North Morecambe Terminal, Barrow: pile design for seismic conditions

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Very poor ground conditions and soils that were susceptible to liquefaction to depths of up to 20 m beneath the site were major problems for the construction of the largest natural gas processing terminal in Europe. Foundations were required to cater for liquefaction and the need to design for defined magnitudes of earthquake. An innovative foundation solution combining deep vibrocompaction with short cast-in-place piles driven into the treated soils was adopted as both cost- and programme-effective. The design and build project included preliminary site preparation and civils work comprising earthworks, drainage and road construction. The contract started in September 1991 and was completed by October 1992, with the process plant programmed to be on stream by October 1994. This paper presents details of the unusual pile design and foundation solution proposed to overcome the earthquake risk.

Keywords: piles & piling; foundations; seismic engineering

Introduction

Keller Ground Engineering successfully bid for civil and foundation works for the process plant at the British Gas North Morecambe Terminal site in Barrow. The design and build project included preliminary site preparation and civils work comprising earthworks, drainage and road construction, soil densification using vibroreplacement techniques and piling works. The contract started in September 1991 and was complete by October 1992. A generalized description of the works and the construction activities is given by Ground Engineering.[1] Background information about the overall project including the offshore platform and pipeline is given by Juren[2] and Spicer.[3]

2. For the UK, historical records show that the Morecambe Bay area has been subject to a relatively high level of seismic activity. Because of the critical nature of the site and its sensitive location, British Gas carried out seismic risk assessments as part of the early project feasibility studies[4,5] and during the conceptual design stage.[6] These concluded that two levels of earthquake should be applied to the design of the process plant and foundations. The assessment also highlighted the very real possibility that liquefaction could occur in the natural soils to depths up to 20 m below ground and foundations were required to allow for this. This paper presents details of the unusual pile design and foundation solution proposed to overcome the earthquake risk. Detailed results of extensive on-site pile testing will form the subject of a future paper.

The site and geology

3. The North Morecambe Terminal site is located to the north of the existing British Gas South Morecambe Terminal about 2 km south-east of Barrow-in-Furness (Fig. 1). The site is approximately 750 m long in the north–south direction and about 250 m wide, bounded by the coast to the west and undulating pasture land to the east (Fig. 2).

4. Before development, the majority of the 20 ha (50 acre) site comprised three former settlement lagoons containing saturated pulverized fuel ash (PFA) (Fig. 3). The PFA was a waste product from the adjacent coal-fired Roosecote power station. The north and south PFA lagoons were generally at a higher level than the central lagoon, with better drainage resulting in a crust of firmer PFA. Ground level was at 12 m OD (Ordnance Datum) in the north lagoon and 10 m OD in the south lagoon. Within the lower central lagoon, the PFA was extremely soft with water at the ground surface typically at 7 m OD.

5. Extensive site investigation, including boreholes and static cone penetration tests (CPTs), revealed ground conditions beneath the PFA to comprise very loose silty gravelly alluvial sand over more dense glacial sands, with glacial till and sandstone at depth. Ground conditions were locally variable and simplifications were necessary for design purposes. A typical geological section is shown in Fig. 4.

6. The PFA extended down to a level of about 4 m OD. It was usually extremely soft with standard penetration test (SPT) N values as low as 2 blows/300 mm (Fig. 5). Parts of the north and south lagoons had been formed to higher levels where some degree of pozzolanic reaction had occurred within the upper layers. Here N values were generally higher. Particle size grading of the PFA indicated a sandy silt which was often clayey.

7. Immediately beneath the PFA were alluvial deposits up to 7 m thick which varied between a soft silty clay with bands

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of peat in the upper levels, becoming a loose to medium dense silty, occasionally gravelly and clayey sand at depth. The sands had measured N values ranging between 5 and 30 blows/300 mm (Fig. 5).

8. The glacial soils were extremely mixed, encountered generally as a fluvioglacial sand, or sand and gravel, over lacustrine glacial lake clay and glacial till. The sands often contained thin layers of clay up to 0·5 m thick. The glacial sands typically had N values varying between 5 and 30 blows/300 mm as shown in Fig. 5. Measured N values for the lake clay were about 15–20 blows/300 mm, while N values in excess of 50 blows/300 mm were recorded in the glacial till.

9. Sandstone bedrock of Triassic age was found at about −20 m OD.

10. Groundwater was perched within the PFA generally at about 7–8 m OD and close to ground level within the lowest part of the central lagoon. In the natural soils, groundwater varied between 4 m OD and 7 m OD.

The problem

11. Foundation design and construction faced technical problems due to extensive PFA lagoons underlain by loose granular soils. Foundations had to cater for soil liquefaction and the need to design for defined magnitudes of earthquake.

12. Seismic risk assessments carried out as part of the early project feasibility studies were refined following extensive geotechnical investigations.[4–6] Two levels of earthquake were specified for the design. General operating needs required a foundation solution able to withstand a 1-in-500-year design earthquake, equivalent to a Richter scale event of 5·25 at a depth of 10 km with an epicentre at 15 km distance. Certain selected critical plant items or shutdown structures (Fig. 6) had to survive a 1-in-10 000-year seismic event, equivalent to 6·0 on the Richter scale.

13. The seismic risk assessment anticipated accelerations of 0·05 g and 0·20 g at bedrock with amplification due to soil conditions to about 0·08 g and 0·28 g at the ground surface. Seismic studies used these values to establish the potential for soil liquefaction following the methods of Seed and Idress,[7] and Seed *et al*.[8] Evaluation was carried out using both characteristic and probabilistic methods based on SPT and CPT design profiles.[9,10] Studies concluded that the loose alluvial and glacial sands could liquefy to depths of up to 20 m below ground during a 0·20 g earthquake, and that the PFA would liquefy during both levels of earthquake.

The solution

14. The foundation problems were complex and the period available for outline design and tendering was limited. The scheme put forward was based on the use of vibrodensification techniques and short cast-in-place piles driven into the treated soils. This is illustrated schematically in Fig. 7. Although vibro ground treatment had been used many times in the past and was a well-proven technique to prevent soil

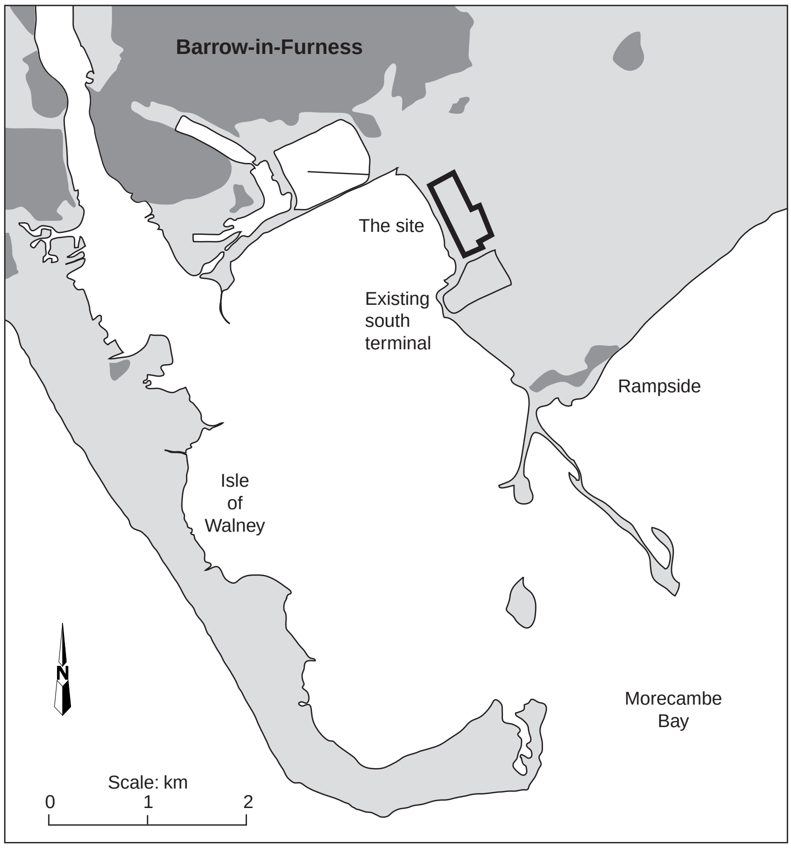

Fig. 1. Site location plan

Fig. 2. View looking north showing the site bounded by the existing South Morecambe Terminal to the south and Roosecote power station to the north, with Barrow-in-Furness beyond

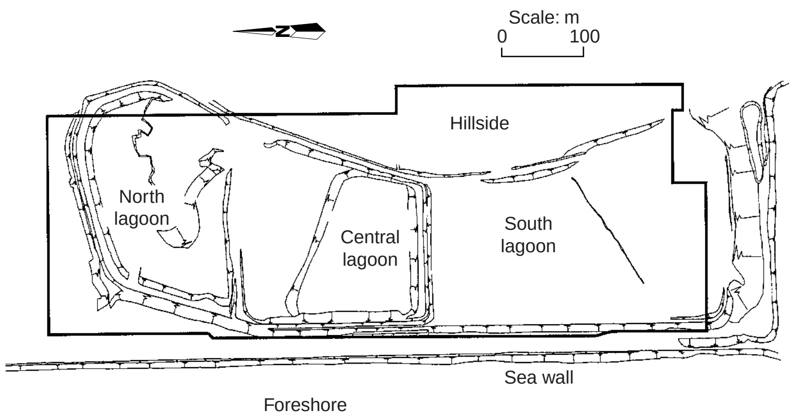

Fig. 3. Site layout plan

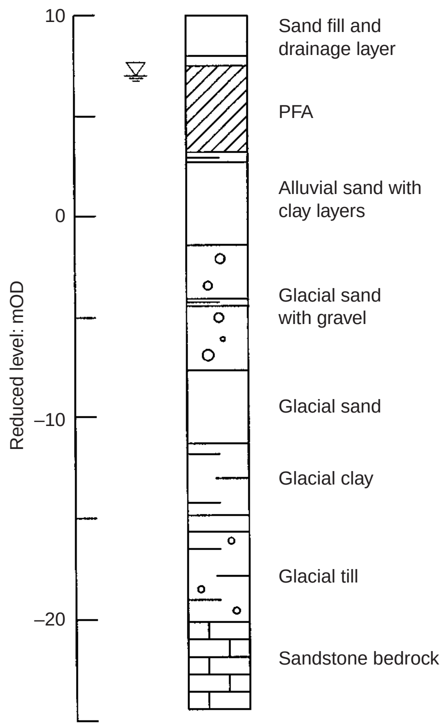

Fig. 4. Typical geological section

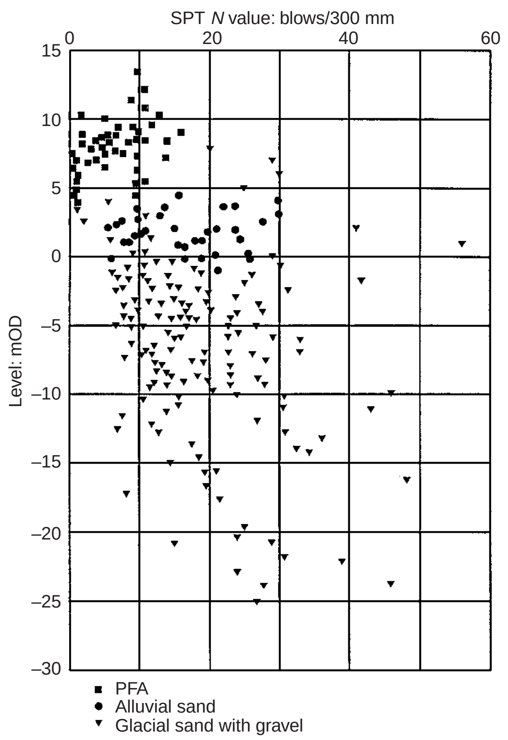

Fig. 5. Plot of standard penetration test results

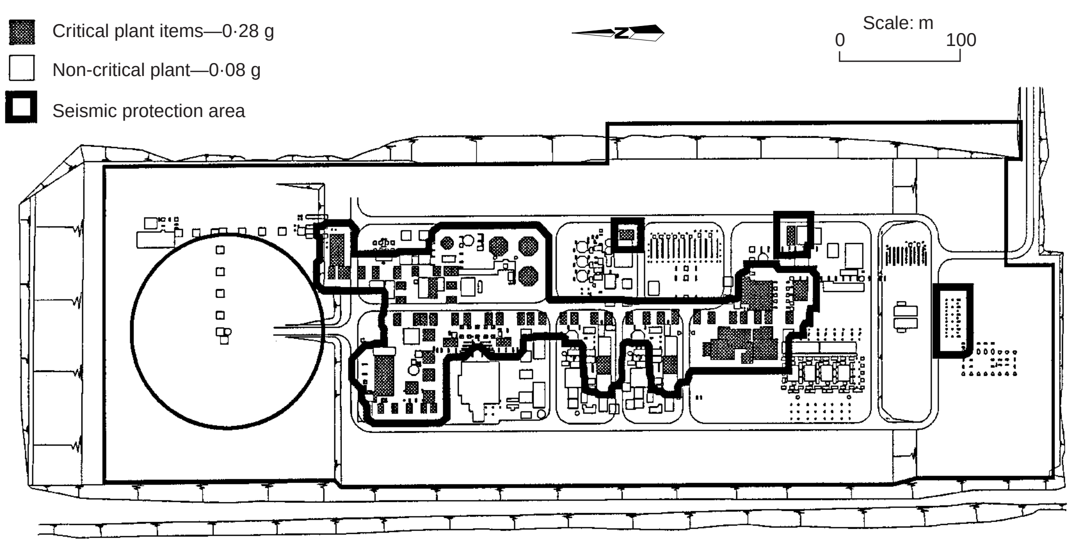

Fig. 6. Critical plant items and seismic protection area

liquefaction, this was the first time that ground improvement had been used in this manner together with piling. The result was an innovative and more efficient solution, with the improved density of the soil giving a better foundation for piles and major savings in length, pile section and reinforcement.

15. Vibroreplacement was not a suitable technique to eliminate liquefaction of the very soft PFA fill. Following the recommendations of the seismic risk assessment, a partial dig and replacement scheme with drainage was proposed to control groundwater. This would prevent liquefaction during the low-level 0·05 g design earthquake, and had the added advantage of providing a very stable working platform for the vibro and piling rigs.

16. Advantages of the dual system solution to the client included an assured construction programme, due to linking two low-cost high-production techniques, and a more predictable seismic behaviour. The approach was flexible, enabling additional low-cost piles to be installed to cater for design changes without the need for more vibro stone columns.

Pile design requirements

17. During a 1-in-500-year earthquake, effective stress site response computations showed that liquefaction of the natural soils was unlikely.[11] This was indicated by the small changes in pore pressure computed for the 0·05 g design earthquake (Fig. 8). Liquefaction of the very soft PFA fill was a serious possibility but could be prevented by a partial cut and replacement scheme. The remaining PFA was totally unsuited for foundations, and piling was required to transfer structural loads through the weak PFA to the underlying soils. All piles had to withstand the earthquake loadings and control shakedown settlement or lateral movement without disrupting normal operation of the process plant.

18. It was not possible to prevent liquefaction of the PFA during a 1-in-10 000-year seismic event, but site response computations showed that treatment by vibrodensification techniques would prevent liquefaction of the natural soils (Fig. 9).

19. Piles installed for non-critical plant were not required to survive. However, piles for all critical plant items had to survive sufficiently intact to limit vertical and lateral movement from shakedown, and to cater for the effects of increased pore water pressures and the dynamic motion of the plant items. Piles had to provide support continuity through the partially to totally liquefied PFA, cope with reduced lateral support from the soil, and deal with possible horizontal lurch movements up to 100 mm.

20. Post-earthquake consolidation of the liquefied soil was also expected which would result in major downdrag loads on the pile shafts. Throughout the seismic event, piles had to maintain sufficient axial load support capacity and limit structural movements. Piles were not required to be reused following a 0·20 g design earthquake.

21. The client's primary requirement for shutdown structures was that damage be limited to avoid release of any inventory gas into the atmosphere. To this end, structural analyses and plant design carried out by British Gas and their process plant designers defined maximum pile settlement and deflection performance criteria as shown in Table 1.

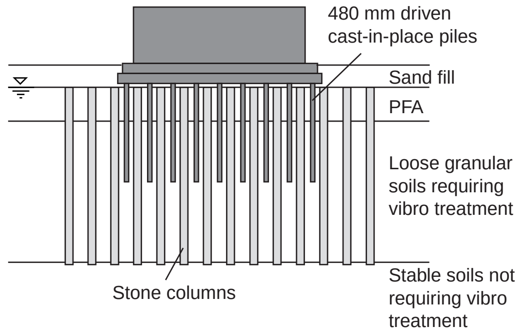

Fig. 7. Proposed foundation solution

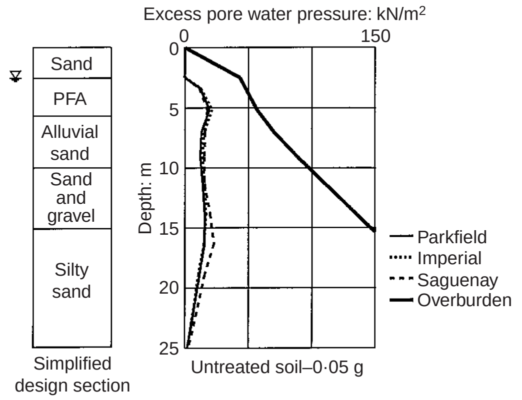

Fig. 8. Results of effective stress site response computations: untreated soil, 0·05 g earthquake

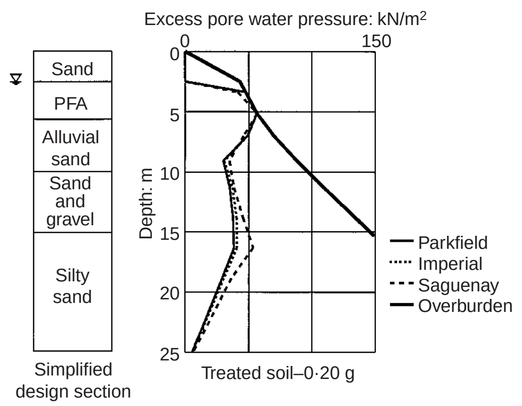

Fig. 9. Results of effective stress site response computations: treated soil, 0·20 g earthquake

Piling proposals

22. For the foundations, a nominal working pile capacity of 600 kN under normal operating conditions was offered for all plant items and

Table 1. Pile settlement and deflection criteria

Description	Operational conditions: mm	0·05 g earthquake	0·20 g earthquake
Total settlement of plant items	25	Not specified	50 mm
Total settlement of tanks	40	Not specified	Not specified
Differential settlement*	10	Not specified	Not specified
Lateral deflection	Not specified	10 mm	100 mm

*Between piles in a group, but also between adjacent pile caps.

Table 2. Proposed pile design working loads and predicted performance

Pile description	Axial working load: kN	Tension working load: kN	Lateral working load: kN	Pile settlement: mm	Horizontal movement: mm
Cast-in-place:					
Operational	600	300	30*	5	2·5
			100†		8
0·05 g earthquake	750	350	125	10	10
0·20 g earthquake	850	400	150	50	100
Precast:					
Operational	600	300	25*	5	2·5
			75†		8
0·05 g earthquake	750	350	90	10	10

*General load case for most plant items.
†Load case for a few specified structures.

other structures. Driven cast-in-place piles of 480 mm diameter were proposed for critical plant items where the 1-in-10 000-year 0·20 g design earthquake was to be applied.

23. Driven cast-in-place piles utilize a temporary steel drive tube and disposable steel base shoe which is driven to depth using a top drive hammer. Reinforcement is placed within the tube before concreting, extraction, and reuse of the tube. The 480 mm diameter was required to provide sufficient flexural stiffness and to allow 254 mm × 254 mm × 85 kg/m universal bearing pile (UBP) steel sections to be used as shear and bending reinforcement. For the remaining plant items it was intended to use 320 mm square prestressed single-length precast piles. As the project developed, 290 mm and 350 mm precast piles together with 380 mm diameter cast-in-place piles were also used for non-critical plant.

24. To aid the process plant designers, a summary of pile working loads and expected performance was issued. This is reproduced in Table 2.

25. Piles were to be installed for a 600 kN axial working load with a minimum factor of safety of about 2·25. Cast-in-place piles of 480 mm were always to be driven into ground previously treated by vibrodensification. Calculations suggested that depths between 7·5 m and 10 m below pile platform level would achieve the required working loads, dependent on the precise ground conditions. When driven into densified soil, precast piles or the alternative 380 mm cast-in-place piles needed to be slightly longer than the 480 mm piles. Precast or 380 mm cast-in-place piles driven in locations outside of the vibrotreatment areas were expected to be longer and to show much more variation, with pile lengths typically between 10 m and 15 m.

26. To aid installation, and to minimize compression and tension due to wind and seismic loadings, pile layouts were proposed with piles on a minimum 2 m grid. An example pile layout is shown in Fig. 10, which also shows the location of the vibro stone columns.

27. It was proposed to use lower factors of safety to cater for dynamic loads during an earthquake with the proviso that settlement control would be maintained. For the 0·05 g magnitude event, a factor of safety of about 1·8 gave a transient axial working load of 750 kN for the precast and 380 mm cast-in-place piles. During a 0·20 g earthquake, a higher seismic axial load of 850 kN was considered acceptable for the 480 mm piles, equivalent to a factor of safety of 1·6.

Pile design

28. Pile design was based on borehole and CPT results provided by the client. These were supplemented with pre- and post-treatment CPT tests carried out as part of the vibro ground improvement works, and full-scale trial pile drives. Site investigation showed extremely variable ground conditions, predominantly sands with varying quantities of silt and gravel, often with frequent thin discontinuous bands of clay (Fig. 11).

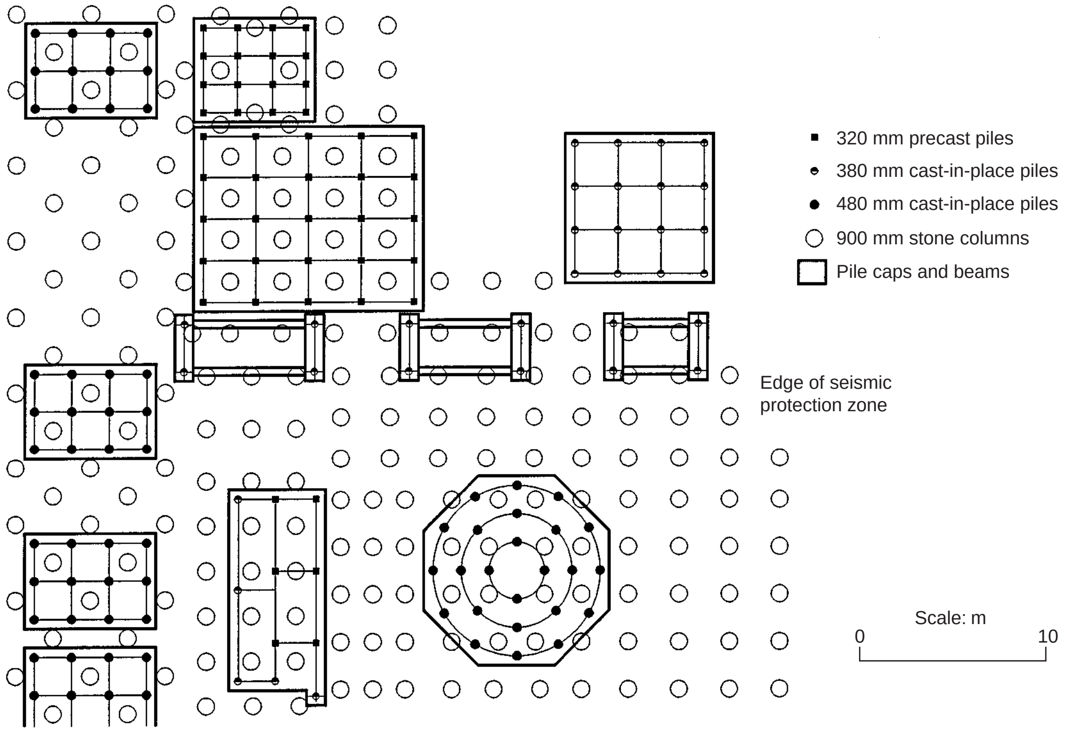

Fig. 10. Typical pile and stone column layout

29. The approach used for final design was analogous to the observational method.[12,13] The available site investigation data were used together with the preliminary load testing and full-scale trial drives to establish drive criteria and proposed pile length for the contract piles. Drive blows were used to monitor the piling with any variations triggering contingency measures, usually to drive piles deeper or harder.

30. Although ground conditions were locally variable, the resulting pile lengths did not change significantly across the site, particularly in the areas previously treated using vibrodensification.

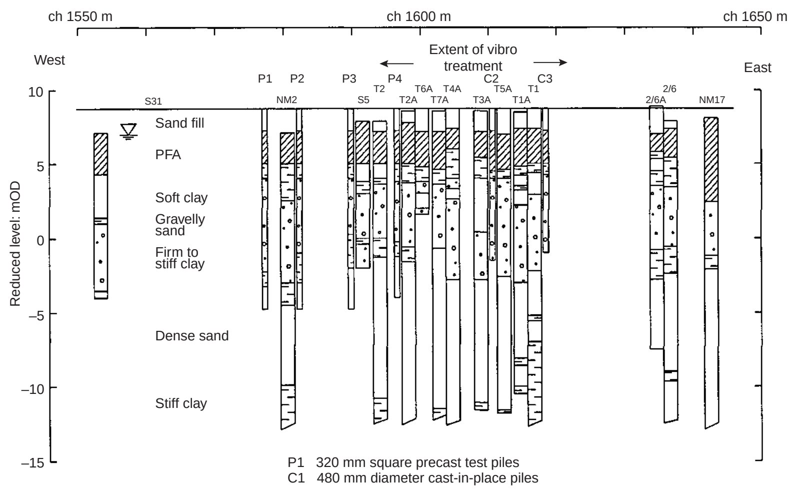

Fig. 11. Section through main test area showing ground conditions

Axial load capacity

31. Bearing capacity design was carried out assuming effective stress conditions. Design parameters used for the calculations were determined from geotechnical considerations and confirmed by the pile testing. These are summarized in Table 3.

32. Pile shaft friction is a function of the radial stress $\boldsymbol{\sigma}_h'$ acting on the shaft and the friction angle δ between the soil and the pile. For a driven cast-in-place pile, the radial stress will lie between an upper bound passive limit stress induced by driving and a lower bound hydrostatic concrete pressure. An allowance for

Table 3. Pile-bearing capacity design parameters

Soil description	Unit weight: kN/m^3	ϕ': degrees	k_s, tan δ cast-in-place	k_s, tan δ precast	N_q
PFA fill	15	20	−0·30	−0·30	—
Untreated, alluvial sand	18	30	1·15	0·85	25
Densified alluvial sand	20	33	1·30	0·90	40
Untreated glacial sand	20	33	1·30	0·90	40
Densified glacial sand	20	38	1·60	1·10	120

the densification effect of driving and the subsequent relaxation on withdrawal of the drive tube can also be made.

33. Because of the difficulties in predicting $\boldsymbol{\sigma}_h'$, the radial stress is usually computed as a function of the vertical effective stress $\boldsymbol{\sigma}_v'$ times an assumed earth pressure coefficient K_s. Choice of K_s is difficult as it is dependent on the existing soils, type of pile and pile geometry, and is usually determined empirically. In this case, design values were based on previous experience in similar soil conditions and then confirmed by an extensive programme of pile testing. Choice of friction angle δ was less problematic. For a rough concrete interface, δ will be close to the angle of internal friction ϕ'.

34. For the smaller precast piles the initial radial stress will be smaller, but with less subsequent stress relaxation. For precast piles the interface friction angle will be closer to 0·75 ϕ'.

35. Because of the presence of thin discontinuous clay layers within the alluvial and glacial sands, preliminary design of piles assumed a clay toe. From the on-site testing and the extensive programme of full-scale trial drives, it was clear that clay layers at toe level could be identified by low measured drive blows (Fig. 12) and suitably chosen driving criteria. In most cases, enhanced end-bearing capacities were used for the final design based on effective stress assumptions assuming sand at the toe.

36. Soil friction angle ϕ' was assessed from SPT and CPT data together with measured pile drive blows. Bearing capacity coefficients N_q were based on the theory of Brinch Hansen,[14] which includes for the depth of embedment of the pile. Similar design values for N_q were used for both the driven cast-in-place and the precast piles, as shown in Table 3. As pile centres were proposed at a minimum of four pile diameters, no group reduction factors were applied.

Tension load capacity

37. Pile capacity in tension was computed using the same design parameters as used for axial compression loads. Pile layouts were chosen to minimize potential tensions due to wind, dynamic or thermal effects. Under normal operating conditions, tension loads in the piles were rare. Under the transient seismic loading conditions, tension loads were possible in some piles for which reduced factors of safety were used.

Lateral load capacity

38. Under normal operating conditions lateral wind, dynamic or thermal loads were nominal. These load cases were checked for all plant items but seismic conditions usually dominated and governed the required reinforcement. The effect of soil liquefaction complicates the seismic loading and the assumed conceptual behaviour is therefore now described.

39. During an earthquake, three stages of behaviour were expected

(*a*) an initial pseudo-elastic phase
(*b*) a partial liquefaction phase where soil liquefaction occurs only in thin bands allowing relative shear movements
(*c*) a complete liquefaction stage.

These three stages are illustrated in Fig. 13. Each of these stages was considered for the 1-in-10 000-year piles. Only the elastic phase is relevant to the 1-in-500-year piles although partial PFA liquefaction was also checked with a smaller assumed relative movement.

40. During the pseudo-elastic phase, seismic inertial loadings can develop between pile and structure to a level governed by the structural

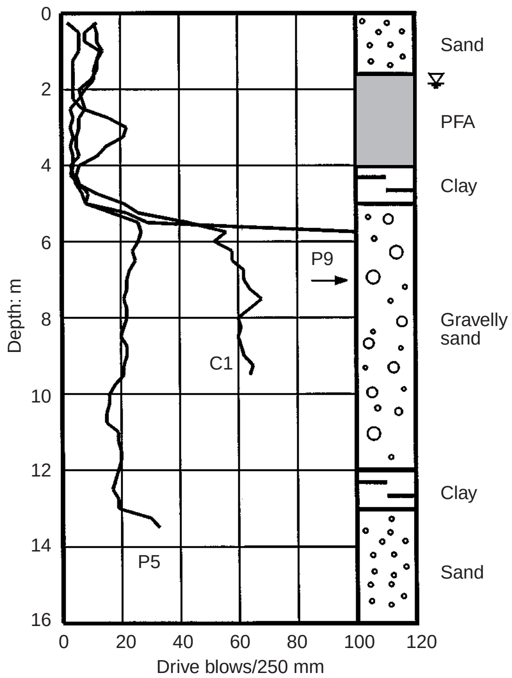

Fig. 12. Measured pile drive blows for piles C1, P5 and P9

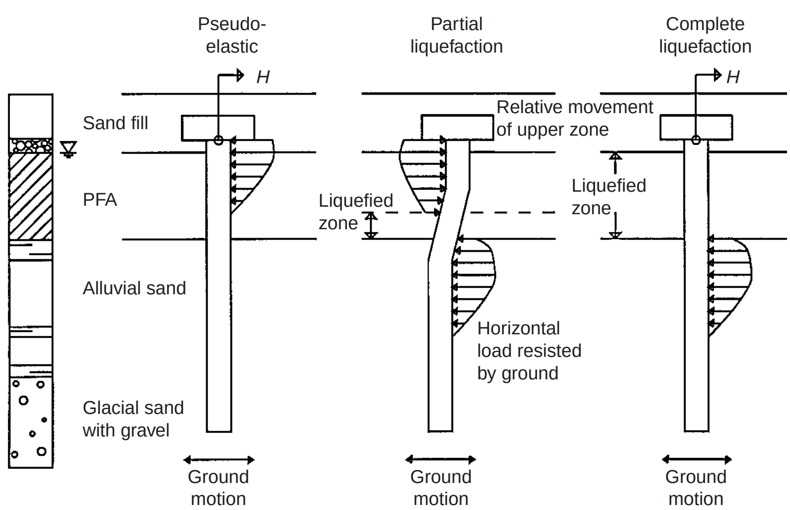

Fig. 13. Pile behaviour during a 0·20 g earthquake

flexibility or natural frequency of the combined soil, pile and structure system. Routine design was carried out using inertial loads equal to the structure mass times either 0·08 g or 0·28 g acceleration. Generally, this approach is inappropriate as the inertial load will depend on the earthquake response spectra at ground level and the natural frequency of both foundation and superstructure. However, most of the plant items have massive pile caps and substructure with small fundamental periods. Substructure was expected to dominate the seismic behaviour and the design simplification was considered acceptable. Certain structures were checked more rigorously to investigate the effect of the natural frequency of the superstructure and foundation system.

41. Pile deflection and bending moment behaviour was investigated using the Oasys Limited soil–pile interaction analysis computer program ALP.[15] Typical design parameters used in the interaction analyses are given in Table 4. Inertial loads to be applied were assumed to act at the head of the pile. Allowance was made for head fixity, and relationships between lateral load and the resulting pile deflection and bending moment were established. Though the PFA fill was very soft, it was shown that it provides more than adequate support to piles keeping moments and deflections to acceptable levels. The pseudo-elastic phase was found to be less onerous than subsequent partial and complete liquefaction phases.

42. Partial liquefaction of a thin liquefied layer at the base of the PFA was also considered. This could allow the sand fill, drainage blanket and upper layers of PFA to move relative to the underlying sands, applying a shearing load to the piles. As a simplifying assumption, the shear movement could be approximated to the maximum displacement from the site response assuming a very long period single degree of freedom elastic system.

43. However, it was accepted that the actual movement must also depend on the ground velocity of the upper soil levels prior to liquefaction, the post-liquefaction ground response of the soil beneath the liquefied layer and the residual strength of the PFA. Analyses assumed maximum shear movements of either 20 mm for the 0·05 g earthquake, or 100 mm for the 0·20 g earthquake.

44. As a design check, non-linear effective stress site response analyses were carried out to assess potential shear movements and any resulting post-liquefaction lurch displacement across the liquefied zone.[11] Analyses included post-liquefaction yield of the surface layer and assumed a residual shear strength of the liquefied PFA of 10 kN/m^2. All analyses indicated movements less than the design assumptions.

45. For the 1-in-10 000-year event, soil shearing movements were of sufficient magnitude to cause development of a plastic hinge at the junction between pile and pile cap, and possibly a second hinge at a lower level in the pile section. Although a two-hinge mechanism could form, this would not result in a collapse failure because of the supporting action of the fill and the residual strength of the liquefied PFA. As indicated earlier, it was not intended to reuse piles following an earthquake. Shear movements were analysed using the ALP soil–pile interaction software and imposing soil shear movements over a 1 m thick zone at the base of the PFA fill. Figure 14 gives typical results for the 1-in-10 000-year earthquake case where shear movements up to 100 mm were considered possible.

46. This shows a large bending moment at the head of the pile, with the shear force reaching a maximum at the interface between the liquefied zone and the underlying densified sands.

Fig. 14. Horizontal load analyses for partial liquefaction case

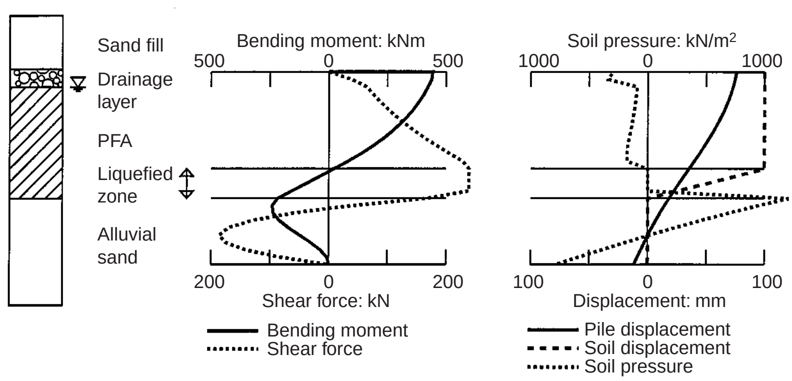

Table 4. Lateral interaction analysis design parameters

Soil description	Unit weight: kN/m^3	ϕ': degrees	Brinch Hansen coefficient: K_q	Young's modulus: MN/m^2
PFA fill	15	20	9	2
Liquefied PFA fill	15	0	0	0·01
Untreated alluvial sand	18	30	10	20
Densified alluvial sand	20	33	15	40
Untreated glacial sand	20	33	15	40
Densified glacial sand	20	38	20	80

47. Complete liquefaction behaviour was investigated by assuming the pile section was largely unrestrained by the soil and loads would depend on the natural frequency of the pile. A relationship between the foundation natural frequency of vibration and resulting inertial load at the pile head was developed assuming no soil support to the pile shaft. However, complete liquefaction will not occur and soil will always retain some residual strength. Based on published test results and back-analyses,[16] a residual shear strength of 10 kN/m^2 was assessed. ALP soil–pile interaction analyses show that this case was less onerous than the partial liquefaction situation unless major lateral flow or lurching movements take place.

48. Behaviour during complete liquefaction was also dependent on whether the previous partial liquefaction stage resulted in the formation of plastic hinges within the pile shaft section. If so, the pile behaviour would be much more flexible and would depend primarily on post-liquefaction displacement of the upper layers of soil.

Structural design

49. Typical seismic lateral loads of up to 50 kN for the 0·05 g and up to 150 kN for the 0·20 g earthquakes were expected based on nominal working pile loads of 600 kN. The results of pile–soil interaction analyses carried out for typical geological soil sections were used to establish the maximum pile bending moments and shears.

50. Detailed pile section analyses were then performed to establish suitable pile reinforcement details, dimensions and properties. Figure 15 shows typical details of the various pile types used during the contract. Figure 16 shows bending moment plotted against axial load relationships for the three main pile types used, and Fig. 17 gives a plot of computed bending moment against flexural stiffness *EI*. Similar computations were made for 290 mm and 350 mm size precast piles, and for longer 320 mm precast piles prestressed with eight strands.

51. Structural design for piles for the 1-in-500-year 0·05 g design earthquake was relatively straightforward, comprising a simple check to ensure an adequate partial factor. This was done by comparing the maximum computed seismic bending moment obtained from the ALP interaction analyses with the computed ultimate moment capacity. A shear check was also made.

52. Piles for the 0·20 g 1-in-10 000-year design earthquakes were more difficult. Seismic loadings were greater, requiring a larger pile section just to deal with bending. For partial liquefaction of the PFA, potential shear movements were expected sufficient to generate bending moments close to the ultimate moment capacity of the section allowing the formation of a plastic hinge. (Compare the maximum bending moment shown in Fig. 14 with the ultimate moment capacities indicated by Fig. 16.)

53. Of prime importance to the performance of the pile was the ability to maintain axial capacity after development of a plastic hinge. Although a traditional steel reinforcement cage could provide similar moment capacity, it was decided to use a UBP steel section reinforcement because of the improved ductility and post plastic hinge performance. Checks were made using the approaches given in the 1988 draft of Eurocode 8[17] and BS 5950[18] for bare steel sections. Particular attention was paid to detailing to ensure that the UBP section would act in a ductile manner under load. This was

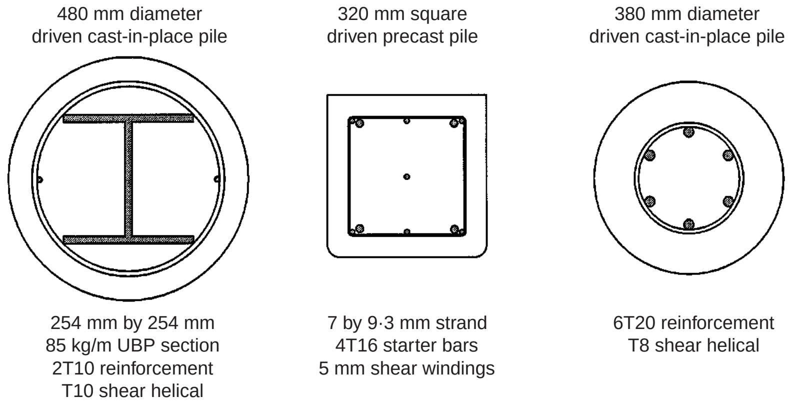

Fig. 15. Section details of driven piles

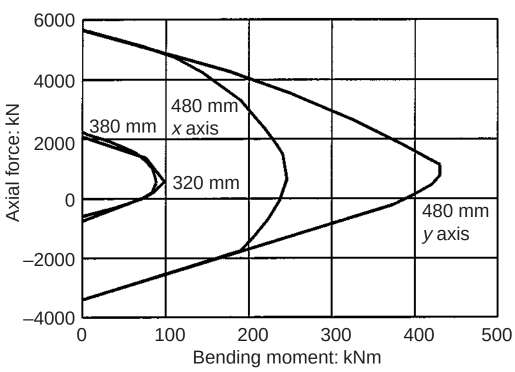

Fig. 16. Axial force against bending moment relationship for proposed piles

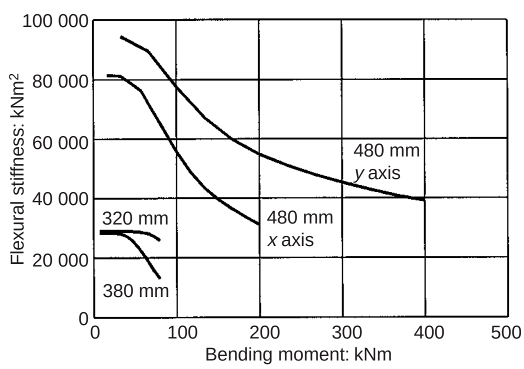

Fig. 17. Bending moment against flexural stiffness relationship for proposed piles

possible using the pile concrete to prevent buckling, which in turn required shear helical to provide confinement to the concrete and to prevent spalling (Fig. 18).

Pile settlements

54. A summary of pile load capacities and predicted settlement behaviour is given in Table 2. Typically, the anticipated settlement of isolated piles was better than the specified performance criteria (Table 1). Group behaviour had to be looked at on an item-by-item basis, but settlements were considered unlikely to exceed specification.

55. Pile settlements under operational conditions and during the 0·05 g design earthquake were estimated using the Oasys Limited computer program PILSET which is based on the approach given by Poulos and Davis.[19] Settlements included for the effect of foundation overturning due to the seismic loadings, although no account of the cyclic nature of the loading was taken. Effects of shakedown of the soil or increases in pore water pressures were not significant. Estimates based on the method of Tokimatsu and Seed[20] suggested these would add less than 10% to the computed settlement.

56. Settlements during the 0·20 g design earthquakes were expected to be much higher due to a number of effects in addition to the overturning motion of the plant items under seismic loading. Increases in pore water pressures, although insufficient to cause liquefaction, were expected to modify soil effective stresses, thereby reducing soil stiffness and the pile load capacity. Shakedown of less dense layers of the treated natural soils, mainly the more silty sands, was also expected, either within the depth of the pile shaft or below the toe. This would result in direct settlement when below the pile toe, or indirect settlement where soil movements cause downdrag loadings. Liquefaction of the very soft PFA fill would also add to the downdrag loads.

57. Because of the dangers to the piling, much effort was made during vibrodensification to ensure that no thin layers of soil remained untreated to a minimum relative density, as defined by CPT cone resistance. All post-treatment CPT results were processed to assess possible shakedown settlements, and where unacceptable, re-treatment was carried out on a localized basis without affecting the construction programme.

Lateral movements

58. Predictions of pile head horizontal displacement were possible using the results of the ALP interaction analyses carried out to check lateral capacity. Under operational conditions, during the 0·05 g design earthquake, and during the early stages of the 0·20 g earthquake, computed head movements were generally small. Estimation of post-liquefaction displacement was more difficult. As discussed earlier, post-liquefaction movements would depend on the ground velocity of the soil prior to liquefaction, the post-liquefaction ground response of the soil beneath the liquefied layer, and the residual strength. However, movements were not expected to exceed the maximum displacement for the site response of a very long period single degree of freedom elastic system. Based on seismic assessment work carried out in conjunction with the University of Southern California,[11] a maximum figure of 100 mm was thought to be of the right order.

Fig. 18. Detail of 480 mm diameter cast-in-place driven piles showing the UBP steel sections and shear helical

Pile testing

59. To confirm the preliminary pile-bearing capacity computations and anticipated pile behaviour, an extensive programme of pile testing was carried out. A series of non-working preliminary test piles were installed following completion of the trials to finalize column spacing, treatment depth and effectiveness of the vibroreplacement. Testing included traditional static axial compression, tension and lateral load testing in both treated and untreated ground.

60. Dynamic CASE and CAPWAP testing[21] was also performed during installation of test piles to investigate pile load capacity and pile driving. CASE testing is carried out by monitoring force input, acceleration and strain response of the drive tube during driving. Analysis based on wave theory enables dynamic pile resistance to be predicted for each drive blow or set. The CAPWAP approach extends the CASE method by an iterative numerical approach where pile dynamic behaviour is matched to site measurements. Often the CAPWAP method allows development of a load settlement relationship for a pile.

61. Installation of test piles started with full-scale trial drives used together with pre-

and post-vibrotreatment CPT data to confirm the ground conditions and test pile design lengths. Figure 19 shows a layout plan of test area C showing locations of trial stone columns, test piles and dummy piles.

62. Test piles were installed both in treated and untreated soils. 480 mm cast-in-place piles were always installed into ground previously compacted using deep vibroreplacement techniques. Five non-working cast-in-place test piles were constructed and tested, one of which was installed with a soft toe to eliminate end bearing (Fig. 20). A total of eight 320 mm precast test piles were driven, five into natural untreated sands, the remainder into densified soil. One 380 mm cast-in-place test pile was installed into untreated ground.

63. All test piles were instrumented with various combinations of embedded concrete strain gauges, inclinometer access tubes and rod extensometers (Fig. 21). All testing was carried out using computer logging and control systems following specified loading and unloading cycles with a final constant rate of penetration cycle for axial compression tests. All piles were taken to failure where possible. In total, twelve axial compression, seven axial tension and ten lateral load tests were completed.

64. Each pile test was fully back-analysed to determine relevant design parameter information. Full use was made of existing borehole data, both pre- and post-treatment CPT logs and the pile drive records to assess the soil profile for each pile. Results of all tests were used to confirm the design lengths and to develop suitable drive criteria for installing the contract piles.

65. Full details of the testing programme, test results and back-analyses will be presented in a companion paper to be published later. However, typical examples are presented in the following sections.

66. As the contract progressed, additional non-working pile tests were carried out in other areas of the site. During the contract a total of twelve working piles were tested to 1·5 times their nominal working load. In all cases they confirmed load settlement behaviour similar to the preliminary test piles.

Axial compression

67. Cast-in-place piles of 480 mm diameter were always intended to be driven into ground previously treated by vibrodensification. Figure 22 shows a typical pile load against head settlement relationship measured during testing of pile C1 located in test area C. Figures 23 and 24 show corresponding plots for precast pile P9 driven into densified soil, and precast pile P5 driven about 20 m away into untreated soil. These plots include end-bearing and shaft friction components determined using strain gauge instrumentation.

68. Pile test results clearly show the significant improvement in soil performance due to the vibrodensification. In general it was not possible to measure an ultimate end-bearing capacity for the 480 mm cast-in-place piles, although use of the strain gauge instrumentation enabled shaft capacity to be estimated with

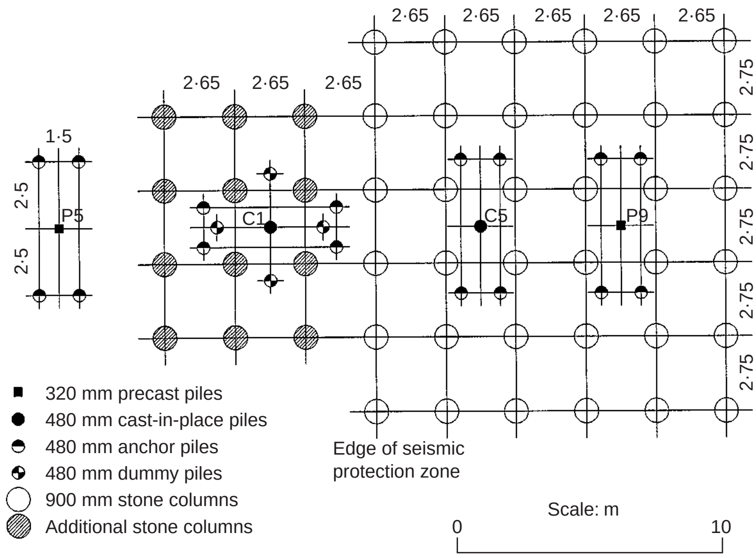

Fig. 19. Layout plan for test area C (dimensions in metres)

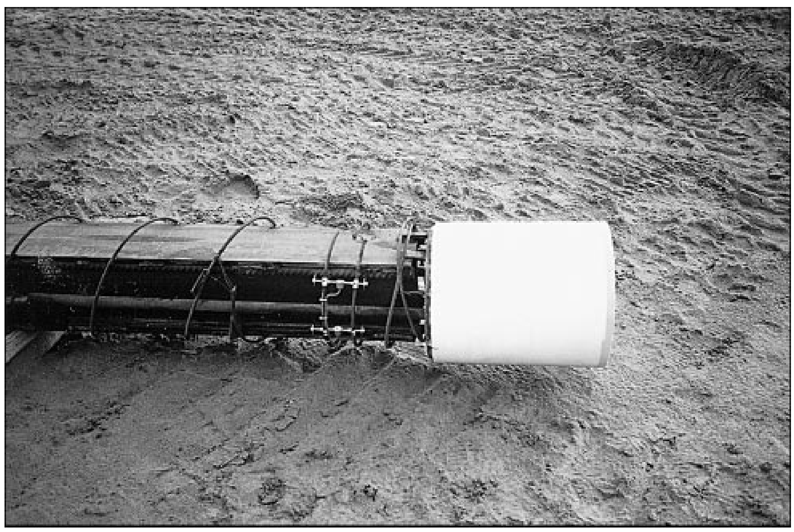

Fig. 20. Soft pile toe formed from polystyrene with access tubes for solvent to remove toe after concreting

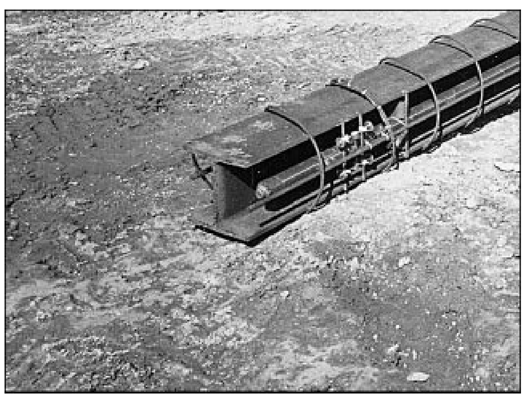

Fig. 21. Preliminary test pile reinforcement fitted with strain gauges, inclinometer access tubes and extensometer bars

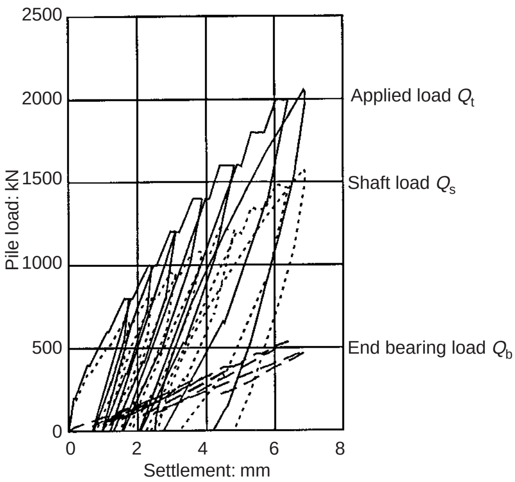

Fig. 22. Pile load plotted against settlement for pile C1

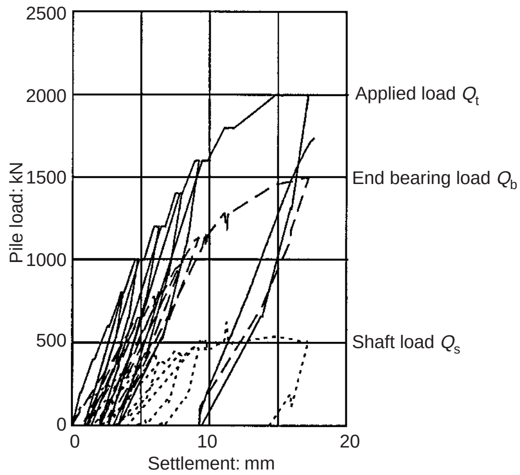

Fig. 23. Pile load plotted against settlement for pile P9

a reasonable degree of accuracy. Back-analysis of test results in general confirmed higher bearing capacities than those computed using the tabulated design parameters. Results for the precast test piles were generally more successful in measuring ultimate load capacities. These again confirmed the suitability of the design parameters.

Axial tension

69. A total of seven piles were tested to check tension behaviour. In all cases, piles were tested following completion of axial compression tests. Tension tests were not considered representative of working behaviour, because of the degree of pre-loading in compression, although they were able to confirm a minimum tension capacity. Tests carried out on precast piles in both treated and untreated soils gave similar results.

Horizontal loading

70. Testing of piles to confirm horizontal load capacity was more problematic, particularly when considering the seismic condition. It was not possible to reproduce full liquefaction or the working pile head fixity. However, analysis had shown that the partial liquefaction case was the more onerous design during which the upper shaft was expected to be supported by the sand fill and weak PFA.

71. The approach adopted was therefore to carry out load testing on a free headed pile supported solely by the fill and PFA. Back-analysis enabled confirmation of design parameters which could then be applied to the seismic design condition. Figure 25 shows a typical horizontal load deflection relationship recorded during testing of pile C1. Displacement profiles for each peak load stage measured using an inclinometer are shown in Fig. 26.

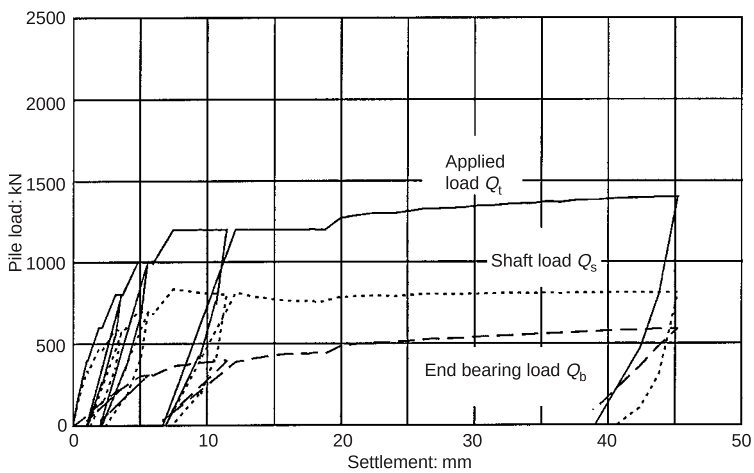

Fig. 24. Pile load plotted against settlement for pile P5

72. Test results were able to confirm the adequacy of the design assumptions.

Pile construction

73. Pile layout design was carried out using details of structural loads, plant locations and foundation footprints provided by British Gas

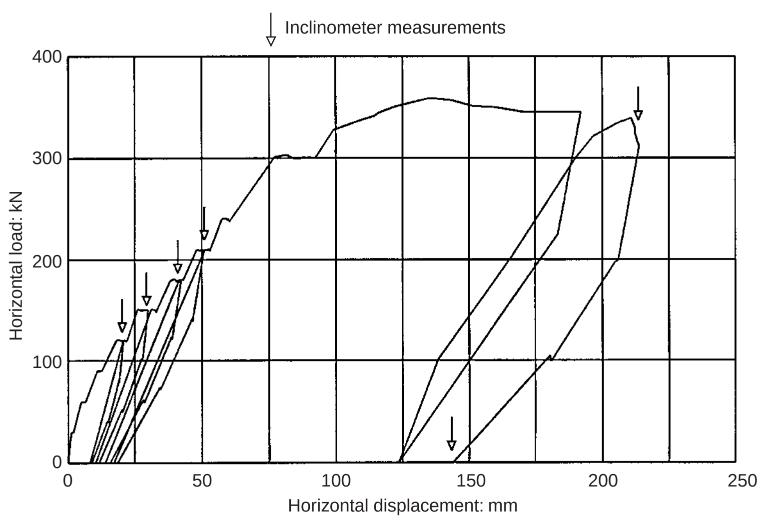

Fig. 25. Horizontal load plotted against deflection for pile C1

and their process plant designers (Fig. 27). Because of the need to ensure that stone columns did not obstruct pile positions, layout arrangement of the piles had to be determined before the positions of the stone columns could be finalized and working drawings prepared (Fig. 10). Design of the piles was straightforward using the operational and seismic load capacities obtained from the section analyses and from the detailed pile design. Layouts were determined to keep loads below the defined capacities for each load case (Table 2).

74. Adjustments to the required drive criteria and minimum pile lengths were defined based on the results of the site investigation, additional CPT profiles and trial drives carried out at various locations across the site.

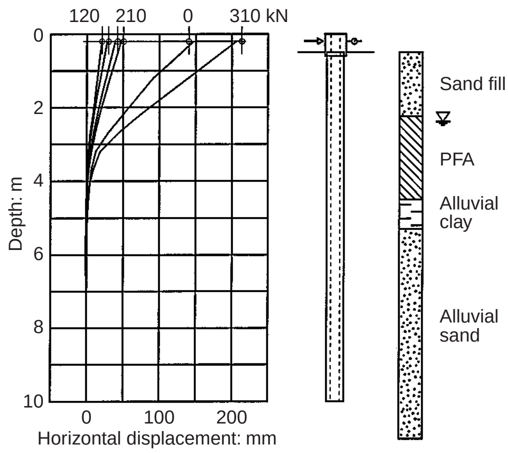

Fig. 26. Inclinometer measurements for pile C1

Fig. 27. Foundation footprint to a storage tank showing the pile heads and steel UBP sections

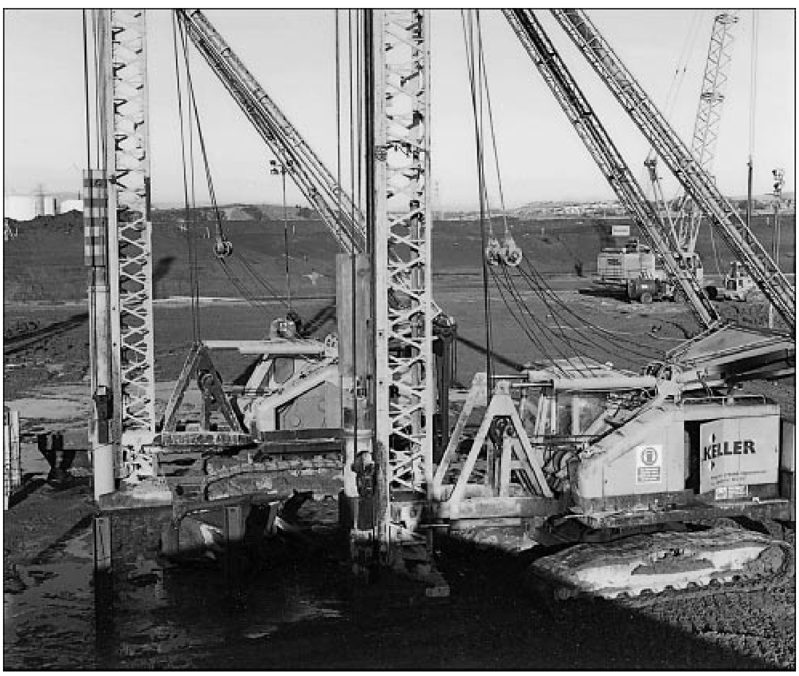

Fig. 28. Pile driving works showing installation of 480 mm diameter cast-in-place piles

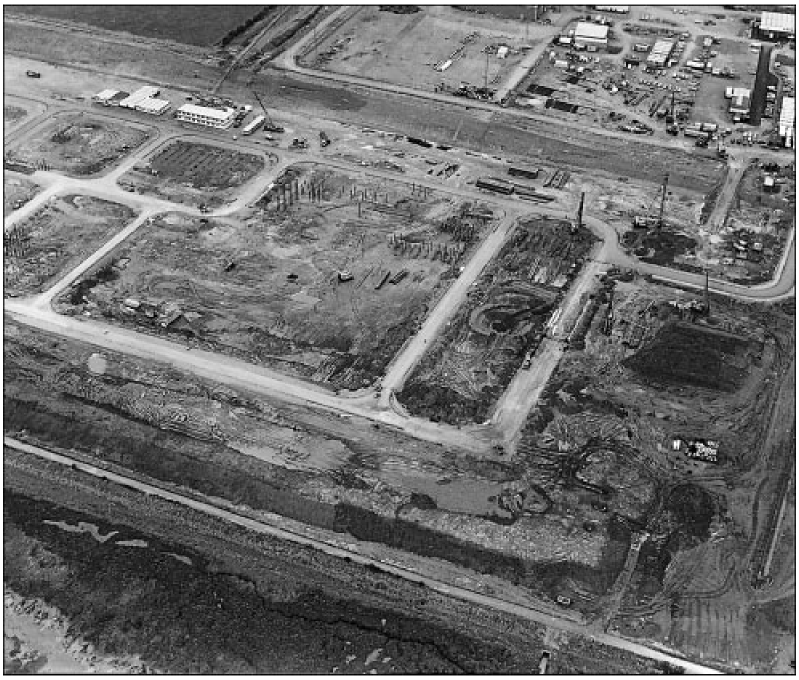

Fig. 29. Aerial view of the site from the south-west showing the piling works nearing completion

75. During the works, continual feedback took place to ensure foundations were being installed as designed. In particular, full drive records were taken for all precast and many driven cast-in-place piles, with sets for the last 2·5 m of the remaining piles being recorded. Pile lengths within the areas of ground improvement were also reviewed regularly as a further check on the treated density of the soils.

76. During the works, around 60 000 m of piling was installed. This comprised about 2400 precast, 1700 480 mm cast-in-place and 1600 380 mm cast-in-place piles, together with test and anchor piles (Fig. 28). Piling works were completed within a nine-month programme using up to ten piling rigs (Fig. 29). A general description of the site works is given by Ground Engineering.[1]

Conclusion

77. This paper has described the solution developed by Keller to the problems of poor ground conditions and the need to design for specified levels of seismic risk. It has also presented the background to a complex and unusual pile design, both in terms of geotechnical and structural engineering. An outline of the design justification by extensive on-site testing has also been given.

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