# The Testing of Large Diameter Pile Rock Sockets with a Retrievable Test Rig

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SUMMARY This paper describes the design, construction and use of a retrievable test rig for evaluating the performance characteristics of large diameter pile rock sockets. The test rig is prefabricated at ground level, lowered into a contract socket, and concreted into position. Large flat jacks react against the base of the socket to force a concrete annulus, representing a section of the pile shaft, upwards eventually to failure. The side shear resistance characteristics to failure and the base resistance characteristics well in excess of design load may be determined. After testing, by use of special design features, the rig and concrete may be rapidly removed from the socket to allow the contract pile to be completed. The results of two such tests conducted in a contract socket of a major construction project in Melbourne are reported to demonstrate the effectiveness of the technique.

## 1 INTRODUCTION

The Rail Overpass from Flinders Street Station to Spencer Street Station of the Melbourne Underground Rail Loop Project included the construction of 105 bored piles of which 85 were to be socketed into the Silurian Mudstone underlying the poor quality near surface soils. These piles were generally of 1000 mm and 1250 mm nominal diameter to carry working loads of between about 3.5 MN and 6.0 MN. The depths to the upper surface of the mudstone varied from about 2 metres to 35 metres from ground level with socket lengths of between about 5 and 15 metres.

During December 1975, the Department of Civil Engineering at Monash University was approached by John Connell-Mott, Hay & Anderson, Hatch, Jacobs, Principal Consultants for the project and entered into discussions concerning the possibility of developing and conducting a limited test programme to verify the design adequacy of, and to provide further performance information on the pile sockets to be formed the following year.

However, from the inception of the test programme, it was recognised that a number of constraints had to be imposed on the development of the programme, the most important of which follow :-

(a) The tests should take place in one of the contract sockets at a considerable depth below ground level.

(b) The tests were to take place during the execution of the piling for the contract and therefore had to be designed to cause a minimum of interference to the contract.

(c) On completion of the test programme, the pile socket was to be restored and handed back to the contractor for completion as a contract pile.

(d) It was desirable to conduct the tests to well in excess of working loads, preferably to failure.

(e) Although a number of fully instrumented pile loading tests have been reported in the literature, it was the authors' opinion that information from such tests was subject to a variety of problems including interpretation, time delays, and instrument reliability. Therefore it was considered that the tests should be as definitive as possible.

## 2 PRINCIPLE OF TESTING TECHNIQUE

During the development stages, a great number of possible solutions were examined but the overriding constraint concerned the need to minimise delays to the contract. It became evident that the technique should involve the minimum possible operations in the socket to set up the test. This could be best achieved by prefabricating the test rig, including all loading devices away from the contract area. When the socket became available, the test rig could be located in the socket, the test conducted and the test rig immediately removed. The general principle of testing is illustrated in Figure 1.

A base plate was located immediately beneath three 920 mm diameter flat jacks, each of about 8600 kN capacity. The reaction frame, resting on top of the flat jacks had two purposes. Firstly it provided a space between the base of the socket and the socket wall test section so as to remove the likelihood of significant interaction effects. Secondly it provided a location for a large water pump which was considered necessary to remove the quantities of ground water likely to seep into the socket. Located on top of the reaction frame was the section of the test which was to be concreted against the socket wall to represent a section of pile shaft. However, had this cylinder of about 1250 mm diameter consisted of in-situ concrete, considerable difficulties in both time and effort would have been experienced in its removal after testing. Therefore, the technique adopted was to centrally locate an inverted truncated cone, or bucket, on top of the reaction frame. The bucket was made of thin steel plate into which concrete was cast with sufficient ducts to allow the passage of various hydraulic and pump connections.

At the base of the bucket was a 220 mm diameter flat jack of 400 kN capacity. The purpose of this flat jack was to release the bucket after testing from the thin concrete annulus cast around the bucket to form the side shear resistance section. Once the bucket was released, the thin concrete annulus

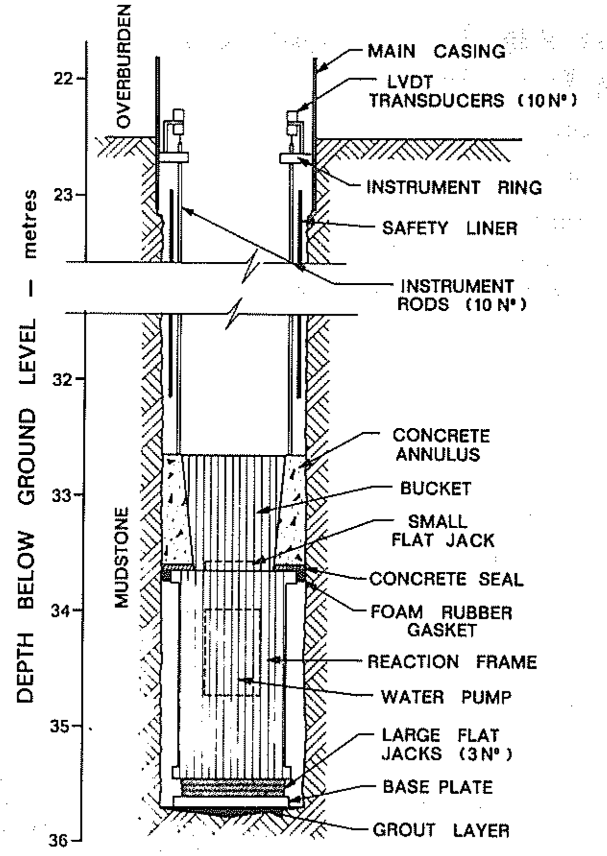


Figure 1 Diagrammatic principle of testing technique

could be easily broken out and removed to allow the remainder of the test rig to be lifted from the socket.

Since on inflating the large flat jacks it was difficult to ensure that both base and side shear resistance became fully mobilised, the dimensions of the side shear section were fixed at one metre length so that this section could be failed with the base pressure reaching about three times the design stress.

The vertical deflections at the base of the socket were to be measured at two diametral points. The deflections of the side shear section were measured at four points each at the bottom and top surfaces. It was considered that LVDT transducers suitably waterproofed could relay the measurements to the surface, but if they were attached to the rig itself, the socket wall or the safety liner, they would be susceptible to movement. The nearest rigid object was considered to be the main pile casing which was sealed into the top of the rock socket several metres above the test location. Therefore, the solution adopted consisted of transmitting the deflections from the ten points of measurement to the level of the bottom end of the pile casing by means of steel rods. These ten rods extended through the instrument ring which was rigidly bolted onto the pile casing. The transducers were mounted on the instrument ring. The instrument ring had a 800 mm internal diameter to allow the passage of a mancage for operations required in the socket itself.

Two tests were conducted in one socket; the first with the test rig located at the base of the socket and the second some four metres above the first location by making use of a four metre long concrete spacing column placed at the base of the socket.

Full details of the design and construction of the rig may be found in Johnston and Donald (1979).

## 3 TEST PROCEDURE

Site operations commenced with a "windowed" safety liner in the socket for a detailed inspection and sampling of the rock socket surface. A water pump was then removed to allow the placement of a thin bedding layer of grout at the base of the socket. The safety liner was removed and the test rig lowered to the base of the socket (Figure 2). This was followed by the lowering of a second shorter safety liner so that its lower end was about 0.5 metre above the test rig and was secured by slings to the top of the pile casing. Although there were no rock falls from the sides of the socket, the safety liners were used at all times.

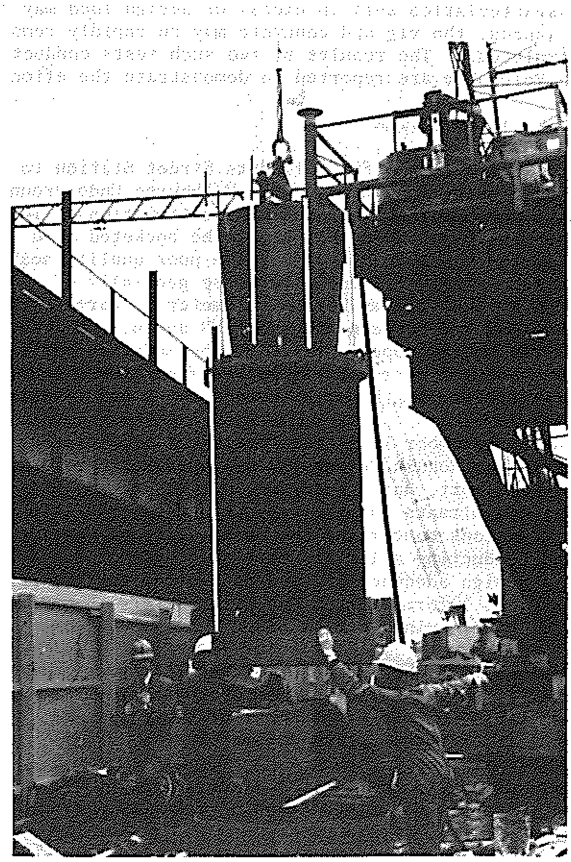

Figure 2 Lowering of test rig into the socket.

The top plate of the reaction frame was 1100 mm diameter and the socket was nominally 1250 mm diameter. This left a gap of approximately 75 mm which was plugged with strips of high density foam rubber. Immediately after placing this gasket, a 50 mm layer of rapid setting concrete was placed over the visible portion of the top plate of the reaction frame to complete the seal. Once set, the remainder of the concrete annulus was placed and vibrated. The instrument ring was then positioned, the instrument rods placed, the hydraulic connections made, and finally the transducers were installed and adjusted.

Once all these operations were completed, the mancage and "down-hole" operator were lifted from the pile casing and the test loading was ready to commence. The pressurising equipment of electric and manual pumps, and the recording equipment consisting of digital data logger, chart recorders and pressure gauges were all located at the ground surface.

The test itself was conducted by pressurising the large flat jacks and recording the transducer outputs. The load was applied in increments, with some load cycling, until excessive creep movement in the side shear section occurred, making it impossible to maintain a constant pressure in the flat jacks.

Immediately after the test, the rig was removed from the socket by a reverse procedure to installation. The main variation concerned the removal of the concrete annulus. Once the instrumentation was removed, the small flat jack at the base of the bucket was inflated to release the bucket which was then lifted clear. The concrete annulus could then be broken out before lifting the rest of the rig from the socket.

The total duration of each test from initial inspection to final removal of the rig was 11 and 8 working days respectively for tests 1 and 2.

## 4 TEST RESULTS

Two tests were conducted in the socket of pile No. 7A, immediately south-east of the Flinders St - Spencer St. intersection. A much simplified geological profile at that location consisted of some fill, soft clays and silts overlying stiff and very stiff clays and silts to a depth of about 22.5 metres below ground level. Below this depth, the Silurian Mudstone was encountered and was generally found to be moderately weathered with clean tight jointing of about 60 mm spacing and a dip of approximately 80°.

Based on borehole data and on socket inspections, a general description of the test locations, classified according to the zone gradings defined by Neilson (1970) is as follows :-

(a) Base of the socket.
This was located at a depth of 35.7 metres and was immediately underlain by medium grey mudstone of Zone 3 weathering. At a depth of about 36.0 metres, the mudstone appeared to become more fractured increasing its weathering designation to Zone 2. At about 36.4 metres the mudstone appeared to be of Zone 1 until about 36.7 metres. At this depth the mudstone quality improved to Zone 3, becoming Zone 3-4 at about 36.9 metres to at least about 39.0 metres.

(b) Socket wall - Test 1.
The concrete test section was cast against the socket wall between depths of 32.7 and 33.7 metres. The rock at this location was generally a medium grey mudstone of Zone 4 weathering. There appeared to be insignificant softening of the socket wall during the period between the drilling process and immediately prior to casting the test annulus.

Samples recovered from this location yielded an average moisture content of 9.3%, which corresponded to a compressive strength ($\sigma_1$-$\sigma_3$) of about 5600 kPa for cores tested under a confining pressure of 690 kPa. Measurements of the roughness of the socket wall at this location indicated that the surface was undulating with asperity heights of between about 5 and 10 mm at wavelengths of about 100 mm.

(c) Socket wall - Test 2
The second test location was between 28.7 and 29.7 metres depth. The mudstone was generally dark grey and pyritic. Weathering was such that at 28.7 metres, the mudstone was of Zone 2, with a clay Zone 1 fault between 28.73 and 28.83 metres. Below this level, the mudstone was generally Zone 3 to 29.7 metres with a thin layer of Zone 2 mudstone between 29.31 and 29.56 metres. Samples recovered indicated an average moisture content of about 11.7% which corresponded to a confined compressive strength ($\sigma_1$-$\sigma_3$) of about 3600 kPa. The roughness of the socket at this location was similar to the location of test 1.

The average load-displacement curves for side shear resistance and for base resistance are shown in Figures 3 and 4 respectively.

## 5 DISCUSSION OF TEST RESULTS

### 5.1 Side shear resistance - Test 1.

Figure 3 shows that the initial portion of the load-displacement curve is very steep with negligible displacement at loads up to about 500 kN. The average displacement of the annulus at the design load of 785 kN (200 kPa) was about 0.3 mm, and at three times design load, it was about 2 mm. The socket section exhibited work strengthening behaviour without actually reaching a peak load at an average displacement of 12.5 mm. However the curve is relatively flat at displacements beyond about 10 mm, at which the load, 4144 kN, may be taken as the failure load. This load corresponds to a maximum side shear resistance of about 1050 kPa.

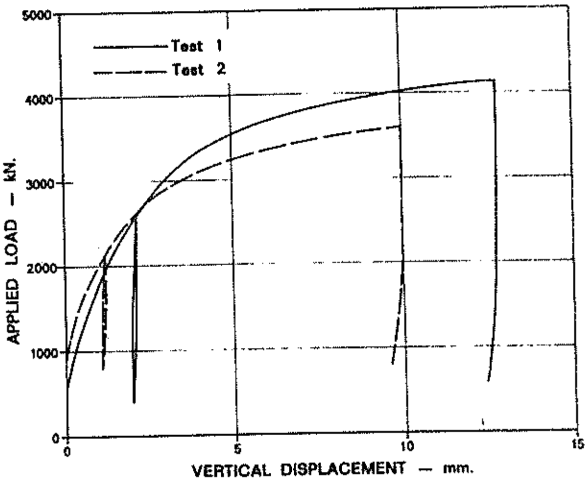


Figure 3 Side shear resistance load-displacement curves.

In conventional design, the saturated mudstone is treated as a $\phi$ = 0 material giving an undrained "cohesion" of 2800 kPa. Therefore, the side shear resistance reduction factor, $\alpha_c$, is given by:

$$\alpha_c = \frac{\text{shear stress at failure}}{\text{undrained "cohesion"}} = \frac{1050}{2800} = 0.37$$

### 5.2 Side shear resistance - Test 2

Figure 3 also shows that the average load-displacement curve for Test 2 is similar to Test 1, except that the initial behaviour is slightly stiffer. The displacement at the design load of 393 kN (100 kPa) was too small to be measured, and even at three times design load, it was only 0.4 mm. Using 10 mm displacement as a criterion, the failure load for Test 2 was 3600 kN, equivalent to a side shear stress of 943 kPa. As before, the estimated undrained "cohesion" of the rock would be on average about 1800 kPa.

$$\therefore \quad \alpha_c = \frac{943}{1800} = 0.52$$

The unload-reload curves for both tests show very stiff behaviour and little hysteresis, and creep behaviour was insignificant at stress levels below failure. It can be concluded that a socket in this material would not exhibit work softening behaviour at acceptable levels of deformation, and that the component of deformation due to live load would decrease markedly after the first full loading.

5.3 Base resistance - Test 1

Figure 4 shows the average load-displacement curves for the base. However, for this component of the test the grout layer at the base of the socket did not achieve the desired rigidity. Consequently, this layer was relatively compressible and its decrease in thickness under load represented at least 70% of the total measured base displacement of 19.9 mm. The value of 70% was estimated from the indentation on a thin base tray (Figure 2), originally incorporated in the design as a protection against grout penetration of the flat jacks when the rig was to be seated initially on wet grout.

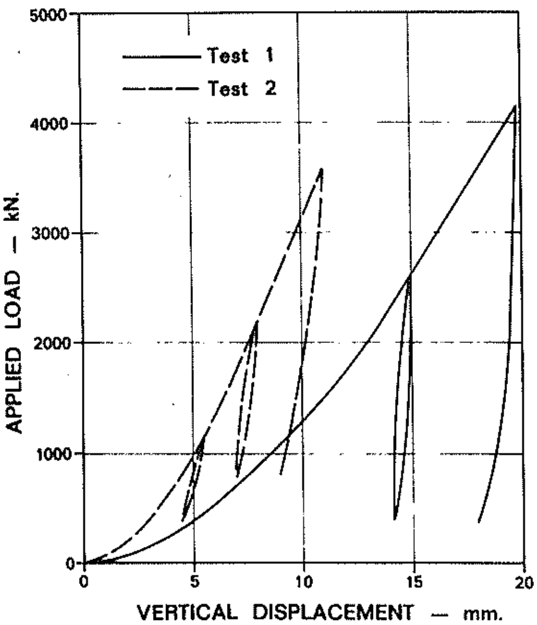


Figure 4 Base resistance load-displacement curves.

The average maximum normal stress beneath the base plate was 4360 kPa, which was 2.73 times the design stress of 1600 kPa. The load-displacement curve is quite linear over its final section, and the base behaviour was still within the elastic range. It is impossible to predict from this curve what the ultimate base failure load would have been, but it can be stated with confidence that the factor of safety against end bearing failure would be significantly higher than 2.73.

5.4 Base resistance - Test 2

The reinforced concrete spacer column used for this test was placed in the depression in the grout caused during Test 1. The load displacement curve shown in Figure 4 does not show the large irrecoverable grout compressions of Test 1. However, based on the shape of this curve it is likely that some bedding compression was still present. The average stress under the base plate, at maximum load, was 3750 kPa at 11 mm total displacement. This was equivalent to a safety factor of 2.34 on the design stress. As before, the base behaviour is still essentially elastic and there is no indication of incipient failure.

6 CONCLUSIONS

In this paper, the design, construction and use of a retrievable test rig for the determination of the performance characteristics of large diameter rock socketed piles has been described.

The test rig may be used to evaluate load-displacement curves for side shear resistance to failure and for base resistance to at least twice the design load. The test rig has the advantages of allowing a complete test to be made in a matter of days, without significant delay to a contact, at minimal disturbance to contractors, in a contract socket and without the need for costly external loading systems.

The results obtained from the two tests showed that the design of the rock socketed piles was adequate with side shear resistance reduction factors, $\alpha_c$, of 0.37 and 0.52 for failure of the mudstone of confined triaxial shear strengths ($\sigma_1$-$\sigma_3$ at $\sigma_3$ = 690 kPa) of 5600 kPa and 3600 kPa respectively.


7 ACKNOWLEDGEMENTS

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The work described in this paper forms part of a general programme of research into the performance of rock socketed piles under the direction of the first two authors.


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