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Behavior of Piles in Liquefiable Soils During Earthquakes: Analysis and Design Issues

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ABSTRACT

A general picture of the current state of the art and the emerging technology for dealing effectively with the seismic design and analysis of pile foundations in liquefiable soils is presented. Two distinct design cases are considered and illustrated by case histories. One is the static response of pile foundations to the pressures and displacements caused by lateral spreading of liquefied ground. The other is the seismic response of piles to strong shaking accompanied by the development of high pore water pressures or liquefaction. Design for lateral spreading is examined in the context of developments in design practice and the findings from shake table and centrifuge tests. Response of piles to earthquake shaking in liquefiable soils is examined in the context of 1.5m cast in place reinforced concrete piles supporting a 14 storey apartment building.

INTRODUCTION

The seismic design of pile foundations in liquefiable soils poses very difficult problems in analysis and design. The pile foundation may undergo substantial shaking, while the soil is in a fully liquefied state and soil stiffness is at a minimum. During this shaking phase, the pile is prone to suffering severe cracking or even fracture. Liquefaction may lead also to substantial increases in pile cap displacements above those for the non-liquefied case. After liquefaction, if the residual strength of the soil is less than the static shear stresses caused by a sloping site or a free surface such as a river bank, significant lateral spreading or down slope displacements may occur. The moving soil can exert damaging pressures against the piles, leading to failure. Such failures were prevalent during the 1964 Niigata and the 1995 Kobe earthquakes.

Lateral spreading is particularly damaging if a non-liquefied layer rides on top of the moving liquefied soil. It is only in the last few years that the profession has begun to deal effectively with these critical design issues. The progress is due to developments in analysis and findings from shaking table and centrifuge tests. These developments have allowed more fundamental and comprehensive evaluations of case histories, and a greater appreciation of design problems.

The objective of the paper is to convey a general picture of the current state of the art and the emerging technology for dealing effectively with the design and analysis of pile foundations in liquefiable soils taking into account the lessons from case histories, the effects of earthquake shaking and the lateral pressures from post-liquefaction displacements.

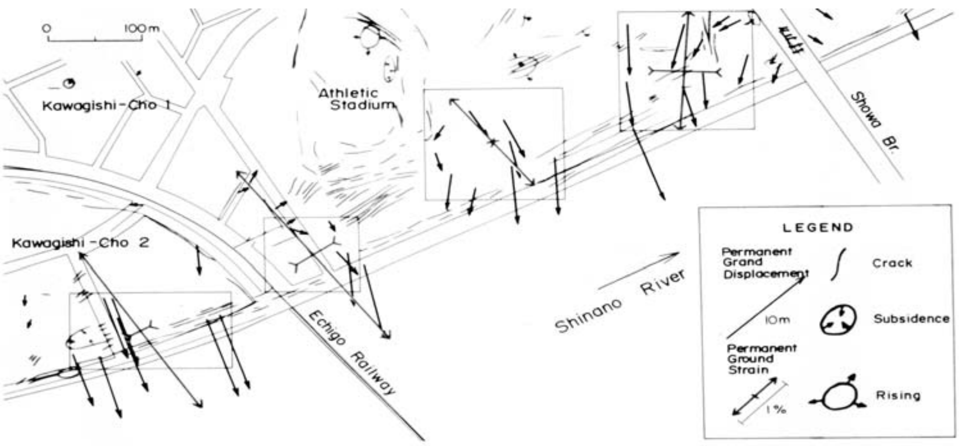

Fig.1. Ground displacements in 1964 Niigata earthquake (adapted from Hamada et al.1986).

Driven Piles

During liquefaction, large ground displacements can take place on sloping ground or towards an open face such as a river bank. Displacements from lateral spreading during the 1964 Niigata earthquake are shown in Fig. 1. (Hamada et al 1986).

Displacements as large as 10m occurred towards the Shinano River. Such displacements were very damaging to pile foundations and caused the failure of two major bridges. Damage to a pile under a building in Niigata caused by about 1m of ground displacement is shown in Fig 2 (Yasuda et al 1990). Complete shearing of a pile supporting a warehouse on Port Island near Kobe, by about 1.5m of ground displacement during the 1995 Kobe earthquake, is shown in Fig. 3. The function of these piles was to control settlement. They were designed primarily for vertical loads and could not carry the moments and shears caused by strong seismic shaking and lateral spreading.

Fig.2. Damage to pile by 2m of lateral ground displacement during 1964 Niigata earthquake (Yoshida et al. 1990).

However piles can be designed to carry the moments and shears generated by earthquake shaking or post-liquefaction ground displacements. Figure 4 shows a bridge on pile foundations. The foundation soils liquefied during the 1983 Nihon-Kai-Chubu earthquake. This led to a failure of the approach embankments by lateral spreading but the pile foundations survived without damage. A pile supporting a crane rail on Port Island, just offshore of Kobe City, is shown in Fig. 5. The ground moved more than 1.0m in this location after liquefaction occurred during the 1995 Hyogo ken Nanbu (Kobe) earthquake. The relative motion between the ground and the pile is clearly evident in Fig. 5. However the pile was designed to carry significant shears and moments and survived without damage.

Performance of CIDH Piles

Matsui and Oda (1996) evaluated the damage to the foundations of five major elevated expressways in the Kobe region, Japan, caused by the 1995 Kobe earthquake. They focused on cast-in-deep-hole (CIDH) reinforced concrete piles, as these comprised 80% of all foundation types. The piles were all over 1m in diameter.

Damage was classified into the four categories given in Table 1 and estimates of the residual load resisting capacities of piles in each damage class are also given. The damage was assessed by direct observation of pile shafts, examination of cores taken from the piles and observations made using borehole television (BHTV) cameras. The BHTV system was very effective, even hair cracks could be observed in the images. Non-destructive methods such as velocity logging, impact wave and electromagnetic wave methods were also used.

Fig.3. Shearing of a pile by ground displacements in 1995 Kobe earthquake (Finn and Fujita 2002).

Fig.4. Bridge on undamaged pile foundations with failed approaches due to liquefaction (Finn and Fujita 2002).

Fig.5. Undamaged pile supporting a crane rail in ground that moved about 1m (Finn and Fujita 2002).

Matsui and Oda (1996) found that pile damage correlated with sub-soil conditions. Damage was largely confined to areas of liquefaction with and without lateral flow of liquefied soil.

Despite the extensive liquefaction and the severe damage to the elevated super-structures, damage to the CIDH piles was negligible. The most extensive damage was along the No. 5 Bay Route of the Hanshin Expressway : 11% B, 37%C and 52% D. On the No. 3 Kobe route, the damage was 16% C and 84% D. There was no instance of A category damage. Matsui and Oda (1996) explained cracking pattern as follows. The cracks near the top of the pile are to be expected as this is usually the location of maximum moment. The cracks lower down the pile occur at the location of the second largest moment, at an interface between soft liquefied soils and a harder formation or where there is an abrupt change in the density of reinforcement.

The comments on the residual capacity of the damaged piles in Table 1 were based on tests conducted on CDIH piles, 1m in diameter by the Hanshin Expressway Public Corporation in 1993 (Kimura et al, 1994). The tests involved single piles and a 3x3 pile group. Data from a typical load test is shown in Fig. 6. The piles showed cracking at around 10cm of displacement. At 40cm displacement, the piles still retained "sufficient lateral capacity." Figure 7 shows a photo of the

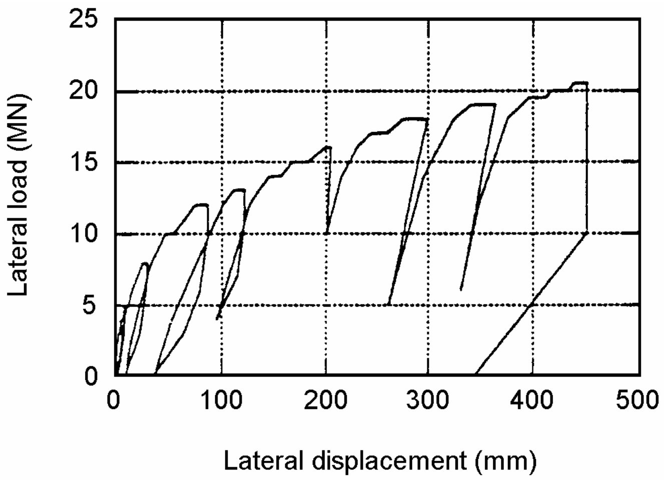

Fig.6. Lateral load test on 1m diameter CDIH pile (Kimura et al. 1994).

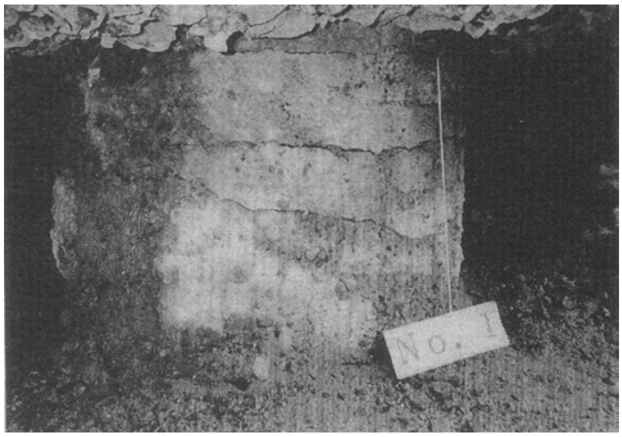

Fig.7. Damage to 1m diameter CDIH pile at 40cm lateral displacement (Kimura et al. 1994).

Table 1. Classification of pile damage (adapted from Matsui and Oda, 1966)

Damage Type	A Severe	B Heavy	C Light	D No Damage
Damage Description	Many cracks with concrete separation all over pile Buckling of main reinforcement Pile shaft separation	Many cracks with concrete separation near pile top Many cracks around middle and lower end of pile	Some cracks with separation near top Some cracks in middle and lower end of pile	Almost no cracks
Residual Pile Capacity	Totally inadequate	Probably adequate vertical capacity, Partial lateral capacity	Adequate capacity	Adequate capacity

external conditions at the head of one of these piles corresponding to a displacement of 40cm.

Clearly the CDIH piles behaved very well, more particularly as they were designed for much less intense ground shaking than they experienced during the Kobe earthquake.

The review of case histories has clearly demonstrated the design problems posed by pile foundations in liquefied soils. To cope with these problems it is essential to have a reliable method of calculating the effects of earthquake shaking and post liquefaction displacements on pile foundations. An overview of the methods used in practice will be given which indicates some of the advantages and limitations of the various methods. The aim of the review is to present a reasonably integrated up to date assessment of the state of the art.

ANALYSIS OF PILE FOUNDATIONS IN LATERALLY SPREADING GROUND

In the case histories section, it was shown that large post liquefaction displacements can occur and that these can be very damaging to pile foundations. These potential deformations can control design but they are very difficult to predict reliably. In engineering practice, the displacements at the top of the liquefied layer are often estimated by empirical formulas based on field data from past earthquakes. The first predictor equation was developed in Japan by Hamada (1986). Very comprehensive predictor equations have been developed by Youd et al (1999) in the USA which are used in practice in North America. An updated version of the Hamada equation has been adopted by the Japan Water Works Association (JWWA, 1997) based only on ground slope and the thickness of the liquefied layer. Bardet et al (1998a, b) have developed a method for predicting post–liquefaction displacements on a probabilistic basis. In engineering practice, the free field displacements are assumed usually to vary linearly from top to bottom of the liquefied layer. The deformed shape of a pile foundation caused by these post-liquefaction displacements is illustrated in Fig. 8.

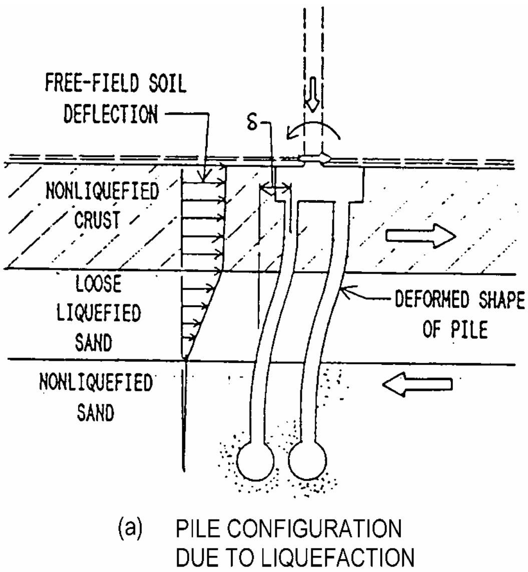

Fig.8. Distortion of pile foundation by lateral soil displacement.

Force Analysis

A force based analysis is recommended in a number of Japanese design codes for analysis of piles foundations in liquefied soils, undergoing lateral flow (JWWA 1997, JRA 1996). The underlying concepts are rational and simple. An unliquefied surface layer, which is transported on the moving liquefied soil is assumed to apply passive pressure on the foundation. A liquefied layer is assumed to apply a pressure less than the equivalent hydrostatic pressure on the piles because of the internal flow resistance of the liquefied sand. The transmitted lateral pressure was found to average about 30% of the overburden pressure on the basis of back analysis of case histories. The pressure distribution against the foundation for design is shown in Fig. 9.

Dobry and Abdoun (2001) and Ramos et al (1999) have studied the behavior of piles in laterally flowing soils by centrifuge tests. The setup for a typical centrifuge test is shown in Fig. 10. Typical test results for moments in the piles are given in Fig. 11. In order to simulate the moments they adopted the two different pressure distributions: inverted triangular and a uniform distribution. The adoption of the inverted triangular distribution may have been influenced by the inverted triangular distribution of displacements in the liquefied soil. However when there is lateral restraint at the pile head, both distributions seem to overestimate the bending moments in the upper part of the pile. Abdoun and Wang (2000) studied the effects of lateral spreading of ground with a upper lightly cemented layer on piles in centrifuge tests. They concluded that the moments in the pile were dominated by the lateral pressures from the cemented layer.

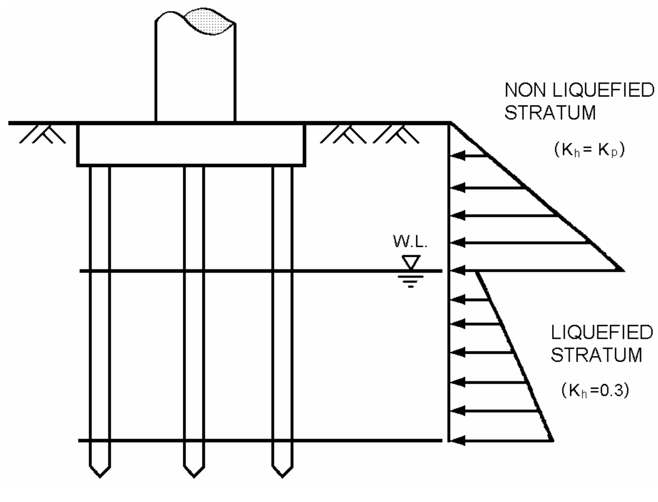

Fig.9. Design pressures against piles in laterally flowing liquefied soils (JWWA 1997).

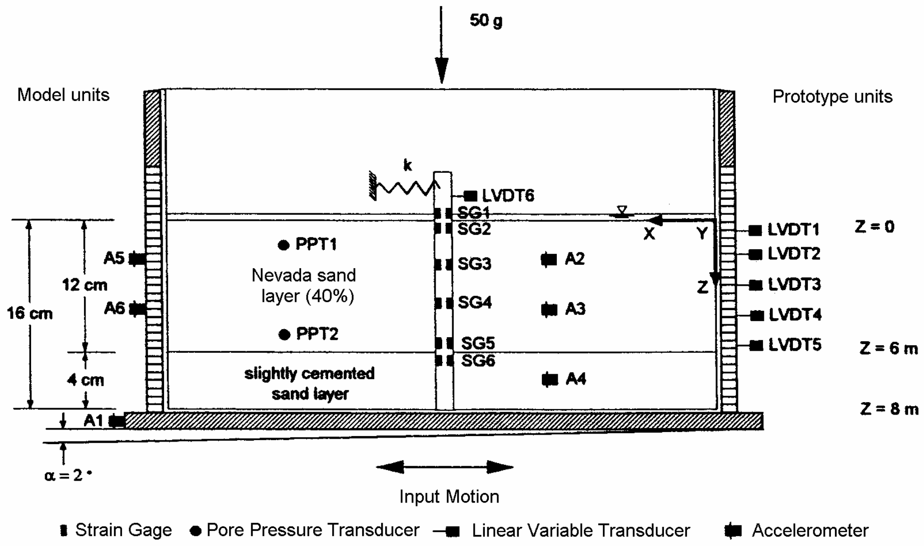

Fig.10. Centrifuge test on pile in flowing soil (Ramos et al. 1999).

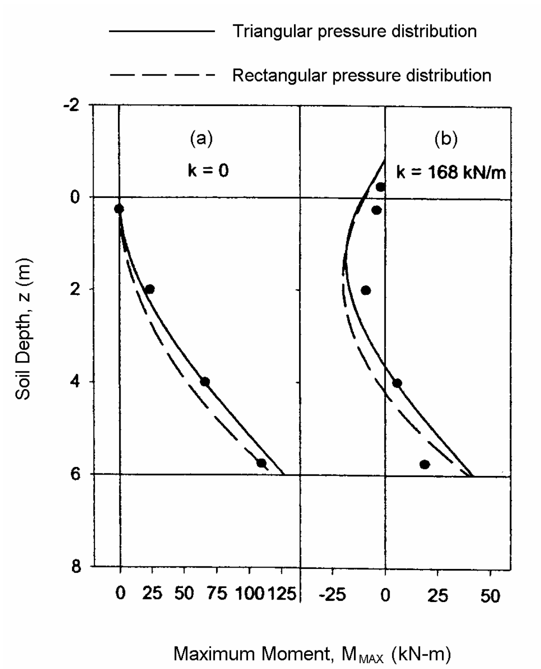

Fig.11. Computed and measured pile moments (adapted from Ramos et al. 1999).

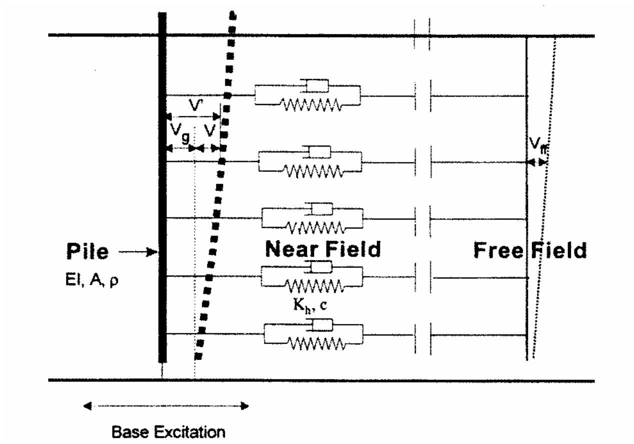

Fig.12. A Winkler spring model for pile foundation analysis.

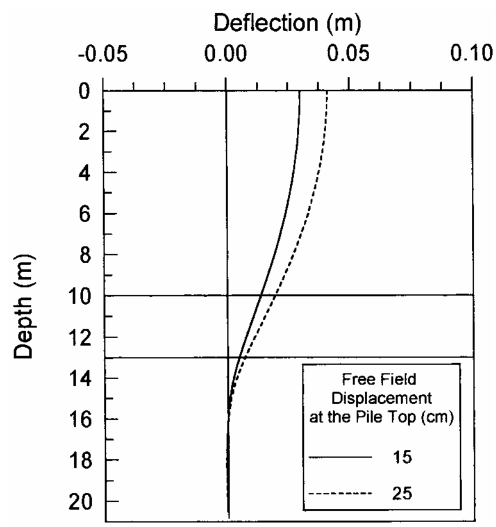

Fig.13. Calculated pile displacements for specified ground flow (Finn 1999).

<u>Displacement Analysis</u>

The first step in the analysis is to estimate the post-liquefaction free field displacements using one of the empirical formulas. These displacements are usually assumed to vary linearly from the top to the bottom of the upper liquefied layer. These linearly distributed displacements are then applied to the free field ends of the near field springs in the very general Winkler model shown in Fig.12 and a static analysis is performed (Finn and Thavaraj, 2001). Degraded p-y curves have usually been used for this kind of analysis.
The effects of lateral spreading on 1.5m diameter CDIH piles

supporting the structure shown in Fig. 29 were analyzed as described above. The free field displacements at the surface were estimated to be between 15 cm and 25cm. The computed pile displacements, assuming that the pile head is fixed against rotation, are shown in Fig. 13. The resulting bending moments are shown in Fig. 14. Note that the maximum bending moment is near the interface between the liquefied and non-liquefied layers.

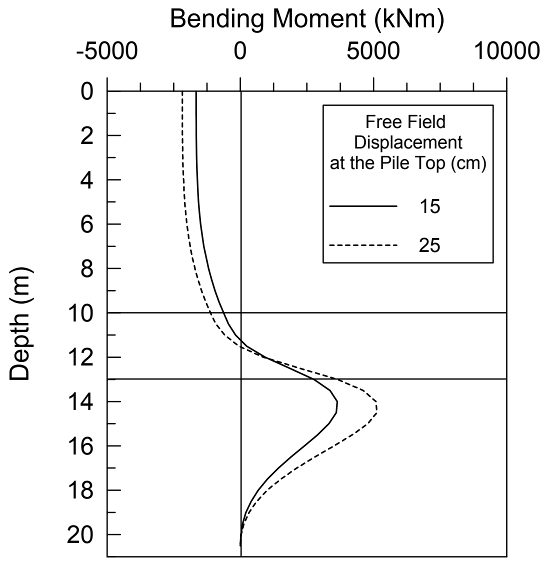

Fig.14. Pile moments induced by field displacements (Finn 1999).

Soil Properties for Displacement Analysis

The selection of soil properties for post–liquefaction flow deformation analysis will be discussed in the context of engineering practice.

Japanese practice. In Japanese practice the springs in the Winkler model are linearly elastic-plastic. The elastic soil stiffness is determined by semi-empirical code formulas related to the elastic modulus of the soil. This modulus is evaluated by plate loading tests or correlations with the SPT-N measurements and therefore includes some nonlinear effects. In some design offices the spring constant K is taken as zero in liquefied soil. The typical Japanese computational model for pile groups is shown in Fig.15.

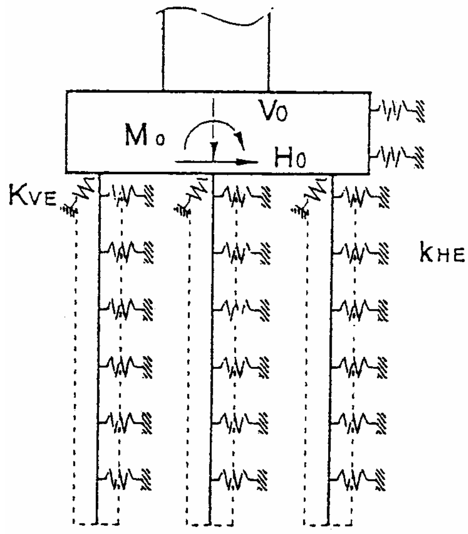

Fig.15. Japanese computational model for pile groups (JRA 1996).

The JRA (1996) code for highway bridges recommends reductions in the spring stiffness for use in liquefiable soils that depend on the factor of safety, F_L, against liquefaction. The reduction factors are given in Table 2. The resistance to liquefaction, R_L, is determined by cyclic triaxial tests on undisturbed samples obtained by in-situ freezing techniques. This strength is modified depending on whether Type 1 or Type 2 motions are used in design, by a factor c_w. Then R= c_w R_L is the dynamic shear strength ratio in Table 2. The factor c_w has a value of 1 for Type1 motions and a value in the range 1.0-2.0 for Type 2 motions. The code should be consulted for details of the 2 types of motions. Generally Type 1 motions are the design motions before the Kobe earthquake. Type 2 motions were introduced to provide protection against another earthquake like Kobe.

Table 2. Reduction coefficients for soil constants due to liquefaction (JRA 1996)

Range of F_L	Depth from the Present Ground Surface x (m)	Dynamic Shear Strength Ratio R	
		$R \leqq 0.3$	$0.3 < Ra$
$F_L \leqq 1/3$	$0 \leqq x \leqq 10$	0	1/3
	$10 < x \leqq 20$	1/3	1/3
$1/3 < F_L \leqq 2/3$	$0 \leqq x \leqq 10$	1/3	2/3
	$10 < x \leqq 20$	2/3	2/3
$2/3 < F_L \leqq 1$	$0 \leqq x \leqq 10$	1/3	1
	$10 < x \leqq 20$	1	1

North American Practice. There is no general consensus in North American practice on the appropriate modeling of the Winkler springs for post-liquefaction analysis. The basis of most analyses is a degraded form of the API (1995) p-y curves or curves due to Reese (1974). The practice is to multiply the p-y curves, by a uniform degradation factor p, called the p-multiplier, which ranges in value from 0.3 to 0.1. This follows from the original work of Dobry et al. (1995). They found that bending moments could be predicted adequately using a Winkler analysis, if the commonly used p-y curves were uniformly degraded by multiplying by a degradation factor p that appeared to diminish with increasing pore water pressure to a value of 0.1 at 100% excess pore water pressure. Wilson et al (1999) confirmed these results but showed that the p-multiplier for fully liquefied soil depended also on relative density, ranging in value from 0.1-0.2 for sand at about 35% relative density and 0.25-0.35 for a relative density of about 55%.

They also found that the resistance of the loose sand did not pick up even at substantial strains but the denser sand, after an initial strain range in which it showed little strength, picked up strength with increasing strain. This finding suggests that the good performance of the degraded p-y curves which did not include an initial range of low or zero strength, must be test specific and the p-multiplier may be expected to vary from one design situation to another.

The very low initial strength range in the laboratory p-y curves followed by a range of increasing strength is related to the dilatancy characteristics of sand at low effective stresses. Similar behavior is observed in tests in which undrained monotonic loading is conducted on sand specimens after cyclic loading to liquefaction. Typical examples of this phenomenon are shown in Fig. 16 (Yasuda et al 1999). Vaid and Thomas (1995) found similar results and also showed that the strain range of very low undrained resistance after liquefaction depends on the number of cycles of stress reversal the sand experiences after liquefaction, before the undrained monotonic loading is applied.

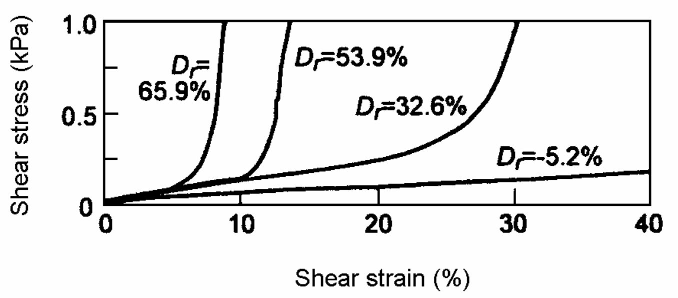

Fig.16. Post-liquefaction undrained stress-strain behavior of sand (Yasuda et al 1999).

Brandenburg et al (2001) conducted a very comprehensive series of tests to determine the effects of various parameters on pile performance in laterally spreading ground. Centrifuge tests on single piles and 2-pile groups were conducted on the centrifuge at UC Davis. Pipe piles were used. The single piles had prototype diameters of 0.36m, 0.73m, and 1.45m: the piles in the pile group were 0.73m in diameter. The foundation soil profile sloped gently towards a channel at one end of the shear box as shown in Fig. 17. It consisted of a non-liquefiable layer of clay, with a thin sand cover, underlain by a liquefiable layer of sand with a relative density of 35% and a base layer of dense sand at a relative density of 85%.

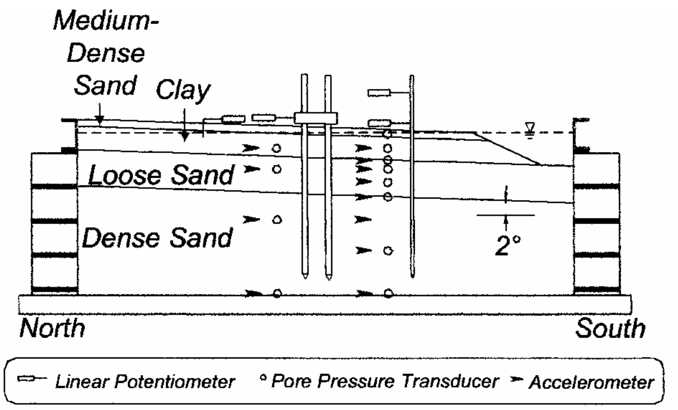

Fig.17. Centrifuge-model-test (Brandenburg et al. 2001).

The responses of the piles to lateral spreading were analyzed using a Winkler model based program, LPILE (Reese et al 2000). Matlock's 1970) static p-y relation for soft clay and Reese's (1974) static p-y relation for sand were used to represent the non-linear springs. A p-multiplier p=0.1 was used for fully liquefied sand.

The responses of the piles to lateral spreading were analyzed using a Winkler model based program, LPILE (Reese et al 2000). Matlock's (1970) static p-y relation for soft clay and Reese's (1974) static p-y relation for sand were used to represent the non-linear springs. A p-multiplier p=0.1 was used for fully liquefied sand.

Three cases were considered: (1) original p-y curves for loose sand with p=0.1 and only the properties in the loose liquefied sand were degraded for pore pressure effects ; (2) original p-y curves for loose sand with p=0.1and reductions in p-y stiffness and capacity of the dense sand due to pore water pressures in that layer; (3) the same as case (2) except that the standard p-y adjustment factors to the static p-y curves for cyclic loading were made also.. As Brandenburg et al (2001) point out these latter adjustments were developed for the large number of water wave generated stress cycles associated with a major offshore storm and are probably not applicable to the far fewer significant stress cycles associated with earthquake shaking. Comparison of measured and computed responses led to a number of important conclusions. The three most important ones are quoted verbatim below.

- the recorded responses of the three single piles and the one group of two piles could be modeled within the range of parameter variations that were studied, but all the responses could not be accurately modeled with the same set of input parameters.

- the parameter studies also showed that the standard adjustments to p-y relations for cyclic loading would have resulted in substantial under-prediction of lateral loads from the clay layer
- the calculated bending moments were more sensitive to the strength and p-y parameters for the upper clay and sand cover layers, and less sensitive to the p-multiplier assigned to the liquefied layer.

These findings pose clear warnings for anyone contemplating analyses of piles in laterally spreading soils using the standard North American p-y curves. The crucial factors seem to be; the dominating role of the non-liquefiable layer, the inappropriateness of using the standard cyclic loading reduction factors for earthquake shaking and the large uncertainty associated with the results of any analysis.

Some of the problems of arriving at a generally acceptable set of Winkler non-linear p-y curves for analysis arise from the assumed form of the curves. If the form is incompatible with the actual stress-strain behavior of the soil, problems in simulating the responses of different pile foundations with one set of p-y curves is not surprising. The North American p-y curves are concave downwards and this is not compatible with the post-liquefaction undrained behavior of liquefied sand under monotonic loading which is concave upwards as shown in Fig. 16 above.

Weaver et al (2002) conducted full scale cyclic loading tests in the field on a 0.6m diameter cast-in steel-shell (CISS) pile in liquefied soil to assess the accuracy of the p-y type of analysis. The test site is on Treasure Island in San Francisco Bay which is the location of the National Geotechnical Experimentation Site. Therefore soil conditions at the site are very well known. Liquefaction was caused by blasting and the cyclic loading was conducted using a high speed hydraulic actuator. The back figured p-y curves for the liquefied sand differed significantly in shape from the standard p-y curves modified by the p-multiplier. The slope of the standard p-y curve is greatest at small displacements and eventually decreases to zero at large displacements. The back calculated p-y curves show no resistance for a range of displacements between 20mm and 50mm. The soil resistance increased thereafter and was still increasing after 150mm. The shape of the back-calculated p-y curves are shown in Fig. 18. The standard p-y curves including the p-multiplier effect are also shown for comparison. The two sets of p-y curves have distinctly different shapes and give different estimate of soil resistance. The shapes of the Weaver et al (2001) curves are consistent with the post-liquefaction undrained monotonic loading test data from Yasuda et al (1999).

The p-y curves from the full scale field test share some characteristics with the p-y curves obtained by Wilson et al (2000) from centrifuge tests. The Wilson data for one cycle of loading at different depths are shown in Fig. 19. The hysteresis loops are very attenuated showing almost no pressure being exerted against the pile. The standard p-y curves, with the p-multiplier effect included, are shown for comparison. Again the shapes are radically different. Liu and

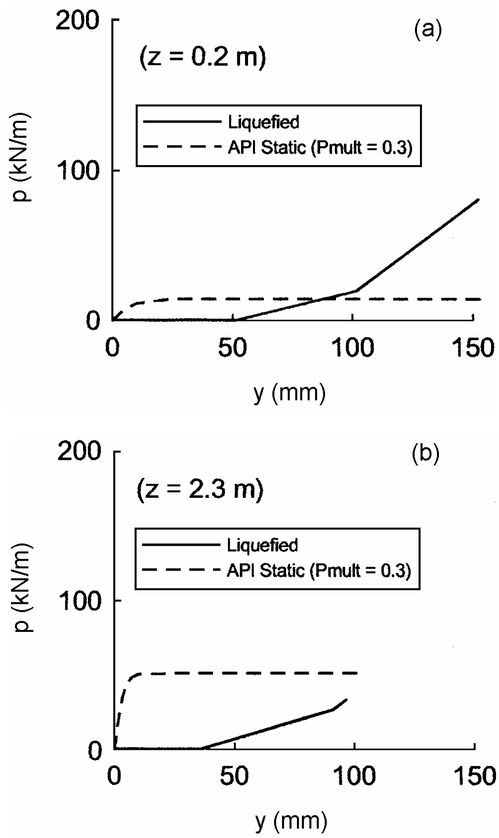

Fig.18. Comparison of standard p-y curves with curves back-figured from test data at depths of (a) 0.2m and (b) 2.3m from a full scale pile test (Weaver et al 2002).

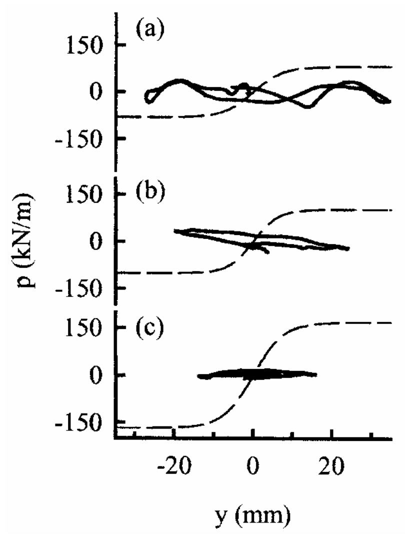

Fig. 19. Comparison of standard p-y curves with curves back-figured from centrifuge data (Wilson et al 2000).

Dobry (1995) found similar results and concluded that liquefied loose sands provide very little resistance. In this case it may be reasonable to ignore the effects of liquefied loose sand as far as pressure on the piles is concerned even in the force analysis. This is consistent with the judgment of some Japanese designers who assign zero stiffness to the elastic springs in their form of Winkler displacement analysis as discussed earlier.

DYNAMIC ANALYSIS OF PILE FOUNDATIONS IN LIQUEFIABLE SITES

In the previous section, the more or less passive response of piles to pressures from laterally spreading ground due to liquefaction was investigated. The dynamic response of piles in liquefied soil in response to earthquake shaking will now be considered. The issues will be explained in the context of the behavior of CDIH piles. A major research project on the seismic behavior of these piles is underway at Kagawa University, supported by Anabuki Komuten, a major construction firm with headquarters in Takamatsu. The company uses CDIH piles almost exclusively for supporting their buildings on reclaimed land. Such land is highly susceptible to liquefaction during earthquake shaking. Potential methods of analysis will be reviewed and some examples from building studies will be presented.

Overview of Analysis

The pile foundation-structure system vibrates during earthquake shaking as a coupled system. Logically it should be analyzed as a fully coupled system. However this type of analysis is not feasible in engineering practice. Many of the popular structural analysis programs cannot include the pile foundation directly into a computational structural model. Therefore various approximate methods of analysis are used.

The most common approach to the analysis of pile foundations is to use Winkler springs to simulate soil-pile interaction. The springs may be elastic or nonlinear. Some organizations such as the American Petroleum Institute give specific guidance for the development of nonlinear load-deflection (p-y) curves as a function of soil properties that can be used to represent nonlinear springs [API 1995]. The API (p-y) curves, which are the most widely used in engineering practice, are based on data from static and slow cyclic loading tests in the field. The reliability of these (p-y) curves for the analysis of pile foundations even under static and slow cyclic loading has been questioned (Murchison and O'Neill 1984). The effectiveness of p-y curves for seismic loading conditions is poorly established. Researchers trying to simulate the seismic response of piles in centrifuge tests usually resort to back-figured p-y curves and, even then, find that no one set of p-y curves can be used for general analysis (Brandenburg et al. 2001). Finn and Thavaraj (2001) have shown by analysis of the response of single piles in dry sand in centrifuge tests that a Winkler computational model with API p-y curves gave poor results for strong shaking but very good results for low level shaking.

A general Winkler dynamic model is shown in Fig. 12 above.. The near field interaction between pile and soil is modeled by springs and dashpots. The near field pile-soil system, together with any structural mass included with the pile, are excited by the seismic base motions and free field motions applied to the end of each Winkler spring. The free field motions at the desired elevations in the soil layer are computed by 1-D dynamic analyses using a computer analysis program such as SHAKE (Schnabel et al. 1972).

An alternative to the Winkler type computational model is to use a finite element continuum analysis based on the actual soil properties. Dynamic nonlinear finite element analysis in the time domain using the full 3-dimensional wave equations is not feasible for engineering practice at present because of the time needed for the computations. However, by relaxing some of the boundary conditions associated with a full 3-D analysis, it is possible to get reliable solutions for nonlinear response of pile foundations with greatly reduced computational effort.

Since seismic response analysis is usually conducted assuming that the input motions are horizontally polarized shear waves propagating vertically, the PILE-3D model retains only those parameters that have been shown to be important in such analysis. These parameters are the shear stresses on vertical and horizontal planes and the normal stresses in the direction of shaking. The soil is modeled by 3-D finite elements as shown in Fig. 20. The pile is modeled using beam elements or

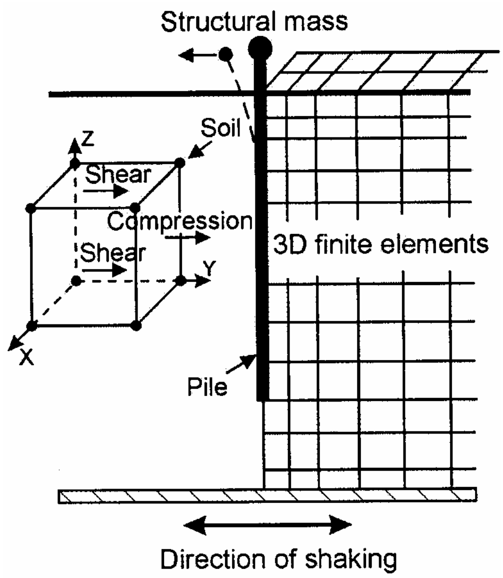

Fig. 20. Computational model in Pile-3D.

volume elements. The pile is assumed to remain elastic. This assumption is in keeping with the design philosophy that the structural elements of the foundation should not yield. In the analysis of concrete piles, the cracked section moduli are used, when deformations exceed the cracking limit. A full description of this method, including validation studies, has been presented by Wu and Finn (1997a, b). The method is incorporated in the computer program PILE-3D. The results are quite accurate for excitation due to horizontally polarized shear waves propagating vertically.

An effective stress version of this program, PILE-3D-EFF, has been developed by Finn and Thavaraj (1999) and validated by Finn et al (1999) and Finn and Thavaraj (2001) in cooperation the geotechnical group at the University of California at Davis. In support of the subsequent analyses of CDIH piles in liquefied sands, some excerpts from the validation study with UC Davis for a single pile are given here.

<u>Analysis of Centrifuge Tests at UC Davis</u>

Dynamic centrifuge tests of pile supported structures in liquefiable sand were performed on the large centrifuge at University of California at Davis, California. The models consisted of two structures supported by single piles, one structure supported by a 2×2 pile group and one structure supported by a 3×3 pile group. The typical arrangement of structures and instrumentation is shown in Fig. 21. Full details of the centrifuge tests can be found in Wilson et al. (1997). The model dimensions and the arrangement of bending strain gauges for the single pile are shown in Fig. 22. Model tests were performed at a centrifugal acceleration of 30g.

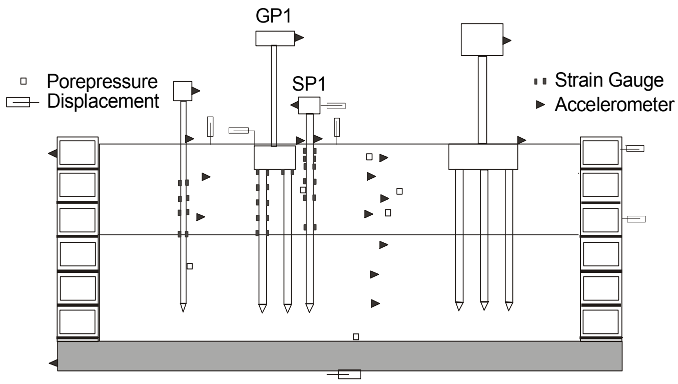

Fig. 21. Layout of models for centrifuge tests.

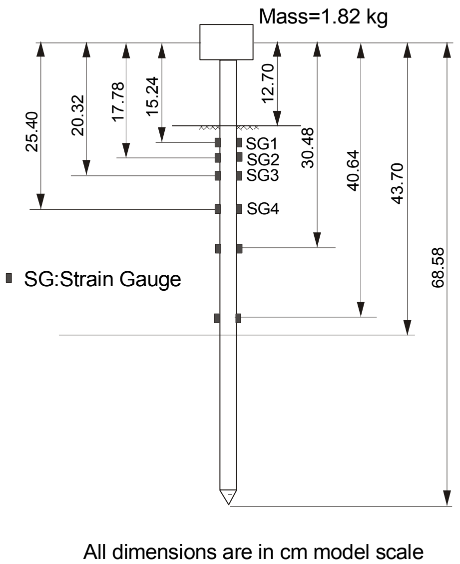

Fig. 22. Instrumented pile for single pile test.

The soil profile consists of two level layers of Nevada sand, each approximately 10m thick at prototype scale. Nevada sand is a uniformly graded fine sand with a coefficient of uniformity of 1.5 and mean grain size of 0.15 mm. Sand was air pluviated to relative densities of 75%-80% in the lower layer and 55% in the upper layer. Prior to saturation, any entrapped air was carefully removed. The container was then filled with a hydroxy-propyl methyl-cellulose and water mixture under vacuum. The viscosity of this pore fluid is about ten times greater than pure water to ensure proper scaling. Saturation was confirmed by measuring the compressive wave velocity from the top to the bottom of the soil profile.

The shear strain dependencies of the shear modulus and damping ratio of the soil were defined by the curves suggested by Seed and Idriss (1970) for sand. The friction angles of the upper and the lower sand layers were taken as 35° and 40°, respectively. Increments in seismic pore water pressures at any time were generated in each individual element depending on the accumulated volumetric strain prevailing in that element at that time and the current increment in volumetric strain, using the pore water pressure model proposed by Martin et al (1975). The moduli and shear strengths of the foundation soils were modified continuously to account for the effects of the changing seismic pore water pressures.

<u>Results of Single Pile Analysis</u>

<u>Acceleration Response.</u> Figure 23 shows the measured and computed acceleration response of the superstructure. There is generally good agreement between them, especially around the time period of peak response.

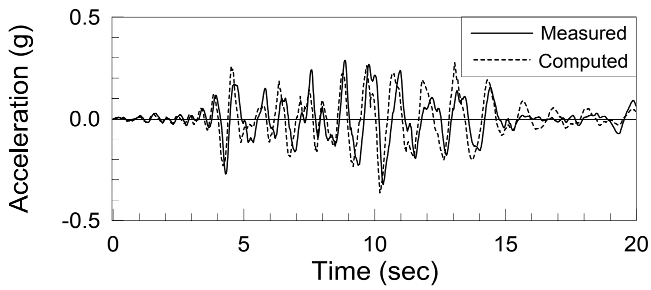

Fig. 23. Comparison of measured and computed superstructure acceleration time histories.

<u>Pore Water Pressure Response.</u> Figure 24 shows comparisons between measured and computed pore water pressures at three different depths; 1.14 m, 4.56 m, and 6.78 m in the free field. There is generally good agreement between the measured and computed pressures.

<u>Bending Moment Response.</u> Figure 25 shows the measured and computed bending moment time histories at two different depths; 0.76 m and 1.52 m. There is a very good agreement between the measured and computed time histories. Figure 26

shows the profiles of measured and computed maximum bending moments with depth. The comparison between measured and computed moments is adequate for engineering purposes, although the maximum moment is overestimated by 10%-15% between 1 m and 4 m depths.

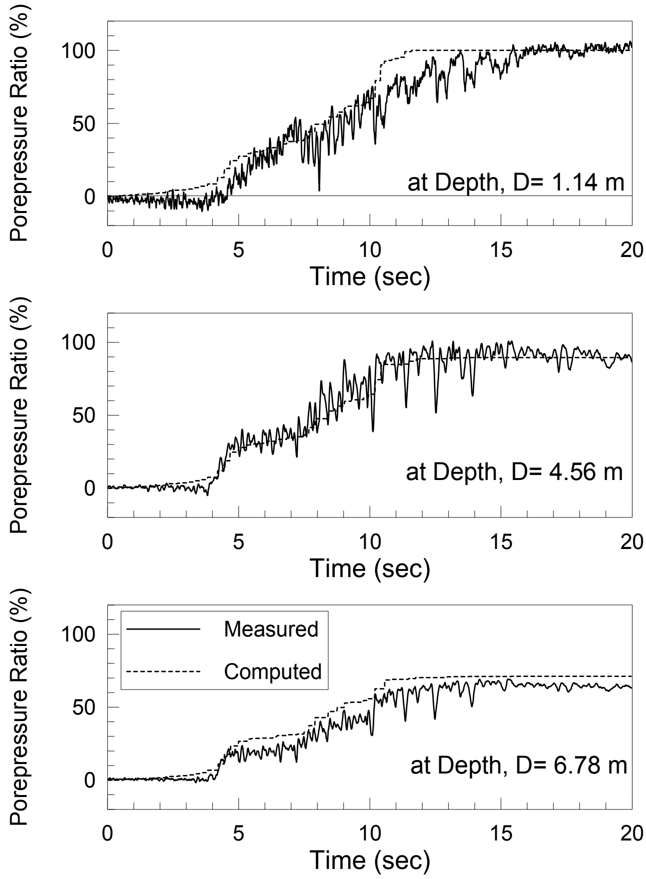

Fig. 25. Comparison of measured and computed porewater pressure time histories at three depths.

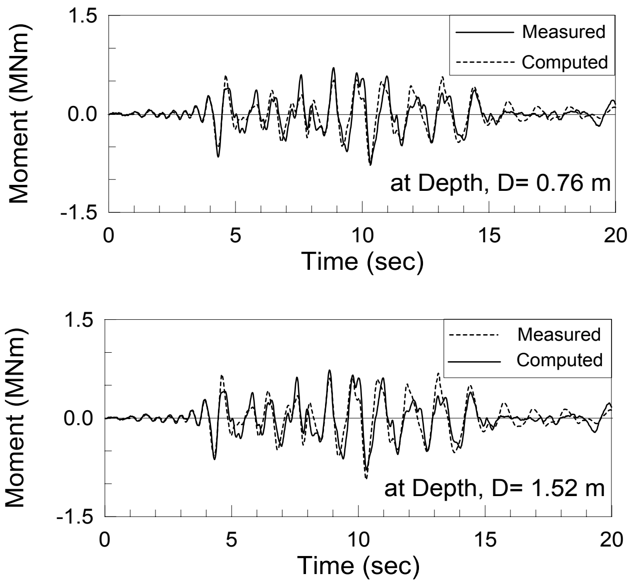

Fig. 26. Comparison of measured and computed bending moment time histories at two depths.

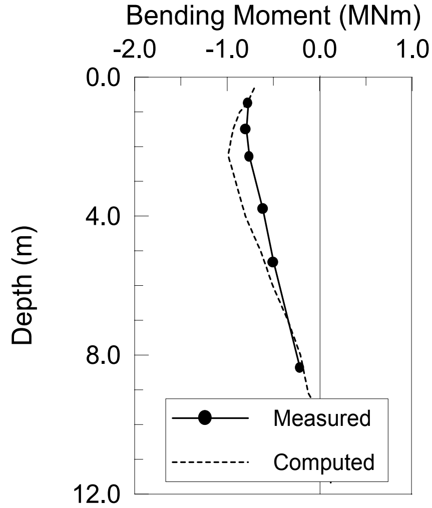

Fig. 27. Comparison of measured and computed maximum bending moments profiles along the pile.

ANALYSES OF CDIH PILES

Seismic response analyses were conducted on a 1.5 m diameter cast-in-place reinforced concrete pile supporting a column of the 14 storey apartment building using PILE 3-D-EFF. The soil conditions and pile are shown in Fig.28. Slightly idealized site conditions shown in Fig. 29. The upper 10m are expected to liquefy during the design earthquake. The mass mounted on the pile in Fig. 29 represents the portion of the total mass supported by the pile. The purpose of placing the mass on the pile is to model approximately the inertial interaction between the super-structure and the pile foundation. It is mounted on the pile head by a flexible support that gives the mass a period of vibration of 1.4s that is the estimated fundamental period of the prototype structure.

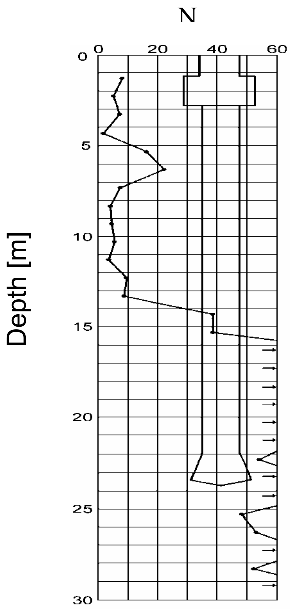

Fig. 28 Site in reclaimed land.

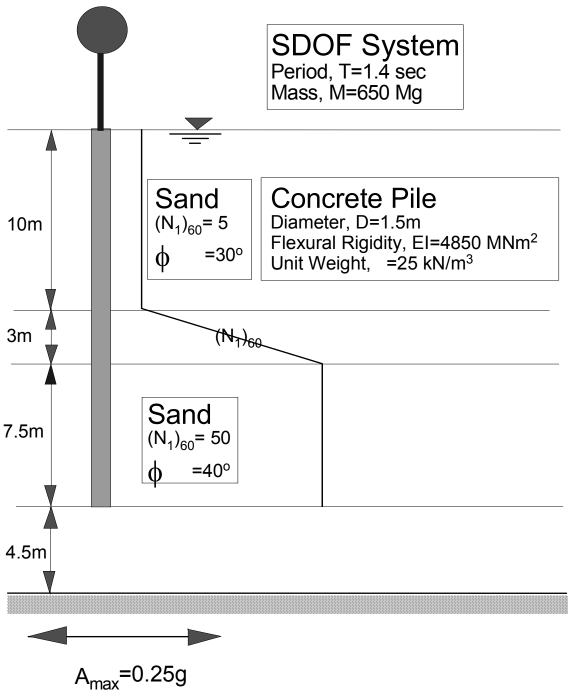

Fig. 29. Model of soil-pile-structure system.

Two kinds of analyses were conducted; total stress dynamic analysis in which seismic pore water pressures and liquefaction are ignored and effective stress analysis that automatically takes the seismic pore water pressures into account. In general, soil properties are adjusted continuously to maintain compatibility with current pore water pressures and shear strains. The peak acceleration of the input acceleration record is 0.25g and is amplified to 0.4g at the surface. The surface accelerations become negligible after liquefaction has occurred. Dynamic effective stress analyses of this system were conducted for two conditions: including both inertial and kinematic interaction, and with kinematic interaction only. The latter analyses did not include the mass of the superstructure. Data from these analyses are compared to evaluate the significance of kinematic interaction.

RESULTS OF ANALYSES

Analyses with Inertial interaction

Pile displacements and moments for the 14 storey building, at the instant of maximum pile head displacement, are shown in Fig. 30 and Fig. 31 respectively. Approximately the top 10 m liquefy or develop very high pore water pressures during earthquake shaking. Results are shown for two conditions; the pile head is fixed against rotation and the pile head is free to rotate. There is generally a greater degree of fixity in Japanese buildings because much deeper grade beams are used to tie adjacent pile caps together than in North America, as shown in Fig. 32. The large grade beams provide considerable restraint against rotation and so they mobilize much higher inherent structural stiffness in the pile.

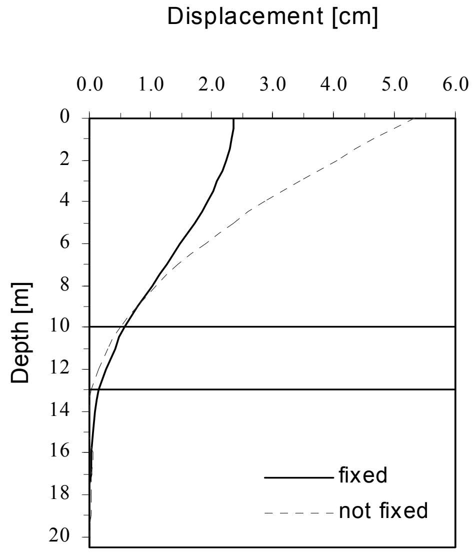

Fig .30. Pile deflections at maximum pile head displacement.

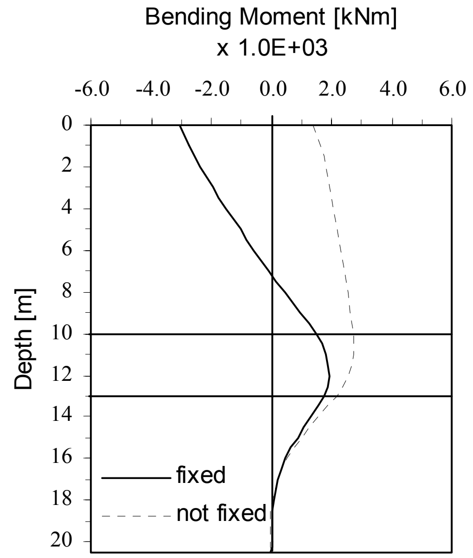

Fig. 31. Pile moments at maximum pile head displacement.

The displacements are more than twice as large when the pile head is free to rotate. The maximum moment occurs at the pile head, when the pile head is fixed against rotation, but significant moment also occurs at the boundary between the softer and stiffer soils. When the pile head is not fixed against rotation, the maximum moment occurs at the boundary between the stiffer and softer soils. This moment is approximately equal to the pile head moment, when the pile head is fixed against rotation. The results show that when designing piles or evaluating pile foundations in potentially liquefiable soils for earthquake loading, it is important to make a realistic assessment of pile head restraint against rotation and to be aware of the potential for large moments at the interfaces between soft and hard layers.

Fig. 32. Large grade beam for 14 storey building.

At some sites a thick surface layer of non-liquefiable soil may lie over the liquefaction zone. A stiff upper layer is incorporated into the original site of the 14 storey building. Deflections and moments for this case, at the instant of maximum pile head displacement, are shown in Fig. 33 and Fig. 34 respectively. As before, the results are shown for two pile head conditions, no pile head rotation and the pile head is free to rotate.

Deflections and moments for this case, at the instant of maximum pile displacement, are about the same whether the pile head is fixed against rotation or not. Also the deflection of the pile head, when the pile head is fixed against rotation,

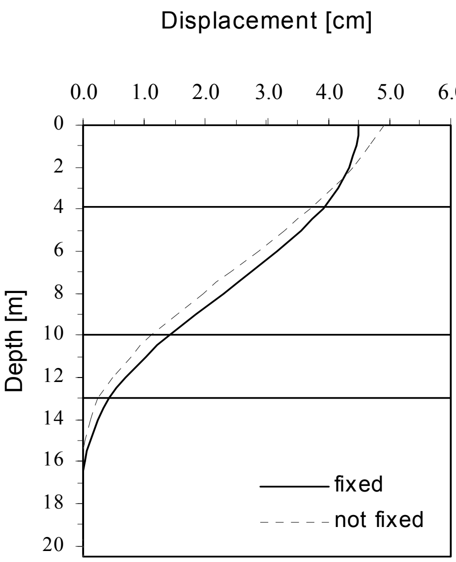

Fig. 33. Pile deflections at maximum pile head displacement.

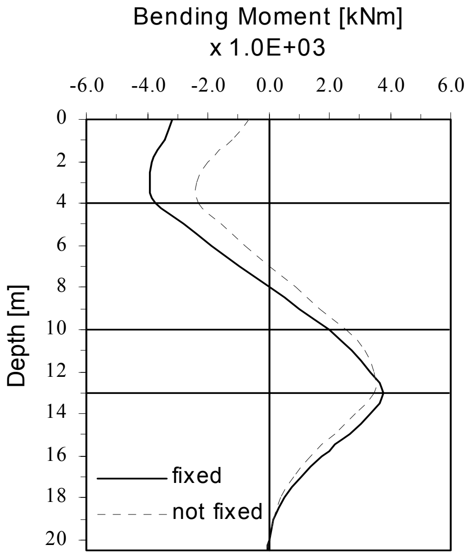

Fig. 34. Pile moments at maximum pile head displacement.

has more than doubled compared to the previous case of no stiff upper layer. This is due to the restraint of the upper layer and the movement of that layer as a rigid body after liquefaction develops. The stiff upper layer greatly increases the bending moment demands on the pile during earthquake excitation.

The moments at the pile head and at the interface between the soft and stiff soils have increased by 30%, compared to the case without the upper layer. When the pile is fixed against rotation the moments at the pile head and the interfaces between layers are about the same. The behavior of the upper layer is clarified further in the next section which presents results from kinematic analyses.

KINEMATIC ANALYSIS

Kinematic analyses were conducted on the 1.5 m diameter pile to assess the importance of kinematic interaction. Analyses were conducted with and without the stiff surface layer and, in each case, the pile head was considered either fixed against rotation or not. The kinematic analyses were conducted after removing the superstructural mass in Fig. 29.

The pile and free field displacements at the instant of maximum pile head displacement are shown in Fig. 35 for the case when there is a stiff surface layer. It is evident that the stiff surface layer is moving as a rigid body at the time of maximum pile head displacement which occurs after the incidence of liquefaction. At this time it also appears to be driving the pile, so that the pile and surface layer undergo about the same displacements. Consequently when the stiff surface layer is present, the kinematic pile head moments shown in Fig. 36 are about the same as the moments, when both inertial and kinematic interactions are included (Fig. 34). This indicates that, in this case, the kinematic moments dominate the moment response of foundation.

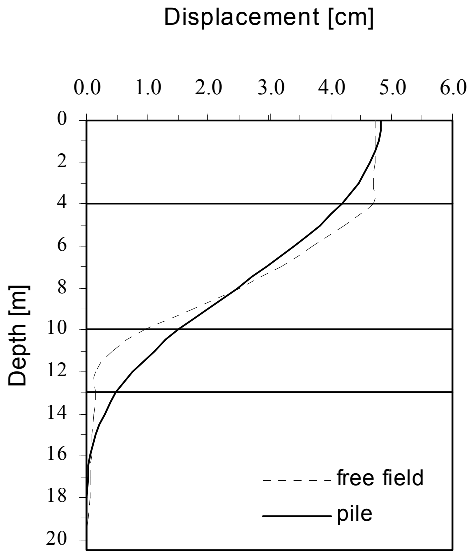

Fig. 35. Displacements of pile and free field at maximum pile head displacement.

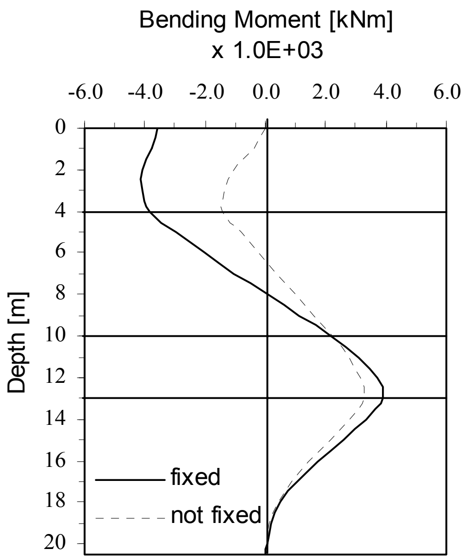

Fig. 36. Kinematic moments at maximum pile at maximum pile head displacement.

Clearly analyses that neglect kinematic effects may in some situations underestimate significantly design moments and shearing forces in foundation piles.

CLOSING REMARKS

A general picture of the current state of the art and the emerging technology for dealing effectively with the seismic design and analysis of pile foundations in liquefiable soils is presented in this paper. Two distinct design cases were considered and are illustrated by case histories. One is the static response of pile foundations to the pressures and displacements caused by lateral spreading of liquefied ground. The other is the seismic response of piles to strong shaking accompanied by the development of high pore water pressures or liquefaction.
Design for lateral spreading is examined in the context of developments in design practice and the findings from shake table and centrifuge tests. Response of piles to earthquake shaking in liquefiable soils is examined in the context of 1.5m cast in place reinforced concrete piles supporting a 14 storey apartment building.

Two methods for design against lateral spreading, a force based method which is specified in Japanese codes and a displacement based method which is sometimes used in North America are presented. The Japanese method is based on studies of case histories from past earthquakes, especially the pile foundation failures caused by lateral spreading during the Kobe earthquake and is very simple to apply. The pressures on the pile foundation are specified as follows; liquefied soil exerts a pressure equal to 30% of overburden pressure and an unliquefied surface layer exerts passive pressure. Data from simulated earthquake loading of model piles in liquefiable sands in centrifuge tests indicate that the force method is an adequate design method.

The displacement method requires the prediction of surface displacements which are then distributed linearly over the liquefied layer and the analysis of pile response to these displacements by a static analysis using a Winkler model or a finite element method. Two factors make this method appear quite unreliable. The surface displacements are predicted by empirical formulas which can err by a factor of 2 and there is no agreement yet on a standardized set of p-y curves or stress-strain curves for representing the post-liquefaction stress-strain behavior of the soil. Recent centrifuge and shake table tests are contributing significantly to a framework of understanding about how piles and soils interact after liquefaction during lateral spreading.

The behavior of piles in liquefied ground was studied in the context of large diameter CDIH reinforced concrete piles. These piles are often used to support buildings in reclaimed land in Japan and as combined foundation–piers for bridges worldwide. Analyses show that large bending moments develop in critical areas such as at the pile head, when it is fixed against rotation, and the boundary between liquefied and non-liquefied layers. The analyses also demonstrate that if a stiff surface layer overlies the liquefied zone, then the moment and deflection demands on the pile may be substantially increased over the case when the stiff upper layer is not present.

Restraint against pile head rotation has a significant effect on the response of piles in liquefied soils, when the surface layer liquefies. The pile cap displacements may be up to three times larger, if the restraint is low compared to full fixity.

If an unliquefiable surface layer covers the liquefied stratum, large kinematic moments may develop in the pile, especially if the surface layer is stiff and relatively thick. After liquefaction the surface layer tends to move as a rigid body and drives the pile to greater displacements. The increased displacements and the greater fixity against rotation of the pile head are responsible for the increase in moments.

The keys to good design are reliable estimates of environmental loads, realistic assessments of pile head fixity and the use of methods of analysis that can take into account adequately all the factors that control significantly the response of the pile-soil-structure system to strong shaking and /or lateral spreading in a specific design situation. Not all factors are important all the time but an informed background is essential in making decisions about what can be ignored.

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