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Piling in Coral Sediments

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ABSTRACT The paper describes two projects for wharf piling in the Pacific Islands. The logistics of construction in these areas necessitates a reasonably accurate prediction of pile lengths, but in the reef flat deposits typically encountered, the presence of frequent coral fragments can significantly influence in situ test results and invalidate conventional pile length estimates.

The paper describes site investigation results, together with pile driving performance and pile test load results.

1 INTRODUCTION

Prediction of piling in coral lagoon reef flat deposits can be complicated by the presence of frequent coral fragments, which may be of sufficient size to influence results of conventional in situ tests in small diameter boreholes and give a false indication of overall density. Caution must therefore be used in adopting commonly used relationships between in situ tests and pile design parameters.

The aim of this paper is to provide documentation of two projects for wharf construction, in which there has been prior borehole and SPT data, comprehensive recording of pile driving data and, in each case, the results of test loading of two piles. Design parameters back-figured from this information will be of value in estimating future performance in similar situations.

A particular problem for construction of facilities in remote areas such as instanced here, is the pre-ordering of pile materials. Local stocks are not available and a shortfall in ordering could result in a delay in construction of up to several months - an expensive penalty once an expatriate contractor is established on site. Over supply is equally wasteful as there is little potential for utilising excess material on nearby forthcoming projects.

2 FUNAFUTI WHARF

2.1 General Description

A general purpose cargo wharf was proposed for Funafuti, the main atoll in Tuvalu (the former Ellice Islands). The wharf was to be constructed within the lagoon on the edge of the reef flat in up to 8 m of water. The use of driven steel tube piles was proposed, with unit load capacities of up to 500 kN.

2.2 Site Investigation

Site investigation (by Ground Test Pty. Ltd.) comprised five over water boreholes drilled with a light rotary drilling rig mounted on a pontoon barge. Bores were drilled by mud flush techniques and included regular standard penetration tests (taken with a trip hammer). They were taken to depths of up to 17 m below seabed.

Subsurface conditions comprised coralline sands and coral fragments for the full depth of investigation. The sands, slightly silty and of medium grain size, contained frequent coral fragments of up to cobble or boulder size. Standard penetration test results are plotted against depth in Fig.1. The differing symbols relate to the individual bores. Where tests reached practical refusal at less than full penetration, the results have been "extrapolated" and plotted with a maximum value of 100.

There was no obvious stratification and although there were differences between the SPT's in the boreholes, this appeared to be due to differing frequency with which SPT's intersected larger coral fragments.

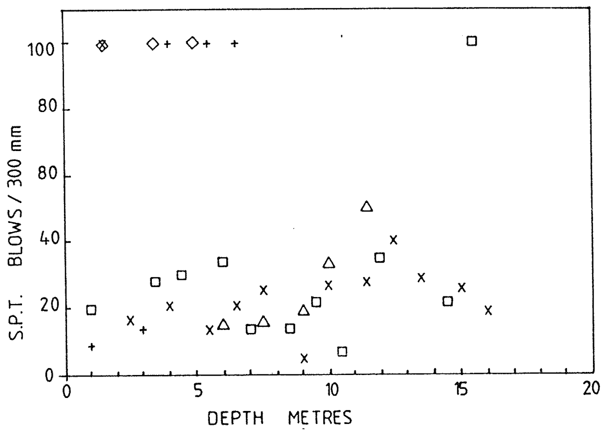

FIG.1 FUNAFUTI WHARF - SPT RESULTS

2.3 Pile Design

The wharf was designed to be supported on driven, closed ended, HP2 hollow segmented piles. These are octagonal steel sections which are rolled in sections and welded together on site to give a finished "diameter" of 433 mm with an average wall thickness of 16 mm.

In making estimates of pile length, it was considered that the use of conventional methods, would not be appropriate if used in conjunction with the actual SPT values. Some allowance would have to be made for the fact that at least some of the test results could be influenced by coral fragments.

Design by the Australian Piling Code (1978), assuming "loose" sands and a factor of safety of 2.5 on the working load of 500 kN, leads to an estimated length for HP2 sections of around 44 m. This method assumes a frictional force on the pile shaft determined from the overburden pressure at any depth, with a constant value below a depth of six pile diameters. End bearing resistance is calculated using an Nq (end bearing factor) value of 60. If "medium dense" sands are assumed (limiting depth 8 diameters Nq = 100) the depth estimate is about 16 m. Quite a difference!!! This design method appears very conservative in submerged soils, where overburden pressures are low and the limiting stress value which effectively governs the design is totally dependent on the fairly arbitrary estimates of the depth at which it applies.

The Meyerhof (1956) method provides for a shaft adhesion of twice the "N" value ("N" in blows/300 mm, Fs in kPa) and end bearing resistance of 400 times the "N" value. For the HP2 piles, the total ultimate load requirement of 1250 kN would be satisfied in end bearing alone with an "N" value of 21. This is about the average value measured on site.

After consideration of various estimates, it was decided to adopt a more conservative approach, to allow for the possible lower than apparent density conditions, and a depth requirement of 11 - 14 m was suggested. This, using the Meyerhof approach, is equivalent to assuming an average "N" value of 12 - 14, with an average shaft adhesion of about 25 kPa and an end bearing resistance of 5000 kPa.

2.4 Experience on Site

The wharf was constructed by Hornibrook Constructions Pty. Ltd. and piles were driven from floating plant using a 4 tonne drop hammer with drops ranging from 1.0 to 2.5 m. Driving resistance was determined on site using the Hiley formula, in conjunction with a factor of safety of 2. Temporary compression values were measured during driving and utilised in the Hiley formula to compute ultimate driving resistance. The pile driving resistances for four typical piles in terms of blows per 500 mm penetration are given in Fig.2.

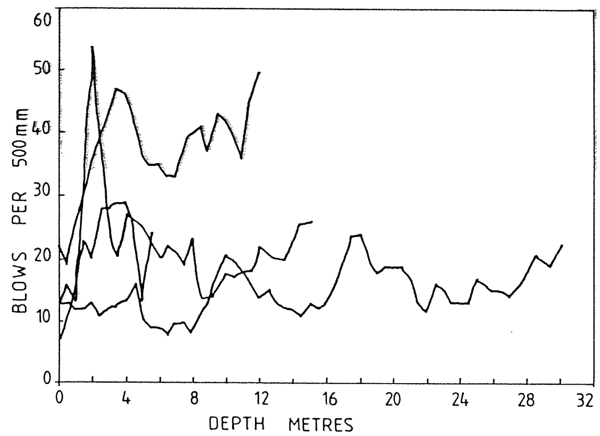

FIG.2 FUNAFUTI WHARF DRIVING RESISTANCE

For the 85 piles driven, the range of depths was from 6.5 to 30.3 m with an average length of 12.9 m. Final sets were in the range 3 - 19 mm.

The extreme range of driven depths was a surprising feature of the work. There was no apparent pattern to the range and adjacent piles sometimes varied in driven length by up to 15 m.

Two piles were tested under static load, the results being given in Figs. 3 and 4. Pile 48 was loaded to twice the working load of 400 kN which could not quite be maintained as settlement was in excess of 100 mm. Pile 66 was loaded to 1.5 times the working load of 400 kN and settlement was a maximum of 11 mm. Ultimate load capacity computed by the Hiley formula for these piles was 920 kN and 970 kN respectively.

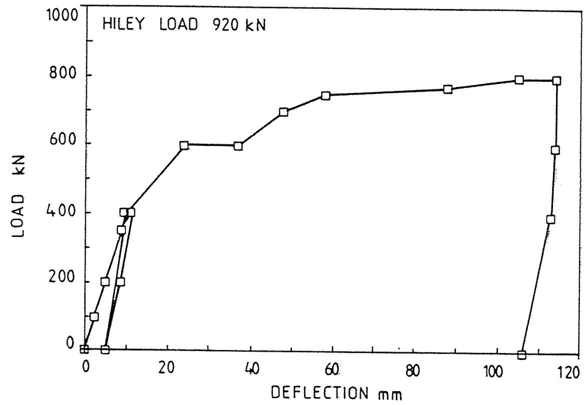

FIG. 3 FUNAFUTI WHARF PILE TEST 48

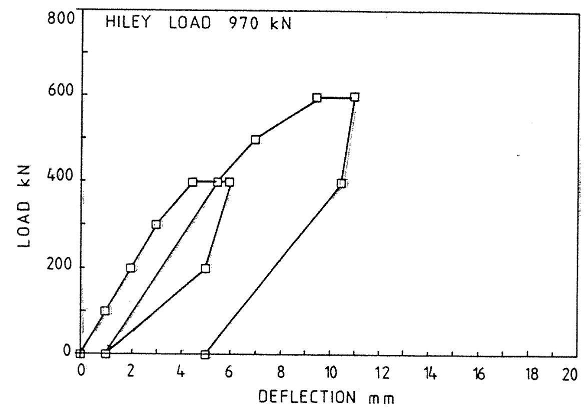

FIG. 4 FUNAFUTI WHARF PILE TEST 66

3 BETIO JETTY

A fisheries and general purpose cargo wharf was proposed for the Republic of Kiribati (on the atoll of Tarawa) at Betio. The wharf was proposed as an L-shaped structure on the lagoon side off the end of an existing breakwater, in about 6 m of water.

The site lies within a complex of coral islands of atoll formation, and subsurface conditions were known (from previous preliminary investigation) to comprise reef flat deposits of sands and coral fragments.

3.1 Site Investigation

Site investigation procedures were similar to those of Funafuti, with six boreholes drilled to depths of up to 30 m. Again, rotary mud flush techniques were used, with SPT's being taken at 1.2 - 1.5 metre intervals in depth. Core drilling was occasionally attempted in denser coral zones, but generally without significant recovery.

(A potential hazard on this site was the possible presence of unexploded bombs on the seabed. The site was the scene of a major encounter during World War II and the adjacent breakwater had been constructed from abandoned military hardware. A diver survey was carried out in advance of drilling and no hazards were encountered.)

Subsurface conditions were also similar to those at Funafuti with coralline sands for the full depth of drilling of 30 m. These were primarily sand size coral and shell fragments, with silty zones and larger coral fragments ranging upwards in size to cobbles, boulders and occasionally larger.

SPT results are plotted in Fig.5. As previously, where refusal was encountered at less than 450 mm penetration, test results have been "extrapolated", plotting with a maximum value of 100.

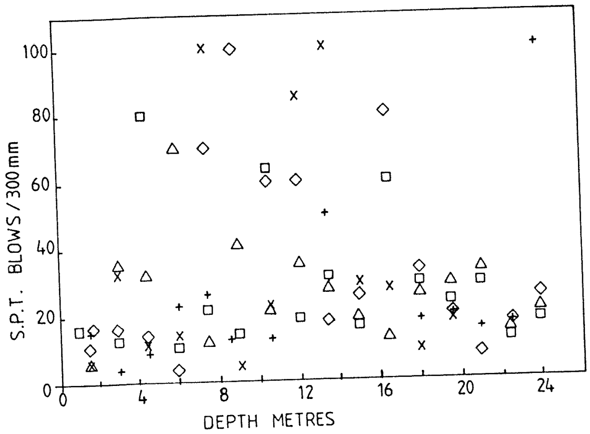

FIG. 5 BETIO JETTY SPT RESULTS

There appeared to be no obvious stratification with apparent random variation, again with the higher values probably being influenced by coral fragments.

3.2 Pile Design

The wharf was designed to be supported on driven, closed ended, steel tube piles, 457 mm diameter x 16 mm wall thickness. Unit working loads for most piles were 500 kN, ranging up to a maximum of 700 kN.

Previous experience at Funafuti confirmed the likelihood that conventional approaches for estimating pile depth would not be applicable. Densities appeared to be slightly higher than at Funafuti, and it was considered that slightly increased stresses could be allowed in making estimates of pile length.

It was considered that estimates of pile length (which would on site be determined from driving resistance and static load tests) should be based on the following ultimate stress values in conjunction with a factor of safety of 2.5.

Shaft adhesion	40 - 50 kPa
End bearing	4000 - 5000 kPa

For 457 mm diameter piles, this leads to a penetration requirement of around 6 - 10 m for 500 kN load and around 13 - 19 m for 700 kN load. Using the Meyerhof approach, this is equivalent to use of an "N" value of about 14 as compared to an average measured value in the depth range 5 - 15 m of 37.

3.3 Experience on Site

Piles were installed by Marples International Ltd. and the steel tubes were driven with a K22 diesel hammer. The driving resistance in terms of blows/500 mm for several typical piles is given in Fig.6.

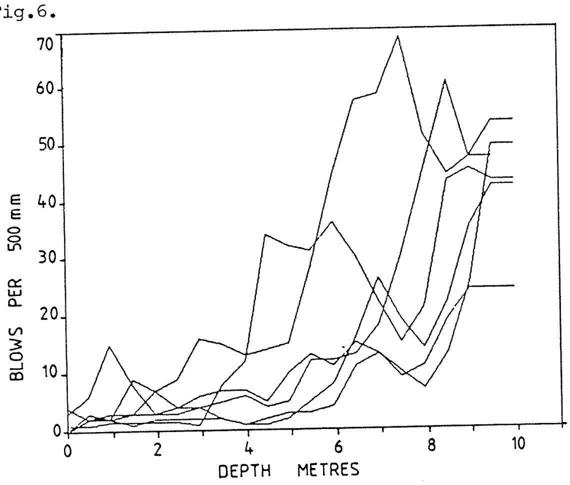

FIG. 6 BETIO JETTY DRIVING RESISTANCE

Pile capacities were again determined by use of the Hiley formula with a factor of safety of 2 and using temporary compression values measured on site. Temporary compressions were in the range 7 - 15 mm with most values in the range 8 - 12 mm. Final sets were in the range 3 - 11 mm with most values in the range 5 - 8 mm.

Driven pile lengths (measured below seabed) were generally around 10 - 11 m, but with occasional piles (mainly for the heavier loads) ranging in depth up to 18 m. Calculated values of ultimate resistance from the Hiley formula were generally in the range 1200 - 1600 kN.

Two piles were tested under static load. Pile B1, initially driven to a depth of 11 m and a set of 20 mm was tested to a maximum load of 900 kN, at which stage, it had suffered a displacement of 25 mm and was considered to have failed. It was re-driven to a depth of 12.5 m with a final set of 9 mm and again tested, this time reaching a load of 1100 kN. The calculated Hiley resistance for this pile was 1160 kN.

A second pile, D27, driven to a depth of 11.3 m with a final set of 5 mm was tested up to a maximum load of 1400 kN and a deflection of 35 mm. The calculated Hiley ultimate load was 1310 kN.

The load deflection curves for the test piles are given in Figs.7 and 8.

An interesting feature of the pile driving performance on this site is the relatively low resistance in the early stages of driving followed by fairly rapid build up of resistance in the depth range 6 - 10 m. This is not mirrored in the SPT results.

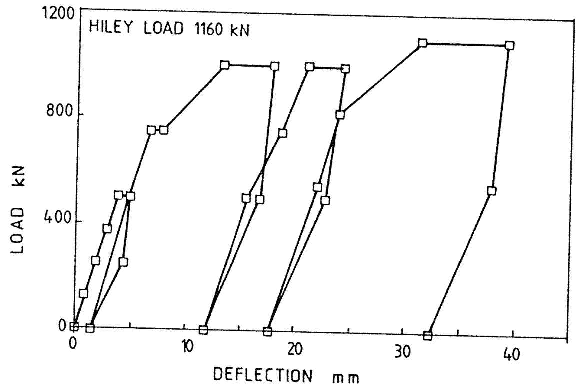

FIG. 7 BETIO JETTY PILE TEST B1

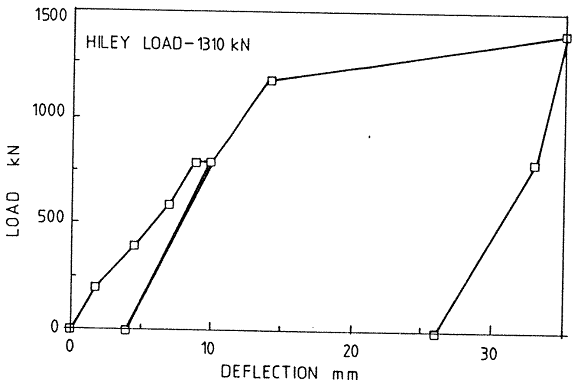

FIG. 8 BETIO JETTY PILE TEST D27

Comparison of static test load results with calculations based on the Hiley formula show surprisingly good agreement.

4 CONCLUSION

The initial assumption that pile driving performance could not be predicted by conventional design methods was borne out on site. Actual predictions which used a strong element of "judgement" to allow for the fact that SPT results were influenced by coral fragments agreed reasonably with performance on site.

Static pile load tests on one site showed good agreement with calculations using the Hiley formula which was used to control the piling work.

Similar conditions are likely to exist elsewhere in the Pacific Islands where there are similar modes of formation of the back reef deposits. The results presented here will serve as a guide for future estimation of piling performance.

5 ACKNOWLEDGEMENTS

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6 REFERENCES

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