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The Uplift Capacity of Steel Piles Driven Into Hawkesbury Sandstone

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SUMMARY An investigation is described in which two steel H-piles were driven in Sydney Harbour through harbour sediment into the sandstone bedrock and the uplift capacity of the "rock sockets" measured as the piles were withdrawn. An on-land driving test designed to confirm that steel piles can be driven significant distances into medium-strength (10-15 MPa unconfined compressive strength) Hawkesbury Sandstone is also reported.

The results indicate that, at the Sydney Harbour site, a substantial contribution to uplift resistance can be reliably obtained from the bedrock using heavy-driven solid section steel piles.

1 INTRODUCTION

When a piled wharf structure is not braceable against the shore, berthing and mooring forces are resisted by the weight of the structure, by raker piles if these are used, and possibly by the uplift resistance of vertical piles provided by soil and perhaps bedrock. In cases where large deck widths and deck weights are fixed by factors other than berthing and mooring forces, the piles may not be required to provide any uplift resistance at all. In general, however, all potential ways of providing such resistance need to be considered in optimising a design.

The contribution of soil strata to resisting upward and downward loading can be calculated by standard soil mechanics methods using site investigation data. If bedrock is overlaid by only several metres of soil the contribution of the soil in supporting downward loads is often small by comparison to that offered by the rock, and detailed soil investigation and analysis is seldom warranted. If uplift is involved, however, the soil is more important because only its contribution is relied upon; there is rarely any reliance placed upon the possible contribution from the rock. Where uplift resistance is demanded from the rock this may be achieved by driving large diameter steel tubes, chopping an inspectable socket, and concreting. Some socketting is, in any case, usually undertaken to seal the tube into rock so that the founding material can be inspected to establish its adequacy to support the downward load. Installation of this type of pile is slow and expensive relative to solid section steel piles driven to practical refusal in rock. The cost differential is due largely to the difference in design bearing pressures adopted for the two systems; 85 MPa for the solid driven pile, and a typical maximum of 3 MPa for the concrete pile. The rationale for the higher stress depends essentially on the driving being regarded as an effective proving test of each pile position. From experience and load tests it has been found that significant movement under a working stress of 85 MPa rarely occurs provided the pile has been driven to practical refusal in rock.

But when steel piles are driven through soils which overlie medium-strength rock (10-15 MPa unconfined compressive strength) it is generally assumed that they will stop at the rock surface rather than penetrate it by any significant amount. Consequently no uplift resistance is expected. If piles penetrate deeper than site investigation data predicts it is usually assumed that the medium-strength rock stratum is at a lower level than expected or that the pile is deflecting or damaged. The latter explanation is prompted by instances of damaged piles having been extracted. In extreme cases, heavy-driven H-piles have split along the web and the flanges have been bent back through one hundred and eighty degrees into a double hook configuration.

The pile tests described in this paper form part of a site investigation of the near offshore area of Garden Island, the site of a Naval establishment in Sydney Harbour. The Island was joined to the harbour shore by reclamation works carried out in 1945. The purpose of this investigation was to obtain sediment and bedrock data to assist in evaluating various options being considered in developing an overall modernisation plan for the Island. Wharf structures costing up to 25 million dollars were included amongst the options and piling work accounted for 6 million dollars of this amount. The estimate was based on the driven steel liner, rock socketted, concrete filled piles described above. An estimated saving of 0.8 million dollars was considered possible provided the site investigation could establish the suitability of solid-section driven steel piles for the site. This paper limits itself to considering the driving of steel piles into sandstone bedrock and the resistance to pile extraction provided by the rock; the investigation of the overlying several metres of sediment is not reported.

2 HAWKESBURY SANDSTONE

Cores were recovered from the bedrock at nine locations. Figure 1 shows the positions of all boreholes and the two overwater pile tests. The results of unconfined compressive strength tests on the cores are listed in Table 1. The average result for cores taken from the top 0.5m was 9.6 MPa. This figure overestimates the average rock strength however because acceptable test specimens could not be obtained from much of the weaker rock in this zone. The average results from 0.5-1.0 m,

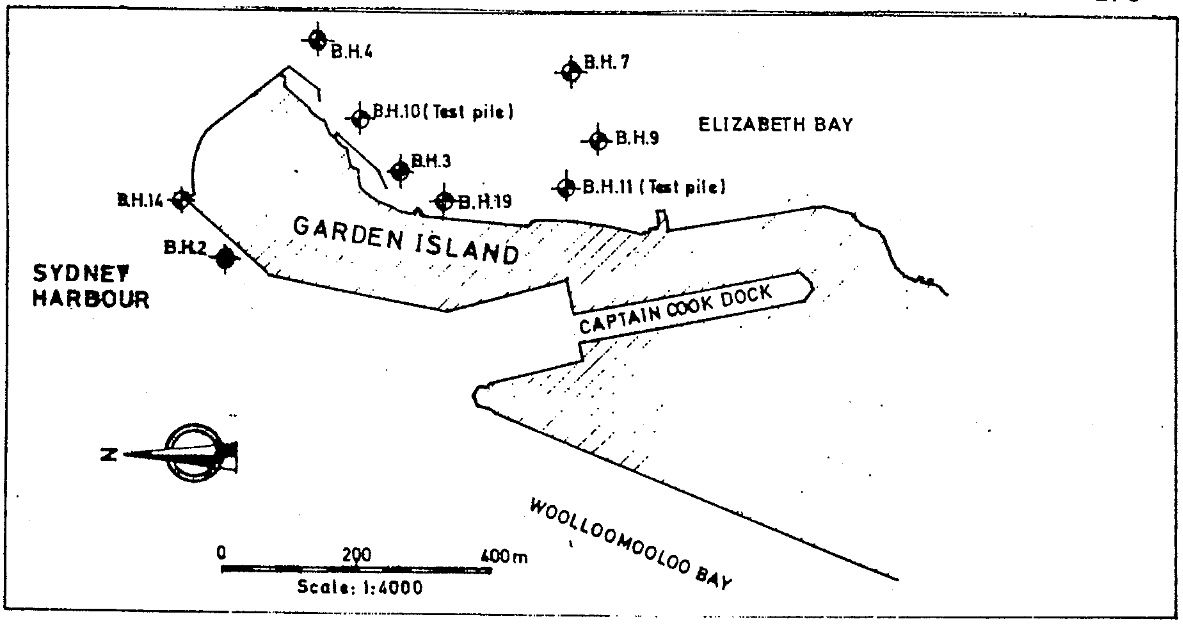

Fig. 1 Locations of boreholes and pile tests

from 1.0-2.0 m and from 2.0-3.0 m into the rock were 10.1, 13.8 and 14.3 MPa respectively. The few tests on deeper cores all gave results in excess of 20 MPa. A picture of the bedrock for the site was developed from these results together with records of past drilling within the subject area, a close-spaced seismic reflection survey, and the well established general characteristics of the Hawkesbury Sandstone geological formation.

The bedrock over the entire site is Triassic Hawkesbury Sandstone which extends for more than 200 metres below Garden Island. The formation consists of 95% lenticular sandstone but significant amounts of shale, shale breccia, clay and sandy claystone occur in the top 60 metres of the formation. The shale may occur finely laminated with fine sandstone in thin lenses but layers up to 30 metres thick have been encountered. Primary bedding within the formation is near horizontal but cross or current bedding and cut and fill structures are common and locally result in rapid vertical and lateral variations in rock type.

Present surface weathering is restricted to outcrops above sea level which are subjected to wetting, drying and oxidation. However, the bedrock adjacent to Garden Island was completely exposed during periods of lower sea level in the late Quaternary so relict weathering profiles are

TABLE 1

UNCONFINED COMPRESSIVE TEST RESULTS ON BEDROCK CORES - MEGAPASCALS

Borehole number	2	3	4	7	9	10	11	14	19
Distance into rock in m									
0									
		10.1			6.9		14.9	7.2	
	4.9	11.4					13.7		4.4
—						5.4			17.1
	11.5	12.6		5.4		10.7	9.3	8.3	
				7.3					14.6
1.0						5.1	16.3		
	22.5			6.4					15.2
		14.2	12.4		4.0		17.6	18.2	
—	11.9		17.7						11.0
				6.2		8.1		17.6	
	15.5				12.7				
2.0		18.8			22.6		14.4		22.1
				10.7				13.4	
			11.4		19.8				
—					11.8				
				16.0		9.4	20.9		
									23.5
3.0		15.3				6.2			

expected below the present sea level. Weathering within sandstone is not usually extensive and is typically limited to one metre, but shale lenses at or very near the bedrock surface may be completely weathered for 6 to 10 metres. Within the sandstone, joints are usually near vertical. Tertiary basaltic dykes have intruded the sandstone, typically through joints,and may be locally weathered to depths in excess of 30 metres.

The results from the drilling carried out for this investigation were in accord with the general features of the Hawkesbury Sandstone formation given above. The drilling indicated that the rock surface was weathered to depths of up to one metre throughout most of the area. Shale was intersected near the bedrock surface in only one location. In this borehole, 6 metres of weathered sandstone and shale were found to overlie solid shale.

Thus the situation was one of a generally very sound bedrock persisting to great depths but possibly containing serious localised weaknesses which occurred randomly at or near the bedrock surface. The implications of this situation for selection of pile type and investigation strategy will be discussed later.

3 OVERWATER PILE TESTS

The two tests were conducted at the positions of boreholes 10 and 11. At borehole 10 the sandstone was weathered, with no parts of the upper 0.5 metre of core testable. Unconfined compressive strength test results on the next metre of core were 5.4, 10.7, 5.1 and 8.1 MPa. Penetration of a driven pile into at least the upper half metre of the rock would normally be expected.

At borehole 11 there appeared to be no significant weathered rock transition. The unconfined compressive strength result at the bedrock surface was 14.9 MPa and test results for the next metre of core were 13.7, 9.3, 16.3 and 17.6. In this situation no significant penetration of a driven pile into the rock would usually be expected; the total uplift resistance of the pile would be attributed to the overlying soil.

Initially a 20 metre long 310 UC 158 steel H-pile was driven into the sediment to a point just short of the underlying bedrock surface at each location. A 3.3 tonne drop hammer falling through one metre was used. The mass of each pile was 3.2 tonnes. Soil profiles consisted of layers of sands and clayey sands of variable density and plasticity. After two days each pile was extracted, recording the forces required. The pile at Borehole 10, which was embedded to a depth of 5.2 metres, sustained a peak pullout load of 320 kN while the pile at Borehole 11, which was embedded to a depth of 7.7 metres, sustained a peak load of 290 kN. The difference between the average shear resistance measured in the two tests was consistent with the differences in soil profile at the two locations.

Each pile was then redriven at its same position and driving was continued to practical refusal using a hammer drop of 2 metres. For the pile at Borehole 10 this involved a penetration of approximately 1.4 metres beyond the point reached in the initial drive. From the borehole information and subsequent inspection of material adhering to the recovered pile it appeared that the upper half of the 1.4 metres was dense sand or highly weathered sandstone and the lower half was soft sandstone of approximate compressive strengh 5 MPa. The pile at Borehole 11 penetrated through a further metre of material on redriving and from the borehole information and subsequent inspection of material adhering to the recovered pile,it appeared to be 0.4 metres of sand overlying 0.6 metres of sandstone of approximate compressive strength 10 - 15 MPa. Redriving data together with borehole information and test results is given in Table II.

Neither pile could be extracted by a dynamometer - measured pulling force of 600 kN or by a force of 750 kN (calculated from submergence measurements of the ballasted pulling barge) even though jetting was used to lessen soil resistance. At this stage of the operation it was confidently expected that the high resistances would be seen, on extraction, to be due to severe distortion of the piles. The pile at Borehole 11 was finally removed using a heavy extracting vibrator in conjunction with a 750 kN pulling force. The remaining pile was removed by a 750 kN pull only after explosives had been used to loosen rock near the toe of the pile. Both piles were straight and completely undamaged apart from the distortion clearly attributable to the explosives.

Due to the methods necessary to finally extract the piles, the actual ultimate uplift resistance provided to each by the sandstone could not be calculated but a value in excess of 1000 kN is considered probable.

Although the material impacted between the flanges of the extracted piles appeared to be consistent with the investigation drill cores and the driving records, there remained some doubt as to the quality of the rock penetrated. The impacted rock had been altered by the pile driving so tests upon it could not confirm its in-situ compressive strength. Limitations on the accuracy with which the site investigation drilling barge and the pile driving barge could be positioned further compounded the uncertainty as to the strength of the penetrated rock. Consequently an on-land driving test was conducted at Balmain in similar rock using the same pile section, hammer and driving frame.

4 ON-LAND PILE TEST

A 3.4 metre long section of 310 UC 158 piling was driven through 1.2 metres of fill and a further 0.92 metres into bedrock. The rock was observable in cross-section in outcrop approximately 3 metres from the test position and appeared to be sound, uniform, seam-free sandstone of medium strength. Four cores were drilled immediately adjacent to and surrounding the test position. These were tested in unconfined compression. The test results for the upper one metre of sandstone ranged between 9.9 MPa and 16.3 MPa with an average value, from 15 tests, of 13 MPa. The test results together with driving resistances are presented in Table III. The pile was driven to practical refusal with an average set over the last 50 blows of 0.4 mm per 2 m hammer drop. Details of pile penetration over the final 75 mm are given in Table IV for both the on-land and offshore tests.

Excavation of the overlying fill revealed that the surface of the sandstone around the pile was free of any joint or defect that could have affected the test. Only two cracks emanated from the pile and both terminated within 200 mm of the pile. The apparent zone of influence of the pile upon the sandstone was very limited: the rock within approximately 10 mm of the pile had been pulverized but was very compact and there was no indica-

TABLE II

OVERWATER PILE DRIVING TESTS - GARDEN ISLAND

Depth below harbour floor in m	BOREHOLE 10		BOREHOLE 11	
	Driving resist. Blows x Drop (m) per 333mm set	Notes	Driving resist. Blows x Drop (m) per 333mm set	Notes
0				
		Loose sand Sand % 88		Loose sand Sand % 88
1				
		Soft clayey sand Sand % 65, P.I. 17		Stiff clayey sand Sand % 75, P.I. 23
	5 x 0.3			
2				
	4 x 1			S.P.T. 10
	6 x 1	S.P.T. 10 Clayey sand		Medium density sand
	6 x 1	S.P.T. 8	2 x 0.3	
3				
	8 x 1		4 x 1	
	8 x 1	Clayey sand Sand % 65, P.I. 20	5 x 1	Medium density sand Sand % 85
	11 x 1	S.P.T. 12	7 x 1	
4				
	10 x 1		7 x 1	S.P.T. 27
	15 x 1	Dense sand Sand % 70, P.I. 20	16 x 1	Dense clayey sand
	15 x 1	S.P.T. 15	15 x 1	
5				
	15 x 1	Limit of initial drive	15 x 1	
	50 x 1	Dense sand Sand % 70 S.P.T. 40	10 x 1	
	50 x 1 + 32 x 2		10 x 1	Sand % 65 S.P.T. 8
6				
	60 x 2	Auger refusal-sandstone	15 x 1	
	130 x 2	Driving Refusal U.C.S. 5.4 MPa U.C.S. 10.7 MPa	15 x 1 15 x 1	Medium density clayey sand
7				
		U.C.S. 5.1 MPa	15 x 1	Sand % 75, P.I. 16 Dense sand
			15 x 1	
				Limit of initial drive
8		U.C.S. 8.1 MPa	20 x 1	Auger refusal-sandstone U.C.S. 14.9 MPa U.C.S. 13.7 MPa U.C.S. 9.3 MPa U.C.S. 16.3 MPa
			20 x 1	
		U.C.S. 9.4 MPa	35 x 1 + 60 x 2 180 x 2 for 100mm	Driving refusal U.C.S. 17.6 MPa
9				

tion of rock distortion beyond this zone. Heavy pulling gear was not available so rock had to be excavated from around the pile before it could be extracted. The pile was essentially undamaged with one leaf of one flange bent slightly from its plane at the pile toe.

5 DISCUSSION

Hawkesbury sandstone presents the foundation designer with a frustrating problem in that it has a very high load capacity almost generally but the presence of some localised weaknesses may preclude him from depending upon the high capacity at any particular position. A similar situation exists with respect to the uplift capacity of driven piles. Even when a site investigation indicates that a penetrable rock zone is present over most of the area, the possibility of the bedrock surface being harder and impenetrable at some spots may preclude the designer from depending upon uplift resistance at all.

TABLE III

ON-LAND PILE DRIVING TEST-BALMAIN

Depth Below Ground Level in m	Test Results Unconfined Comp. Strength in MPa	Driving Resistance Number of Blows x Hammer drop (m) per 100 mm Set	Notes
			Loose fill
		15 x 0.3 total	
1.2			Top of sandstone
	15.7	12 x 1	
		20 x 1	
1.4	15.5		
	15.1 11.7	31 x 1	
	10.4		
		31 x 1	
1.6	13.7		
	16.3	5 x 1 + 20 x 2	
	10.0		
	12.4	40 x 2	
1.8	15.4		
	14.2	40 x 2	
		50 x 2	
2.0	11.0 11.1		
	9.9	100 x 2	
	13.3	55 x 2 for 20 mm	2.12 m Refusal
2.2	15.3		

TABLE IV

TERMINAL DRIVING RESISTANCES OF TEST PILES

	Pile		
	On-Land Balmain	BH 10 Garden Island	BH 11 Garden Island
No. of blows* per 25 mm set over the last 75 mm	23, 35, 55	7, 13, 50	20, 25, 35
Distance driven beyond nominal refusal**	162 mm	15 mm	195 mm
No. of blows after nominal refusal	170	40	180
Average set over last 50 blows	0.4 mm	0.5 mm	0.9 mm

* A blow = a 3.3 tonne hammer falling through 2 m

** "Nominal Refusal" is assumed to correspond to a set of less than 25 mm per 10 blows over 20 blows.

This problem of variation across the site has been compounded in wharf construction by the increasing use of single high capacity piles because the associated increased pile spacings reduce the extent to which loads can be redistributed from a defective pile through the deck structure to adjacent piles. Consequently it becomes more important to ensure that each and every pile is capable of resisting the forces for which it has been designed. However, the uplift resistance of individual piles is less critical than their performance under downward loading. This is because the capacity of deck structures to redistribute load differs significantly for upward and downward loading: mooring and berthing forces are sub-horizontal. The deck, too, is horizontal and consequently stiff in this plane so considerable spreading of the uplift-producing forces occurs. This is not the case with the large concentrated forces which result from cranes and other live loads.

For the Garden Island site, and for others where localised variations in rock quality are expected, the selection of a piling system is strongly influenced by the extent to which the installation process supplements the site investigation by proving the founding material at each pile position. Large diameter driven steel tubes socketted into the rock and filled with concrete fulfill this function well but installation is slow and expensive. In addition there are significant practical problems associated with arranging down-the-hole inspections and drilling of proving holes ahead of the founding depth for every socket, especially when there is persistent water ingress into the socket. Delays to the construction programme are not infrequent.

The driving tests showed the heavy section steel piles can be driven into 10 - 15 MPa Hawkesbury sandstone. This information, taken together with the bedrock picture built up by the site investigation, indicates that a driven pile system would provide the designer with an assurance of the capability of individual piles to support downward loads that is comparable to that provided by the dearer, slower system described above.

The uplift tests are open to some interpretation. The ultimate loads sustained by the piles can be only estimated because of the methods finally needed to extract them. Also some assumption needs to be made of the effectiveness of jetting in reducing the soil restraint. However, the most conservative interpretation suggests an ultimate average skin friction in the rock of at least 230 kPa. It appeared that the piles could be driven at least 0.5 metres into the sandstone over the subject area. Based on these figures the uplift resistance of the rock would add 50% to that contributed by the average 7 metres of sand and clayey sand sediments which overlie the rock. Final design figures would need to be chosen having regard to the capacity of the particular wharf structure to distribute berthing and mooring forces amongst the piles. The amount of the soil's contribution would also be an influencing factor.

For the 310 UC 158 piles used in the tests it is considered that the rock uplift resistances would be realized with acceptable reliability when the piles are driven to practical refusal using a hammer of similar mass to the pile and a hammer drop of 2m. In this context practical refusal is considered to correspond to either:

(i) a set of less than 10 mm per ten blows over the last thirty blows;

OR

(ii) a penetration of 150 mm beyond nominal refusal where nominal refusal is defined as a set of less than 25 mm per ten blows over the last twenty blows.

Consideration would need to be given to the driving energy required to achieve adequate penetration of other pile sections and to the possibility that the required energy may be such as to cause damage to the piles.

Little generality should be assumed for the above conclusions. Extrapolation of the results beyond the investigated site and to other rock types could not be undertaken without specific corroborative investigations.

6 CONCLUSIONS

The investigation showed that heavy section steel H-piles could be driven up to 1 metre into medium strength (10 - 15 MPa unconfined compressiv strength) Hawkesbury sandstone and that, at the Sydney Harbour site, substantial uplift resistance could be achieved with an acceptable degree of reliability by driving piles to practical refusal in the bedrock.

With respect to downward loading it is considered that, for this particular site, where the major risk is the presence of localised weaknesses in an otherwise strong rock, heavy driven solid section piles provide an assurance of individual pile performance comparable to that produced by the slower, far more expensive system of driving steel casings to rock, chopping an inspectable socket, drilling an inspection hole ahead of the founding depth, then concreting.

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