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PAPER 9637 ROAD PANEL

Elevated section of the West Gate Freeway, South Melbourne, Australia. Part 1: design and construction of foundations

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The construction of West Gate Bridge during the 1960s and 1970s reinforced the need for an elevated freeway across the South Melbourne and Port Melbourne areas to remove heavy freight vehicles and through traffic from the overloaded network of commercial and residential streets. Difficult soil conditions and a recent history of bridge collapses (Kings Bridge and the West Gate Bridge itself) meant that the design and construction of the new elevated structures would come under unprecedented scrutiny from all sections of the community. Quality Assurance had to begin at the time of conceptual design, and to be implemented all the way through the design and construction process into the service life of the structures. The introduction of a construction technology—short line match casting of segments—which was new to Australia at the time, also emphasized the necessity for the highest possible level of technical control throughout the project.

Introduction

The West Gate Freeway in South Melbourne was opened late in 1988, providing improved access to Melbourne's ports and rail facilities and to its western suburbs and beyond. It incorporates two parallel bridges which elevate the freeway for a length of 1·85 km over the busy urban area of South Melbourne. The locality is commercial/industrial in nature, and contains many of Melbourne's major arterial roads as well as minor streets, two suburban railway lines, a major container handling yard and tram lines in Clarendon Street and Kings Way (Fig. 1).

2. The freeway connects to the West Gate Bridge, leading to Melbourne's major western traffic corridor. Before the freeway was opened, traffic access to and from West Gate Bridge was through the local streets of South Melbourne, Port Melbourne and the residential bayside suburbs. The average 24 hour two-way daily traffic volume on West Gate Bridge in the early 1980s was 45 000 vehicles, but when the tolls were removed in December 1985, the traffic volumes leapt rapidly to about 60 000 vehicles per day. The traffic volume has now risen to around 90 000 vehicles per day.

3. The complex urban environment, together with the difficult ground conditions which exist at the site, led to the adoption of a method of construction

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Written discussion closes 15 October 1991; for further details see p. ii.

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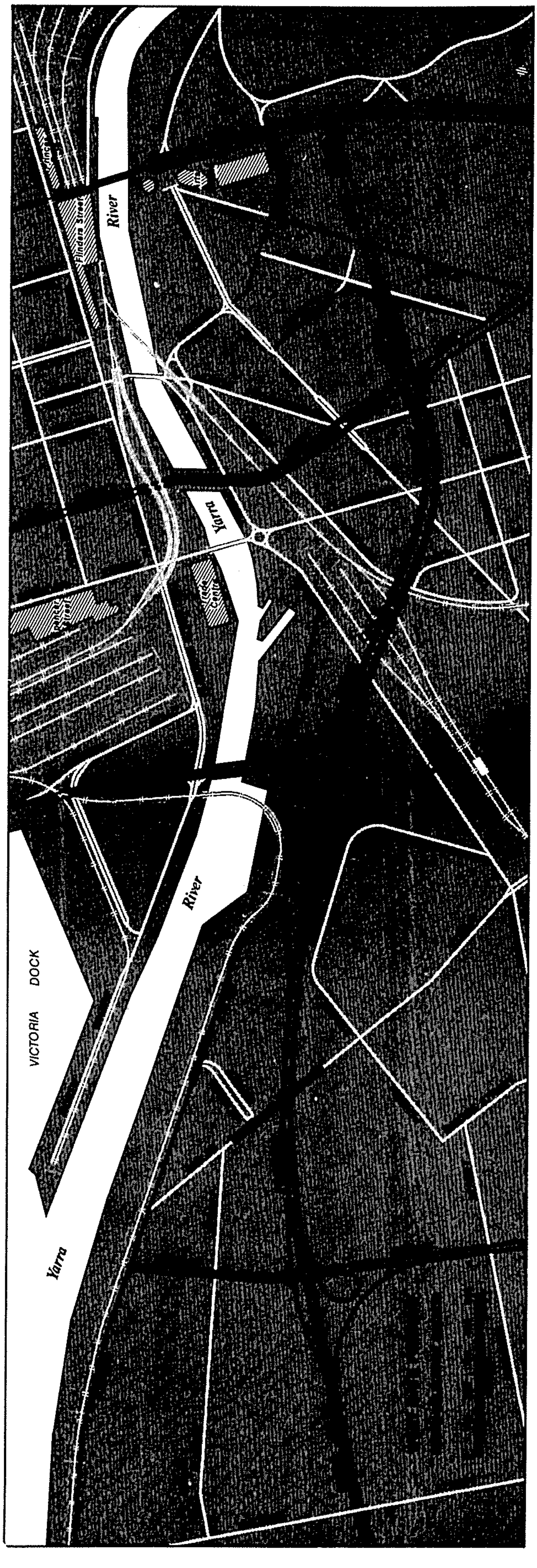


Fig. 1. Locality plan

comprising prestressed concrete box girder bridges formed by the match-cast segmental method and erected in balanced cantilever with glued joints between segments.

4. Quality Assurance procedures were implemented from the start of the design process in the form of proof engineering and extensive testing and certification for all major technical aspects of the project.

Ground conditions

5. The ground conditions at the site had an important influence on the form of construction adopted. In early times, this area of South Melbourne was a swamp with sparsely scattered sand ridges. Over the years, the area has been filled to a height of 1–1·5 m, so that the existing surface is now about 2 m above sea level.

6. The site geology along the route of the West Gate Freeway through South Melbourne comprises an alluvial sequence of soft to stiff clays, sands and gravels overlying Silurian aged claystone, siltstone and sandstone bedrock (known locally as Melbourne Mudstone) at depths of between 20 m and 40 m (Fig. 2). The strength of the rock varies from very low to medium, the sandstone beds usually being the stronger materials. The Silurian bedrock has weathered variably, and has been subjected to both folding and faulting.

7. Swarms of steeply dipping porphyritic dykes traverse the bedrock at some locations. In some areas, Tertiary aged basalts exist as flows above the bedrock, separated from it mostly by another alluvial sequence of stiff clays and dense sands up to 16 m thick, also of Tertiary origin. The basalts are deeply weathered and vary in character from the consistency of clay to nearly fresh rock. Within the basalt strata, extensive clay seams and patches were found. The basalt provided an alternative founding material to the mudstone, but also caused considerable hindrance to construction where it was either too thin or too weathered to support foundations.

8. The highly compressible silts over most of the site could not be relied on to support any imposed loads from falsework supports. However, balanced cantilever construction does not rely on any support from the ground because the main support is provided by the permanent bridge foundations at the piers at all times during construction.

9. Wherever appreciable thicknesses of the soft silty clay exist, long-term settlements are in evidence. These settlements, which have been measured at up to 15 mm per year, are the result of a combination of delayed compression of the clays under self-weight and creep beneath the fill crust which had been placed over the last 150 years.

10. The generally poor ground conditions, large pile loads and stringent differential settlement requirements for the elevated structures resulted in the adoption of bored piles socketed into the Silurian bedrock or into the overlying basalts as the preferred foundation type.

Pile foundations

General

11. The foundations of the elevated freeway structures comprise 341 piles founded in Silurian mudstone and 79 piles founded in basalt. Three pile diameters were used. The approach structures at the ends of the bridges and ramps were

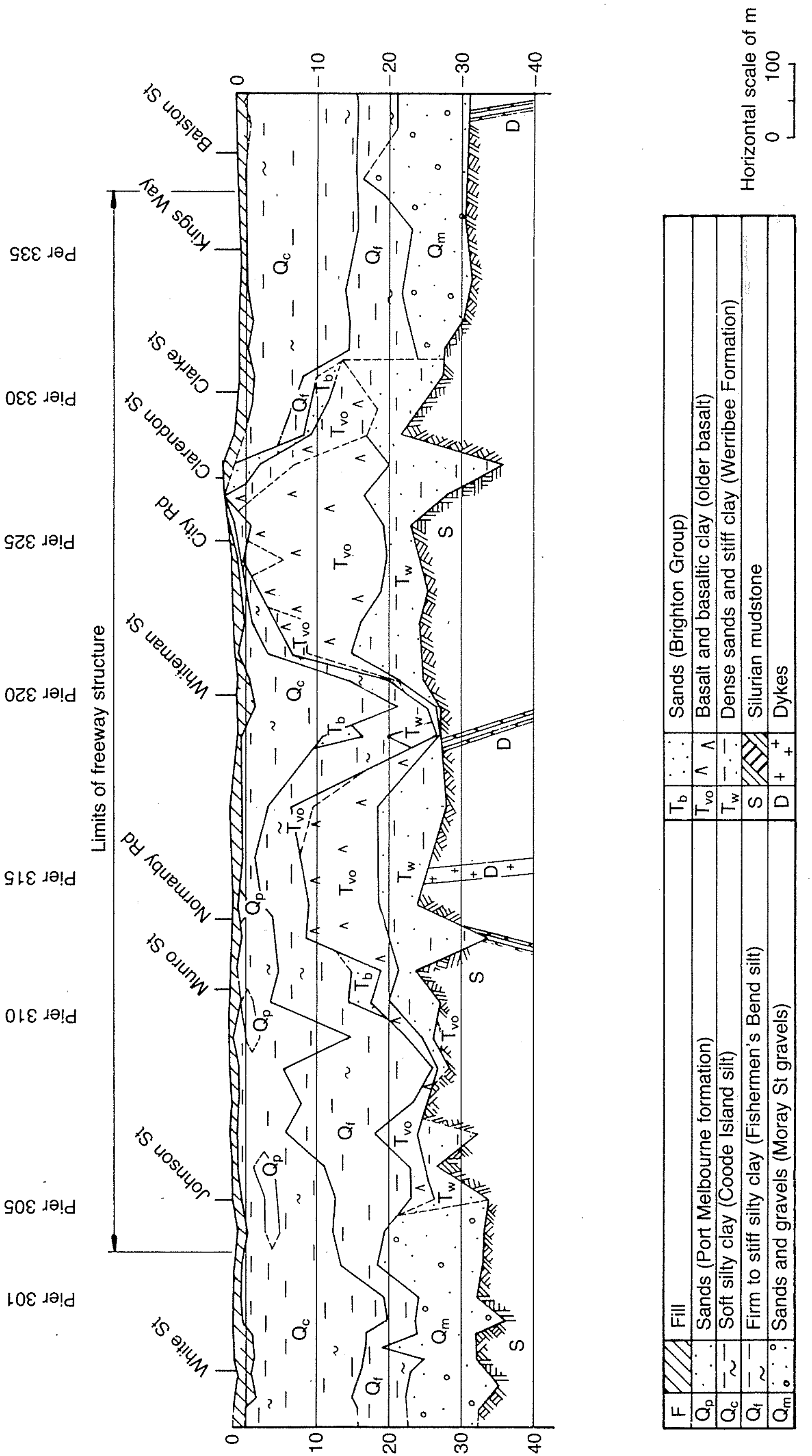


Fig. 2. Geological section along freeway alignment

supported on 1·1 m dia. piles, while the main structures were supported on 1·3 m and 1·5 m dia. piles.

12. The 1·3 m dia. piles are located under the piers which support spans comprising single cell segments: i.e. under the ramps and under the four-lane sections. The 1·5 m dia. piles are under the piers, carrying twin cell segments (refer to part 2 of the Paper[1]).

13. The selection of 1·3 m dia. piles against 1·5 m dia. piles was not dependent on vertical load capacity but was based on lateral stiffness requirements.

14. Each pile is of reinforced concrete construction, with lengths varying from a minimum of 5·0 m to a maximum of 62·7 m.

Site investigations

15. Investigation bores were drilled at every one of the 420 pile positions, with additional bores being drilled when cores from adjacent holes indicated a large variation in rock quality. This was to be expected in view of the extensive folding of the bedrock which produced steeply dipping beds. Holes were wash bored through sediments, using rotary drilling methods to depths of up to 70 m. Conventional 75 mm dia. thin-walled tube samples were taken in the soft to firm alluvial clays, employing a stationary piston sampler to minimize sample disturbance and to improve sample recovery. Standard penetration tests (SPTs) were taken in stiff to hard clays, and also in cohesionless soils. Fig. 3 shows typical bore logs.

16. Because of the importance of lateral stiffness of piles, pressuremeter tests were carried out in the alluvial materials, using both Menard and self-boring instruments to determine soil modulus.

17. To assist in defining stratification of the alluvial soils along the freeway route, electric friction cone penetrometer tests were carried out to refusal at every pier location, using the Roads Corporation's equipment. Fig. 3 also shows typical penetrometer traces. These penetrometer probes enabled accurate identification of the depths of soft to firm silty clay over the freeway route so that negative skin friction loads on piles could be assessed.

18. Within rock strata, 54 mm dia. cores were obtained by means of diamond coring bits. These cores were subjected to a range of tests, including moisture content, point load strength index, unconfined and triaxial compression tests. The prime objective was to obtain rock modulus values for use in settlement-based pile designs, with strength parameters used to check overall capacity. Pressuremeter tests were also conducted within rock strata to provide information on strength and stiffness, and to establish rock design parameter correlations with simpler laboratory tests. Figs 4 and 5 show correlations developed between Melbourne Mudstone modulus, strength and moisture content.

19. Despite a large scatter in test results attributable to the inherent variability of the basalt, correlations were developed between rock pressuremeter modulus and the visual weathering grade of the core. Over 250 pressuremeter tests were carried out in the basalt to give satisfactory correlations.

20. Where bores penetrated through the basalt and into underlying saturated sandy materials, a standard practice of post-grouting investigation bores with a cement grout was adopted to minimize water inflow into pile sockets during construction.

Pile load tests—axial loading

21. At the time of this project, Monash University was involved with investiga-

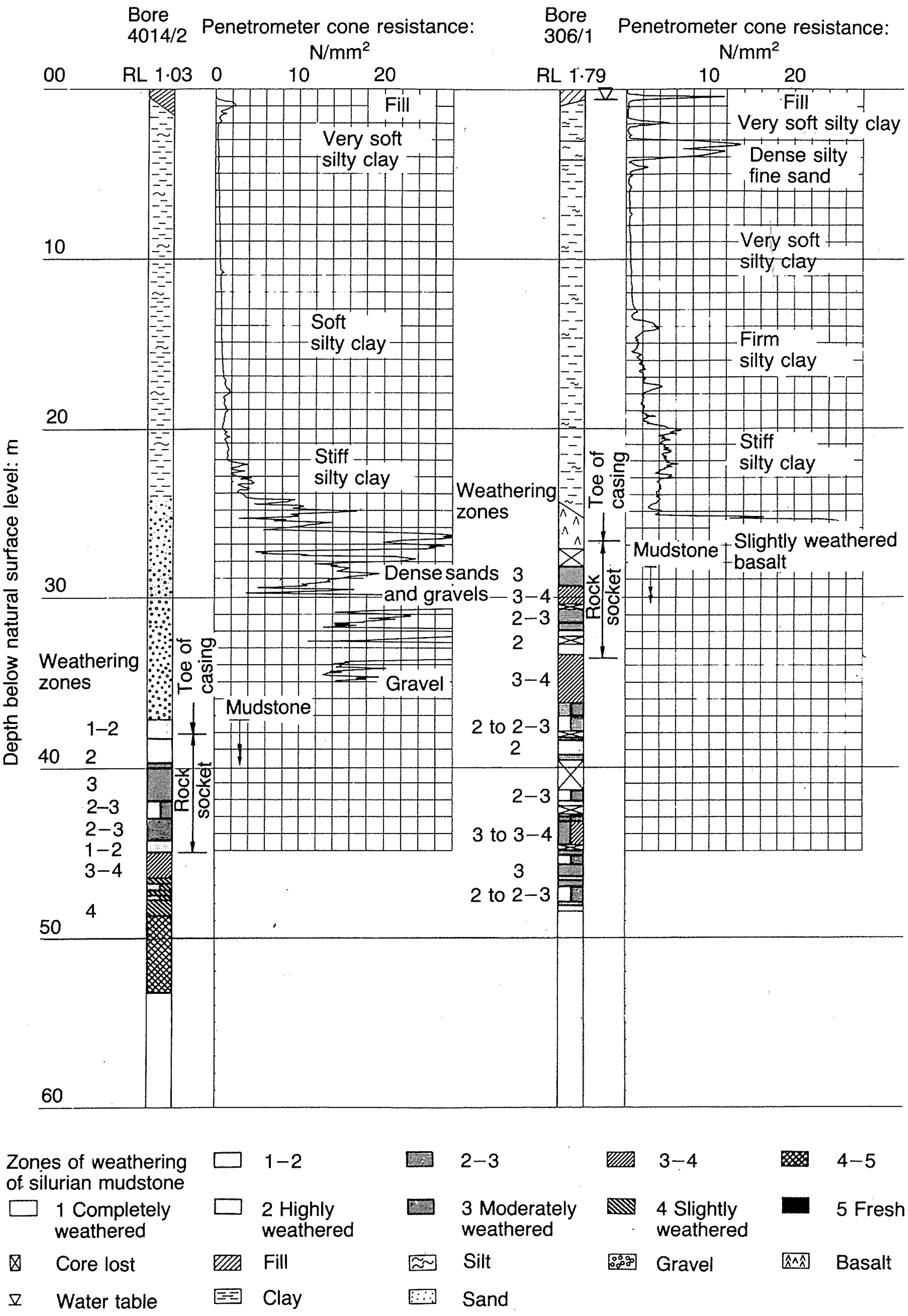


Fig. 3. Typical bore logs and electric cone penetrometer plots

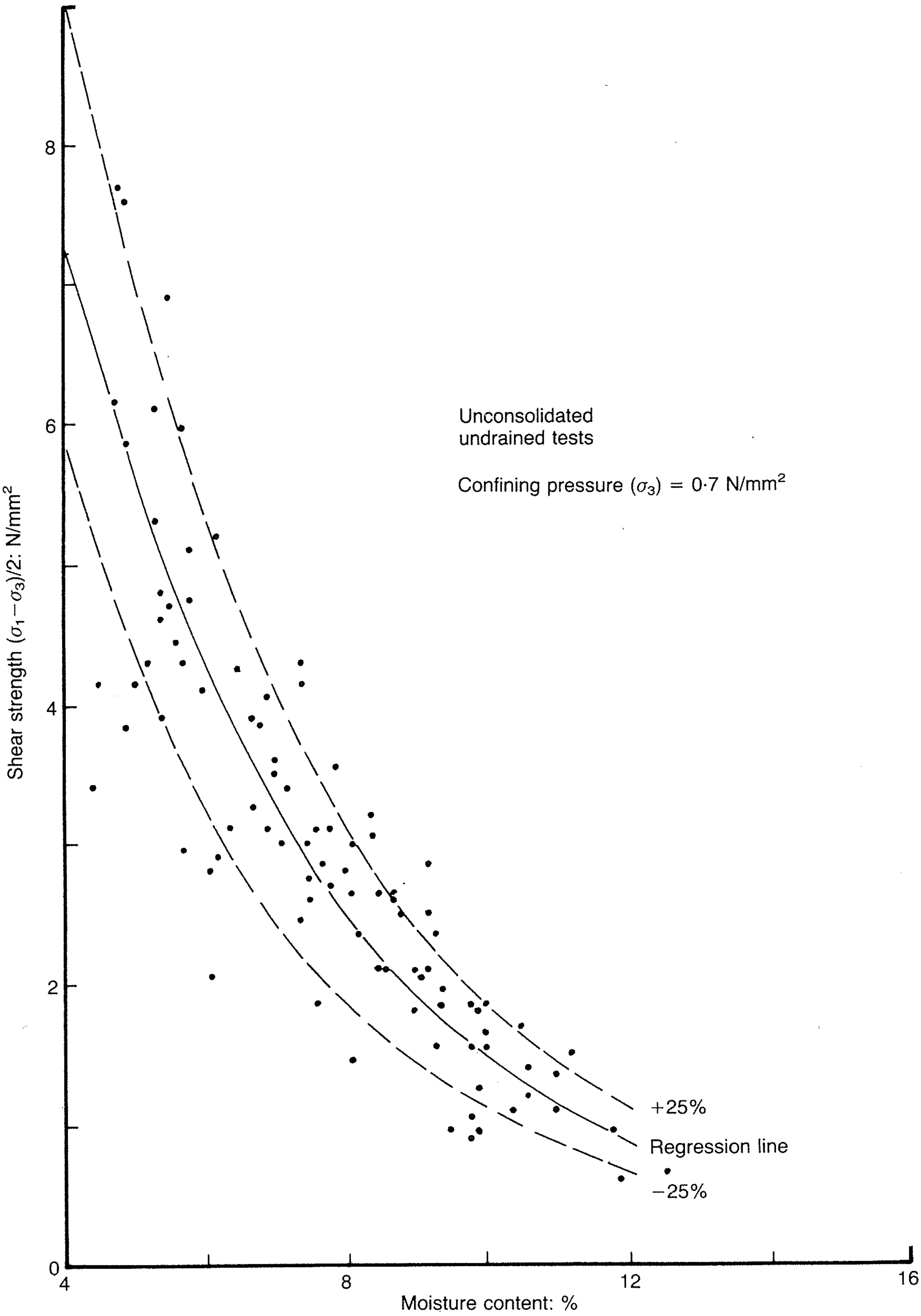


Fig. 4. Correlation between moisture content and undrained shear strength for Melbourne mudstone at West Gate site

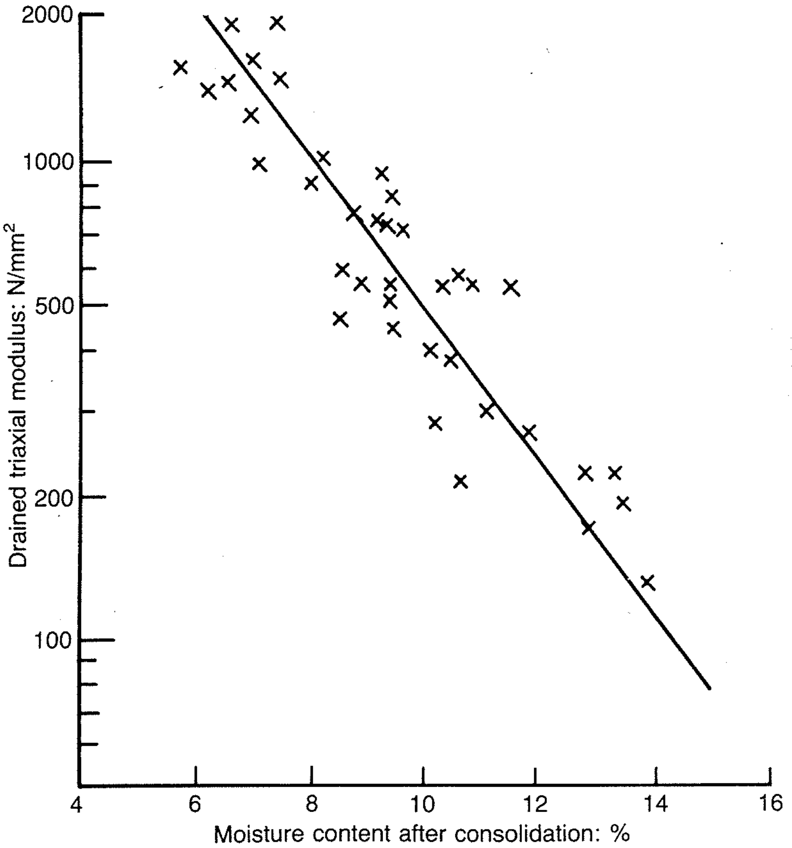


Fig. 5. Correlation of drained secant modulus to 50% strength with moisture content—West Gate site

tions into the load bearing performance of large bored piles socketed into the Melbourne Mudstone. The Roads Corporation embarked on a collaborative investigation program with Monash University into the design and performance of piles socketed into mudstone. The program, which is described fully by Williams,[2] included 41 field pile load tests.

22. Some of the field tests involved the use of retrievable test rigs, which were cast into the shafts of bored pile sockets and loaded against the bases of the holes by inflating a series of large flat jacks. A column separating the side resistance section from the base enabled independent assessment of the side friction and end bearing components of capacity. One of the two test rigs used is shown in Fig. 6. Following testing, the cone of precast concrete was jacked away from the outer annulus of the side resistance section, using the single 480 mm dia. flat jack, enabling retrieval of the lower pedestal and flat jack assembly for further use. Sockets of 1 m and 1·5 m nominal dia. were tested using these rigs, the larger of which had a capacity of over 1500 t. The results of all these tests were used along with data from laboratory tests on the mudstone to establish actual pile design curves for the project.

23. A separate program of investigation was conducted into the behaviour of bored piles founded in the basalt flows, and included six pile load tests. The essential difference between these tests and those for the mudstone founded piles was that the basalt test pile information was used to confirm the adequacy of the design method and the rock strength and modulus values proposed, whereas the mudstone pile tests were used directly to produce design curves for the production piles.

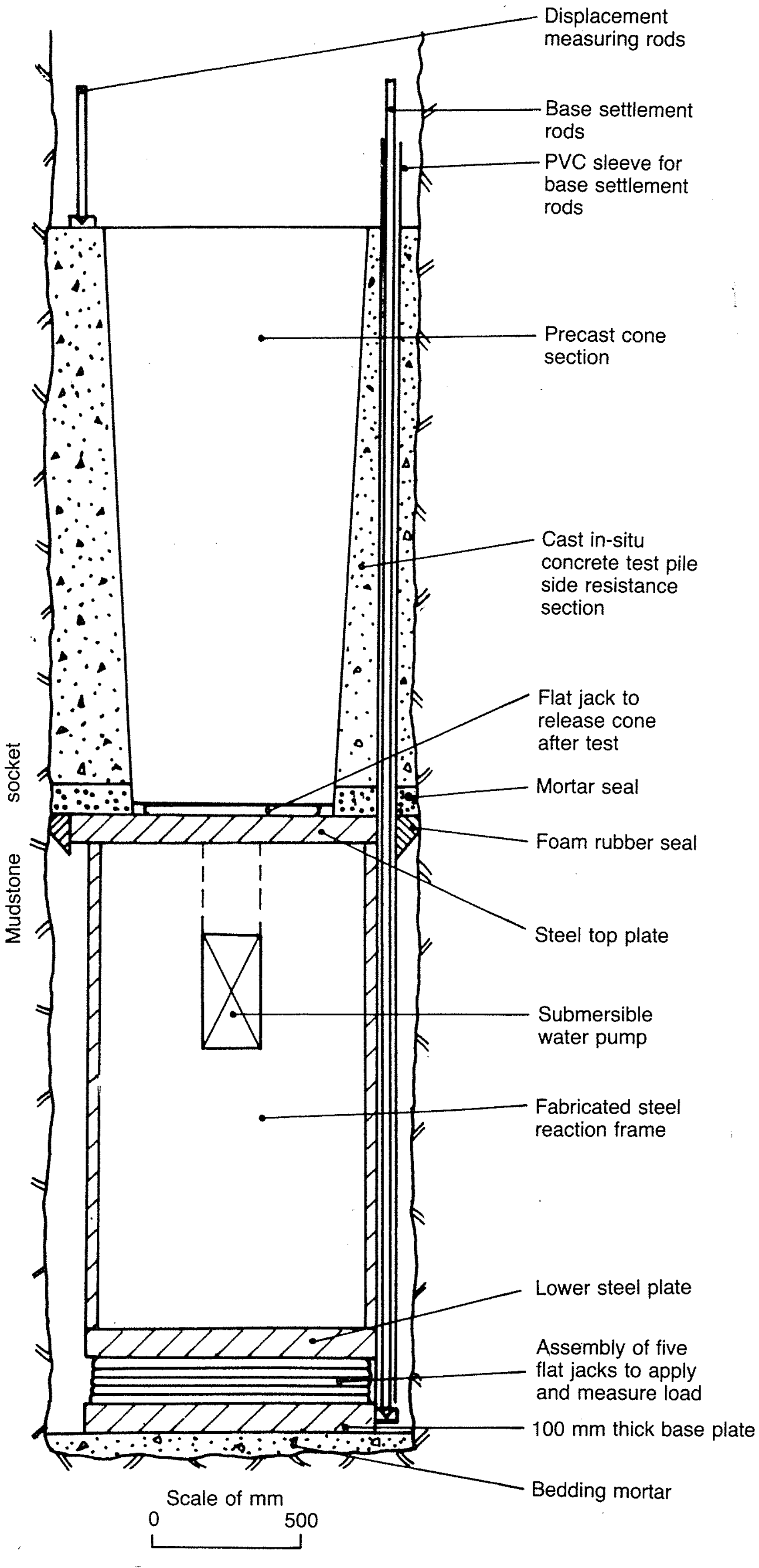


Fig. 6. Retrievable test rig for socketed piles

Fig. 7. Strain gauged reinforcement cage and flat jack assembly being lowered into basalt socket

24. Rock anchored reaction beams were used to provide the test reaction for three of the basalt pile tests. For the other three tests, a series of four 1·15 m dia. flat jacks were used to provide the reaction. One of the flat jacks was sealed and instrumented with a pressure transducer to record load, while the other three cells were used to increase load. Fig. 7 shows a strain gauged reinforcement cage and flat jack cell assembly being lowered down one of the basalt test pile sockets. Following the test, all flat jacks were grouted to replace the oil so that pile socket

side resistance and base sections acted monolithically in service. The tests are described in greater detail by Evans *et al.*[3] and Addis.[4]

Pile load tests—lateral loading

25. During the early stages of the project, several critical design and construction aspects were identified, which required a reliable estimate of the lateral load–deflexion behaviour of the large bored piles. These aspects included the amount and location of reinforcement within the piles, elastomeric bearing design, bridge expansion joint spacing, anchor pier spacing, and the way in which precast superstructure segments could be cantilevered out from each pier.

26. The lateral load–deflexion behaviour of piles is particularly difficult to predict, although various theoretical models exist to assist design. The Poulos[5] model was used initially to conduct sensitivity analyses for lateral load behaviour, but it was soon found that design and construction options were greatly influenced by the soil stiffness and strength profiles used. A series of full-scale lateral load tests was undertaken to establish suitable soil models. A special jacking frame was constructed to enable pairs of piles to be pulled together by means of cables. In this way, piles 45 m apart could be loaded simultaneously. The arrangement for the lateral load tests is shown in Fig. 8.

27. A total of five series of lateral load tests were carried out, the load being applied at or above ground level as required. Lateral loads of up to 45 t were applied. Some piles were strain gauged to determine pile bending moment distributions and, in all cases, accurate measurements of pile lateral movements were recorded, using a system of displacement transducers and invar wires attached to datum columns behind the piles. All instrumentation was connected to a data logger, with output to a computer which facilitated data reduction.

Design of foundations

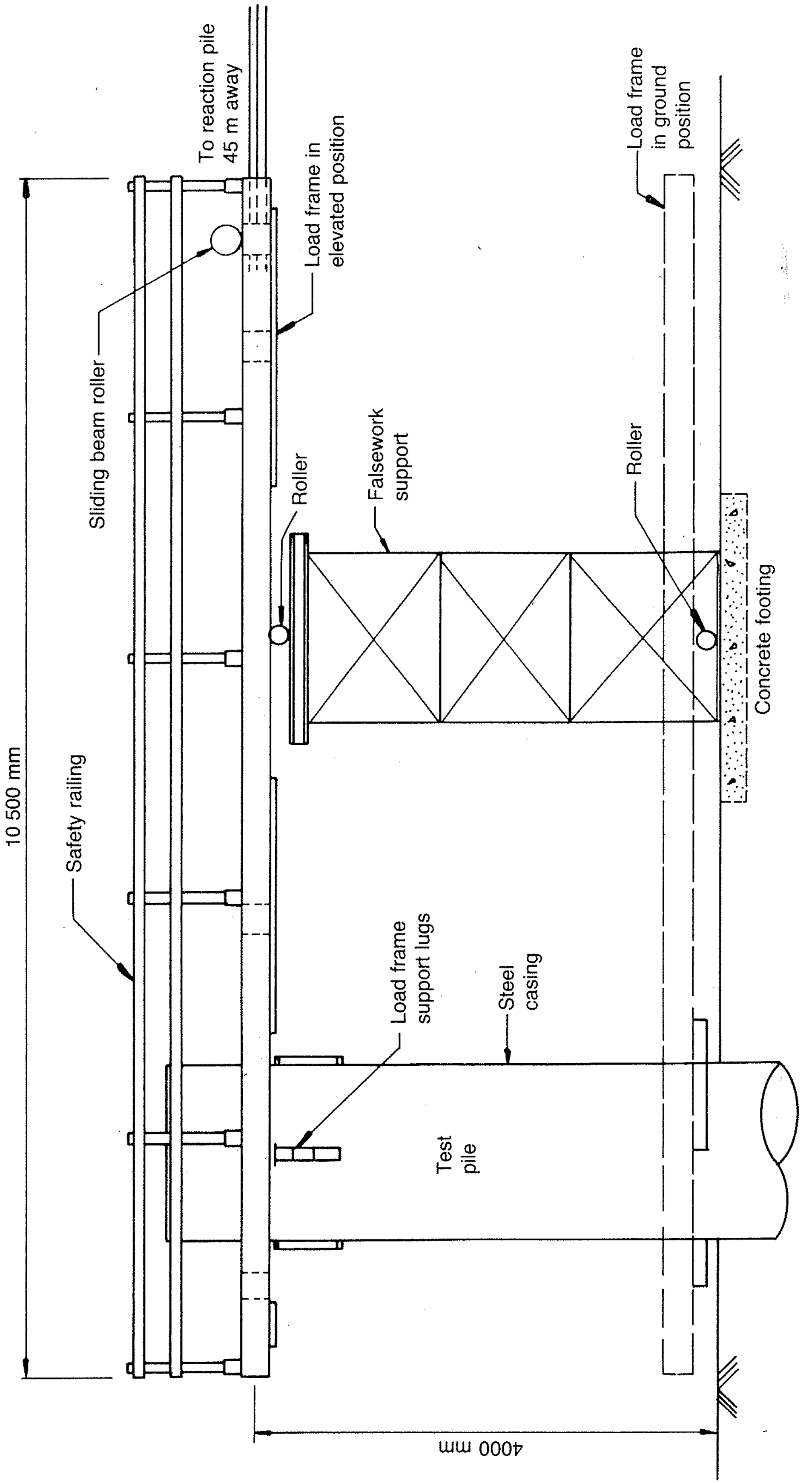
Design criteria

28. The design of the bored piles supporting the elevated freeway structures was controlled primarily by permissible settlement limits rather than by pile capacity. Torsion in the single spine sections was a major factor. The strict differential settlement limits listed in Table 1 emphasized the need for a detailed understanding of the load–settlement behaviour of the bored piles.

29. Pile socket loads comprised live and dead loads from the structure, together with an allowance for negative skin friction (NSF) loads that arise from long-term settlement of the soft to firm alluvial clays. In some cases, the NSF loads were effectively absorbed by positive skin friction in stiff or dense sediments overlying the rock. However, in most cases for the mudstone founded piles, some

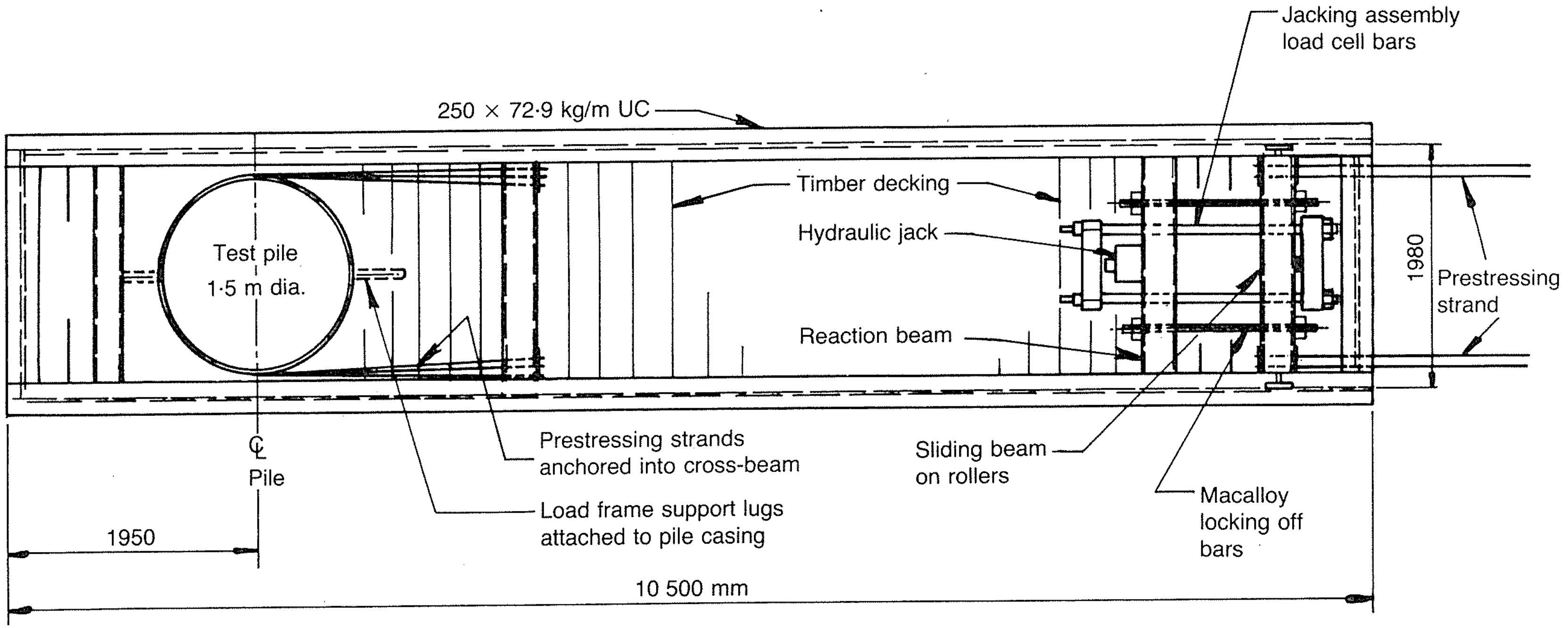
Table 1. Differential settlement limits for foundations

1. Between piers along structure after span continuity	15 mm
2. Between adjacent piles beneath a common pile cap prior to span continuity	5 mm
3. Between adjacent piles beneath a common pile cap after span continuity (short term)	3 mm
4. As for 3, but creep and relaxation effects acting to reduce bending moments and torsional moments (long term)	6 mm



(a)

Fig. 8 (above and below). ***Lateral load test system: (a) elevation; (b) plan***



(b)

Fig. 8—continued

NSF load was calculated to reach the socket. These additional loads of up to 6300 kN occasionally exceeded combined live and dead loads from the structure. The maximum socket design load for any pile was 14 800 kN, with most piles having design loads in the range 3000–10 000 kN.

Rock socket design

30. *General.* Before this project, socketed piles were designed on the basis of ultimate strength, with factoring down to an allowable design load.

31. In rock sockets, as with driven or bored piles in clay, the mobilization of base resistance occurs at greater pile settlements than the mobilization of side resistance. Unlike the situation with clays, variable weathering, effects of rock jointing and clay seams, socket wall roughness and variables associated with construction practice make the analysis and design of rock-socketed piles a more complex issue.

32. If base resistance is neglected, as a pile socket is loaded, the load–settlement behaviour may be assumed to be initially elastic, with no slip or yielding at the socket wall. At higher load levels, some yielding of the rock occurs. With very smooth walled sockets, an abrupt loss of capacity can be experienced once yielding has started. These mechanisms are shown in Fig. 9. In rough sockets, dilatation of the socket occurs under load, resulting in an increase in normal stress and shear resistance capacity until there is shearing through the rough socket asperities.

33. Surface softening or smearing of the walls with mudstone tailings or residual bentonite can have a significant influence on the above mechanisms, weakening the rock at the critical concrete/rock interface and lowering the wall system stiffness and strength so that the normal stress increase due to dilation is inhibited.

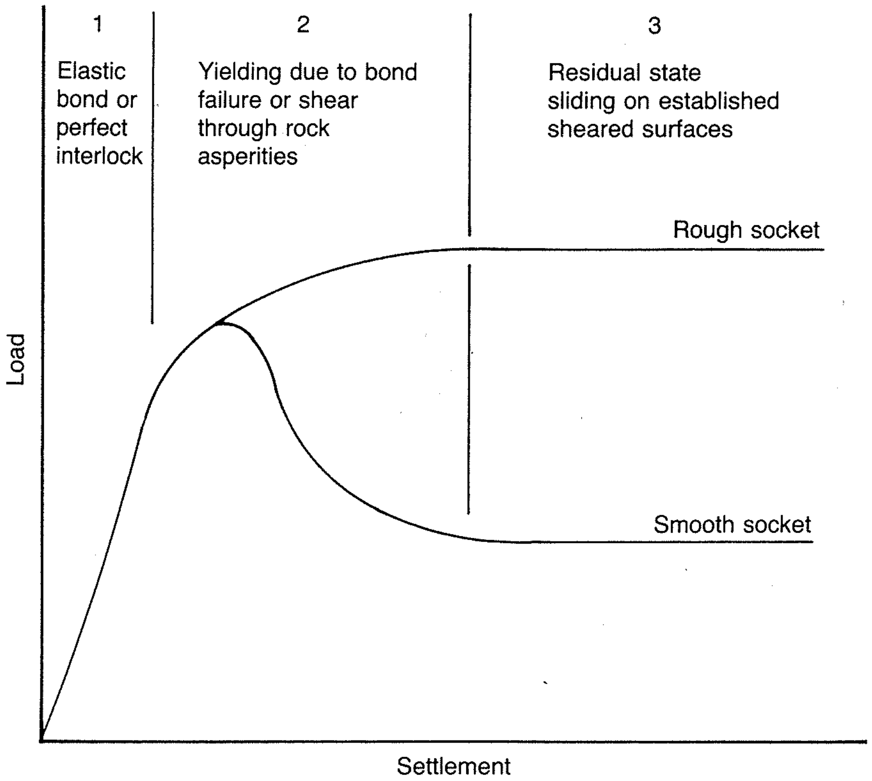


Fig. 9. Load–settlement characteristics for socketed piles—side resistance only

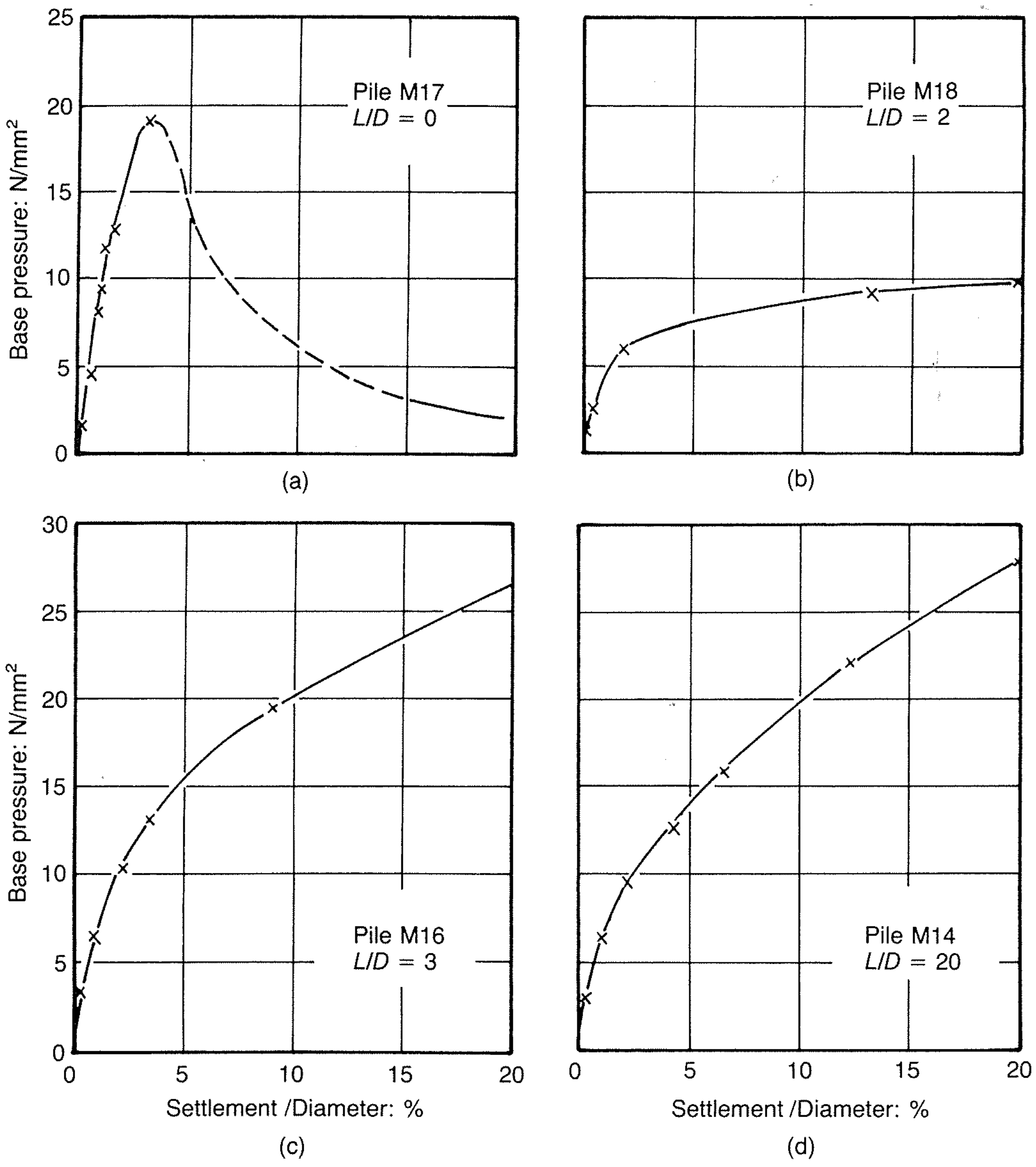


Fig. 10. Effect of embedment ratio on load–settlement curves for end-bearing piles

34. For the base resistance of rock-socketed piles, Williams[2] found from pile tests that the depth of embedment of the pile base below the rock surface had a marked influence on the load–settlement behaviour. In these tests, the sides of the piles were isolated from the socket walls, where appropriate, so that base resistance characteristics alone could be studied at various embedment depths. For bearing at the rock surface ($L/D = 0$), an abrupt failure occurred with a loss in sustainable base load to about 15% of the peak value. For embedment depths of greater than about two pile diameters, abrupt failure did not occur. Rather, a plastic yielding developed in which the load–settlement curve flattened off gradually, without a clearly defined point of rupture (see Fig. 10).

35. The designs of mudstone and basalt founded piles were similar, in that theoretical elastic solutions were used to determine side and base resistance components of capacity. Separate elastic solutions were used for each rock type, the basalt-founded pile solutions incorporating a provision to model the alluvial sediments beneath the rock.

36. *Basalt founded piles.* The basalt founded piles were designed to operate in a pseudo-elastic range, using rock mass modulus values established by pressuremeter testing. Design side and end bearing stresses were assigned by factoring down estimated ultimate side resistance and base resistance values by 2 and 6 respectively. Reliable ultimate resistance values were not available for basalt founded piles, and were therefore estimated on the basis of pressuremeter test results, rock strength/side resistance correlations developed for other rock types and, in the case of highly weathered basalt, limiting strength/side resistance reduction factors established for strong soils.

37. The factor of 6 on assumed ultimate base resistance for the basalt piles reflects the slow development of base resistance with settlement and the emphasis on small design differential settlements for the structure. Design values (Table 2) were reviewed in the light of the results of load tests on six basalt founded piles. While design stresses for side and base resistance proved to be reasonable, the rock stiffness proved to be about half that adopted for design on account of the effects, with time, of non-linearity and creep. However, the compression of alluvial sediments beneath the basalt flows proved to be negligible and had been significantly overestimated in the designs.

38. *Mudstone founded piles.* For the mudstone founded piles, the design initially involved elastic theory to establish load–settlement relationships and the distribution of side and base resistance components of capacity. Suitable design charts were developed from finite-element solutions for rock socketed piles, in which side resistance only, base resistance only, or combined side and base resistance piles were modelled.[6]

39. At this point, side and base resistance components of capacity were reduced to take into account non-linear load–settlement behaviour shown to be present at relatively low stress levels and attributable to plasticity effects (see Fig. 11). Load–settlement curves from field pile load tests were used to prepare normalized side shear resistance and base resistance curves for design purposes. The ultimate side resistance value (f_{su}) and the 1% of diameter base settlement stress (f_{bl}) were the parameters selected to non-dimensionalize the side and base resistances. It was found, after normalizing the field test curves, that pile behaviour for a wide range of rock strengths and pile embedment ratios lay within a narrow band of upper and lower bound curves, from which average design curves were constructed (see Fig. 12).

40. The mudstone founded pile design procedure was iterative, starting with an assumed socket length and design settlement, then working through elastic solutions and relaxation curves to establish final side load and base load resistance components. If the sum of these components was not close to the nominated design socket load, a new socket length was chosen and the design process repeat-

Table 2. Design stiffness and resistances—Basalt

Degree of weathering	Allowable side resistance: N/mm^2	Allowable base resistance: N/mm^2	Elastic modulus: N/mm^2
Highly weathered	0·125	1·5	440
Moderately weathered	0·40	2·2	750
Slightly weathered	0·70	4·5	1400

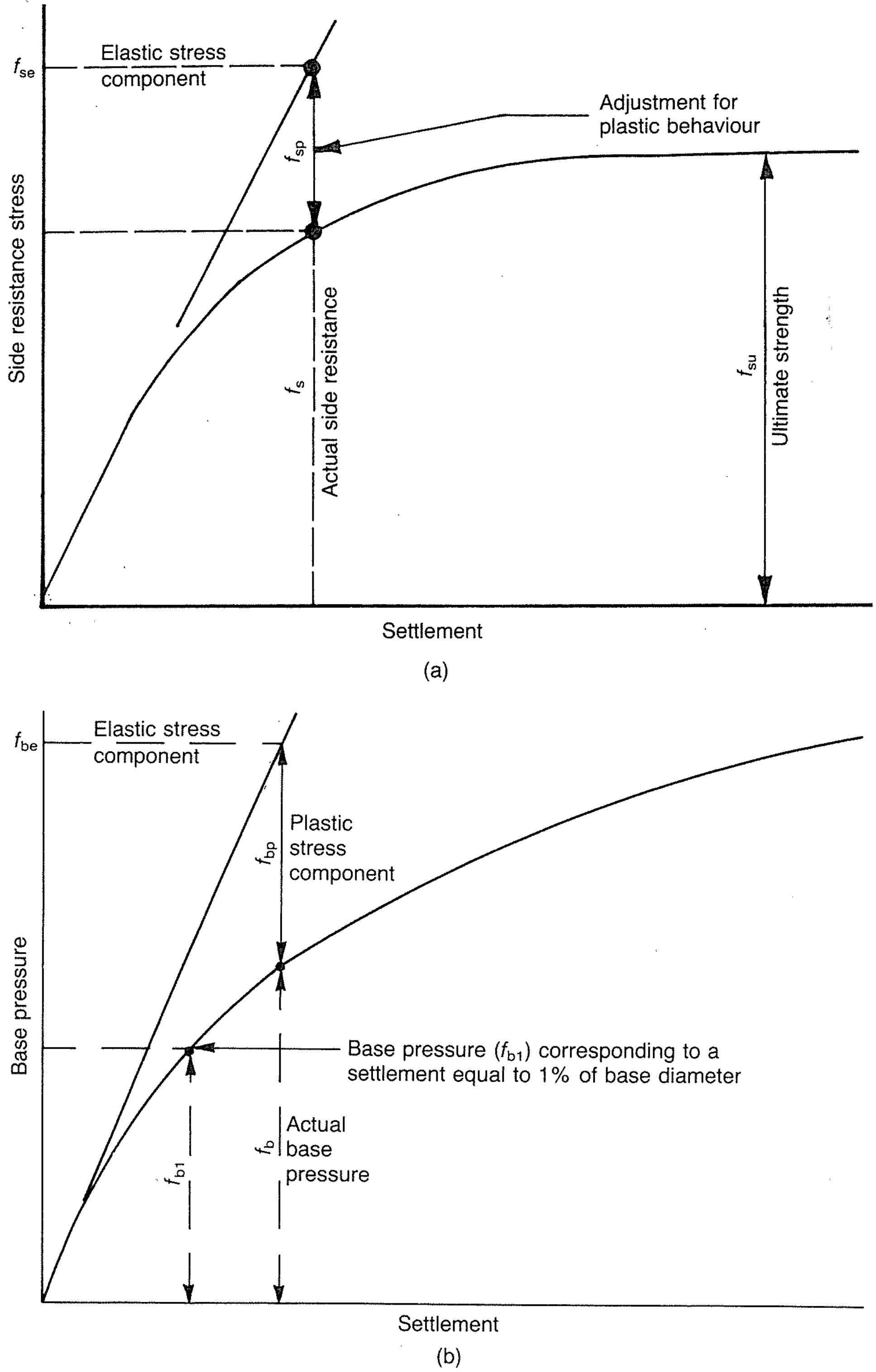


Fig. 11. Definition of terms used to normalize socketed pile design curves allowing for plasticity effects: (a) side resistance curve; (b) base resistance curve

ed. A design settlement of half the specified maximum was adopted to account for variation in rock strength and modulus values, and the spread of experimental data used to construct relaxation curves. The settlement based design approach usually controlled socket proportions, but in the case of large pile loads in very weak rock, capacity considerations were used to establish socket lengths. In such

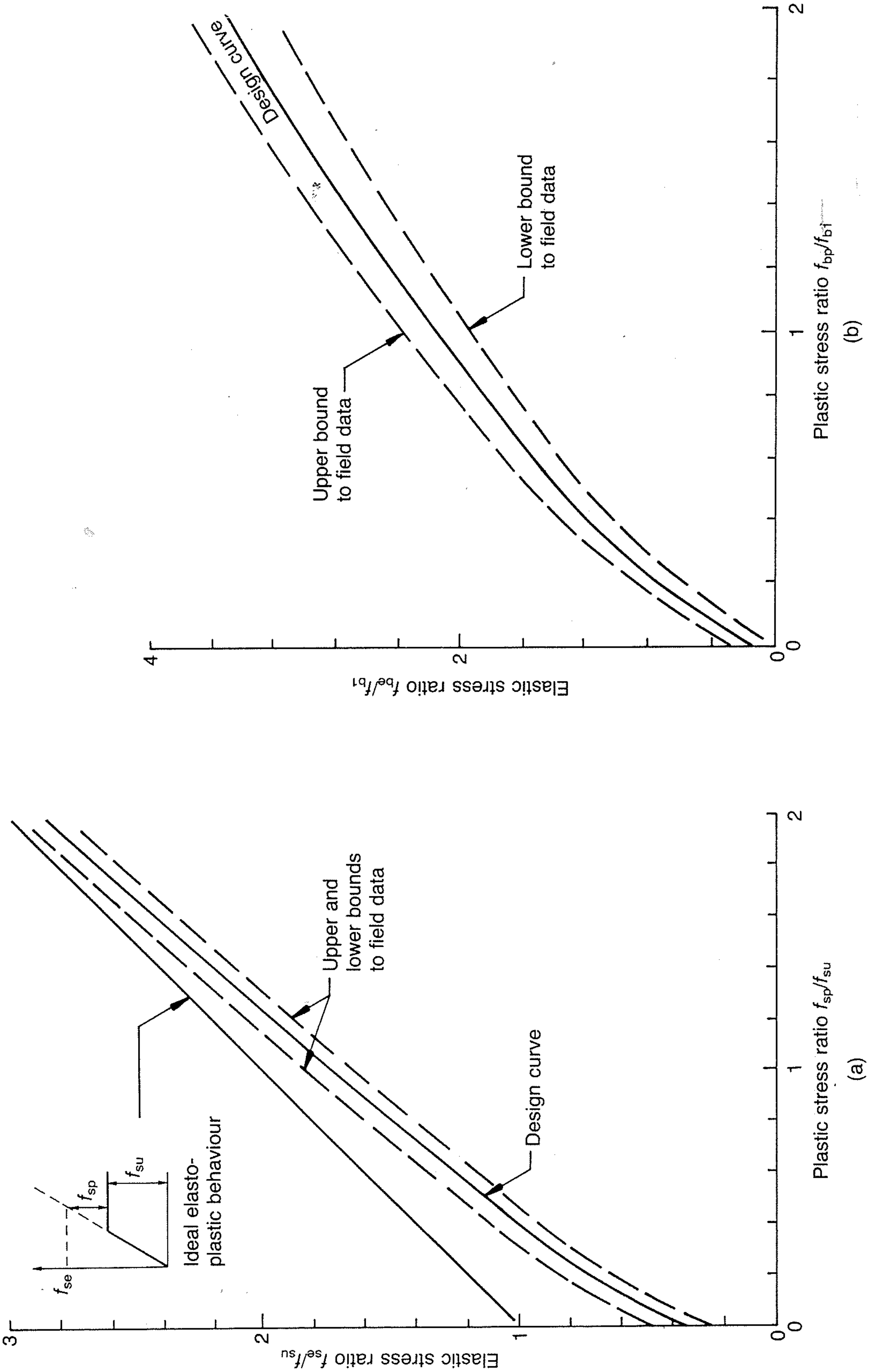


Fig. 12. Normalized design curves for socketed piles in mudstone: (a) side resistance; (b) base resistance

cases, the settlement based approach required significant extrapolation of design curves, which was judged to be inappropriate.

41. All designs for basalt founded piles were carried out by the Roads Corporation's geotechnical engineers and were proof engineered by consultants. For the mudstone founded piles, designs were carried out by both geotechnical consultants and the Roads Corporation, using the design method described previously. Proof engineering of these designs was also carried out by consultants.

Driven piles

42. During investigation drilling at the four pile locations at pier 315 in the basalt flow under the railway yards site, the northernmost bore revealed only 2 m of basalt. This sudden thinning of the basalt flow to the north of the pier resulted in the drilling of a series of 13 additional bores to define accurately the extent of the flow within the pier area. At the southern edge of this pier, the basalt was 11 m thick.

43. Because of the strict differential settlement tolerance for adjacent piles beneath a common pile cap, the option of having one pile of the northern pile cap founded on the basalt flow and one pile founded in the bedrock about 20–25 m deeper was not favoured. To complicate matters further, extensive zones of weak porphyritic dyke material existed at this location in the mudstone bedrock. Finally, a group of 24 prestressed driven piles was designed to replace the northern pile at the pier. This group of driven piles had a similar axial stiffness to the bored pile at the south side of the pier.

Lateral load resistance

44. Pile design for lateral load resistance was carried out using the elastic continuum method of Poulos.[5] A refinement of the purely elastic idealization of the pile–soil system allows soil elements near the ground surface to be modelled by means of limiting, or yield, pressures. A reduction in soil modulus values can be used to approximate non-linear behaviour and creep effects. Poulos took an active interest in the laterally loaded pile test program, and was engaged by the Roads Corporation to advise on critical aspects of the load test program as well as to give an independent interpretation of the load test results to determine appropriate soil stiffness and yield pressure distributions.

45. Soil investigations revealed differences in soil modulus of more than a factor of 10 for the same soil layer, depending on the type of test performed. The lateral load test results demonstrated that lateral deflexions and ground line rotations were only about one tenth of the values calculated initially for any given combination of lateral load and applied bending moment at the heads of piles. Self-boring pressuremeter soil modulus values gave the best prediction of behaviour.

46. Some of the lateral load tests involved maintaining load periods of 2–5 days to assess time-dependent effects, and it was found that if backfigured short-term soil modulus values were halved, a reasonable agreement with long-term pile lateral deflexions was obtained. Fig. 13(a) shows one of the lateral load–deflexion curves for a 1·5 m dia. pile subjected to a load of 450 kN applied about 4 m above ground level. Further details of the lateral load test program are described by Addis.[4]

47. Strain gauge instrumentation down two of the test piles allowed bending moment distributions to be evaluated. Fig. 13(b) shows bending moment curves

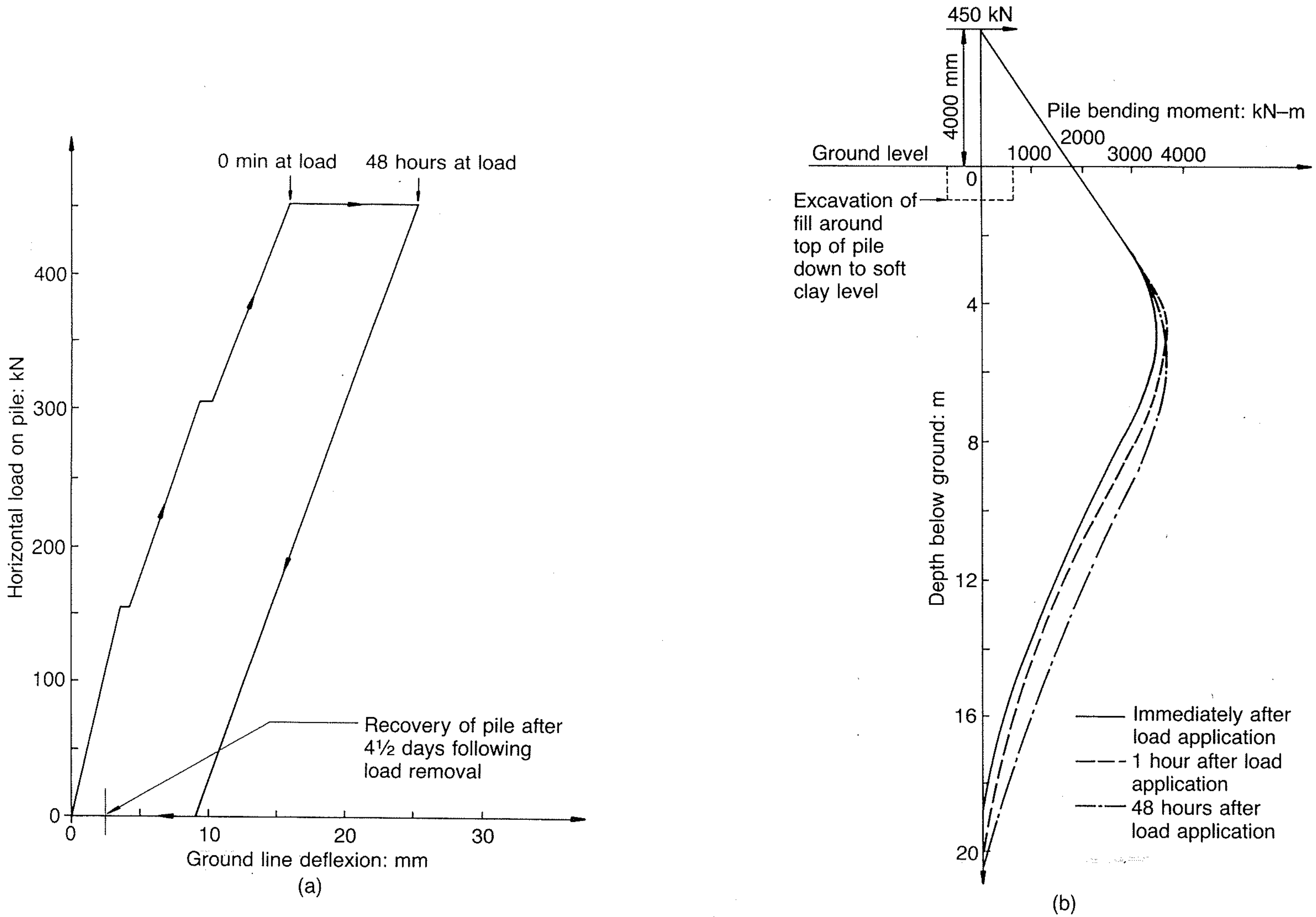


Fig. 13. Laterally loaded pile test results for load applied 4 m above ground level: (a) load–deflexion curve; (b) bending moment distribution

for one of the test piles. The tendency towards increasing bending moment at any particular level with time was attributed to creep and relaxation of the most heavily stressed soil zones near the ground surface. Analysis of the strain gauge information resulted in a change in reinforcement cage proportioning over the upper part of the piles. Socket regions were unreinforced.

Construction of piled foundations

General

48. Pile socket lengths in the mudstone vary from 2·2 m to 25·3 m, and are straight-sided. Socket lengths for the basalt founded piles vary from a nominal 0·8 m in nearly fresh rock to 12·6 m in highly weathered rock with clay seams. Each pile consists of a steel casing which was driven through the overlying silt/sand/gravel strata and into the founding rock. Casing lengths were 12 m and were joined by full strength butt welds. Material from within the casings was excavated progressively, as necessary, to assist in the penetration of the casing. Excavation of soil and rock within the casings and the socket regions was by rotary drilling and/or chopping and grabbing techniques.

Basalt founded piles

49. These piles were excavated by means of bucket augers and chop and grab techniques, and were usually dewatered for inspection. The basalt, on account of its closely spaced joints, was permeable enough to require concentrated pumping during inspections, but, in most cases, water inflow to the socket could be controlled temporarily to allow adequate inspection. Exceptions occurred where investigation bores had not been grouted after drilling, and the combined inflow of water from the socket region and through the open bore hole was excessive.

50. Extensive clay seams and patches through otherwise sound rock were found to be very susceptible to softening and deterioration following pile excavation, this being exacerbated by socket water inflow through the rock joints. In several cases, the socket wall deterioration caused some overbreak but, if minimum time was permitted between pile excavation and concreting, overbreak was generally not severe.

Mudstone founded piles

51. The steel casings were seal driven a minimum of 1·0 m into mudstone, but at some pile locations where the quality of the mudstone was poor, casings were driven up to 4 m into the bedrock.

52. The casings were fabricated from 12 mm thick plate, but in some instances, 16 mm thick plate was used for the leading section of casing. The toe, or cutting edge of the casing, was stiffened with an additional 25 mm thick plate for a length of 1 m to resist the high driving forces required to seal the casings into the mudstone.

53. The steel casings acted as excavation liners through the sand and silt layers, and contributed to pile stiffness during out-of-balance stages of segment erection. They do not contribute to the pile stiffness under service load conditions, and while they enhance the durability of the piles in the short term, they cannot be relied on for long-term durability. Concrete of cylinder strength grade 30 N/mm^2 together with sulphate resistant cement and cover to reinforcement of 75 mm were

considered sufficient to provide adequate durability. Soil conductivity tests taken during foundation investigations revealed that the ground conditions were relatively passive.

54. Casings were installed by percussion (using an independently mounted 'bell mouth' diesel hammer), vibration or oscillation techniques. The technique used for a particular pile was determined by two criteria: the limitation of ground vibration, and the noise emanating from the construction works.

55. Where piles were located close to buildings or sensitive underground services, the measured ground peak particle velocity was limited to 5 mm/s or less at the edge of nearby buildings, or to 10 mm/s or less at the location of the service. The contractor was required to adjust his casing installation procedure accordingly. Peak particle velocities were measured directly from geophones installed 150 mm below the ground surface next to buildings and at the level of the underground conduit for services. For piles near some major services, oscillatory methods were mandatory after pre-drilling through the surface fill.

56. Properties adjacent to the freeway reserve were inspected and monitored during construction. All masonry buildings and those containing sensitive equipment on 39 properties located partly or wholly within 30 m of the freeway were inspected by an independent Architect. The condition of buildings was recorded photographically, and where significant cracking was present, the width and length of cracks were measured and recorded. The inspection reports were compiled with the permission and assistance of the property owners, and were signed by the owner, the Architect and the Resident Engineer as a true and accurate record of the condition of the building before the start of construction work. Copies were retained by each party.

57. Levels were taken at two-month intervals on the corners of all buildings to check for any foundation movements. Widths of selected cracks were monitored for any movement that could have been related to construction activities. Although some complaints were received from owners, after discussion with them, which included an examination of the vibration records and pre-construction inspection reports, they were in all cases satisfied that no damage had been sustained. The effort involved in taking these measures was of benefit to both the owners and the Roads Corporation, and was also welcomed by the construction contractors.

58. In order to reduce the effects of noise to which the neighbourhood was exposed and which emanated from the installation of casings, driving was permitted only during relatively noisy peak traffic periods. In some areas, percussion of casings was not permitted and casings were advanced by oscillatory or vibratory methods.

59. Early in the project, it was decided to construct eight mudstone founded piles by separate contract, principally to take advantage of the closure of Montague Street during the upgrading and reconstruction required as part of freeway access improvements. Four of these piles were located in the Montague Street median. The other four piles were constructed for the purposes of axial and lateral load testing to provide design and performance information. This early pile construction also permitted assessment of construction techniques used to excavate piles and to form rock sockets in unstable fault zone material, revealed by test boring at one of the pile locations. The contractor for the construction of these piles proposed to use bentonite to maximize socket stability, a technique unfamiliar at that time to the Roads Corporation. Several other poor quality rock areas

had been discovered at pier positions along the route, and so the opportunity was taken to assess bentonite construction procedures.

60. It was proposed to inspect two of the early contract pile sockets to check wall roughness and rock quality. One socket collapsed after removal of bentonite and before inspection, thus highlighting the potential problem of unstable sockets. The other socket, in stronger rock with more favourable jointing, showed that the walls had adequate roughness to satisfy design requirements; therefore, additional work using reaming teeth fitted to the drilling bucket was not required.

61. Diamond coring of these piles after construction revealed segregated bands of concrete at some levels, indicating that better control of tremie operations would be required during the ensuing main piling contracts. Also, a review of design loads before the first main piling contract yielded significantly larger loads than had been designed for these piles. Static and dynamic testing confirmed that seven of the eight piles were not capable of sustaining design loads within the settlement limits, and these piles were replaced.

62. For the ensuing piling contracts, four methods for constructing the sockets of the piles were specified; in fact, only three methods were adopted.

(*a*) Classification 1 piles permitted dewatering and down-hole inspection of the excavated pile founded in rock considered to be stable. Several Class 1 sockets were reclassified during construction to Class 3B or 3W, on account of the partial collapse of sockets during excavation or dewatering. Inspection enabled a correlation between core information and actual conditions, as well as a verification of the cleanliness of socket walls and bases.

(*b*) Classification 2 piles permitted hand excavation and concrete lining in 1·5 m stages in founding material likely to be unstable. No sockets were actually built using this method.

(*c*) Classification 3B piles required excavation and concreting to be performed under bentonite in material likely to be unstable.

(*d*) Classification 3W piles were constructed as for Class 3B, except that water was used instead of bentonite.

In the case of Class 3B and 3W piles, the level of the stabilizing water or bentonite slurry was not permitted to fall more than 1 m below ground level during excavation. This minimized the likelihood of socket wall collapse.

63. For Class 1 sockets, because the bases were inspected for cleanliness, the design assumed full end-bearing resistance and side friction in the socket.

64. Classifications 3B and 3W applied to piles where the bore information indicated that the walls of the socket were likely to be unstable and it was therefore too dangerous to dewater the piles to inspect the sockets. Since the bases of these piles could not be inspected physically to ensure their cleanliness and geology, the initial socket designs ignored end-bearing resistance and relied solely on side resistance. For piles of this classification, the socket was excavated with the pile filled with water or bentonite to natural surface level. On completion of socket inspection, the base was airlifted and, where bentonite was used, the bentonite was recirculated to remove rock fragments and fines within the pile. The concrete was then cast in two stages in the same manner as for Class 1 piles described below.

65. Procedures for inspecting sockets and cleaning the bases of Class 1 piles were rigidly specified because of the inherent dangers. Inspection was not allowed to begin until at least one hour after the completion of the dewatering operation

and only after soundings had been taken which indicated that there was no inflow of material into the socket. Immediately before inspection, venting of the hole with compressed air was carried out to flush out any foul air (mainly carbon dioxide).

66. An inspection shield was specified for each pile diameter. This was a circular casing with a length of 15 m and a wall thickness of 16 mm. Its diameter was 75 mm less than the internal diameter of the casing. Supporting legs, 1 m long, were attached to the bottom of the shield to keep it clear of the base of the socket. Small openings in the shield, reinforced with steel bars, enabled geotechnical engineers to inspect and sample the material in the socket region.

67. The inspection shield had to extend at least 2 m up into the permanent steel casing of the pile, and personnel were trained thoroughly in operating and emergency procedures. If there were any doubts about the condition of a socket or of the likelihood of a 'blow-in', or if the operating procedures could not be performed, then the classification of the socket was changed to a Class 3, the socket was redesigned and construction completed under the revised status.

68. Following inspection, the pile was refilled with water and the base cleaned again by air lifting. The unreinforced socket was cast to a level at least 3 m into the steel casing by the underwater tremie method, using concrete of 175 mm slump. Water was then pumped out and the top surface of the concrete was broken back to sound concrete. The prefabricated reinforcement cage was then inserted and the remainder of the pile was cast in the dry, using 80 mm slump concrete with internal vibration.

69. In all cases, careful attention was given to examination of the rock quality and the nature of jointing in order to determine the most suitable method of construction and, therefore, of pile classification. Only one pile could not be completed successfully owing to problems that occurred during construction.

Formation of sockets by ream and line technique

70. The ream and line technique for forming sockets in unstable mudstone was developed to minimize the risk of major collapse of the socket walls. The method involved the progressive excavation and concrete lining of the socket in stages without the need for dewatering. The concrete lining was formed by over-reaming the partly excavated socket, filling with concrete and, after initial hardening, drilling through the concrete to the specified socket diameter, thus leaving a residual annulus or liner of concrete supporting the unstable socket wall.

71. In one case, that of pile 422/2, foundation investigation revealed that the socket would be located in fault zone material, and that the founding mudstone was broken and very closely jointed. Joints dipped steeply and were slickensided. Accordingly, the pile design required the steel casing to be driven 17 m into mudstone, instead of the normal 1–2 m, resulting in an installed length of casing of 43 m, with a socket length of 17 m.

72. Unfortunately, attempts to install the casing to the design toe level had to be abandoned 5 m above level when it was found that the cutting edge was distorting into an oval shape. Excavation of the socket to the final base level in one operation was unacceptable because of the instability of the socket material and the risk that the capacity of the adjoining piles (422/1 and 422/3 which were under construction) could have been affected by a major collapse of the socket (Fig. 14). The construction of the socket by means of the ream and line technique was considered to offer the best solution to this problem, and was successfully implemented.

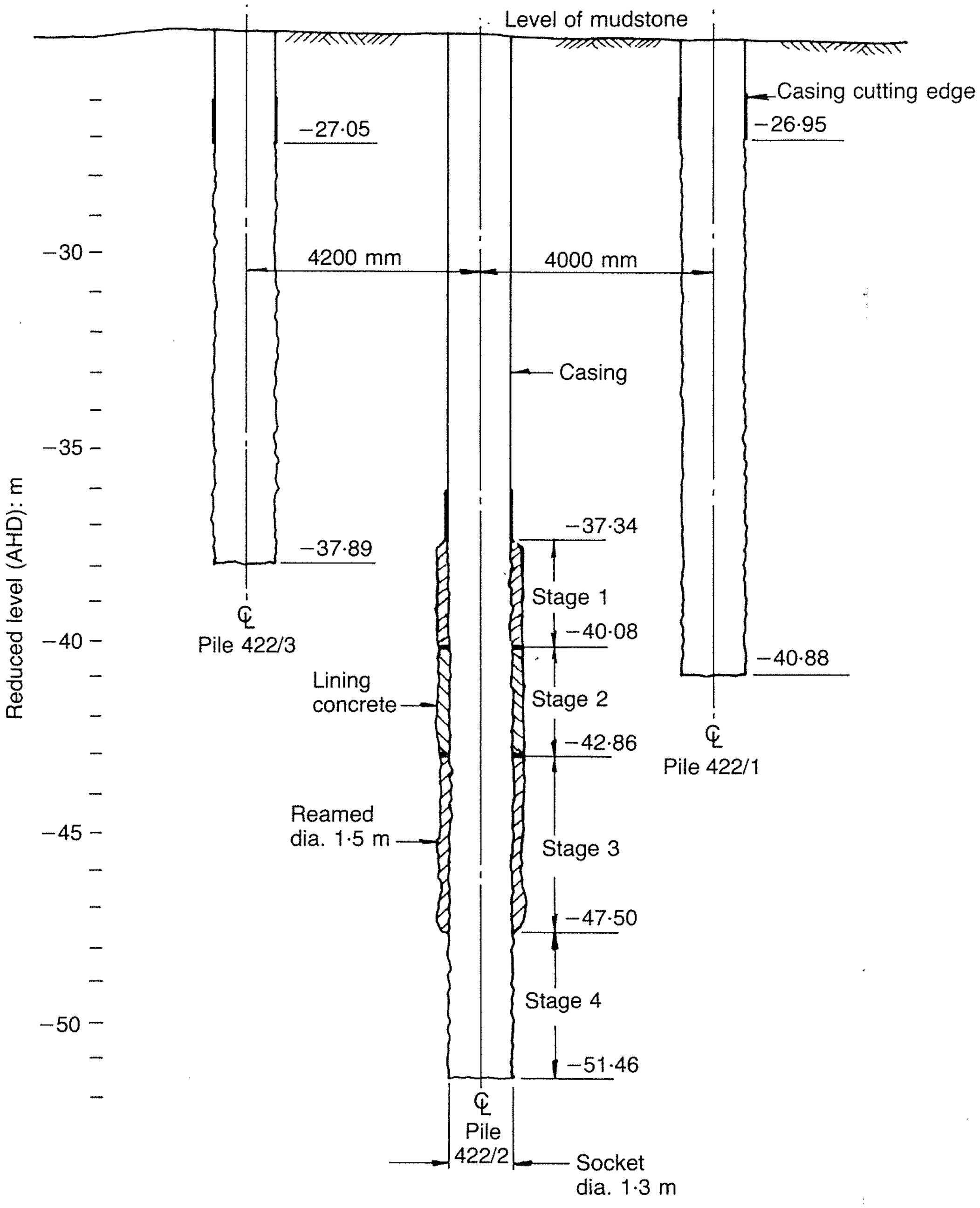


Fig. 14. Ream and line technique for construction of pile 422/2

Construction defects

73. To study the effects of deficiencies in construction practice, two trial piles 1 m in diameter and 10 m long were excavated in mudstone close to the vertical face of a disused clay pit, cast underwater and subsequently excavated for examination. The usual practice for constructing rock sockets in the mudstone was adopted by drilling with a bucket auger and then air lifting to clean all loose material before concreting under water using a tremie tube.

74. In the case of the first pile, after drilling and air lifting, the socket was allowed to stand overnight before concreting by means of 200 mm dia. tremie pipe. Immediately before placement of the reinforcing cage followed by the tremie pipe, 0·15 m^3 of excavated mudstone spoil (including fines and rock fragments up to 50

mm in diameter) was dumped into the pile. Midway through the concreting operation, about 0·5 m^3 of silty clay was tipped into the socket.

75. Following excavation, inspection of the completed pile showed that very little of the mudstone spoil had been displaced upwards from the base of the pile during the concreting process, although the base spoil had assumed a concave shape with some thickening around the edges of the base. A minor amount of spoil was found through the concrete up to 2 m above the pile base. A segregated band of concrete existed above the 2 m point, indicating a tremie operation problem which had not been deliberately induced. It would appear that the tremie pipe had been briefly lifted clear of the concrete at this point.

76. At the pile midpoint, the silty clay inclusion remained as a bulk defect with sound concrete above and below. There was no indication that the tremie operation tended to displace the silty clay upwards or disperse it through the surrounding concrete.

77. For the second trial pile, excavation was carried out by bucket auger underwater, and air lifting was performed with a 150 mm dia. pipe to clean the base. After the socket had been allowed to stand overnight, a reinforcing cage was placed before concreting, using a 200 mm dia. tremie pipe. The tremie pipe was kept immersed in the concrete, following conventional practice, except at a point 3 m from the base, where the pipe was lifted momentarily above the level of the concrete that was being placed.

78. On inspection of the completed second pile, two important observations were made. Firstly, the base of the pile was only relatively clean over the central area with about 5 mm thickness of fine debris; appreciable mudstone spoil remained around the circumference of the base, to the extent that only about 50% of base area was effective. Secondly, as might be expected, 3 m above the base a segregated band of concrete (essentially very dense sand and aggregate) was found where the tremie pipe had been lifted clear of the concrete.

Socket base cleaning and inspection

79. The principle of ignoring the contribution of base resistance for Class 3 sockets led, in many circumstances, to pile designs with extremely long sockets in rock of very doubtful stability. A video camera socket inspection device (SID) was developed for inspecting pile sockets underwater or in bentonite. The SID is essentially a sealed diving bell equipped with a glass window at the bottom for viewing (Fig. 15). Video images are transmitted to a surface monitor by way of an 'umbilical' cable to the stainless steel housing for the camera. Features incorporated in the SID include water jets to flush the base within the viewing area, and a graduated depth gauge which indicates the depth of soft base debris. The SID equipment is described fully by Holden.[7]

80. Inspections of the bases of piles showed further evidence of base debris and decreased confidence in the excavation and air lifting techniques used to clean the pile bases. A number of base cleaning buckets were developed to overcome these problems. Most of these buckets employed angled base scrapers, some equipped with rubber flaps, which gathered up soft material and rock fragments as the buckets were rotated in the socket. The most effective cleaning bucket of the many trial buckets used on the project consisted of a spiral shaped scraper, which swept base debris into the bucket and towards the centre. An air lift pipe at the centre of the bucket removed the loose material to the surface. A comprehensive description of the development of cleaning buckets is given by Holden.[8]

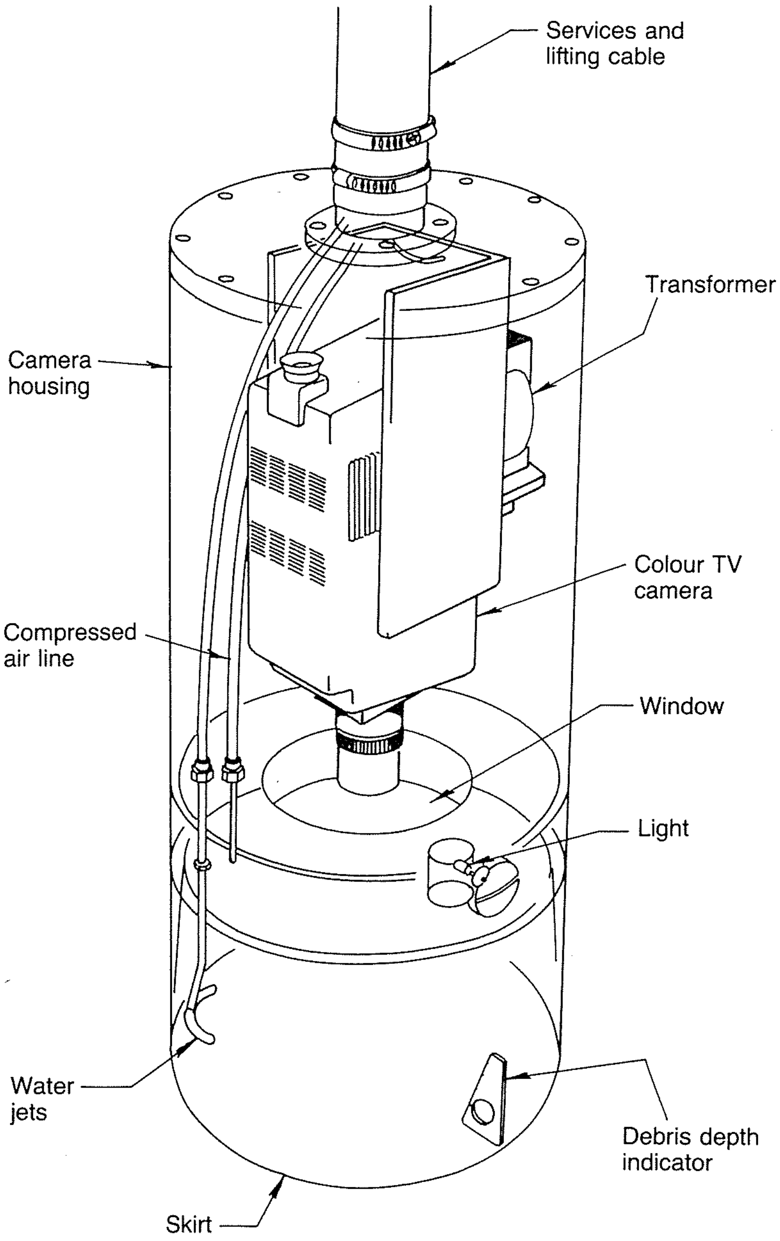


Fig. 15. Socket inspection device (SID)

81. The developments of the SID and improved base cleaning methods allowed inclusion of base resistance capacity in Class 3 piles. Generally, around 80% base cleanliness could be achieved, using the best of the cleaning buckets, but designs allowed for 50% reliable end bearing.

Assessment of foundations

Socket wall inclusions

82. During one of the many SID inspections of pile bases, just before concreting of a socket and after bentonite recirculation, the stainless steel camera housing had apparently scraped up the socket sides on withdrawal from the pile. A considerable quantity of bentonite gel had accumulated on top of the SID housing. The presence of the gel on the socket walls was of considerable concern because it had a similar shear strength to plastic concrete and there was no assurance that the gel would be displaced during concreting.

83. The Class 3B piles were designed mainly for side resistance, and if significant bentonite gel remained along socket walls during concreting, the capacity of the piles could have been appreciably reduced.

84. The build up of bentonite gel on socket walls is a function of the bentonite slurry properties, construction time, time between completion of excavation and concreting, and the permeability of the rock (greater thicknesses of bentonite developing against the more permeable beds of sandstone). A detailed study of construction records was carried out for all Class 3B piles to establish the time that the bentonite slurry remained in the piles between excavation and concreting.

85. A program of deflexion drilling was also initiated in which diamond cored boreholes were drilled down from the ground surface beside piles to the top of the uncased socket level. At this point, using a range of downhole wedges, the alignment of bores was changed in an attempt to intersect the socket wall. These operations involved meticulous drill string alignment and detailed calculations at various stages to maximize the likelihood of intersecting the socket walls. At each pile location where deflexion drilling was carried out, construction records were studied carefully to establish the as-built pile alignment.

86. The deflexion drilling program was slow but surprisingly successful, with many metres of intact core retrieved, in which the concrete–rock interface was sandwiched neatly along the core length. Observation of these cores confirmed suspicions that bentonite gel contamination of socket walls was often significant. The evidence was sufficient to indicate that design assumptions could be severely compromised.

Static load testing

87. By the time the bentonite problem emerged, about seven piles had been static load tested, five of which had been excavated and cast under a bentonite slurry. Fig. 16 shows one of the static load test arrangements using a deep rock anchor reaction system. Initial inspection of load test results indicated no problems with reduced capacity.

88. Three of the test piles appeared to have an average wall gel thickness of 1–3 mm, based on the deflexion drilling results. This was considered insufficient to adversely affect development of side shear stresses under axial loading because of the roughness of the socket walls.

89. Average bentonite gel thicknesses of about 8 mm were found along the rock socket–concrete interface of the two remaining Class 3B test piles. This was of about the same magnitude as the socket roughness. Also, during deflexion drilling of one of these piles, the drill string had actually dropped a short distance in the region near the top of the socket. However, the mudstone at this location was of very low strength, and the pile had a socket length of 17·4 m with a mobilized average side shear stress at the design load of only 0.054 N/mm^2. Any loss of capacity resulting from bentonite wall smear would be expected to be less significant where such low side shear resistances were developed at the design load.

90. The remaining Class 3B pile which was load tested had been strain gauged so that side and base load components of capacity could be evaluated. Close examination of the side load–settlement curve did indeed reveal uncharacteristically low capacity. A lower bound side resistance capacity of 11 500 kN was calculated, whereas the pile test results revealed an ultimate side resistance capacity of 7000 kN, about 40% below the design value.

91. A further nine Class 3B piles were proof load tested by means of rock

Fig. 16. Static pile load test

anchor reaction systems, and additional static load tests were proposed to test other Class 3B piles. The estimated cost of a static load programme, using kentledge to test all 80 of the Class 3B piles, was AUS$1·75 million and would have taken up to three years to complete. Even with duplicate load frames and additional kentledge, the delay to the freeway construction programme was unacceptable. Up until this time, a total of 22 static load tests had been carried out on the project, six of which were for basalt-founded piles.

Dynamic load testing

92. Before this project, dynamic load testing of piles had been carried out for some years on driven piles with ultimate capacities of 3000–4000 kN. Investigations were carried out with the assistance of the firm Goble and Associates (now Pile Dynamics, Incorporated) of Cleveland, Ohio, to determine what hammer mass would be required to mobilize the ultimate capacities of the project bored piles. Practical aspects concerning pile head preparation and access for attaching strain gauge and accelerometer instrumentation were also considered.

93. It was found from computer modelling that a drop hammer of 20 t falling up to 2·5 m was necessary in order to mobilize required capacities of up to 26 000 kN, including allowances for skin friction through soil strata, some of which would, in the long term, constitute a downdrag load. Using a drop hammer fabricated by a local piling contractor and a release mechanism and special hammer leaders designed and fabricated by the Roads Corporation, a pilot dynamic testing program was undertaken to assess the technique.[9] Fig. 17 shows a pile being tested by means of the 20 t drop hammer. Some of the piles previously statically load tested were included in the pilot study of 12 piles to provide a comparison for capacity predictions using the dynamic method. Goble and Associates were engaged to carry out this work but they were not supplied with static load test results until after predictions using the dynamic method had been made.

94. In all cases, the ultimate pile capacity mobilized during dynamic load tests was less than that calculated for the static tests. In most cases, static loading was not carried to a well-defined failure or gross settlement point. Hyperbolic extrapolation of static load curves was used, where appropriate, to compare static and dynamic load capacities. Table 3 shows the results of these comparisons. The loads in Table 3 include total capacity arising from socket base and side load contributions, plus casing skin friction through the alluvial sediments overlying the rock. The comparisons gave confidence that the dynamic method tended to produce conservative estimates of static capacity.

95. The dynamic testing pilot program revealed factors of safety significantly lower than designed on two of the piles, both of which had been cast under bentonite and one of which had revealed significant bentonite gel contamination at the socket wall interface during deflexion drilling. On the basis of these results, it was decided to test all remaining Class 3B piles by means of the dynamic method. The cost per pile was AUS$6000 compared with the estimated cost per pile of

Table 3. Comparison of static and dynamic capacities

Pile	Estimated static capacity:* kN	Mobilized dynamic capacity: kN
302/1	25 000	20 000
303/1	32 300	21 000
403/2	37 600	32 300
3385/1	17 300	14 000
428/4E17	15 000	10 000

* Estimated on the basis of hyperbolic extrapolation of static load curves.

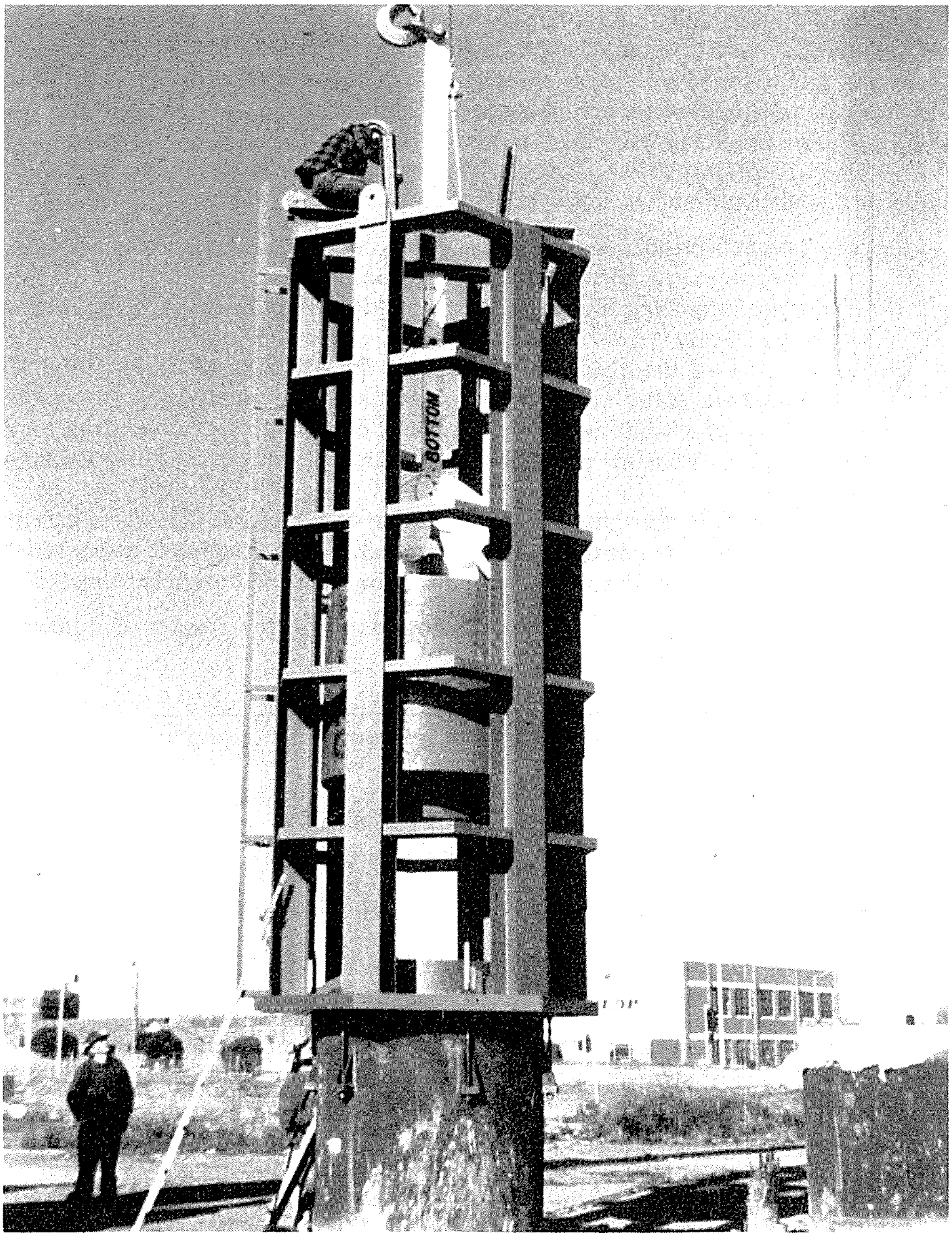


Fig. 17. Dynamic pile load test

AUS$22 000 for static load proof testing using kentledge. Load testing by means of a reaction beam and rock anchors would have been more expensive—around AUS$80 000 per pile—and more time-consuming.

96. Some additional non-3B piles for which some construction difficulties had occurred or which were considered marginal from a design or construction perspective, together with some piles selected at random, were also included in the program for further dynamic testing. Eventually, 146 of the 420 bored piles on the

project were dynamically tested at an average rate of testing of three piles per day. A small number were retested, using repeated blows of the 20 t hammer to increase settlement and to mobilize a greater proportion of base socket capacity, effectively stiffening the pile load–settlement response. Mobilized ultimate capacities of up to 48 000 kN were predicted with the dynamic technique, using the 20 t hammer.

97. The acceptance criteria adopted for the dynamic testing program were based on consideration of the following factors

(*a*) short-term frictional support from soft to firm alluvial strata which would, in the long term, become a downdrag load
(*b*) frictional support from stiff clayey and dense sandy or gravelly deeper alluvial strata
(*c*) uncertainty of the absolute accuracy of the dynamic testing method to reflect true static capacity; a partial factor of safety of 1·25 on the factored combination of live load and dead load plus downdrag load was used to ensure that the factored load did not exceed the ultimate load
(*d*) the possibility that creep would occur at low factors of safety whereby allowable settlements would be exceeded; a partial factor of safety of 1·5 was used to guard against creep under the factored design loading.

98. The final acceptance criteria used for assessing the results of dynamic testing of piles were

$$R_u > 1{\cdot}25\ [1{\cdot}1(DL + LL) + DD]$$

(total factored load combination)

and

$$R_u > 1{\cdot}9\ [0{\cdot}85(DL + LL) + DD]$$

(permanent factored load combination)

where R_u is the ultimate static load capacity arising from the dynamic test and analysis
$DL + LL$ = *dead load + live load*
DD is the downdrag load resulting from the settlement of soft alluvium

Further requirements were that no significant losses of shaft integrity should be detected in the analyses and that large permanent sets (greater than 5 mm per hammer blow) must not occur.

99. A factor working in favour of pile acceptance was that a detailed review of design loads at socket level showed that the downdrag loads calculated at the start of the project were conservative. The provisions of the, at that time, recently published Australian Piling Code[10] were certainly less stringent, to the extent that some revised downdrag loads were only 1/3 to 1/2 the original downdrag loads.

100. As a result of all quality assurance procedures involving static and dynamic load testing of piles, pile concrete coring and socket interface deflexion drilling, a total of eight piles were replaced on the project.

Embankment considerations

101. The poor ground conditions over the length of the project led also to a limiting height of embankment of 1·5 m at the abutments of the approach structures. The embankments were built on soft silty clay, known locally as Coode

Table 4. Embankment settlement predictions

Embankment height: m	1·5	2·0	4·0
Immediate settlement: mm	50	65	135
Primary consolidation: mm	100	280	1040
Secondary consolidation settlement (for 100 years): mm	150	300	Not calculated
Total settlement: mm	300	645	2000

Island Silt. This material, up to 20 m deep, is very compressible and exhibits significant long-term settlement under constant load. Settlement predictions for various embankment heights were made and these are shown in Table 4. The predictions were based on laboratory tests of undisturbed samples and the performance of other embankments in the area, most notably the approaches to Kings Bridge on Kings Way.

102. In addition to limiting the height of embankment to 1·5 m, it was also decided to surcharge the embankment areas to accelerate settlement and to minimize long-term movement. This required the placing of additional filling 2–2·5 m deep over the 1·5 m high embankment. Stability considerations limited the maximum height of any surcharged embankment with 2 : 1 batter slopes to 4·0 m.

103. To check the rate of settlement and the accuracy of settlement predictions, a trial section of embankment 3·5 m high was constructed on the western approaches and instrumented with remote reading settlement gauges and piezometers. A predicted primary consolidation settlement time of about 1 year was allowed for initially, but field observations indicated very much slower consolidation rates. Almost two years after filling, excess pore pressures in the middle of the soft clay had shown negligible dissipation. Since 600 mm of settlement had occurred at this stage, it appeared that the original estimates of settlement were seriously in error.[11]

104. At critical locations, such as bridge abutments and a major culvert structure beneath the freeway approach embankment to the west, vertical wick drains were installed to accelerate excess pore pressure dissipation. Wicks were installed in a triangular array, with spacings of 1·5–3 m and extending to depths of around 19 m. At the culvert site, surcharge was removed after nine months to meet the requirements of the critical path in the roadworks programme. The predicted consolidation time using wick drains was 18 months. It became clear from further settlement measurements after culvert construction and completion of the roadworks that the time under surcharge with wicks operational had been insufficient (Fig. 18). Therefore, surcharges at other wick drained areas at bridge abutments were left in place for periods of up to three years to ensure minimal settlement of bridge abutment fills after construction.

105. The limitation on the height of the embankments directly influenced the length and form of construction. The 2·0 m deep superstructure could not extend to the abutment because of the limiting embankment height, and so approach structures were introduced as a transition between the abutment and the start of the box girder superstructure. The approach structures varied in length from 53 m to 123 m and they comprised conventional prestressed concrete beam and slab bridges with spans of 17·5 m continuous over the supports.

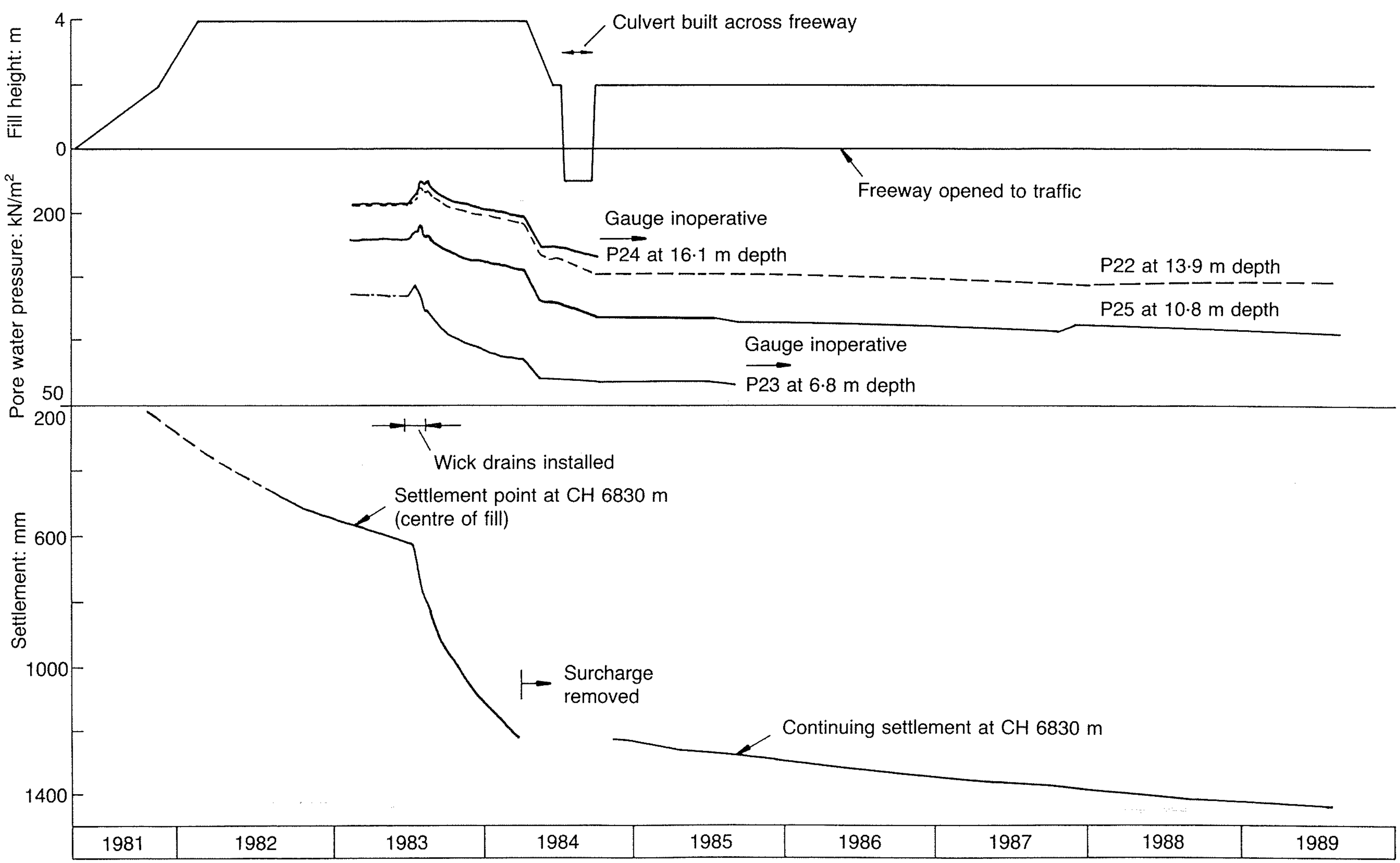


Fig. 18. Settlement of structure approach embankment at Hartley Street (Ch. 6830)

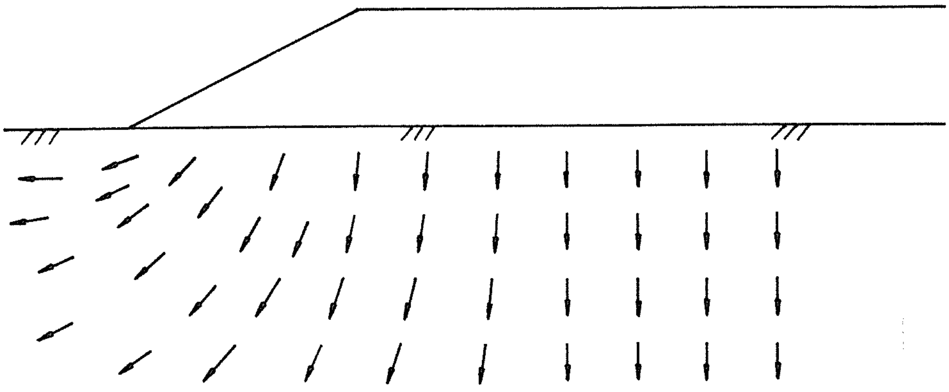


Fig. 19. Lateral displacements in soft subsoil under embankments

Effects of ground movements at abutments

106. The settlement of bridge approach embankments had been identified as a problem at an early stage in the project, but surcharge times turned out to be very much longer than predicted.

107. The longer surcharge time than originally expected meant that surcharges were still in place for up to three years after abutment pile construction. This was not initially considered to be a problem since piles had been designed for full downdrag loads. What had been overlooked, however, was the effect of lateral soil displacements that occur at the edges of embankments. In soft subsoils beneath embankments, such lateral soil movements can be very substantial.[12] The characteristic of soil displacement beneath embankments is shown diagrammatically in Fig. 19.

108. During a routine survey of completed western abutment piles about a year after construction, lateral movement of the heads of the piles was detected. Excavation around the completed piles, within 2·3 m dia. steel casings to a depth of 2·4 m and bentonite slurry to a depth of 10 m caused the piles to move back towards the embankments by 14 mm. Soil inclinometer tubes were installed at the toes of embankments. Readings confirmed over a period of 3–4 months that continuing lateral soil movements away from embankments were occurring under the surcharge fills to depths of 10–12 m in the soft subsoil.

109. Outer sleeve steel casings 2·1 m in diameter were installed over the 1·1 m dia. piles to a depth of 10 m, and were mucked out using a combination of augering, water jetting and small buckets passed down between the piles and the outer sleeves. Rebound of piles of between 20 mm and 45 mm was observed, confirming the earlier evidence that pile movements were the result of lateral soil movements associated with embankment settlement. Subsequent abutment piles constructed at the eastern approaches and ramps were installed through similar sleeve casings to negate the effects of lateral soil movements at these locations.

Conclusions

110. To date, the foundations for the elevated structure have performed in accordance with design predictions, as all the design and construction difficulties were overcome before the facility was opened to traffic. The world-wide problems of settlement and lateral movement at bridge abutments, where approach embankments are constructed over deep compressible soils, appear to have been largely overcome by a combination of pile sleeving, embankment surcharging, use

of wick drains and the adoption of relatively shallow structural depth transition structures between approach fills and the main elevated structure.

111. The construction experiences with deep bored piles showed the need for very careful attention to hole cleaning and inspection procedures to ensure that design assumptions were fulfilled in practice. Significant developments such as the remote inspection of pile bases by underwater camera (SID) and special base cleaning devices led to improved techniques for pile construction.

112. Expert and alert on-site supervision, together with a project oriented applied research and development program, permitted many potentially serious foundation problems to be detected or anticipated so that design and construction procedures could be modified during the project period.

113. The comprehensive static and dynamic pile testing carried out was found to be a necessary part of the Quality Assurance program for the foundations. In particular, the application of the very latest developments in pile dynamic testing and numerical analysis techniques proved to be extremely cost effective, in terms of both immediate costs and project time savings.

Credits

114. Foundation design
Roads Corporation of Victoria—VIC ROADS (formerly Road Construction Authority of Victoria)

Geotechnical consultants
Monash University Civil Engineering Department
Sydney University School of Civil Engineering
Golder Associates Pty Ltd
Coffey & Partners Pty Ltd
Goble and Associates (USA)

Proof engineers
Maunsell & Partners Pty Ltd

Foundation contractors
Frankipile (Australia) Pty Ltd
Vibro-Pile (Aust) Pty Ltd
Dillingham Constructions (Aust) Pty Ltd

Acknowledgements

115. Acknowledgements are given at the end of the second Paper.[1]

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