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STAT*NAMIC* Load Testing on Bored Piles in Australia

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Summary Two separate STAT*NAMIC* load testing programmes have been carried out in Australia. The first commercial STAT*NAMIC* testing was for a twenty-six storey apartment building in Melbourne, where three tests were performed on rock socketed piles. The second programme consisted of two STAT*NAMIC* loading tests on a rock socketed pile, at the Olympic Village site in Sydney. The results provided a clear indication of the potential for this recent development in the area of foundation design optimisation. Strain gauge instrumentation of the piles enabled measurement of the load distribution along the pile shaft and at the base. This enabled a comparison to be made between the derived shaft friction parameters and those traditionally adopted by geotechnical practitioners in Australia. Quality assurance and safety protocols, which are necessitated by the nature of the test method, are also discussed.

1. INTRODUCTION

Three 16 MN STAT*NAMIC* pile loading tests were performed on concrete piles on the Quay West project in South Melbourne. The tests were performed in accordance with routine quality assurance requirements of the piling contract, as an alternative to static load testing, the costs of which were seen by the contractor to be prohibitive. The objectives of the STAT*NAMIC* testing programme were the same as those for a static load testing programme, viz.:

1. To confirm the adequacy of the piles tested.
2. To check the design parameters of the rock socket in the siltstone, thus providing confidence in the design of all piles on the project.
3. To confirm the adequacy of the construction procedures of the piles.

The Quay West project involves construction of a 26 storey apartment building located at the corner of South Gate and Jennings Way at Southbank. The specification required static load testing of two piles: one test to confirm compression performance and one test for tension loading. As an alternative, STAT*NAMIC* testing was proposed, to reduce the costs and time associated with pile testing, and it was agreed that three STAT*NAMIC* tests would be performed, using a 16 MN device. STAT*NAMIC* testing had not been performed previously in Australia and its introduction on the project was seen to be innovative by the client and his consulting engineers.

In the second testing programme two 8 MN STAT*NAMIC* tests were performed on a bored concrete pile constructed at the Olympic Village precinct at Homebush Bay, Sydney. Homebush Bay is the central location of the facilities being built for the Year 2000 Olympic Games and is approximately 20 km west of the Sydney CBD. The Olympic Village testing programme was carried out for demonstration purposes only and the test pile was not a production pile. However, the results provided valuable data on the load-displacement performance of large diameter bored piles and in particular the geotechnical pile design parameters.

The test piles at both sites were instrumented with strain gauges to measure the load distribution down the shaft. Construction methods and the geotechnical profile were the primary factors influencing the performance of the test piles. This paper describes the physical STAT*NAMIC* testing programme and the results obtained.

The quality assurance regime and safety protocols adopted for STAT*NAMIC* testing in Australia are also discussed. This includes performing a computer simulation of the test and pre-test checklists.

2. GEOTECHNICAL CONDITIONS & TEST PILE DESIGN

2.1 Subsurface Profile, Quay West

The soils typically comprised stiff silty clays to 21 m depth, overlying sands of medium to dense consistency, in turn overlying Silurian siltstone occurring at a depth of about 30 m. The strength of the rock, inferred from correlations with moisture contents, are shown in Table 1.

Table 1. Inferred rock strengths in test piles

Pile No.	Depth (m)	D.O.W.$^+$	Depth below top of rock (m)	M.C.$^\#$ (%)	$q_u^{\#\#}$ (MPa)
55	30.8	MW*	0	4.0	8
	31.2	SW**	0.4	2.4	15
	32.2	SW	1.4	1.8	21
	35.2	SW	4.4	1.9	20
58	30.4	MW-SW	0.1	4.4	6.5
	31.2	SW	0.8	2.6	14
	31.5	SW	1.1	2.6	14
59	30.5	MW	0	3.0	11.5
	31.3	SW	0.8	1.2	32
	31.8	SW	1.3	1.3	28
	32.0	SW	1.5	3.0	11.5

D.O.W.$^+$ = Degree of Weathering
M.C.$^\#$ = Moisture Content
$q_u^{\#\#}$ = Unconfined compression strength
MW* = Moderately Weathered
SW** = Slightly Weathered

The initial design incorporated conventional design methods, using the following ("allowable") design parameters:

Shaft resistance in clays:	16 kPa
Shaft resistance in dense sands:	35 kPa
Shaft resistance in SW siltstone:	800 kPa
End-bearing resistance in SW rock:	8000 kPa

The design parameters adopted in the rock were significantly higher than those initially recommended by the consulting engineers.

The concept of an "allowable" stress or capacity is based on limiting settlement or deflection to within acceptable limits, such that pile design is based upon a serviceability criterion alone.

To support the specified loads, with appropriate load/ reduction factors, the geometry of the rock sockets were proportioned in accordance with the above parameters, with the overwhelming percentage of the load being resisted by the SW rock. Estimates of settlements were made to demonstrate that differential settlements were within the specified performance criteria.

2.2 Subsurface Profile, Homebush Bay

The subsurface profile at the location of the test pile was geotechnically logged. The shale bedrock was classified with respect to its intact strength and percentage of defects using the scheme introduced by Pells et al. (1978). In brief, the classification scheme ranges from Class V (5) to Class I (1) shale, where:

- Class V shale is defined as "Mainly shaley clay – hard clay with thin zones of weak shale." Typically highly fragmented with an Unconfined Compressive Strength (UCS) that is not normally measurable (i.e. too low).
- Class I shale is defined as "Strong shale – core sections can only be slightly scratched with steel knife." Typically slightly to fractured with a UCS of greater than 16MPa.

The following ("allowable") design parameters were assumed:

Shaft resistance in clays (ignored):	0 kPa
Shaft resistance in Class V Shale:	80 kPa
Shaft resistance in Class IV Shale:	150 kPa
Shaft resistance in Class III Shale:	250 kPa
Shaft resistance in Class II Shale:	500 kPa
Shaft resistance in Class II/I Shale:	600 kPa
End-bearing resistance in Class II/I Shale:	6000 kPa

The allowable shaft adhesion and end bearing parameters used for the test pile design are typical of those adopted by engineers in the Sydney region for the design of bored piles founded in shale of similar quality (Pells et al., 1978). It is noted that the parameters indicated in that paper are generally based on the presumption that the pile base is adequately clean and that the shaft sidewalls are rough and free of smear (i.e. the remoulded material).

3. PILE CONSTRUCTION DETAILS

3.1 Pile Construction, Quay West

The piles tested were bored piles of nominal 750 mm diameter, constructed using Bentonite drilling fluid over the full length of the piles, to ensure the stability of the excavation at all stages.

Drilling for the piles was done with a conventional drilling bucket over the full length of the pile. During construction, the soils were logged by an experienced geotechnical engineer, who also took rock cuttings from the drilling bucket for determination of moisture content in keeping with established Melbourne practice. This enabled an indication of the rock strength to be made, using site specific data in conjunction with a local correlation of moisture content and strength that had been established over many years. In turn, this provided a check of the design parameters to be made, thus ensuring construction to be in compliance with design requirements. Upon completion of drilling, the rock socket was roughened using offset reaming teeth attached to the drilling bucket.

A purpose-built cleaning bucket was used to remove residual debris from the pile base. The socket was then airlifted, with Bentonite being completely replaced with a mix that complied with Specification requirements for concreting.

3.2 Pile Construction, Homebush Bay

A 900 mm diameter triple flight soil auger bored the hole down to the top of the Class II shale, at 8.5 m depth. Below this level it was necessary to replace the soil auger teeth with tungsten carbide (TC) rock cutting teeth, to penetrate the higher strength rock. Due to groundwater ingress and some collapse in a permeable soil layer between 4 and 5.5 m depth, a 5 m long temporary steel casing of 1.05 m diameter was inserted as the pile was progressively reamed out, down to 6.0 m depth. The as-constructed pile depth was 10.2 m. This was increased to 10.7 m by the filling (0.5 m thick) involved with the earthworks preparation around the pile head for the STAT*NAMIC* test.

A full-time geotechnical presence was maintained to identify the stratum and to police base and sidewall cleaning and shaft roughening procedures. The rock socket was roughened and smear was cleaned from the sidewalls using a grooving tool attachment to the drilling auger. The grooves or undulations were visually estimated to be about 20 mm wide and 20 mm deep, at spacings of up to 200 mm. It is noted that this exceeds the dimensions given for an "R4" Roughness Class (Walker and Pells, 1992).

3.3 Pile Instrumentation

For both projects strain gauges mounted on 1 m long (12 mm diameter) steel 'sister bars' were affixed to the main reinforcing cage at varying levels down the cage length.

4. STAT*NAMIC* TESTING PROGRAMME

4.1 Pre-Test Quality Assurance and Safe Work Procedures

STAT*NAMIC* testing involves the controlled burning of solid fuel pellets in a pressure chamber to impart a force to the foundation to which it is connected. Despite the fact that the STAT*NAMIC* fuel is classified as a low-grade pyrotechnic fuel, a comprehensive safe work method is mandatory for STAT*NAMIC* testing. Further, the considerable effort required in assembling the test equipment dictates that a thorough quality assurance procedure is followed. Safety and quality assurance aspects are combined in a standard pre-ignition checklist, which includes the following:

1. Site inspection and site suitability assessment.
2. Collection and review of site specific information (e.g. geology, pile details).
3. Pre-test preparation and condition check of test equipment.
4. STAT*NAMIC* simulation of proposed test configuration.
5. Refer to Section 4.2 for discussion of the STAT*NAMIC* Simulation procedure.
6. Safe working procedures on test site including pre-ignition warning sirens, danger signage and safety fencing.
7. Ignition by certified Powderman.
8. Dismantle test assembly.
9. At the Olympic Village site Sonic Integrity Testing was carried out prior to the STAT*NAMIC* testing.

4.2 STAT*NAMIC* Computer Simulation

An integral part of the planning process for STAT*NAMIC* testing involves performing a computer simulation of the test. The purpose of carrying out a STAT*NAMIC* simulation is to assess and design the following aspects of the test:

- Quantity of STAT*NAMIC* fuel required for desired test load.
- Selection of the vent size for release of pressure.
- Reaction mass required.
- Expected launch height of reaction masses/silencer
- Characteristics of the loading and unloading curve.

The STAT*NAMIC* simulation was carried out using the TNOWAVE simulation package software developed by TNO Building and Construction Research Organization of the Netherlands (Bielefield & Middendorp, 1995). The simulation package is an extension of the TNOWAVE program, which was originally developed to simulate the behaviour of a pile and soil during dynamic pile loading tests.

The simulation performed for the Olympic Village site indicated that a quantity of 6.5 kg of STATNAMIC fuel was required for the 8 MN test load. Further, a launch height of about 1.3m was predicted for the reaction masses, which was within the safety criterion for the STAT*NAMIC* testing.

5. RESULTS AND DISCUSSION

5.1 STAT*NAMIC* Results, Quay West

Three tests were completed in a testing time of six days at the Quay West site. The results of the three STAT*NAMIC* load tests are depicted in conventional load–displacement format in Figure 1. As shown, a displacement of between 18 mm and 24 mm was recorded for the maximum applied test load of about 16 MN. Net pile movements were less than 6 mm. The "measured" results were sufficient to satisfy the requirements of the piling contract. However, the measured results were then "corrected" to account for damping and inertial effects, which were fairly minor due to the founding conditions for these piles.

Table 2. Derived static pile resistance

Pile No.	P.M.F $^+$ (MN)	Meas. Max. Displ. (mm)	P.D.F * (MN)	P.I.F$^\#$ (MN)	Derived Static Force (MN)
55	16.2	18.7	N/A	1.8	15.7
58	16.2	20.5	1.25	-2.0	17.5
59	16.5	23.2	0.3	-2.8	19.2

P.M.F.$^+$ = Peak Measured Force
P.D.F. * = Peak Damping Force
P.I.F. $^\#$ = Peak Inertia Force

When results of STAT*NAMIC* tests have been corrected for damping and inertia, the corresponding load–displacement curve is regarded as the equivalent "static" loading test curve or "derived" curve. The results of those analyses are included in Table 2.

5.2 STAT*NAMIC* Results, Homebush Bay

The testing at the Olympic Village was carried out in heavy rain. Despite the inclement weather two tests were completed in four days. The load–displacement diagram shown in Figure 2 was obtained from the test. A pile head displacement of 2.5 mm is indicated for the maximum applied test load of 8.2 MN. The loading and unloading portion of the "curve" are in the form of straight, sub-parallel lines. Based on this result it is apparent that the load–displacement performance of the pile was completely elastic during this test and that no plastic pile behaviour occurred.

The results indicated that there is no time lag between the maximum applied force and the maximum pile displacement. Thus the damping and inertial effects were insignificant. This is consistent with completely elastic pile load–displacement behaviour, indicated by both the linear load–displacement 'curve' and also the displacement–time graph, which shows no residual pile movement remained after unloading (not present here). On this basis, an equivalent 8 MN static load test would be expected to yield exactly the same load–displacement curve as this STAT*NAMIC* test result.

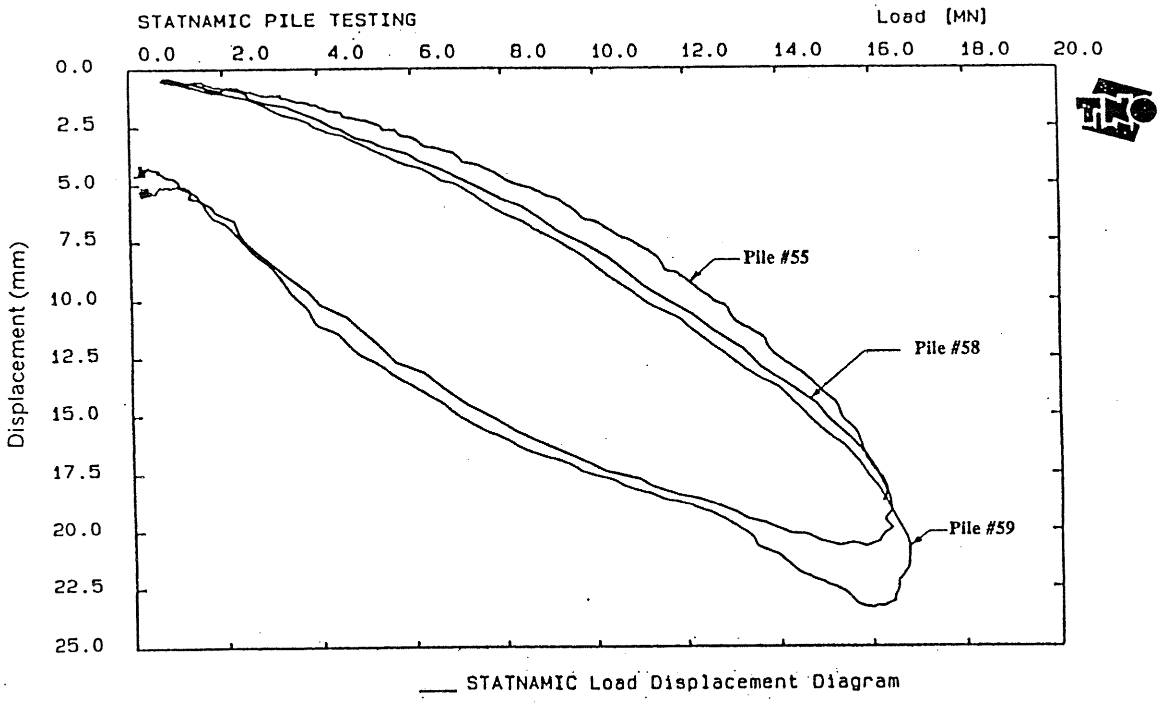

Figure 1. Load–displacement results, Quay West site

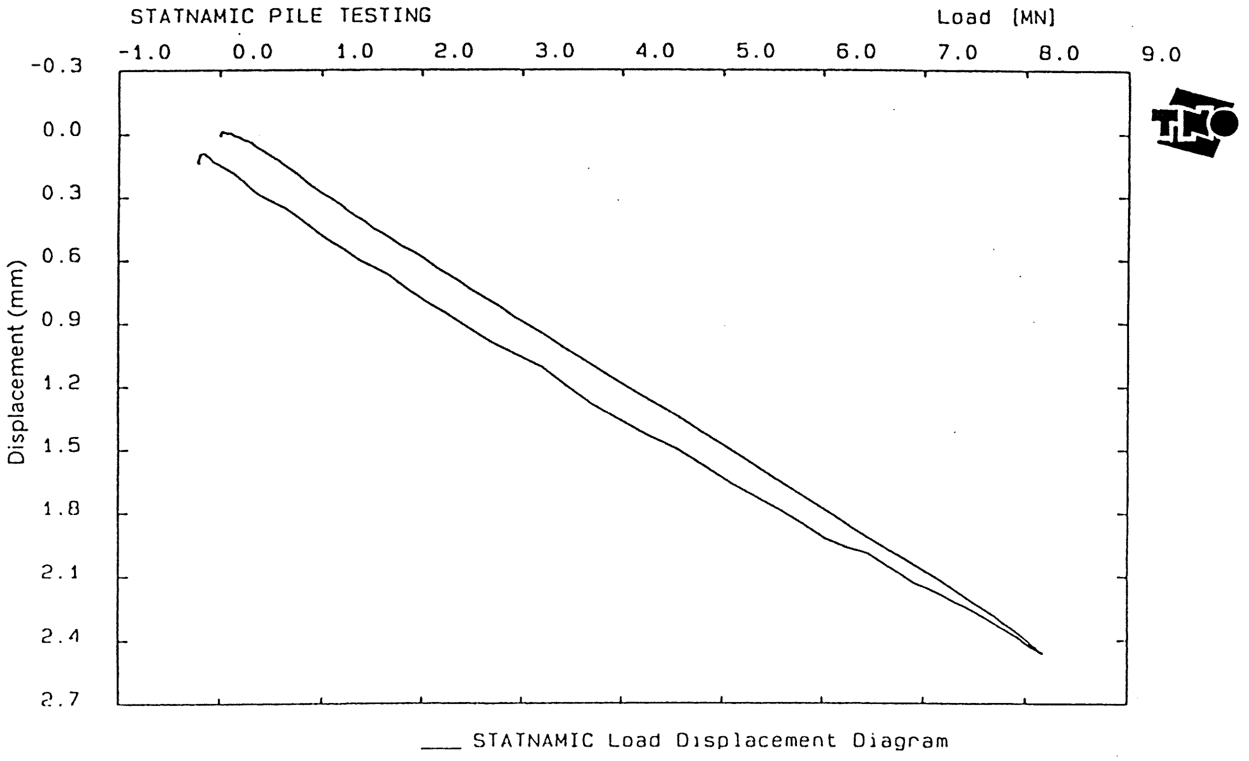

Figure 2. Load–displacement results, Olympic Village site

5.3 Strain Gauge Results Quay West

Two gauges at each of five levels, i.e. ten gauges, were installed in each pile. The gauges were located with the intention of providing an assessment of the load transfer in the soils, and particularly in the rock sockets. Unfortunately, readings from a total of only three levels were recorded that were of sufficient quality to enable meaningful interpretation of load transfer to be made. It is believed that poor results were obtained due to the proximity of two 350 kV underground power lines causing electromagnetic interference to the (resistance type) strain gauges. The power lines were located within 10 m of pile No. 55 and 30 m of pile Nos. 58 and 59. The existence of the power lines was not known at the time of installation of the gauges.

Despite this, it is believed that sound judgements could be made from the limited readings to provide an indication of the load transfer to be made.

5.4 Strain Gauge Results, Homebush Bay

Three gauges were installed at varying levels down the test pile. The calculated loads at the three gauge levels, as a function of time, are shown in Figure 3. The graph indicates that the maximum load in the pile occurred at 158 milliseconds (it is noted that this is a time scale, and is not the same as that of the STAT*NAMIC* monitoring software). The maximum load for the strain gauges at 8.0 m and 9.7 m depth was calculated to be 6.1 MN and 2.7 MN respectively. In other words, only about 33% of the applied load reached the level of the lowest strain gauge (9.7 m depth) and therefore about 67% of the test load had been carried by shaft adhesion above this level.

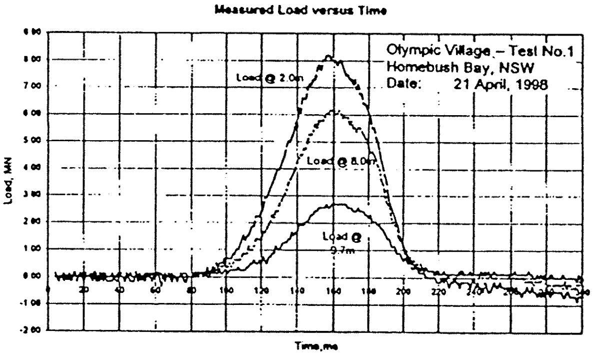

Figure 3. Measured load vs time.

On the basis of the above, load distribution was calculated down the pile for three separate instances; viz., before the maximum load is reached at 140 milliseconds (i.e. during loading), at the maximum load (158 ms) and after the maximum applied load at 180 ms (i.e. during unloading). On the basis of these calculations, it was inferred that less than 10% of the applied load reached the base or toe of the pile.

Based on the assessed load distribution, the average mobilised shaft adhesion can be calculated at the two lower strain gauge levels. Average shaft adhesion of about 700 kPa and 750 kPa was assessed at 7.5 m and 9.7 m respectively. It is noted that at 7.5 m depth, the pile is within Class V shale and at 9.7 m depth the shaft is founded within Class II/I shale. Thus, it is apparent that while the upper rock socket (i.e. at 7.5 m) has been near or fully mobilised during the STAT*NAMIC* test, the shaft at 9.7 m depth is only partially mobilised. This is consistent with the magnitude of pile displacements (2.5 mm max.).

On the basis of the above, the mobilised shaft adhesion within the Class V shale is 7 to 14 times greater than the allowable value indicated by Pells et al. (1978) for this material (50-100 kPa). It must be noted that careful construction control and shaft roughness of greater than R4 was achieved for the test pile. Thus, higher shaft friction parameters than suggested by Pells et al. (1978) could have been expected for the test pile. Notwithstanding this, the potential for substantially higher shaft friction parameters has been demonstrated. The veracity of this relationship should be assessed on a site-specific basis.

6. CONCLUSIONS

STAT*NAMIC* load testing has been successfully carried out during two testing programmes in Australia. The first commercial application at the Quay West site verified pile performance for the construction methods adopted by the piling contractor. The second testing programme was carried out for demonstration purposes, but also showed the potential for the test to economically prove higher design parameters.

During both of the STAT*NAMIC* test programmes STAT*NAMIC* tests have been performed at the rate of 1 to 2 days per test. This compares favourably with static load testing which would have taken about 3 weeks per test, based on normal rates for such testing.

STAT*NAMIC* simulation enables a prediction of the performance of the test configuration and pile to be carried out prior to testing. This allows design of a safe and cost effective test programme.

Pile instrumentation has provided detailed information on the distribution of load along the pile shaft and hence essential design information on shaft friction. By adopting higher shaft friction parameters in the design process, there is the potential for considerable cost savings in the construction of piled foundation systems.

The STAT*NAMIC* test result confirmed that with good construction methods, design parameters significantly greater than those adopted in conventional practice may be achieved.

7. REFERENCES

Australian Geomechanics Society (1985). *Engineering Geology of the Sydney Region,* edited by P.J.N. Pells, A.A. Balkema, Rotterdam.

Bielefeld, M.W. and Middendorp, P. (1995). *Statnamic Simulation*, presented at the First International Statnamic Seminar, Vancouver, Canada.

Brown, D.A. (1994). Evaluation of Static Capacity of Deep Foundations from Statnamic Testing. *Geotechnical Testing Journal*, Dec.

Frankipile (1997). Ground Engineering, Pile Loading Tests, Report for Civil & Civic (unpublished).

Justason, M.D., Mullens, A.G., Robertson, D.T. and Knight, W.F. (1998). Proc. 7th Int. Conf. and Exhibition on Piling and Deep Foundations (DFI), Vienna.

Middendorp, P., Berminghammer, P. and Kuiper, B. (1992). Statnamic Load Testing of Foundation Piles, *Proceedings, Fourth International Conference on Application of Stress-Wave Theory to Piles, The Hague.*

Pells, P.J.N. *et al.* (1978). Design Loadings for Foundations on Shale and Sandstone in the Sydney Region, *Aust. Geom. Jnl.*, Vol. G8.

Poulos, H.G. (1998). Pile Testing – From the Designer's Viewpoint. Second International STAT*NAMIC* Conference, Tokyo, Japan (to be published).

Randolph, M.D. (1991). *RATZ – Load Transfer Analysis of Axially Loaded Piles.* Program Manual.

Tchepak, S. and Chin, M.C. (1998) – *Statnamic Testing of Bored Piles Socketed into Melbourne Siltstone.* Second International STAT*NAMIC* Conference, Tokyo, Japan (to be published).

Walker, B.F. and Pells, P.J.N. (1992). *Draft Specification for the Construction of Bored Piles Socketed into Shale and Sandstone.* Draft Paper (unpublished).