Fourth Australia - New Zealand
Conference on Geomechanics,
Perth, 14-18 May 1984

Monitoring of the Bowen Bridge Foundations

B.C. CHANDLER
Associate, Maunsell & Partners Pty. Ltd., Melbourne
and
R.J. HARVEY
Senior Geotechnical Engineer, Maunsell & Partners Pty. Ltd., Melbourne

SUMMARY Bowen Bridge is the second road crossing of the Derwent River in Hobart. In addition to superstructure loading the bridge piers have been designed to withstand impact from 5000 tonne barges, this design feature being introduced principally as a result of the Tasman Bridge disaster in January, 1975. The bridge is supported on 13.3m diameter caissons, extending to 47m below river level to found on rock. This paper presents details of site stratigraphy, design and general arrangement of the foundations. Monitoring of two aspects of rock performance during construction is included in the paper; water pressure relief for cofferdam dewatering and rock mass stiffness for comparison with design assumptions. Water pressure relief in the rock at and below foundation level was required to permit construction of the caisson walls in the dry inside cofferdams. Pressure relief holes were drilled in the rock and piezometers installed to monitor pressures during and after dewatering. Rock mass stiffness is a significant parameter in assessing the response of the foundation to superstructure and ship impact loadings. A programme of rock coring and pressuremeter testing provided estimates of rock stiffness for design purposes. These estimates are compared with data from direct reading multi-rod extensometers installed up to 60m below river level. Details of the extensometer installations, together with assessments of rock mass stiffness from pressuremeter and extensometer records are presented.

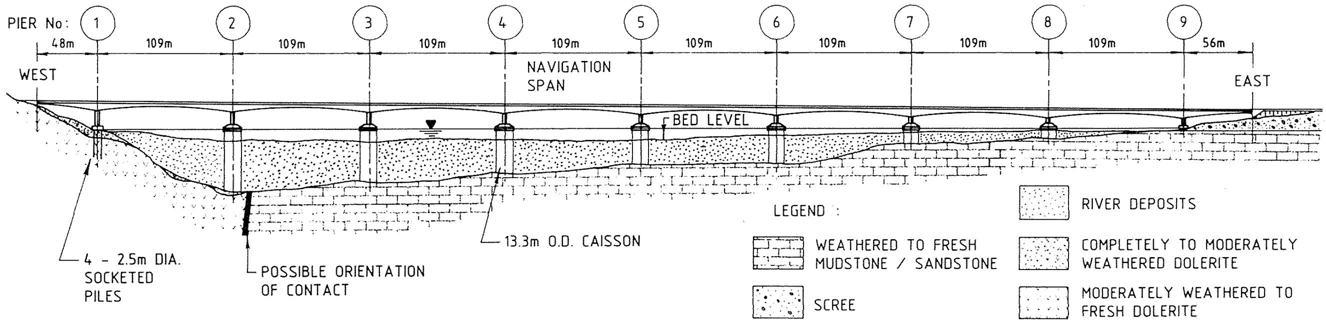

Figure 1 Elevation of bridge and stratigraphy

1. INTRODUCTION

Investigations for a second crossing of the Derwent River in Hobart commenced in 1975 following the partial collapse of the Tasman Bridge which was struck by an ore carrier in January of that year. The Tasman Bridge, although fully restored by 1977, is vulnerable to further ship collisions and to avoid a repeat of the disruption caused by the loss of the bridge a second bridge was approved. Work on the second crossing, Bowen Bridge, was directed by a Joint Committee [1] established to act on behalf of the Commonwealth and Tasmanian Governments. Bowen Bridge was designed to withstand collisions from 5000 tonne barges and have the capability of being upgraded to withstand collisions from barges of up to 10000 tonne. After extensive investigations of various bridge sites and types of structure, a site was selected off Dowsings Point and the final structural arrangement comprised large 13.3m diameter caisson foundations for 7 river piers and a twin cell continuous superstructure having a total length of 976m, [2].

Deep deposits of soft river sediments overly rock along most of the Derwent River. Rock level at the river piers varied between approximately RL-10 and RL-47m. To permit founding of the caissons on the rock and allow construction of the caisson walls to proceed from the bottom up in the dry, it was necessary to install and dewater a cofferdam at each river pier position. A programme for foundation monitoring during construction was established with two principal objectives:

(i) to observe the water pressures beneath the cofferdams and guard against uplift conditions during construction, and

(ii) to accurately record the response of the rock under the application of load throughout construction to confirm design assumptions and provide data for future upgrading of the foundations.

An elevation of the bridge and the underlying stratigraphy is shown in Figure 1.

2. SITE INVESTIGATIONS AND STRATIGRAPHY

Many sites were investigated in the initial planning stages for the second crossing. Geological mapping and seismic surveys were conducted and correlated with known borehole data [3]. When the preferred site was established a Phase 1 borehole investigation was carried out [4]. The investigation was aimed at detailing the stratigraphy and obtaining sufficient data on material properties to select pier positions and proceed with detailed design. In parallel with the detailed design, a Phase 2 borehole investigation [5], involving up to four holes per pier, was undertaken to establish rock levels and rock quality at each pier position and provide sufficient data for tendering the works, and to allow design to be finalised. In addition to these investigations further boreholes were drilled and pressuremeter tests carried out during construction to assist in establishing founding levels. Piezometers and extensometers were installed in these boreholes as part of the foundation monitoring programme.

A total of 56 boreholes were drilled at the site for the Phase 1 and 2 investigations and a further 25 boreholes were drilled and logged during construction. In all, for this project approximately 340 holes and probes (including pressures relief holes) were drilled into the rock at the bridge site.

Very soft organic clayey silts dominate the river deposits. Some interbedded layers of medium dense silty sands and gravels exist in the lower older series of river deposits at depth. However, at the deepest section of river deposits the clayey silt extends the full depth to rock at which level it is of a firm consistency. Testing revealed the clayey silt to be a normally consolidated highly compressible OH material. Measured moisture contents ranged from about 90% to 200% and the average liquid limit was about 180 with a corresponding plasticity index of 130. Vane shear strengths of the clayey silt increase linearly from about 3 kPa at bed level to 30 to 40 kPa below a depth of 15m. Measurements of the modulus of the clayey silt, from both undrained triaxial tests and oedometer tests, were obtained for calculating the response of the caisson under ship impact. Standard penetration tests in the interbedded silty sands and gravels at depth gave N values generally in the range of 15 to 30.

Jurassic dolerite and Triassic mudstone rock formations underlie the river deposits and the contact between the two rock types is coincident with the deepest section of river deposits, Figure 1. The dolerite is fine to medium grained and exhibits variable weathering over short distances from fresh to highly weathered, with moderate to high fracturing and irregular sub-vertical and sub-horizontal in-filled joints. The dolerite beneath the river deposits forms a steep deeply weathered scarp, dipping at an average slope of 25°. The unconfined compressive strength of the fresh intact dolerite, determined from drill core samples, exceeded 100MPa. As expected, the compressive strength was greatly reduced by the presence of joints in the core samples. Closed, tight joints yielded strengths of 25 to 40 MPa.

The mudstone varies in grain size from a claystone to a fine to medium grained quartz sandstone and is predominantly fresh to moderately weathered. Some highly to completely weathered layers in the form of stiff to hard soils exist near the surface of the formation. Bedding planes are frequent, generally tight clean and sub-horizontal. The unconfined compressive strength of the fresh to slightly weathered mudstone, unaffected by visible joints, was determined to be in the range of 15 to 30 MPa while tests on tightly jointed samples gave strengths as low as 3 MPa. No relationship could be established between insitu moisture content and strength.

The orientation of the contact between the dolerite and mudstone was inferred from seismic surveys and boreholes to strike approximately parallel to the river and dip steeply. Highly fractured indurated mudstone, believed to be a result of the thermal influence of the dolerite, extends for up to 20m beyond the contact.

3. FOUNDATIONS

3.1 General

Ship impact was a dominant aspect of the river pier design and, accordingly, the caissons were required to function entirely as embedded gravity structures when resisting lateral loads applied at water level. The design of the 13.3m diameter caissons evolved in response to the site conditions viz: up to 37m of soft to firm soils overlying two rock types having varying and irregular surface slopes. Foundations for Pier 1, located on the west bank, comprised four 2.5m diameter piles socketed into weathered dolerite. Both abutments and Pier 9 were supported on large spread footings cast directly on rock above river level in open excavations.

3.2 Cofferdams

The nature and slope of the rock surface did not lend itself to traditional caisson sinking methods. It was considered that the most viable method was to use a cofferdam in which a tremie plug was cast and the cofferdam dewatered to allow the caisson walls to be built in the dry from the bottom up. The design and construction of the cofferdams became one of the most significant aspects of the foundations despite the fact that they were only required to act as temporary structures.

Due to the scale and degree of difficulty associated with building the cofferdams a design was given in the tender documents, comprising interlocking steel 'H' piles driven to rock level. The Contractor, Leighton Candac, submitted an alternative cofferdam design formed of precast concrete units and this was accepted. Units were lowered to rock by means of hanger bars connected above water level to a steel jacket supported on temporary piles, units in 7m nominal lengths being added progressively during sinking. To overcome the irregular rock surface the Contractor determined rock levels by probing at close centres and then profiled the leading edge of each cofferdam aiming at achieving a gap no greater than 0.5m between the leading edge and the rock surface, Figure 2. The remaining gap where necessary was sealed with sand bags and props placed by divers.

3.3 Foundation Design

The foundations for the piers (Nos. 2, 3 & 4) on stronger rock comprised a 1.9m wide annular trench formed within the cofferdam. At other piers, on weaker rock, the trench width was increased to 4.4m. With variable weathering of the upper layers of mudstone it was necessary to vary the depth of excavation into the rock to achieve a satisfactory bearing capacity. Trench depths ranged from 0.5 to 3.5m in the mudstone and in the dolerite at Pier 2 a nominal trench depth of 0.3m was adopted.

The function of the tremie plug, Figure 2, is to

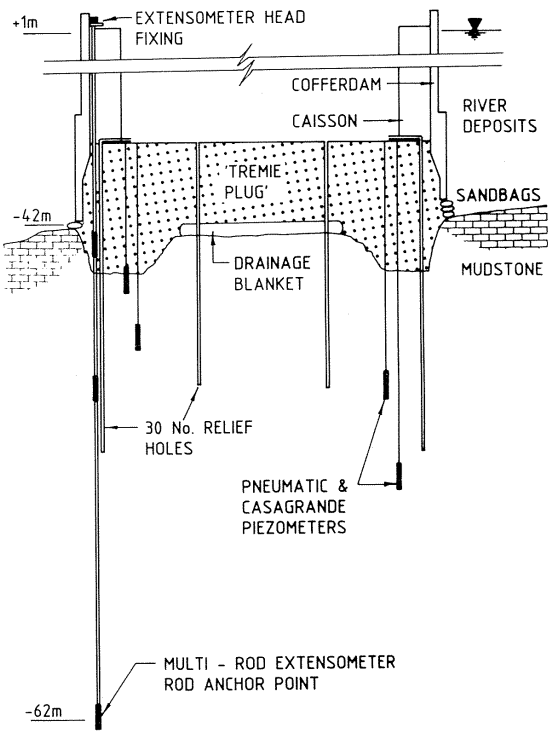

Figure 2 Pier 3 foundation drainage system and instrumentation

provide support and a seal for the dewatered cofferdam, distribute caisson wall loads into the rock, and contribute mass to the completed caisson foundation. Single lift concrete pours of up to 800m^3 were required to form the tremie plugs.

The allowable bearing pressures were determined from a study of the rock weathering, jointing and strength as indicated by drill cores. It was essential that estimates of ultimate bearing capacities and factors of safety were sufficiently conservative to ensure that foundation settlements under working loads would be limited and have negligible effect on the bridge superstructure. An empirical relationship for determining rock bearing pressures [6] was used as a guide in assigning allowable bearing pressures and founding levels for each pier. The relationship takes into account the condition and spacing of discontinuities and the footing size. Factors of safety of at least 3 were required under all normal service loading conditions. For ship impact loading minimum factors of safety of 1.5 for navigations Piers and 1.25 at other piers were accepted with due allowance for the conservative method used to assess the imposed bearing pressures at founding level and the dynamic nature of the loading.

Maximum bearing pressures at founding level were computed for both service load and ship impact conditions. Allowance was made for lateral support of the surrounding river deposits under ship impact loads by using soil modulus from undrained triaxial compression tests. Values of modulus adopted for design were 500 kPa at 3m below bed level, increasing linearly at 225 kPa/m with depth. Rock modulus was also required for the analysis and values were taken in the range of 1000 to 10,000 MPa for the dolerite and 500 to 5000 MPa for the mudstone.

Table I summarizes maximum bearing pressures on the rock at founding level. The range of bearing pressures under ship impact loading relates directly to the range of rock modulus adopted for the analysis.

TABLE I MAXIMUM BEARING PRESSURES

Pier No.	Rock Type	Trench Width (m)	Service Loading (MPa)	Ship Impact Loading (MPa)
2	Dolerite	1.9	2.4	4.3 to 8.5
3	Sandstone	1.9	2.4	4.1 to 6.5
6	Mudstone	4.4	1.3	2.3 to 3.8

4. DEWATERING OF COFFERDAMS

4.1 Relief Holes and Piezometers

Relief of water pressure was required in the rock at and below the foundation level to allow the cofferdams to be dewatered without uplift occurring. Relief was achieved by drilling a total of 30 No. 125mm dia. holes through the tremie plug and up to 10m into rock. A crushed rock drainage blanket was also provided beneath the central section of the tremie plug to add to the pressure relief. The typical arrangement of the pressure relief holes and drainage blanket is shown on Figure 2.

The effectiveness of the pressure relief system was gauged by means of piezometers. Two types of instruments were used, pneumatic piezometers manufactured by Soil Instruments Ltd. and Casagrande stand pipes with porous tips. Generally, both types were installed at each pier, some at the level of the concrete/rock interface and others at selected depths up to 7m below the interface. Each piezometer was installed in a separate borehole within a sand filter approximately 1m long and sealed by a grout plug.

4.2 Monitoring of Piezometers

The design and specification called for a maximum excess water head of 5m at the concrete/rock interface, at any stage of the dewatering, and a maximum rate of dewatering of 3m/hour to limit any build up of excess pore pressures under transient seepage conditions. Figure 3 shows the piezometer observations made during the dewatering of Pier 6. Owing to pump limitations the rate of dewatering did not exceed 1.0m/hour. The steady build-up of excess head from the commencement of pumping is shown for various instrumented depths below the tremie plug. In the fully dewatered state the excess head varied from 3 to 4.5m at the concrete/rock interface to approximately 11m at a depth 7m below the interface. Little change was observed in the piezometer readings when the water level was maintained at any one level indicating that a steady state of seepage was reached rapidly.

Piezometer response at other piers on mudstone was essentially the same as that observed at Pier 6. Excess head at the concrete/rock interface was generally less than the 5m stipulated, although up to 7m was recorded at Pier 3, where the surface of the tremie plug was approximately 38m below river level.

Upon initial dewatering of Pier 2 the piezometers responded in a similar manner as at other piers on mudstone. However, after 10 days a silt inflow occurred in one of the relief holes necessitating recharging the cofferdam with water and grouting the relief hole. Subsequent dewaterings resulted in further silt inflows and a steady increase in recorded excess water head to above the specified limit. The dewatering system was abandoned and

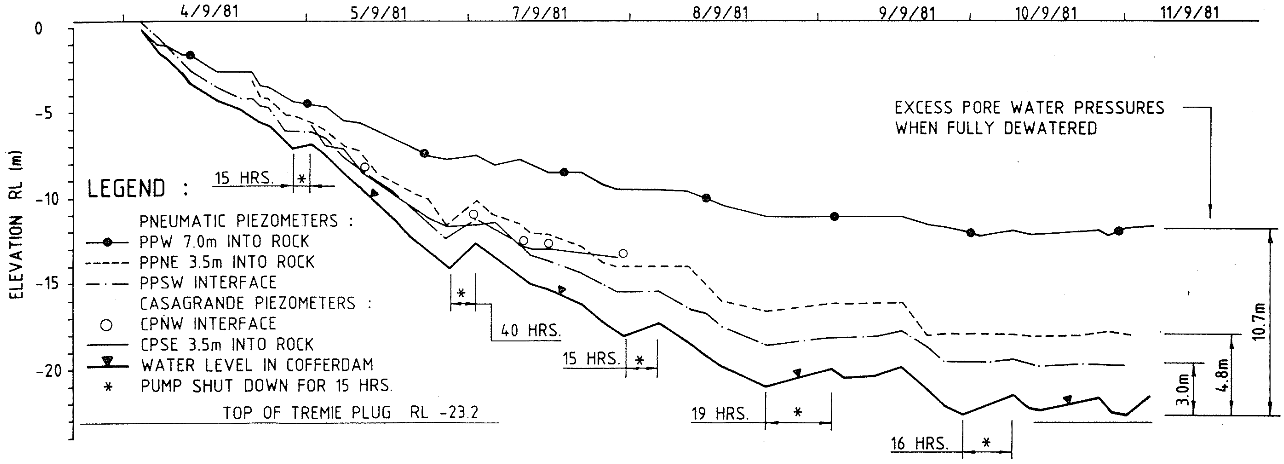

Figure 3 Pier 6 piezometer monitoring during dewatering

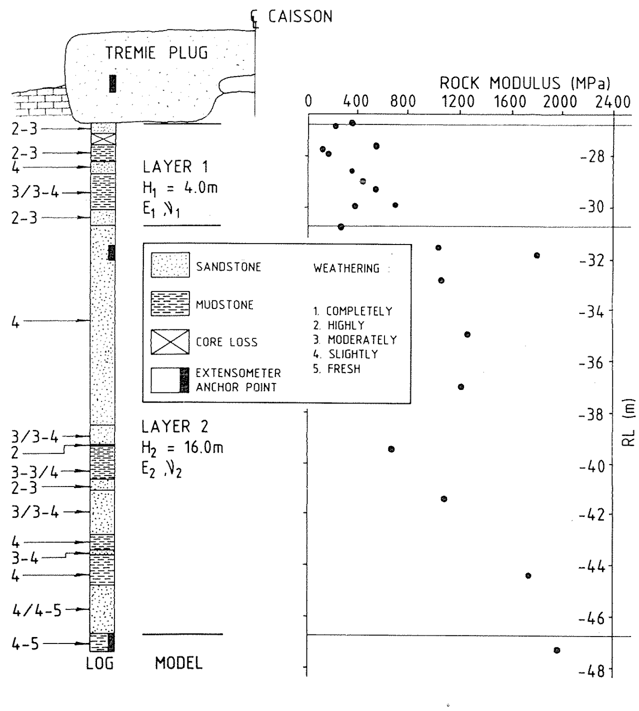

Figure 4 Pier 6 rock quality and modulus

a second tremie plug was cast involving 1600m^3 of concrete, sufficient to offset full hydrostatic pressures at rock level. The decision for this remedial action was based on minimisation of time and cost.

4.4 Inflow Rates

Inflow rates were recorded during dewatering, as part of the programme for monitoring water pressures, and checked periodically during the fully dewatered state to confirm that stable conditions were maintained. As expected the inflow rates steadily increased as the water level was lowered in each cofferdam. The inflow rates at the fully dewatered stage were estimated to range from about 9 to 30m^3/hr. Although it was not possible to obtain accurate measurements, no significant changes to inflow rates were observed over the time the cofferdams were fully dewatered.

5. ROCK MASS STIFFNESS

5.1 General

Rock mass stiffness is a significant parameter in analysing the response of the foundation to superstructure and barge impact loadings. A pressuremeter was used to provide data for checking design assumptions and has been complemented by settlement observations from multi-rod extensometers, installed at Piers 3 and 6, and detailed surveys of caisson caps at all piers. Estimates of rock mass stiffness have been determined from the extensometer settlement observations and compared with the results of insitu testing.

5.2 Insitu Testing

Pressuremeter testing of the rock was carried out at Piers 3 and 6. The pressuremeter and methods of interpretation are described by Hughes and Ervin [7]. At Pier 3, pressuremeter tests were conducted in two boreholes as part of an additional investigation aimed at providing more information on the rock quality for establishing founding levels. Pressuremeter tests at Pier 6 were carried out in the borehole drilled to facilitate installation of the extensometer. The results of the tests were used to position the extensometer anchors to best reflect the response of the rock mass to load and develop simple models of the rock mass below foundation level at each pier. The pressuremeter results and model for Pier 6 are shown in Figure 4.

5.3 Extensometer Installations

The Interfels multi-rod extensometer was selected for its accurate robust means of monitoring pier settlements both during construction and throughout the life of the structure. Three anchors were used, the top anchor being located in the tremie plug above the concrete/rock interface, the middle anchor some 4 to 6m below the interface, and the bottom reference anchor 20m below the interface, Figure 2. Each extensometer was located on the bridge centre-line and the general arrangement of the rods, anchors and head assembly is shown in Figure 5. Extensometers were installed prior to dewatering of the cofferdams, immediately after installation of the relief holes and piezometers. The installation procedure at Pier 6 consisted of the progressive core drilling and pressuremeter testing of an N size hole, subsequent reaming of the hole to 125mm diameter, and placement of the extensometer anchors, rods and oil-filled sheathing. An attempt was made to grout the hole at each anchor level and place sand between each anchor. However, practical difficulties prevented this being achieved and the hole was continuously grouted from the bottom to the top of the tremie plug. The installation procedure at Pier 3 was similar to that adopted

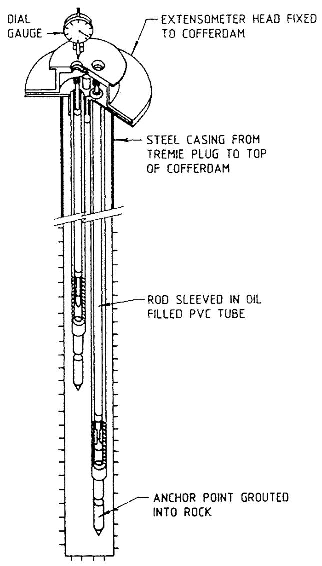

Figure 5 Multi-rod extensometer (after Interfels 1972)

at Pier 6 except that a 150mm dia. hole was drilled using a down-the-hole hammer without the need for reaming.

The extensometer rods are permanently housed in a steel casing from the top of the tremie plug to the caisson cap. The head unit was attached to the inside wall of the cofferdam throughout the dewatering and caisson construction and was subsequently transferred to a permanent mounting in the caisson cap.

5.4 Monitoring of Extensometers

Monitoring of the extensometers continued throughout the dewatering stage while cleaning-up the tremie plug and casting the caisson walls, and during the cap, columns, and superstructure erection. Typical results of the monitoring are presented in Figure 6 for Pier 6, and are summarised in Table II:

TABLE II SETTLEMENTS RELATIVE TO BOTTOM ANCHOR (mm)

Stage of Works	Top Anchor		Middle Anchor	
	Pier 3	Pier 6	Pier 3	Pier 6
Initial Dewatering	1.7	1.8	1.0	-0.1
Caisson Construction and Refilling	2.4	3.7	0.9	-0.9
Caisson Cap and Column Construction	1.0	1.4	0.1	0.2
Superstructure Erection	1.8	5.9	0.3	0.5
Total	6.9	12.8	2.3	0.3

The results show significantly smaller settlement at Pier 3 compared with Pier 6 reflecting the difference in rock quality. Both piers settled slightly when dewatered which was unexpected and continued to do so during caisson wall construction. Pier 3 showed some uplift during refilling and similarly, the rock at depth below Pier 6 showed some tendency to dilate during refilling. However, the behaviour during the dewatering stage at both piers was somewhat erratic and remains unexplained. In comparison, the settlement pattern during cap and column construction and superstructure erection, Figure 6, was most plausible. Both piers exhibited a decreasing rate of creep settlement after completion of the superstructure erection, Pier 6 recording 0.9mm of creep settlement over 10 months.

5.5 Construction Surveys

Pier position and levelling surveys were carried out during all stages of construction. In particular, caisson cap levelling was carried out during erection of both upstream and downstream superstructure cells as each segment was added. Whilst, on occasions, the estimated error associated with this levelling was less than the magnitude of the recorded settlements, the results have provided a useful means of comparison with the extensometer readings. The cap levelling also provided an estimate of the pier rotation resulting from the eccentric loads imposed during erection of the superstructure, Table III.

TABLE III CAISSON CAP SETTLEMENT DURING SUPERSTRUCTURE ERECTION (mm)

Pier No.	Centreline Settlement	Differential Settlement	
		Maximum	Final
3	3	5	2
4	8	7	3
5	7	8	1
6	9	14	7

5.6 Estimates of Rock Mass Stiffness

The pier settlements recorded by the extensometers have been used to calculate rock mass stiffness. The stiffnesses thus obtained provided a comparison with the assumed rock stiffness used in the design, Section 3.3.

Modelling of the rock mass was carried out using a multi-layered elastic analysis programme, ELSYM 5 [8]. The models adopted were simplified to two layers with interfaces corresponding to the extensometer anchor positions, Figure 4. Superposition of circular loaded areas was used to simulate the annular foundation. At both Piers 3 and 6 the rock modulus has been determined for two stages of the monitored load 'vs' settlement curves; cap and column construction, and superstructure erection, Table IV.

TABLE IV COMPARISON OF PRESSUREMETER AND EXTENSOMETER ESTIMATES OF ROCK MODULUS (MPa)

Pier	Layer	Rock Type and Weathering	Pressuremeter	Cap and Column Construction	Superstructure Erection
6	1	Mudstone 2-3 to 3	100- 600	320	110
	2	Sandstone 3 to 4	1000-2000	3010	1700
3	1	Sandstone 3 to 4	1000-1500	1150	550
	2	Sandstone 3-4 to 5	1800-2600	2600	4000

The analyses indicate a decline in rock mass modulus, from the cap and column construction stage to superstructure erection stage, for both layers of the Pier 6 model and layer 1 of the Pier 3 model. The decline in modulus is believed to be a function of the stress path experienced by the

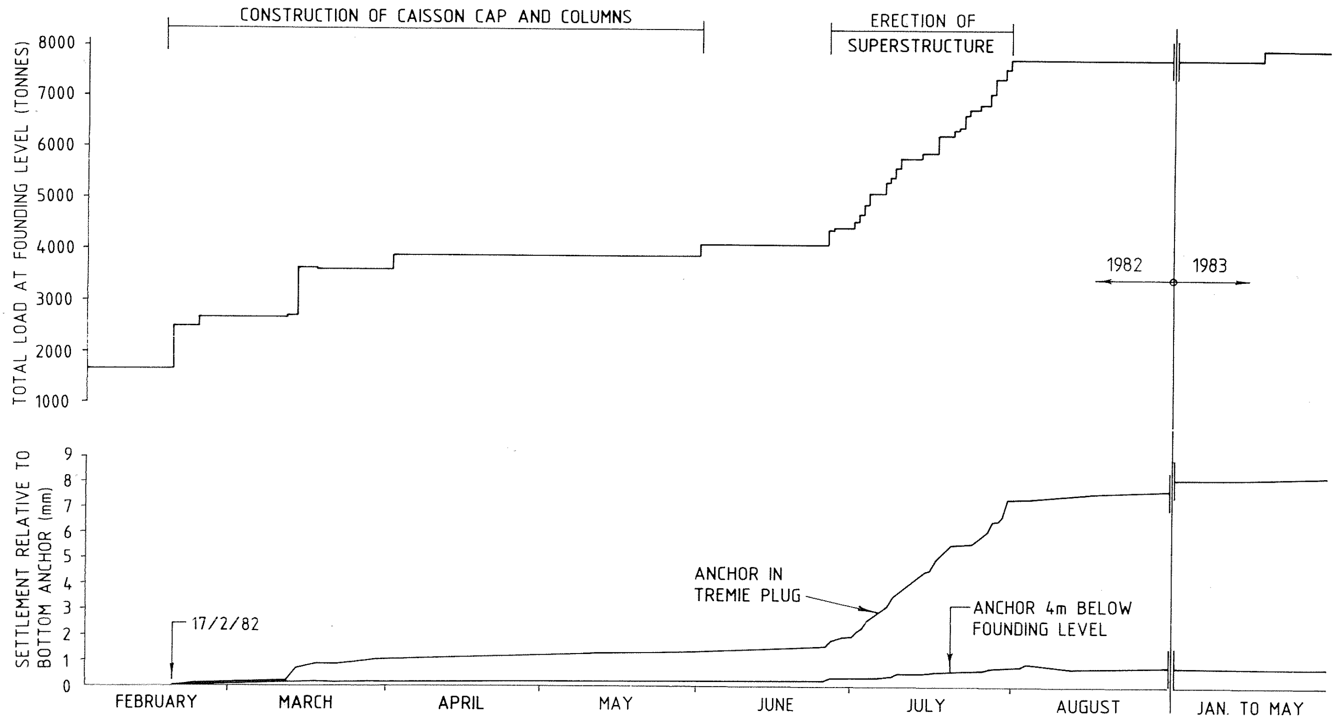

Figure 6 Pier 6 extensometer monitoring during pier and superstructure erection

rock mass under direct compression and eccentric loads applied during superstructure erection and the natural non-linear stiffness of the rock mass.

6. CONCLUSIONS

Reliable means of monitoring pore water pressures and settlements were required for the construction of foundations for the Bowen Bridge. The pneumatic piezometers, installed at depths to 50m below river level, functioned satisfactorily, and are considered most suitable for the application in view of the ease of reading and small volume change required for response. The importance of the monitoring ability provided by the piezometers was well illustrated during construction of Pier 2, where as a result of increasing water pressures a significant change to the method of construction was undertaken to enable works to proceed expeditiously. Whilst identical installation procedures were used for both pneumatic and Casagrande piezometers, the observations of many of the Casagrande type indicated leakage problems and were therefore considered less reliable.

The two multi-rod extensometers installed to depths up to of 60m, have also functioned satisfactorily. It is believed that the differing settlement responses observed during dewatering are attributable to complex stress paths in the rock produced by transient and steady state seepage. Estimates of rock modulus from settlements monitored by the extensometers during cap, column and superstructure erection were generally in the range of pressuremeter measurements of rock modulus. However, the extensometers indicated a definite decline in modulus with increasing load which is believed to be indicative of the non-linear stiffness characteristics of the rock mass. With total pier settlement being less than 10mm, the extensometers were the only practical means of providing accurate meaningful settlement data at the river piers.

7. ACKNOWLEDGEMENTS

The authors wish to thank the Joint Committee on Second Hobart Bridge and Maunsell & Partners Pty. Ltd. for their permission to publish this paper.

8. REFERENCES

[1] LICKISS, K.G., Planning the Bowen Bridge, Hobart, Annual Conference I.E. Aust., April, 1983.

[2] LESLIE, J.A. & ALLEN J.B., Design of Bowen Bridge, Hobart, Annual Conference I.E. Aust., April, 1983.

[3] LEAMAN, D.E. (1975), Second Derwent Crossing Study. Magnetic and Seismic Survey Reports. Unpubl. Rep. Dept. Mines Tasm. Report Nos 1975/32 & 39.

[4] Report on Phase 1 Site Investigation for Second Hobart Bridge, Maunsell & Partners Pty. Ltd. Unpubl., Oct., 1976.

[5] Report on Phase 2 Site Investigation for Second Hobart Bridge, Maunsell & Partners Pty. Ltd. Unpubl., Sept., 1977.

[6] NATIONAL RESEARCH COUNCIL, CANADA, Canadian Manual on Foundation Engineering. 1975.

[7] HUGHES, J.M.O. & ERVIN, M.C. (1980), Development of a High Pressure Pressuremeter for Determining the Engineering Properties of Soft to Medium Strength Rocks, Proc. 3rd ANZ Conf. on Geomechanics, Wellington, NZ.

[8] AHLBORN, G. (1971), ELSYM5, Computer program for determining stresses and deformation in elastic systems. Univ. California, Berkeley.