DESIGN OF PILE FOUNDATIONS IN LIQUEFIABLE GROUND: A CASE STUDY IN FRASER DELTA, BRITISH COLUMBIA

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ABSTRACT

Observations from sites in Japan that suffered liquefaction during the 1964 Niigata earthquake and where large permanent ground deformations occurred beneath pile-supported buildings illustrate convincingly that foundation piles in liquefied soils can pull away from the pile caps and they can break at interfaces between liquefied and nonliquefied soil layers where maximum pile curvatures occur. Data from a case study in the design of pile foundations in potentially liquefiable soils in the Fraser delta are presented. The axial and lateral behaviours of steel pipe piles are discussed. Axial compression load testing on an instrumented 25 m long 508 mm diameter steel pipe pile shows that residual stresses in the pile must be considered if correct shaft and toe resistances are to be determined for the design of piles in liquefied ground. Pseudo-seismic lateral pile analysis, which considers the nonlinearity of both pile and soil, can adequately simulate the behaviour of piles subjected to large horizontal ground displacements. The results show that concrete-filled steel pipe piles can be designed to accommodate the anticipated large ground deformations caused by liquefaction.

INTRODUCTION

Since the early 1980s, much has been written on the amount of permanent ground displacements that have occurred as a result of liquefaction in past earthquakes. The damaging effects of ground displacements on pile foundations were best illustrated when broken reinforced concrete piles were discovered under several buildings in Japan during excavation for reconstruction in the mid 1980s. These sites had experienced liquefaction and suffered permanent vertical and lateral ground displacements of 1 to 2 m during the M7.5 Niigata earthquake in 1964. The observations and subsequent analyses confirmed that the piles were broken at the interface between the liquefied and nonliquefied soils where maximum curvature or change of shear strain occurred. At some sites where liquefaction occurred below pile toes, the piles could not be located upon excavation of the pile caps. It was speculated that the piles might have sunk under their own weight in the liquefied soils. These observations highlight the importance of considering both the axial and lateral behaviour of piles in potentially liquefiable ground for aseismic design of pile foundations.

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This paper presents data from a case study in the design of 508 mm diameter steel pipe piles in liquefiable soils in the Fraser delta. The case study was for an expansion to a manufacturing plant in Tilbury Industrial Park, Delta. The major design considerations were the reduction of axial pile capacity in liquefiable soils and the seismic behaviour of piles subjected to large liquefaction-induced lateral ground displacements. Data from a full-scale instrumented pile load test conducted at the site are presented to address the axial load distribution in a long pile. Results of a pseudo-seismic lateral analysis of the foundation piles, which considers not only the nonlinearity of the stress-strain property of soils but also treats the pile as a nonlinear beam element, are described.

SITE CONDITIONS

The site is located in the Fraser River delta, an area particularly prone to damage in the event of a nearby moderate earthquake or a large subduction earthquake (Rogers, 1992). Typical of the Fraser delta, the site is underlain by deep, Recent deposits of relatively loose soils susceptible to liquefaction. Ground amplification through the deep, soft sediments and strength loss or liquefaction of saturated soils are the primary concerns for aseismic design of structures in the area (Byrne and Anderson, 1987; Lo et al. 1991; Sy et al. 1991). In fact, liquefaction features have recently been observed at several sites in the Fraser delta (Clague et al. 1992; Naesgaard et al. 1992) and guidelines have been proposed for liquefaction assessment in the region (Richmond Task Force Group 1991).

The soil profile at the site consists of 2 m of sand fill overlying 2 m of silt which overlies approximately 4 m of interbedded sand and silt. Beneath the sand and silt deposit is loose to medium dense, fine to medium grained sands extending to 30 m depth. The sand deposits are underlain by a deep, compressible marine silt deposit. Figure 1 shows cone penetration test (CPT) and standard penetration test (SPT) data at this site. The groundwater table varies seasonally but is typically about 1 to 2 m below site grade. Analyses indicate that the site is susceptible to liquefaction to a depth of about 18 m under the design earthquake.

The existing building foundations were not designed against extensive liquefaction. Typical of heavy and settlement-sensitive buildings in the Fraser delta, the existing structure was supported on piles, subsequent to preloading of the ground to minimize future long-term differential settlements. A combination of low capacity timber piles and high capacity cast-in-situ expanded base concrete piles was used for the building and machinery foundations. The ground floor slab was supported on grade.

The expansion site under consideration was also preloaded during the original construction. Preloading had increased the CPT and SPT penetration resistances of the sandy soils underlying the site, but the increase was not sufficient to preclude liquefaction due to the design earthquake.

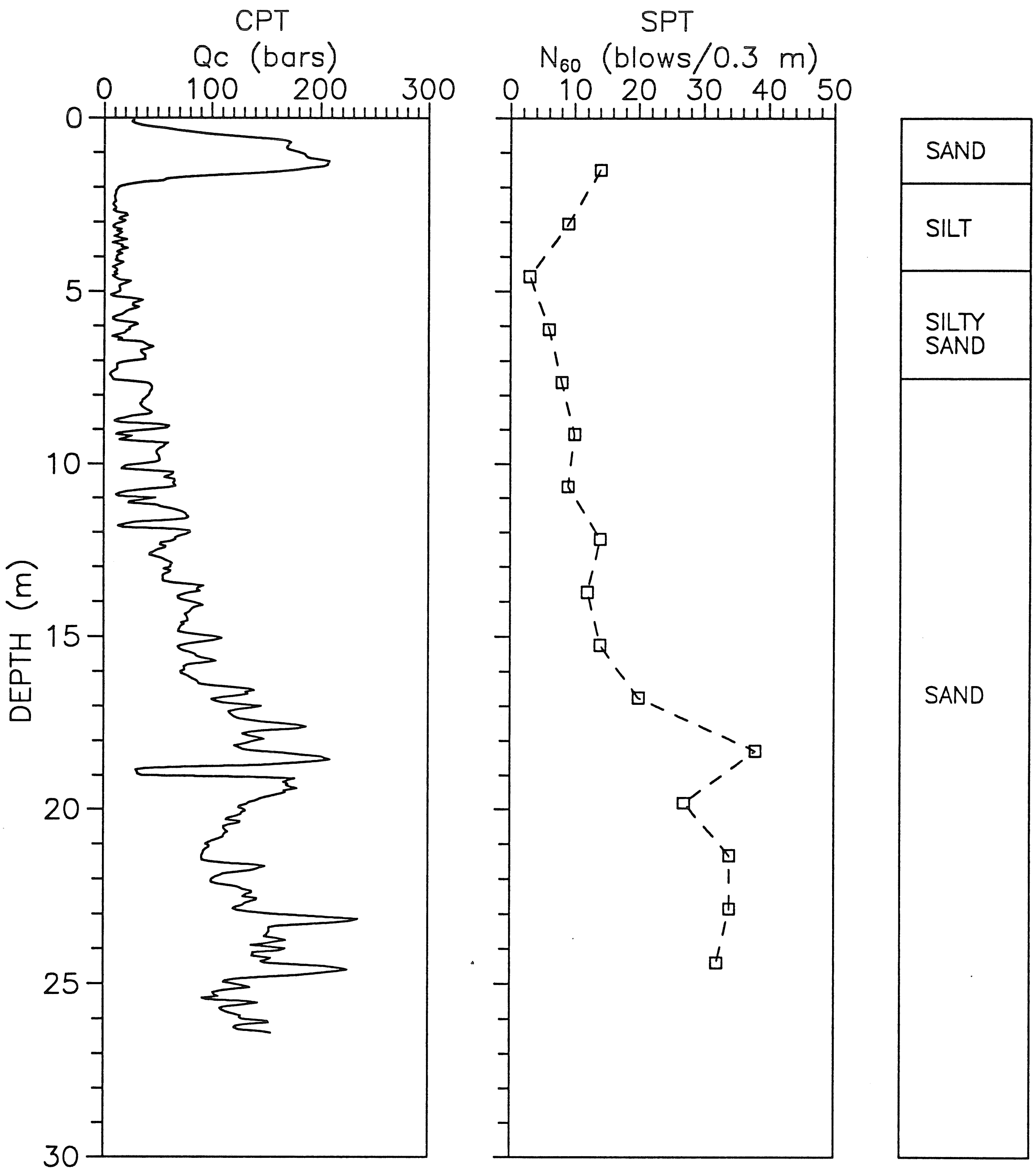

Fig. 1 Cone penetration test (CPT) and standard penetration test (SPT) data

DESIGN METHODOLOGY FOR PILE FOUNDATIONS IN LIQUEFIABLE SOILS

Liquefaction of foundations soils will result in permanent vertical and horizontal ground displacements during earthquake shaking. The magnitude of these displacements depends on many factors including earthquake intensity and duration of shaking, soil density, grain size, thickness of liquefied layer, and ground slope. Although predictions of these ground displacements involve many uncertainties, analytical and empirical solutions are available to provide estimates (Tokimatsu and Seed, 1987; Bartlett and Youd, 1992). Analyses suggest that the site could experience settlements of 0.3 m and horizontal ground displacements of up to about 1 m due to seismically induced liquefaction.

Two basic options are available to mitigate the effects of liquefaction on structure foundations. Firstly, the ground can be improved to prevent liquefaction and, thereby, avoid the large liquefaction-induced ground displacements. In the Fraser delta, vibro-compaction with stone columns and dynamic compaction are commonly used. The alternative is to design the structure and its foundations to withstand the anticipated vertical and horizontal ground displacements without collapse during the earthquake. The latter option was selected for this site based on cost comparison.

Several pile types were considered for the proposed expansion. Steel piles are generally not considered economical for the soil conditions in the Fraser delta because the geotechnical capacities of these piles are substantially less than their structural capacities. In this case, however, because the foundation soil was not densified to prevent liquefaction, flexible ductile steel piles were expected to perform better than timber or concrete piles. This opinion was supported by a series of full-scale lateral pile load tests conducted by Naesgaard (1992) at a site in the Fraser delta. The test program consisted of imposing large lateral pile head displacements on a concrete-filled steel pipe pile and on two reinforced concrete piles, all with a diameter of 406 mm. The results indicated that under large liquefaction-induced lateral displacements, concrete-filled steel pipe piles would perform better than concrete piles with internal steel reinforcement.

As most pile foundations are relatively flexible structural elements, they tend to follow the ground movements instead of resisting them. Large ground movements can impose significant bending moments on the piles as illustrated by the extracted broken piles in Japan. The steel piles are expected to follow the ground and will yield under the anticipated large lateral ground movement. Upon yielding, plastic hinges will form at the top and bottom interfaces between the liquefied and nonliquefied soil layers. Therefore, it is important for the pile section to have adequate ductility upon yielding to withstand the imposed deformation without rupturing. The steel pipe piles should also be filled with concrete to prevent any local buckling of the pipe wall. In the event that the steel pile is cracked by bending, the concrete core, confined by the hoop strength of the steel pipe, can still provide substantial compressive strength.

The toes of the steel piles must be set below the depth of liquefaction. Adequate pile penetration in the nonliquefied soil is required to carry the column loads and the drag load in the liquefied soil as the liquefied layer consolidates. For liquefaction condition, the pile capacity is reduced due to the loss of soil support and the addition of drag load in the liquefied soil. Analysis for this site suggested that piles driven to 24.4 m depth could provide a vertical compressive capacity of 620 kN with adequate factor of safety, even if the top 18 m of soil liquefies. It is important, however, to accurately estimate the load distribution in the piles as discussed later in this paper.

Based on the above considerations, 508 mm diameter concrete-filled steel pipe piles were selected for the new building foundations. The rest of this paper discusses the two major design issues concerning pile foundations in liquefiable soils, namely,

1. Axial pile behaviour in liquefied soils, and
2. Lateral pile behaviour due to liquefaction-induced deformation.

AXIAL PILE BEHAVIOUR

Research at the University of British Columbia (Robertson et al. 1988) has shown that the axial capacities of steel pipe piles in the Fraser delta can be reliably estimated by static pile analysis based on CPT data and correlations proposed by Bustamante and Gianeselli (1982). The research was based on load testing of a series of full-scale steel pipe piles installed to different depths at a research site. Although the pile capacities were well predicted, the axial load distribution along the pile length was not verified by field measurements.

The actual distribution of shaft and toe resistances is important in this case study because some of the soil layers will liquefy and apply a drag load on the pile while some others will lose strength due to pore pressure build-up caused by cyclic loading. Accordingly, a pile load testing program was conducted which included an axial compression test on a 25 m long, instrumented steel pipe pile at this site. The 508 mm diameter by 12.7 mm thick wall steel pipe pile was driven close-ended to a final penetration of 24.4 m below grade with a Delmag single-acting diesel hammer having a manufacturer's rated energy of 89.5 kJ. Figure 2 shows the pile driving resistance profile (blows per 0.3 m) of the test pile, as well as locations of the 5 pairs of electrical strain gauges installed along the pile length.

The test pile was load tested to failure 17 days after it was driven. The load was applied in 89 kN increments with a constant holding time of 10 minutes for each load increment. Readings from the pile strain gauges, load cell, and linear motion position transducers were recorded automatically on a data acquisition system.

Figure 3 shows the pile top load versus pile top movement data, as well as the Davisson's offset limit line. A maximum load of 2850 kN was applied to the pile top during the load testing. Based on the Davisson's criterion, the failure load for the test pile was interpreted as 2400 kN.

The conventional interpretation of load test data assumes that the test pile is stress-free before load testing. Assuming then that there was no load on the 25 m long test pile prior to the load test (i.e. with all the strain gauges zeroed at the beginning of the load test), the axial load distribution at various stages of the load test is shown on Fig. 4. The data suggest that about 18%, i.e. 440 kN, of the failure load was resisted by the pile toe and the remaining load, i.e.

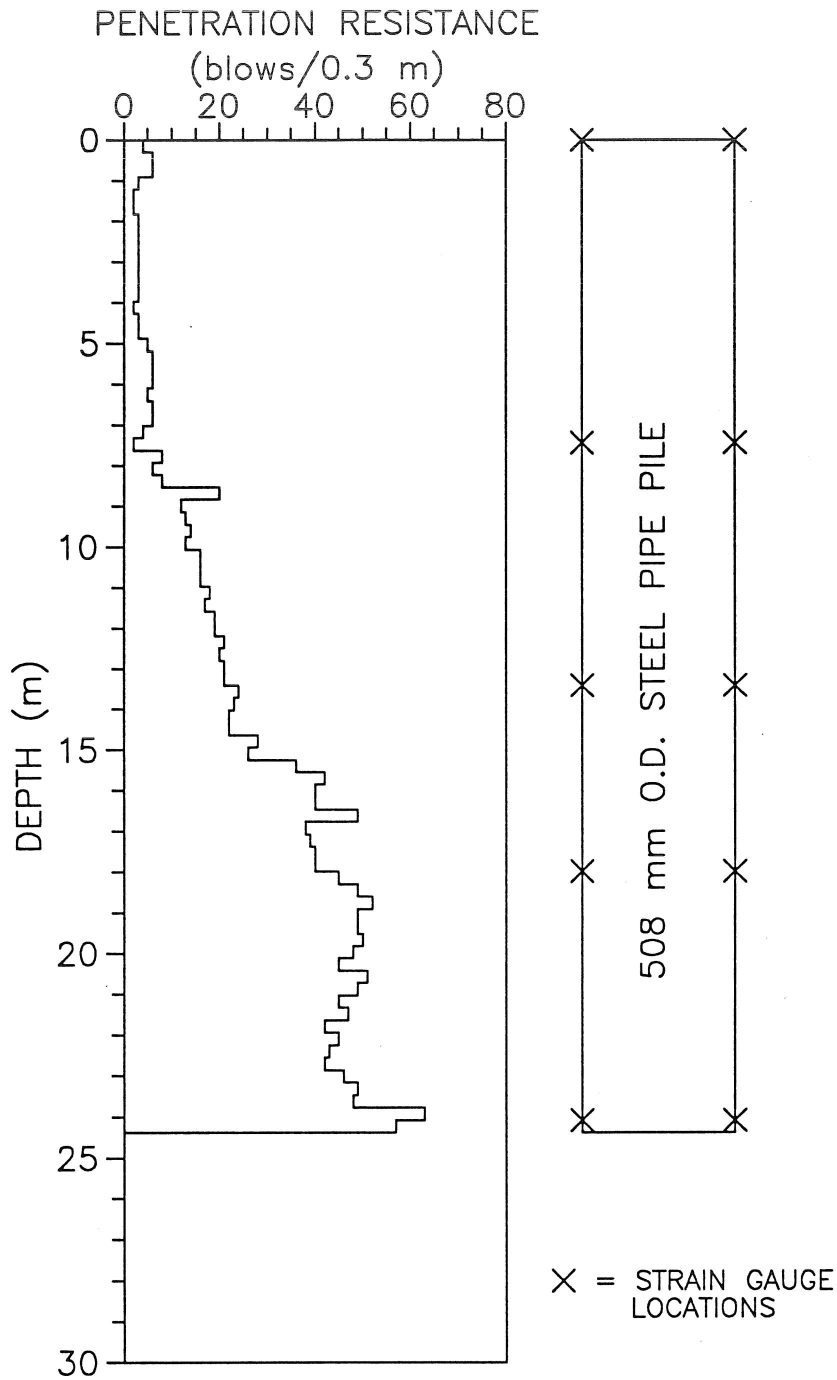

Fig. 2 Pile penetration resistance diagram and strain gauge locations

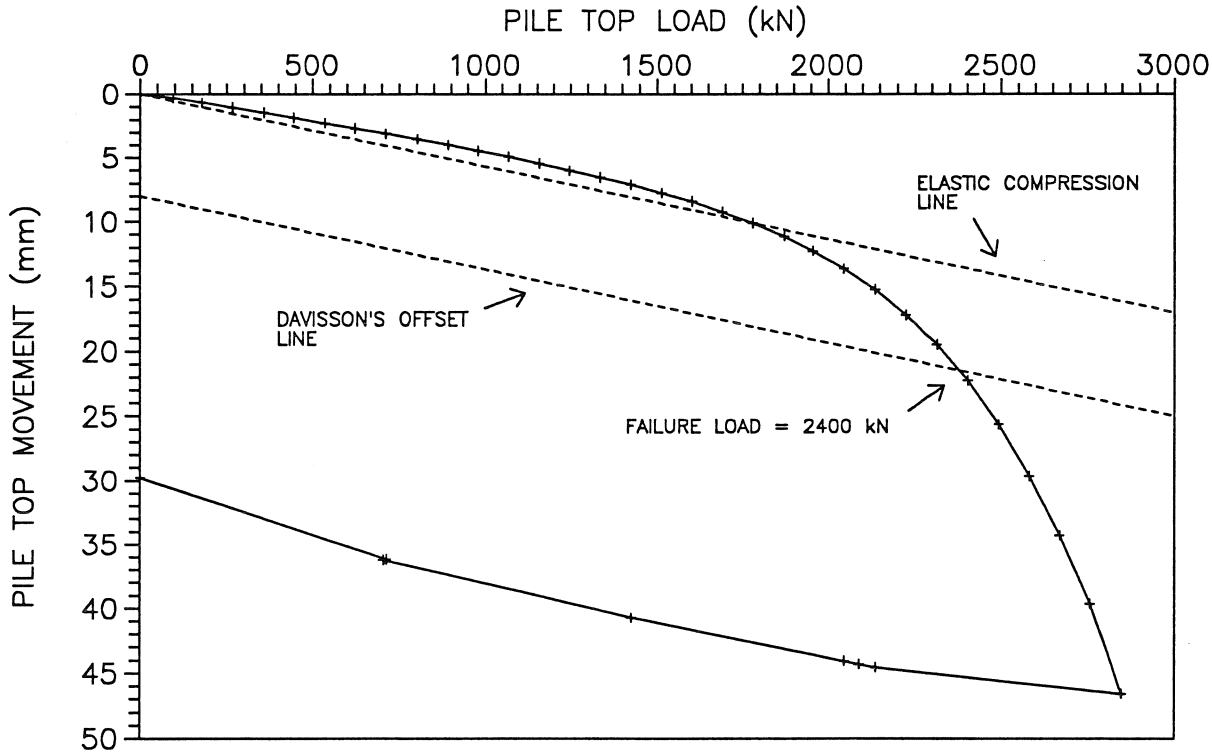

Fig. 3 Pile load test results

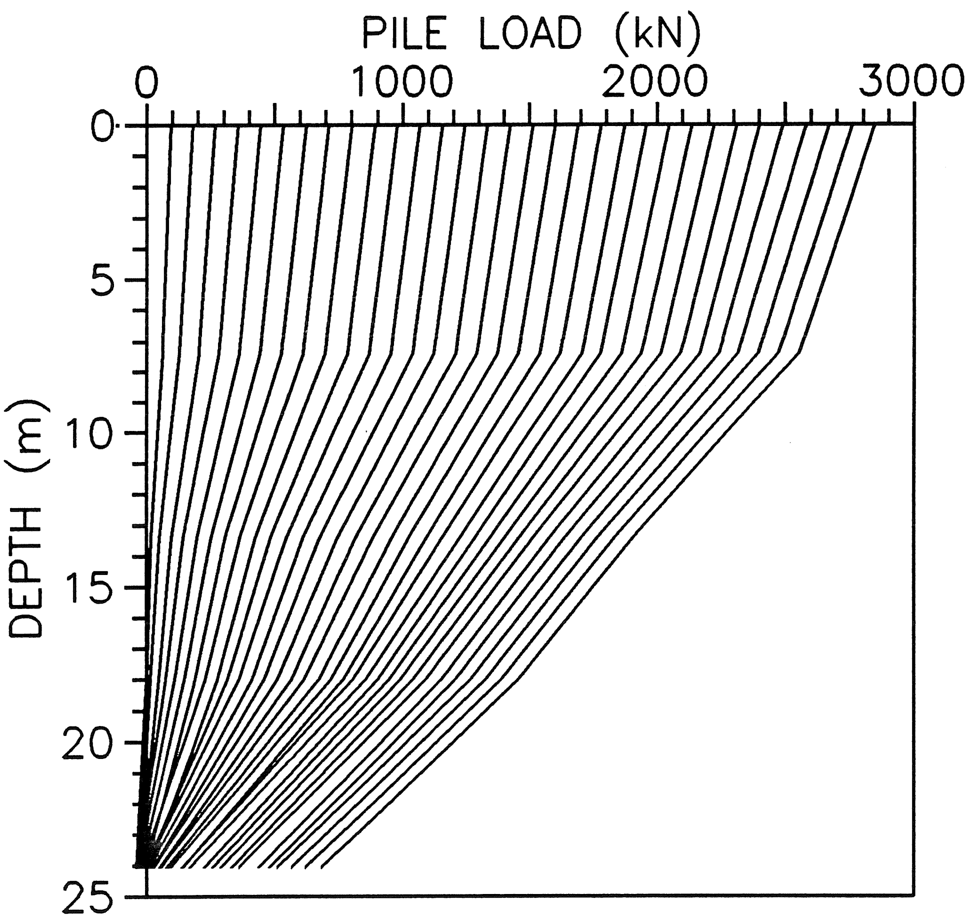

Fig. 4 Axial load distribution without residual load

1960 kN, was resisted along the shaft. The data further indicate that the portion of the shaft resistance in the upper 18 m of the soil was approximately 1250 kN. This shaft resistance value was considered too high since another load test conducted on a nearby steel pipe pile with the same diameter but with only 18 m embedment resulted in a total failure load of only 1050 kN. This incorrect interpretation of load transfer in the 25 m long test pile is due to the neglect of residual load in the pile as discussed below.

The existence of residual stresses in driven piles after installation has been recognized for some time but it has not been routinely included in pile design (Hunter and Davisson, 1969; Holloway et al. 1978; Briaud and Tucker, 1984). The residual load in the pile before load testing can be significant, coming from the impact driving and subsequent reconsolidation of soils around the pile after driving. Although ignoring residual load in the interpretation of the pile load test results does not affect the determination of the failure load or total pile capacity, it results in an erroneous distribution of pile load by underestimating the pile toe resistance and overestimating the pile shaft resistance. This is illustrated nicely in Fig. 5 from Holloway et al. (1978).

Figure 6 shows the axial load distribution considering residual load. There was some problem with zero shifts in the strain gauge measurements during pile driving. As a result, the residual load shown in Fig. 6 represents a best estimate based on a comparison of the actual strain gauge readings, load test data on the nearby 18 m long pile, and estimates based on analytical methods. As shown by comparing Figs. 4 and 6, the effect of including residual load is to increase the pile toe resistance from 440 kN to 860 kN and to decrease the shaft resistance from 1960 kN to 1540 kN. The shaft resistance in the upper 18 m is about 660 kN, which compares well to that inferred from the nearby 18 m pile load test. The average unit soil resistances for the various soil strata are summarized in Table 1. These values are calculated from the load transfer data at the failure load determined by the Davisson's criterion, with and without considering residual load.

Table 1. Unit Soil Resistances Considering Residual Load

DEPTH (m)	SOIL TYPE	PILE COMPONENT	UNIT RESISTANCE (kPa)	
			Without residual load	With residual load
0 to 7.4	SILT	SHAFT	20	11
7.4 to 18.3	SAND	SHAFT	60	32
18.3 to 24.4	SAND	SHAFT	74	89
24.4	SAND	TOE	2150	4240

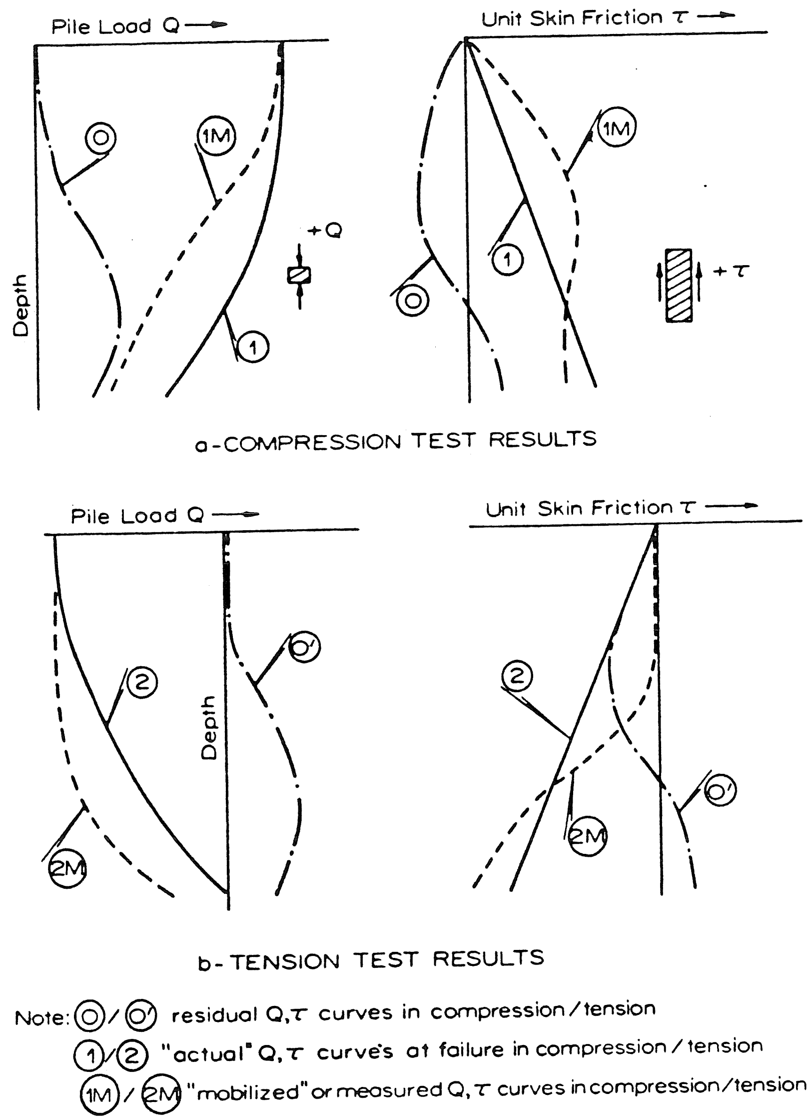

Fig. 5 Effects of residual load on pile load distributions (Holloway et al. 1978)

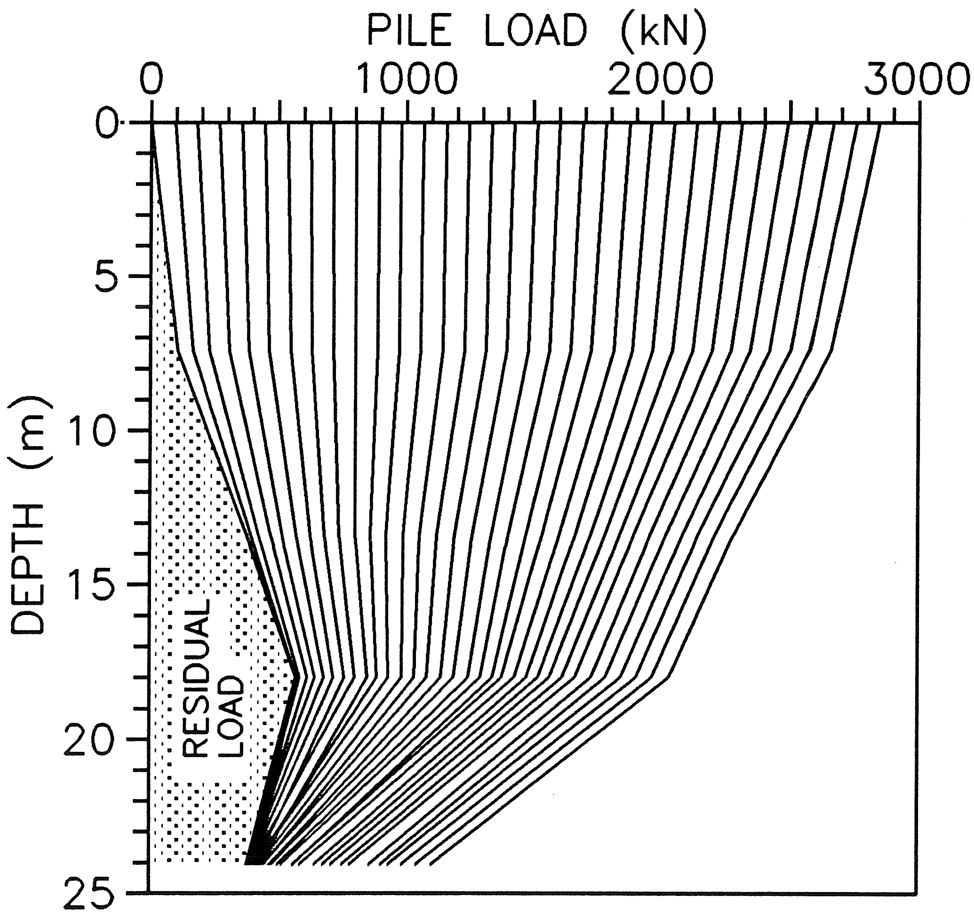

Fig. 6 Axial load distribution with residual load

LATERAL PILE BEHAVIOUR

To assess the lateral seismic behaviour of the concrete-filled steel pipe piles, a pseudo-seismic analysis was carried out using a modified version of the computer program LATPILE (Byrne et al. 1984). LATPILE computes the lateral response of a single pile subjected to pile head loads and free-field ground movements using a finite difference approach. The soil is replaced by a system of horizontal nonlinear springs, called p-y curves. The free field ground movements are then applied onto the pile through the outer ends of these soil springs as shown on Fig. 7. The soil reaction term "p" then becomes a function of the relative displacements between the pile and the free-field movements. Nonlinearity of the soil is accounted for by using a secant soil stiffness of the p-y curve and an iterative procedure.

In conventional static lateral pile analysis, the pile is modeled as linear elastic beam elements. This is appropriate for piles designed to remain elastic under static working loads. However, it is often uneconomical or unfeasible to design piles to remain elastic for earthquake conditions, when subjected to large ground movements or large inertial forces. A modified version of LATPILE, called LATPILEN, was used for this study to account for the pile nonlinearity. Consistent with the procedure for accounting for soil nonlinearity by the use of p-y curves, pile nonlinearity is accounted for by using a secant flexural stiffness, EI, based on the moment-curvature relationship of a pile section (Fig. 7). An appropriate secant flexural stiffness is selected for each pile element depending on the computed pile curvature. As the pile is loaded beyond the elastic range, the secant flexural stiffness reduces as the pile curvature increases. In each iteration, both the secant soil stiffness and flexural pile stiffness are updated depending on the computed relative pile-soil displacements and pile curvatures, respectively, until a prescribed convergence criterion is satisfied.

LATPILEN analyses were conducted to determine the effects of the liquefaction-induced permanent ground movements on the lateral response of 508 mm diameter concrete-filled steel pipe piles. The piles, designed as a combination of friction and end-bearing piles, are 24.4 m long. The 508 mm steel pipe has a wall thickness of 9.5 mm. The concrete has a 28-day compressive strength of 35 MPa and is reinforced with six No. 25 steel bars.

Analyses using both linear elastic pile elements with a constant flexural stiffness (EI) of 137.5 MN-m^2 and nonlinear pile elements with varying secant flexural stiffness were performed. Figure 8 shows the moment curvature relationship for the pile section estimated using the computer program PMEIX (Reese, 1984). This procedure computes the location of the neutral axis of the composite pile section by a trial and error process. Cracking of the concrete is considered in the analysis and, therefore, requires an input of the axial compression load.

Soil nonlinearity was modeled by means of p-y curves. For static conditions, these curves were estimated based on procedures recommended in API (1987). The input soil parameters are summarized in Table 2. The groundwater table was assumed to be at 2 m depth.

No established procedures currently exist to estimate p-y curves for liquefied soils. In this analysis, the soil layers deemed to liquefy during the design earthquake ground motions (i.e. between 4 and 18 m depths) were softened by reducing the soil resistance, p, of the p-y curves

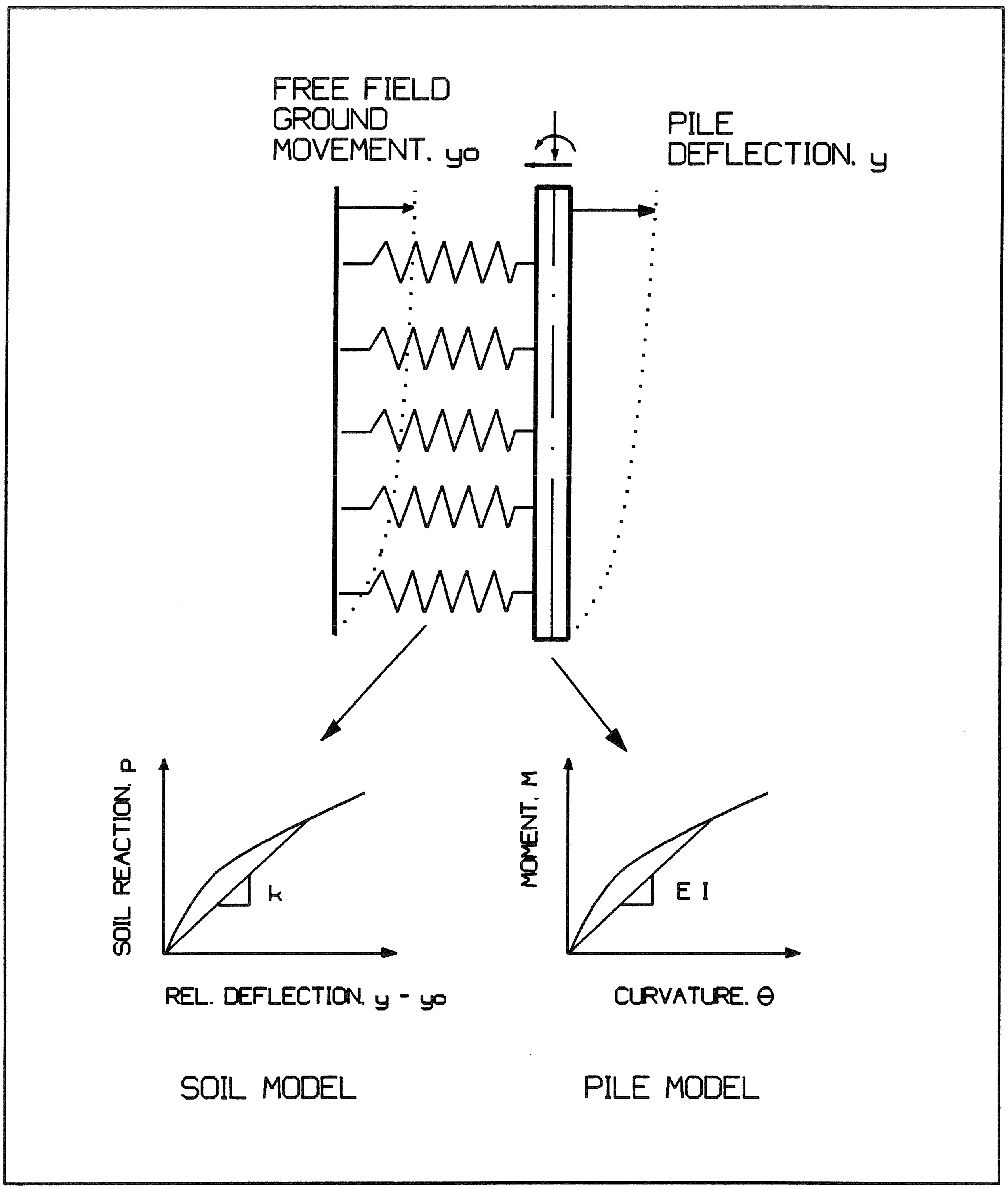

Fig. 7 Lateral pile analysis with nonlinear soil and pile models

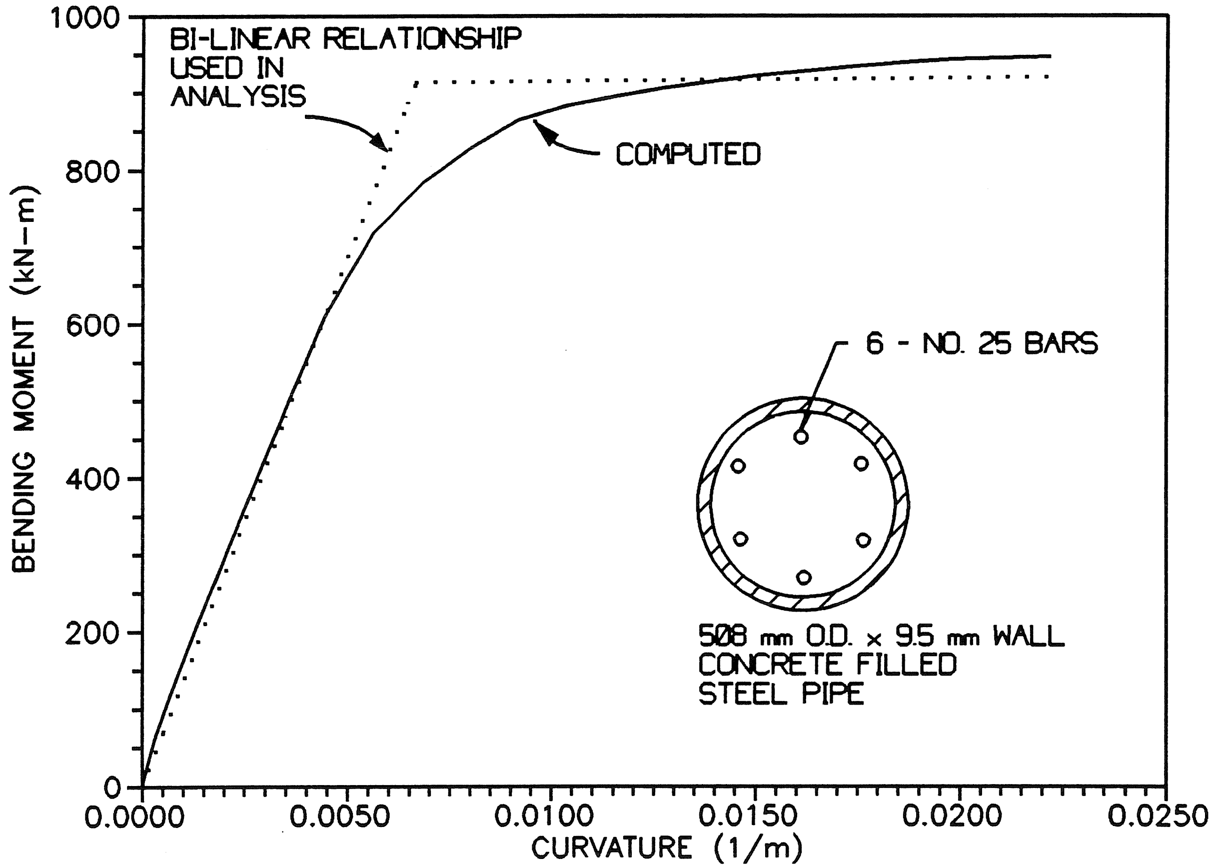

Fig. 8 Moment-curvature relationship for concrete-filled steel pipe pile

by a factor of 1.0 (no reduction), 0.01 and 0.001. This approach appears to be consistent with the anticipated behaviour of liquefied sands that both the soil stiffness and ultimate resistance would be reduced upon liquefaction.

Table 2. Input Soil Parameters for Derivation of P-Y Curves

DEPTH (m)	SOIL TYPE	$(N_1)_{60}$	FRICTION ANGLE
0 - 2	sand fill	14	36°
2 - 4	silt	10	28°
4 - 8	silty sand	6 - 7	31°
8 - 18	sand	10 - 18	33° - 38°
18 - 24	sand	24	40°

Since the site is relatively level, the permanent ground displacements would be dominated by the displacements within the liquefied soils. In the analysis, the free-field ground displacement profile was assumed to increase linearly from zero at the lower boundary of the liquefied layer (i.e. 18.3 m depth) to the full surface value at the top boundary of the liquefied layer (i.e. 4 m depth). Displacement above the liquefied soil layer was assumed to be constant. The six imposed displacement profiles in the analyses, i.e. with surface displacements of 0.05, 0.25, 0.5, 0.75, 1.0 and 1.25 m, are shown on Figure 9.

Results from the LATPILEN analyses for the linear elastic pile elements and nonlinear pile elements are shown on Fig. 10 and 11, respectively. Computed pile deflection and bending moment profiles for the three soil resistance values (i.e. 1.0, 0.01 and 0.001 of p in the liquefied zone) are presented. For clarity, only the results for the imposed surface ground displacement of 0.25 m, 0.75 m and 1.25 m are shown. The pile head was taken to be pinned at the pile cap.

For both cases with and without accounting for pile nonlinearity, the pile tends to move with the ground instead of resisting it. The higher the soil resistance in the liquefied layer, the closer the pile tracks with the free-field ground movements. The differential pile-soil movements within the liquefied soil layer also reduce significantly if pile nonlinearity is accounted for, i.e. nonlinear pile tends to follow the ground movement more closely. This is likely due to the "restraightening" of the pile as plastic hinges form at the upper and lower boundaries of the liquefied soil layer.

For all cases, the maximum pile bending moments occur at the lower and upper boundaries of the liquefied soil layer. The magnitude of these maximum bending moments depends on the soil stiffness in the liquefied soil. The higher the soil stiffness, the higher is the computed maximum bending moment.

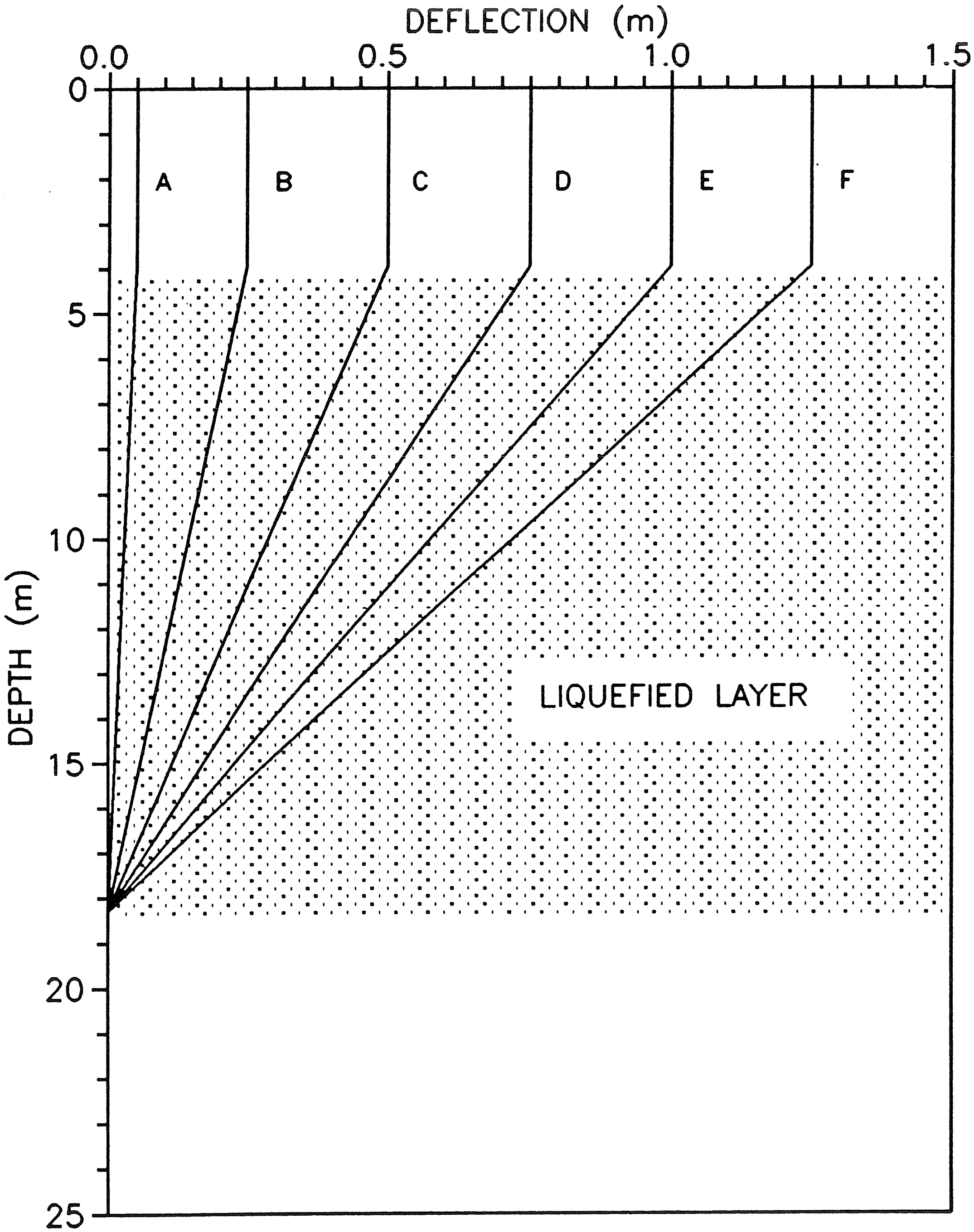

Fig. 9 Free-field ground displacement profiles used in analysis

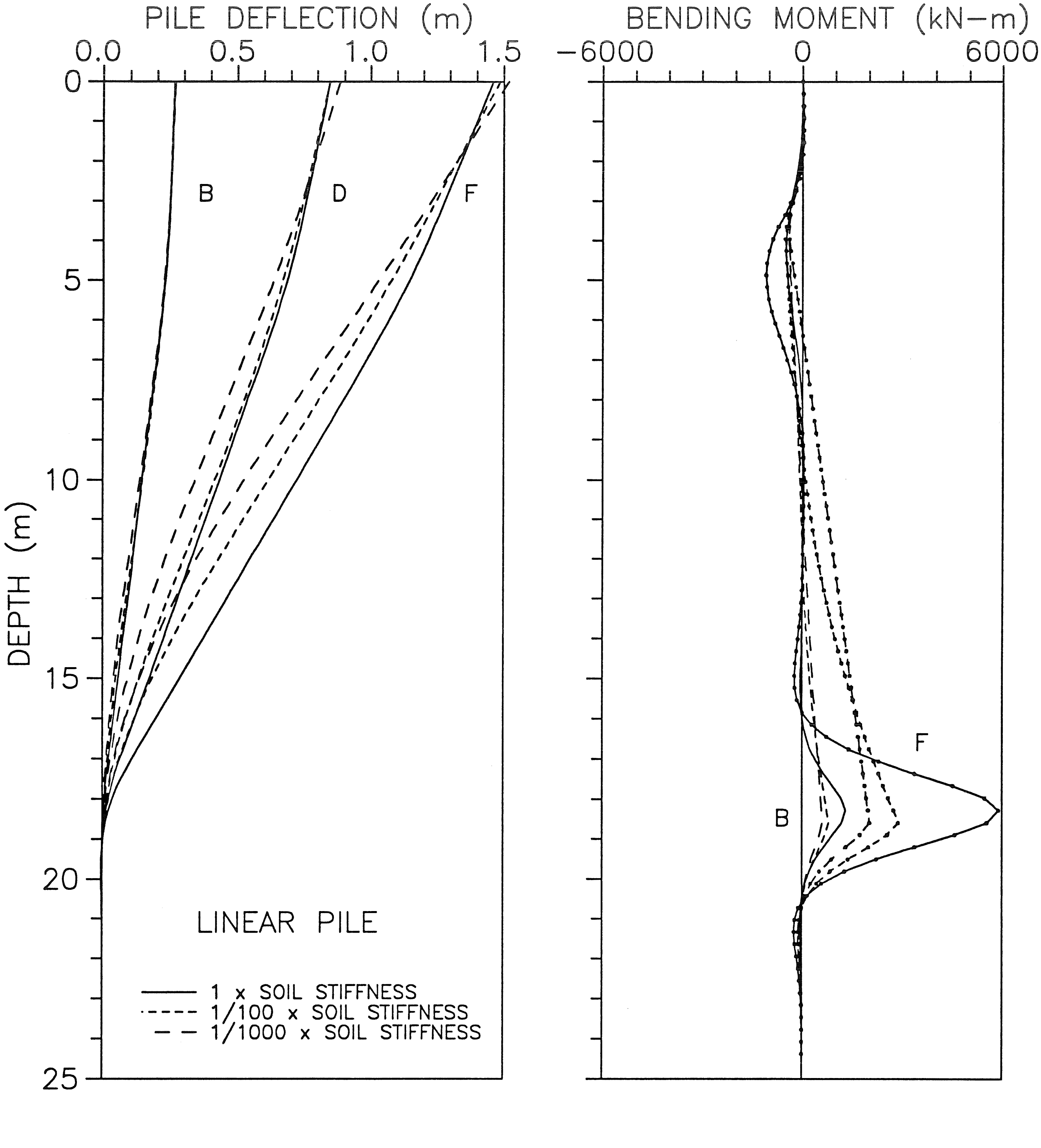

Fig. 10 Computed deflection and bending moment profiles for linear pile

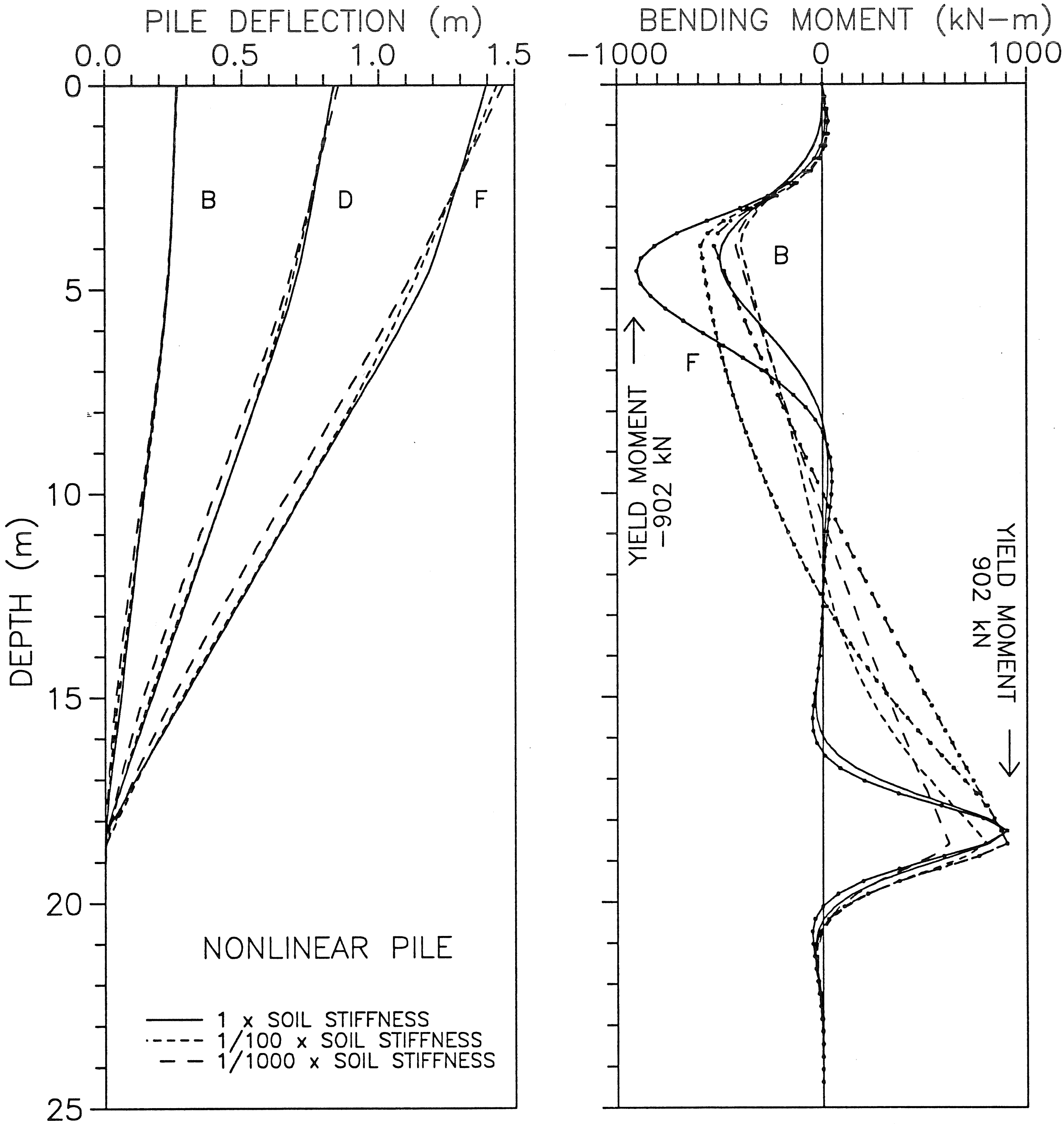

Fig. 11 Computed deflection and bending moment profiles for nonlinear pile

Within the liquefied layer, the pile bending moments are essentially zero for the case with no reduction in static soil stiffness within the liquefied zone. This is because the portion of the pile within the liquefied layer follows the free-field movements almost exactly, with minimal differential pile-soil movements. As the soil stiffness reduces in the liquefied soil layer, the pile tends to move less than the ground, causing the bending moment to increase.

Figure 12 shows the computed pile bending moments at the upper and lower boundaries of the liquefied layer as a function of the imposed surface ground displacements. The results shown are for the nonlinear pile condition and for the static soil stiffness with no reduction. The bending moment at the lower boundary is consistently higher than that at the upper boundary, suggesting that the lower plastic hinge forms first. The analyses suggest that the lower plastic hinge would form at a surface ground movement of about 0.25 m, whereas a displacement of 0.75 m is required before the upper plastic hinge forms. As expected, the assumption of linear elastic pile predicts bending moment which is unrealistic and much higher than the moment capacity of the pile section.

Analyses were also performed for other conditions to check the sensitivity of the results, including fixed-head pile, effect of inertia shear load, thicknesses of nonliquefied and liquefied layers, and different ground displacement profile shapes. Similar general conclusions were reached for these conditions.

SUMMARY AND CONCLUSIONS

Some aspects of the axial and lateral behaviours of pile foundations in liquefiable soils at a site in the Fraser delta are discussed in this paper. An axial static load test was conducted on a 25 m long 508 mm diameter instrumented steel pipe pile to obtain the load distribution along the pile length. The results show that residual stresses in the pile must be considered to obtain reliable shaft and toe resistances for design of piles in liquefied ground. Pseudo-seismic pile analyses using the LATPILEN program, which accounts for the nonlinearity of the pile and soil, can adequately simulate the expected behaviour of piles subjected to large liquefaction-induced horizontal ground displacements. It is concluded that concrete-filled steel pipe piles at this site can be designed to withstand the anticipated large permanent ground deformations caused by soil liquefaction. Another factor that must be considered is the post-seismic performance of the building.

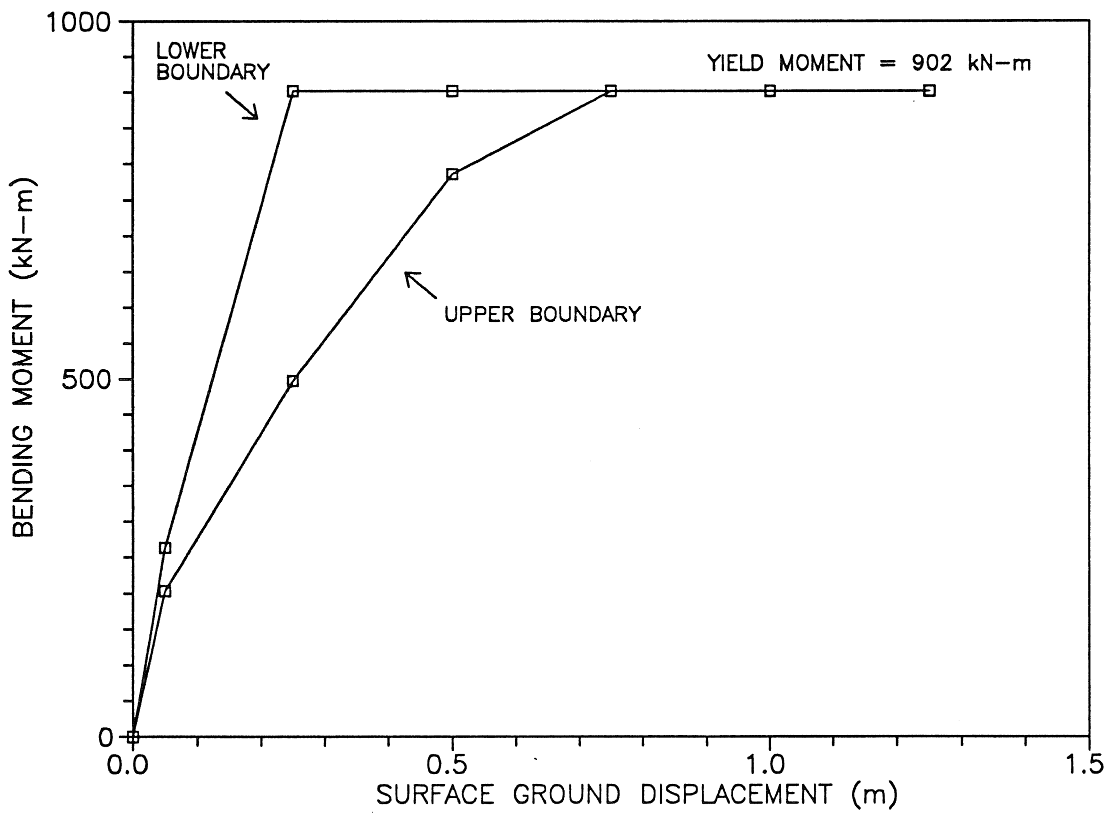

Fig. 12 Surface ground displacement vs. maximum pile bending moment

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