also used to establish the efficiency of energy input to the pile for the driving equipment in use. This then allowed the ultimate capacity of those piles driven without PDA to be calculated with confidence from the pile driving records (set and temporary compression).

6. PERFORMANCE OF WINGED PILES

Five wing designs were developed, based primarily on use of available pile sections with a minimum of welding. The wing designs adopted varied in their total projected base area from 0.15 m^2 to 0.22 m^2.

Forty one piles were driven with the winged modifications. These piles were driven until the required ultimate capacity was indicated to have been achieved, either by PDA or use of driving records and expected energy delivered to the pile (based on earlier PDA analysis). In some instances the calculated capacity was increasing with further penetration, but additional driving was not ordered because the required capacity had been achieved.

For the projected base area of the winged pile sections, the mobilised base resistance for each of these piles was found to range from 11.3 MPa to 18.7 MPa , with a mean of 14.9 MPa and a standard deviation of 1.94 MPa. The required ultimate capacity was achieved for all piles, with the lower mobilised base resistances corresponding to those piles where sufficient capacity was achieved without further driving. As such, it is judged probable the design ultimate base resistance of 15 MPa was reasonable, and could have been achieved if required. In general the adopted minimum factors of safety were exceeded.

The building is now complete, and is performing satisfactorily.

7. DRIVING EFFICIENCY

The PDA data proved invaluable in assessing the efficiency of the driving equipment in use, and hence allowed those piles driven without PDA to be installed with confidence.

The 7 tonne drop hammer used to drive these piles was raised using the power lowering winch of the crane, and a single sheaf pulley system. The PDA indicated the efficiency of this system was about 35%, significantly reducing the benefit of the heavy hammer. Of interest is that during the course of the pile driving the drive chain connecting the winch to the power lowering sprocket broke. After eliminating the inertia associated with this mechanism, and replacing the rope, the efficiency of the drop as transmitted to the pile, increased to slightly better than 50%. This change was clearly significant and confirmed again the value of the PDA, and highlighted the order of energy losses which can occur within the pile driving system.

8. CONCLUSIONS

This project resulted in a number of salutary lessons. Fortunately the required pile support to the building was able to be provided at little extra cost or delay to the project. However, the various factors which were encountered could just as easily have resulted in significant cost and time over-runs.

The significant lessons learnt, or reinforced, through this project included:-

- Be cautious in estimating the likely performance of site investigation equipment when sourced sight unseen from organisations with little relevant or known experience.

- Be generous in the depth to which sites are investigated in coral deposits. It is possible the basement basalt rock was present within a few metres of the adopted founding depth (not proven). To have persevered with the available equipment to depths of about 20 metres would have, with hindsight, been prudent.

- Coral deposits, even if shown to be very dense/strongly cemented are unlikely to provide significant shaft adhesion for driven piles.

- Be very specific when recommending pile types and predicted capacities. The structural capacity of steel piles will usually exceed the geotechnical capacity of such piles unless they are founding on very competent materials. This is particularly so for low displacement piles such as H-piles.

- Conservatism in footing design when working in remote areas may be prudent.

- Dynamic testing of piles (PDA) was invaluable in resolving an otherwise extremely uncertain situation.

- The end bearing capacity of 15 MPa adopted for the very dense coral is not dissimilar to that which would be chosen from routine analysis, for example assuming dense sand. The difficulties arose because the piles were selected on the basis of their structural capacity on the assumption that effective refusal to further driving would occur in these soils.

- The measured efficiency of the drop hammer used was significantly less than might otherwise have been assumed from reference to piling handbooks. As such the use of the Hiley formula could have grossly over-estimated the mobilised pile capacity.