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Recent Studies on Seismic Centrifuge Modeling of Liquefaction and Its Effects on Deep Foundations

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ABSTRACT

The effects of liquefaction on deep foundations are very damaging and costly, and they keep recurring in many earthquakes. The first part of the paper reviews the field experience of deep foundations affected by liquefaction during earthquakes in the last few decades, as well as the main lessons learned. The second part of the paper presents results of physical modeling of deep foundations in the presence of liquefaction conducted mostly in the U.S. and Japan in the 1990's, with emphasis on the work done by the authors and others at the 100 g-ton RPI centrifuge. Centrifuge models of instrumented single piles and pile groups embedded in both level and sloping liquefiable soil deposits have been excited in-flight by a suitable base acceleration. End-bearing and floating piles with and without a pile cap, with or without a mass above ground, free at the top or connected to a lateral or rotational spring to simulate the superstructure's stiffness, with the foundation embedded in two- or three-layer soil profiles, have been tested. Tests with a mass above ground have allowed backfiguring the degradation of the lateral resistance of the loose saturated sand against the pile as the soil liquefies, while tests in sloping ground without a mass have allowed studying the effect of lateral spreading. Interpretations of these centrifuge experiments and their relation to field observations, soil properties, theory and analytical procedures are also discussed.

INTRODUCTION

Liquefaction of loose, saturated granular soil during earthquakes has been and continues to be a major cause of damage to deep foundations. Buildings, bridges, port facilities and other structures are affected. Cracking and rupture of piles at both shallow and deep elevations, rupture of pile connections, and permanent lateral and vertical movements and rotations of pile heads and pile caps with corresponding effects on the superstructure have been observed. Table 1 lists some earthquakes in the last 35 years where deep foundations have been damaged by liquefaction, together with appropriate references for each seismic event. One event not included in Table 1 is the 1987 Edgecumbe earthquake in New Zealand, where lateral spreading caused passive failure of the nonliquefied shallow soil crust against the deep foundation of a highway bridge. There was no significant damage to the bridge and foundation, but analysis revealed that it had been a close call (Berrill et al., 1997). Another event not listed in the table is the 1999 Chi-Chi, Taiwan earthquake, where liquefaction at the Taichung Harbor damaged some piers but the port performed generally well and remained functional; the undamaged piers were those supported by piles (Uzarski et al., 1999).

Permanent ground deformation, and especially lateral spreading, is a main source of distress to piles and other deep foundations and to their supported structures, as are inertial superstructural forces and moments arising during shaking and acting on a soil already weakened by rising water pore pressures. Other sources of distress include reduction in the point bearing capacity of the piles, post-liquefaction compaction settlement, and cyclic deformation of the foundation during shaking. Youd (1993) reviewed case histories of bridge foundations affected by lateral spreading from as far back as the 1868, California; 1886 Charleston, South Carolina; and 1906 San Francisco earthquakes. Mizuno (1987) made a compilation of pile damage during earthquakes in Japan for the 60-year period between 1923 and 1983, and identified sand liquefaction and lateral spreading as main causes of damage. Other compilations and general discussions of case histories including the earthquakes listed in Table 1, have been presented by NRC (1985), Hamada and O'Rourke (1992), O'Rourke and Hamada (1992), and Dobry (1994), while Tokimatsu et al. (1996), Yokoyama et al. (1997) and Tokimatsu (1999) have provided a review of pile foundation damage due to liquefaction in the 1995 Kobe earthquake.

As suggested by the discussion above, and confirmed even by a superficial examination of the case histories, this is a complex soil-structure interaction problem which is very difficult to model analytically. It involves large ground deformations, both cyclic and permanent, inertial effects during shaking, and soil-foundation and foundation-superstructure interactions, all in the presence of rapidly changing soil properties with time. Centrifuge physical modeling has emerged as a main tool to study the problem, understand and quantify the parameters involved, and provide guidance and calibration to both simplified engineering procedures and numerical simulation techniques.

The first part of the paper reviews the experience of deep foundations affected by liquefaction during earthquakes and the main lessons learned. The second part discusses physical modeling of deep foundations in the presence of liquefaction conducted mostly in the U.S. and Japan in the 1990's, with emphasis on the work done by the authors and others at the 100 g-ton RPI centrifuge.

Table 1 Some Earthquakes Where Deep Foundations Have Been Damaged by Liquefaction

Year	Earthquake	Country	Reference(s)
1964	Niigata	Japan	Hamada et al. (1986) Kawashima et al. (1988) Stewart et al. (1988) Yoshida and Hamada (1991) Hamada (1992a)
1964	Alaska	USA	McCulloch and Bonilla (1970) Ross et al. (1973) Bartlett and Youd (1992)
1983	Nihonkai-Chubu	Japan	Hamada (1992b)
1989	Loma Prieta	USA	Benuzca (1990) Seed et al. (1992)
1991	Limon	Costa Rica	Youd et al. (1992) Yoshida et al. (1992) Youd (1993)
1995	Hyoken-Nambu (Kobe)	Japan	Tokimatsu et al. (1996) Matsui and Oda (1996) Hamada et al. (1996) Yokoyama et al. (1997) Tokimatsu (1999)
1995	Manzanillo	Mexico	TGC (1995) Swan et al. (1996)

EXPERIENCES DURING EARTHQUAKES

It is useful to quote first the summary conclusions of two prominent researchers after reviewing case histories of liquefaction effects on foundations and structures.

Youd (1993) assessed the damaging effect of lateral spreading on bridges, founded mostly but not exclusively on deep foundations: "(Ground lateral spreading) displacements generally range from a few centimeters to several meters and are directed down mild slopes or toward a free face such as an incised river channel. Such displacements thrust bridge abutments and piers riverward, generating large shear forces in connections and compressional forces in the superstructure. These forces have sheared connections, allowing decks to be thrust into, through, or over abutment walls or causing decks to buckle. In other instances, connections have remained intact with the deck acting as a strut, holding tops of piers and abutments in place while the bases of these elements are displaced toward the river. These actions have inflicted severe damage and even bridge collapse."

Tokimatsu (1999) reviewed in detail the experience of pile foundations, mainly under buildings, during the 1995 Kobe earthquake and other events. While he focused on lateral spreading he also considered the presence of inertial effects. He concluded that: "(1) damage is concentrated near the top and bottom of the liquefied layer of non-ductile piles, leading to the tilt of their superstructures in many cases; (2) the piles within a building near the waterfront show different failure modes in the direction perpendicular to the waterfront, while those away from the waterfront show similar deformation patterns; (3) pile foundations enclosed by cement mixing walls, diaphragm walls, and cement column walls did not suffer any vital damage; and (4) the earth pressures acting on rigid foundations from non-liquefied crusts overlying laterally spreading soils may be as large as the passive ones, whereas those acting on deformable foundations appear to be considerably smaller."

Figures 1-18 provide examples of the damaging effect of liquefaction and lateral spreading on deep foundations and superstructures, and illustrate the development of limit equilibrium evaluation methods calibrated by the case histories.

Figures 1-4 show the destructive effect of lateral spreading on two bridges during the 1964 Niigata earthquake in Japan. This was a magnitude 7.5 event with epicenter about 50 km from Niigata. The earthquake caused extensive liquefaction damaging buildings, bridges, lifeline facilities, etc., in Niigata City. Figures 1 and 2 illustrate the damage to the abutment and piers of the Yachiyo Bridge. The foundations of both abutments and piers were reinforced concrete piles 30 cm in diameter and a length of about 10 m. Pier 2 was broken at the ground surface level with a permanent deformation of 1.1 m developing between top and bottom of the pier. Figure 2 shows the distorted shape of the pile which was extracted and examined after the earthquake; the pile was destroyed at a depth of 8 m, at the bottom of the liquefiable layer. This damage was clearly caused by a 2 to 5 m free field lateral spreading of the ground toward the river, which pushed the foundations of the piers toward the river while the tops of the piers were restrained because of the resistance of the girders. The difference between the foundation deformation (~ 1 m) and the free field deformation (2 to 5 m) is explained by the stiffness of both foundation and superstructure. Figures 3 and 4 show the collapse of the Showa bridge in the same earthquake, also caused by liquefaction and lateral spreading toward the river. As shown in Fig. 4, five simply supported girders between piers P2 and P7 fell into the water. The piers were constructed by driving steel pipe piles that had considerable flexibility in the direction of the bridge longitudinal axis. Figure 3 shows the deformed shape of one of the steel piles which was extracted after the earthquake. The pile, of a 61 cm diameter, was bent toward the center of the river at a depth about 7 or 8 m below the riverbed, that is, probably close to the bottom of the liquefiable layer. The measured surface ground displacement at the river bank toward the river was about 5 m (Hamada, 1992a).

Figure 5 shows damage to a railway bridge during the 1991 Limon, Costa Rica, earthquake, caused also by liquefaction and lateral spreading. This was a magnitude 7.7 event with epicenter about 30 km from the Rio Bananito Bridge of Fig. 5. The bridge was a 50-m long, single-truss structure supported on elliptically-shaped caissons 1.46 m by 2.16 m across the major axes. The caissons were constructed of a 12 mm cast-steel shell filled with concrete. During the earthquake, the ground displacements caused by liquefaction and lateral spreading pushed the supporting caissons out from under the seating plates on both ends of the bridge. Surveying after the earthquake revealed that the lateral ground displacements toward the river had been 1 to 2 m at both ends of the bridge. This caused loss of support that allowed the truss to tip downstream by about 15 degrees (Youd et al., 1992).

The previous examples from the Niigata and Limon earthquakes clearly show that critical locations in the shear and bending response of deep foundations to lateral spreading are the head of the foundation and the bottom of the liquefied layer. Other examples indicate that when a nonliquefiable layer overlies the liquefied layer, a third critical point is the top of the liquefied layer. This is illustrated by the sketches in Figs. 6a - 6e, which summarize the damage to pile-supported buildings near the waterfront from a number of field investigations after the 1995 Kobe earthquake in Japan (Tokimatsu, 1999). This was a magnitude 7.2 earthquake, with epicenter very close to Kobe City and other affected areas. Extensive liquefaction and lateral spreading developed at port facilities, bridges and buildings including damage to piles as sketched in Fig. 6.

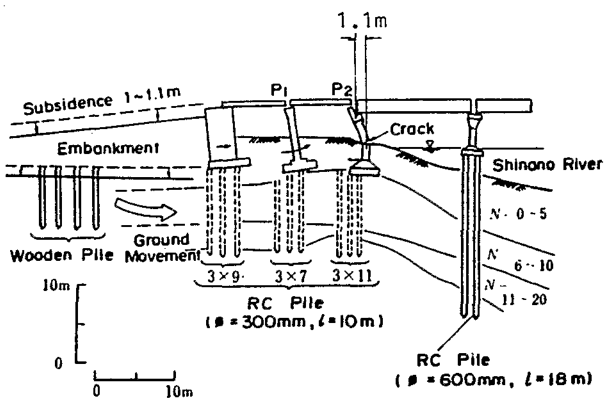

Fig. 1. Damage to Yachiyo Bridge in 1964 Niigata, Japan earthquake (Hamada, 1992a).

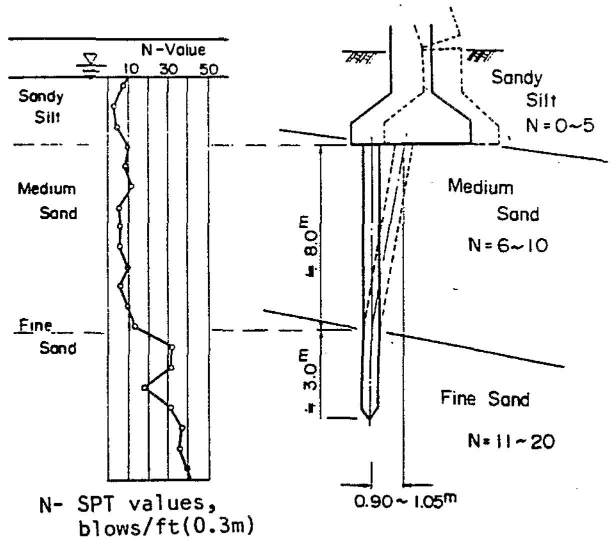

Fig. 2. Damage to a foundation pile under Yachiyo Bridge in 1964 Niigata, Japan earthquake (Hamada, 1992a).

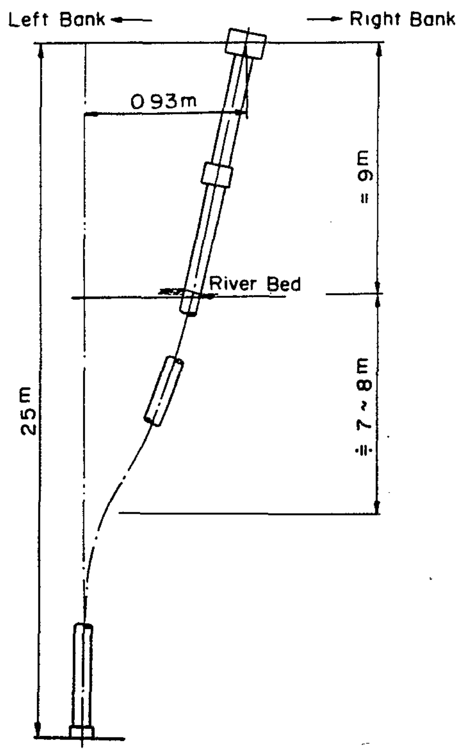

Fig. 3. Deformation of a steel pipe pile under Pier No. 4 of Showa Bridge in 1964 Niigata, Japan earthquake (Hamada, 1992a).

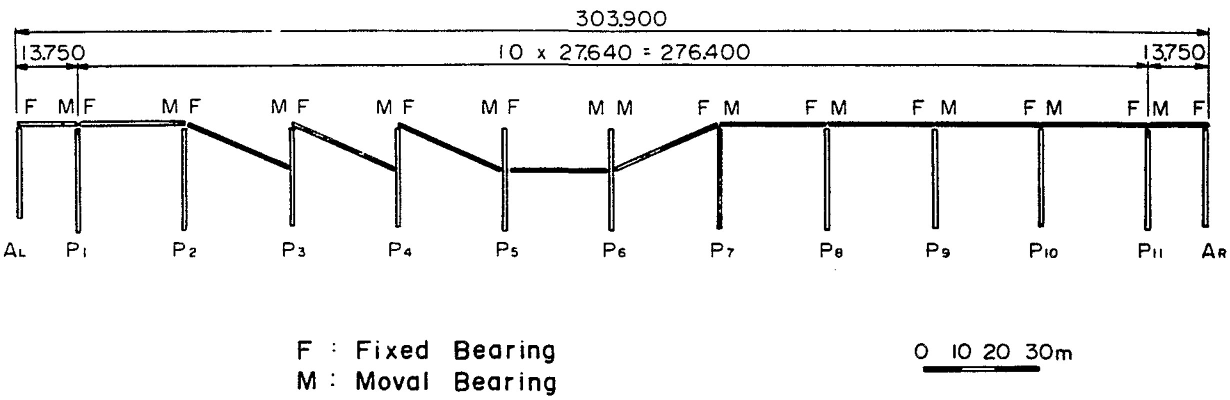

Fig. 4. Collapse of spans of Showa Bridge in 1964 Niigata, Japan earthquake (Hamada, 1992a).

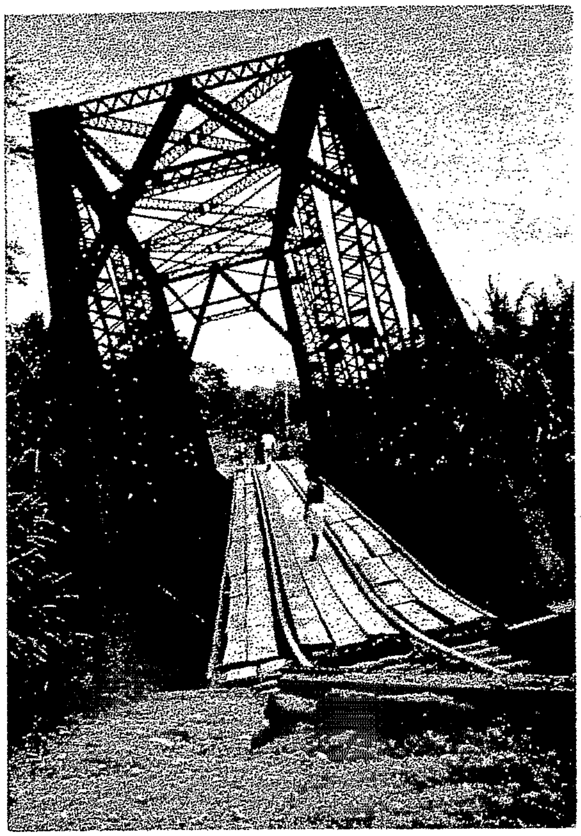

Fig. 5. Railway bridge over Rio Bananito river that tipped due to caissons being pushed out from under bridge seatings, 1991 Limon,, Costa Rica earthquake (Youd, 1993).

The importance of the upper and lower boundaries of the liquefied layers is clearly shown in another example of pile damage during the Niigata earthquake, illustrated in Figs. 7 and 8. This is the Niigata Family Court House (NFCH) building, which experienced about 1 m permanent lateral ground displacement (Fig. 7). Figure 8 shows the soil profile and two damaged piles as they looked after being extracted from the ground. Both piles were hollow concrete piles of 35 cm outside diameter and 7.5 cm wall thickness; with longitudinal reinforcement consisting of twelve 13-mm diameter steel bars and transverse reinforcement provided by a spiral hoop 3 mm in diameter and a pitch of 800 mm. Pile No. 1 was a floating pile that bent and cracked at a depth of about 2 m, that is near groundwater level which is also the upper boundary of the liquefiable soil. Pile No. 2 was an end-bearing pile, which cracked at a depth of about 8 m, that is at the bottom of the liquefied layer, and exhibited a shear dislocation of approximately 0.1 m at 2 m depth. The double curvature shape of Pile No. 2 indicates that the nonliquefied shallow layer pushed the pile laterally with the bottom layer below 8 m resisting this bending action (Hamada et al., 1986; Hamada, 1992a; Meyersohn, 1994).

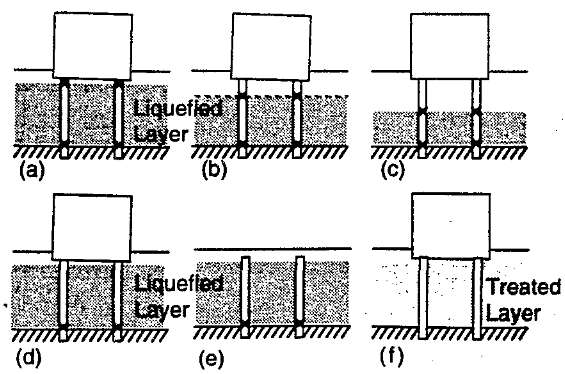

Fig. 6. Typical damage pattern of buildings on pile foundations subjected to lateral spreading in 1995 Kobe, Japan earthquake (Tokimatsu, 1999).

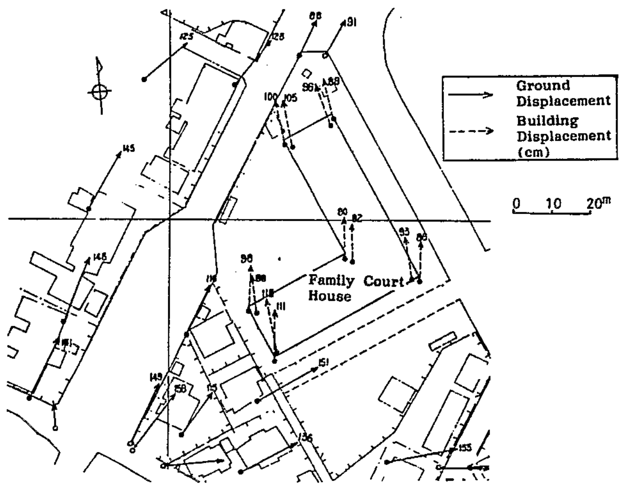

Fig. 7. Permanent horizontal ground surface displacement in the vicinity of Niigata Family Court House (NFCH) building, 1964 Niigata, Japan earthquake (Hamada, 1992a).

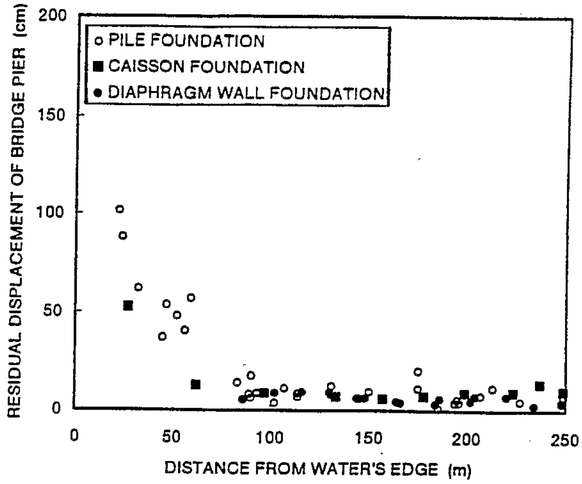

Fig. 9. Permanent horizontal displacements of bridge piers along Route 5 of Hanshin Expressway versus distance to waterfront, 1995 Kobe, Japan earthquake (Yokoyama et al., 1997).

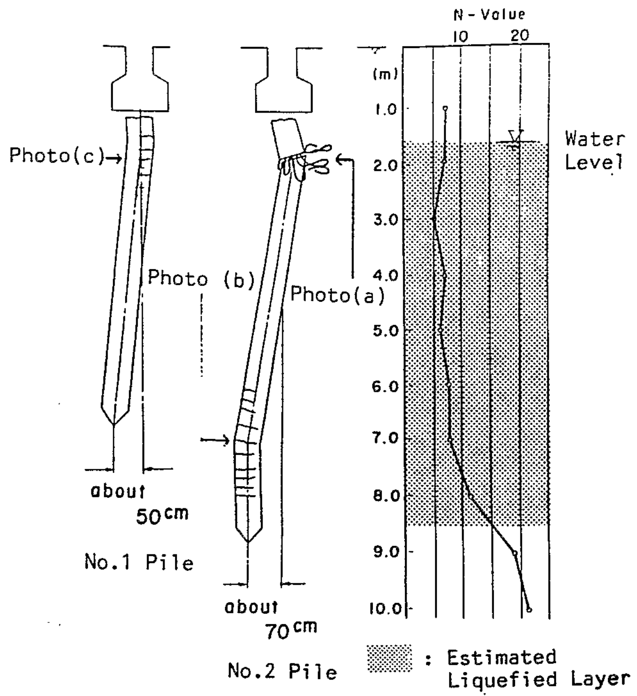

Fig. 8. Damage to floating and end-bearing piles and Standard Penetration Test N-values, NFCH building, 1964 Niigata, Japan earthquake (Hamada, 1992a).

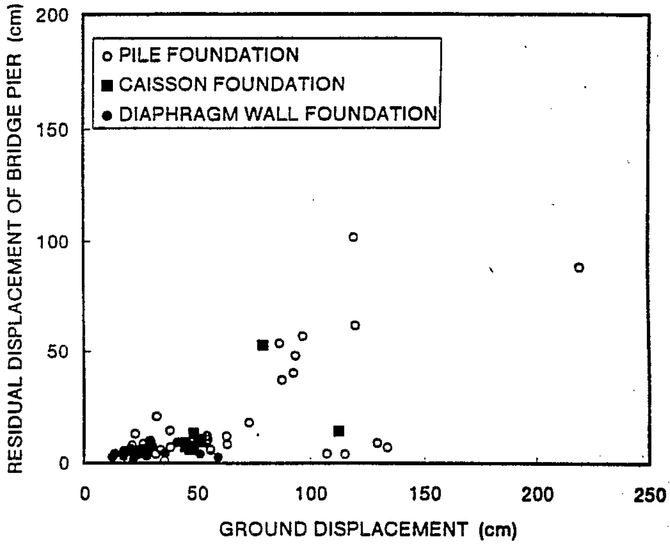

Fig. 10. Permanent horizontal displacements of bridge piers and of the ground, Route 5 of Hanshin Expressway, 1995 Kobe, Japan earthquake (Yokoyama et al., 1997).

Clearly the effect of lateral spreading on deep foundations is a complex, pseudostatic, kinematic soil-structure interaction phenomenon driven by the lateral movement of the ground in the free field. This is illustrated in Fig. 9, which shows the permanent horizontal displacements of bridge piers versus distance to the waterfront, along Route 5 of the Hanshin Expressway after the 1995 Kobe earthquake. This tendency of the pier displacements to decrease rapidly with distance to the waterfront is of course caused by the similar tendency of the free field ground displacement to decrease away from the waterfront, in the Kobe earthquake as well as in other seismic events. However, the two displacements are not necessarily the same (Fig. 10), with the free field deformation typically constituting an upper bound, and with the foundation head deformation ranging anywhere from zero to this upper bound; the value of the deep foundation head/bridge pier displacement depends strongly on the stiffnesses of the foundation itself (and, more generally, also on the superstructure stiffness). (Yokoyama et al., 1997.)

The experience of the Landing Road Bridge during the Edgecumbe, New Zealand earthquake is an extreme case in which the pile foundation and superstructure practically did not move and did not suffer significant damage, despite lateral spreading in the free field and large forces imposed by the ground on the foundation. This was a magnitude 6.3 earthquake having an epicenter 17 km from the bridge site. The structure is a 13-span highway bridge on simply supported spans carrying a two-lane concrete deck and two footpaths. The deck post-tensioned concrete beams are tied down with bolts at all piers and abutments, forming a quite stiff monolithic structure. The concrete slab piers run the whole width of the superstructure and are each supported by eigth 40.6 cm square batter prestressed concrete piles (Fig. 11). The abutments are also on piles. While extensive liquefaction and lateral spreading with 1.5 m lateral ground deformation occurred in the free field toward the river, only minor cracking at the base of the piers and some buckling of footpath slabs occurred, with the deformation limited to a 1° rotation toward the river of two of the piers. In addition, the batter piles under one of the abutments cracked at the top with a 0.5° rotation of the abutment face toward the river. Post-earthquake investigations revealed that the 1.5 m thick cohesive unliquefied crustal soil layer had failed in the passive mode against the buried portion of the slab piers (Fig. 11). Further limit equilibrium analysis indicated that the passive load exerted by this cohesive crustal layer on the foundation must have been of the order of 850 to 1000 kN per pier, with the pressure exerted by the liquefied layer neglected. The collapse load of the foundation system corresponding to the collapse mechanism of Fig. 11 is estimated to be about 950 to 1150 kN. Therefore, the fact that the soil failed before the foundation was a close call, with a factor of safety of one being reached almost simultaneously on both. That is, either a stronger shallow soil layer, a taller slab, or a weaker foundation system could have pushed the foundation into the failure mechanism sketched in Fig. 11 before or at about the same time the soil was experiencing passive failure. Clearly in this case pile batter played a positive role, contributing to both lateral stiffness and strength of the foundation. Important lessons of this case history include, again, the significance of the shallow nonliquefiable layer in controlling the lateral loading imposed on the deep foundation, and the role played by the stiffness of both foundation and superstructure (Berrill et al., 1997).

The potential detrimental effect of batter piles is illustrated by the two case histories in Figs. 12 and 13. Figure 12 corresponds to the 7th Street Terminal Wharf of the Port of Oakland damaged by the 1989 Loma Prieta earthquake in Northern California. This was a magnitude 7.1 earthquake with epicenter about 100 km from Oakland. Mainly due to site amplification, the general area of the wharf was subjected to a maximum ground acceleration of 0.25 to 0.30 g, with liquefaction, ground settlement (up to 0.3 m) and lateral spreading of 0.1 m toward the waterfront. The battered concrete piles with inboard inclination suffered tensile failures near the top, probably due to the lateral spreading and load concentration on the batter piles (Benuzca, 1990; Seed et al., 1990). Figures 13-14 show the soil conditions and the pile-supported quay and loading cranes in the container area of the industrial Port of Manzanillo that experienced the 1995 Manzanillo, Mexico, earthquake. This was a magnitude 7.9 earthquake of epicenter located less than 40 km from the port. The vertical and batter end-bearing piles of Fig. 13 are reinforced concrete, 50 cm square piles. Liquefaction (of at least the sand layer between elevations 8 and 14 m), ground settlement, and lateral spreading of several centimeters toward the waterfront were observed throughout the port. The top connections of about 100 of the 1300 piles were damaged by the lateral spreading, with most of the damaged ones being batter piles; again, clearly the greater lateral stiffness of batter piles produced load concentrations on them that the connections could not take (TGC, 1995; Swan et al., 1996).

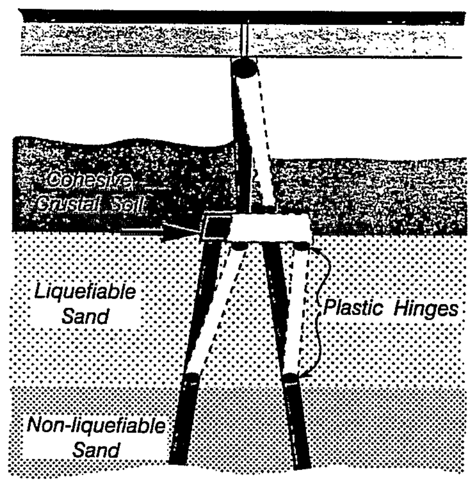

Fig. 11. Limit equilibrium evaluation of foundation of Land Road Bridge during 1987 Edgecumbe, New Zealand earthquake (Berrill et al., 1997).

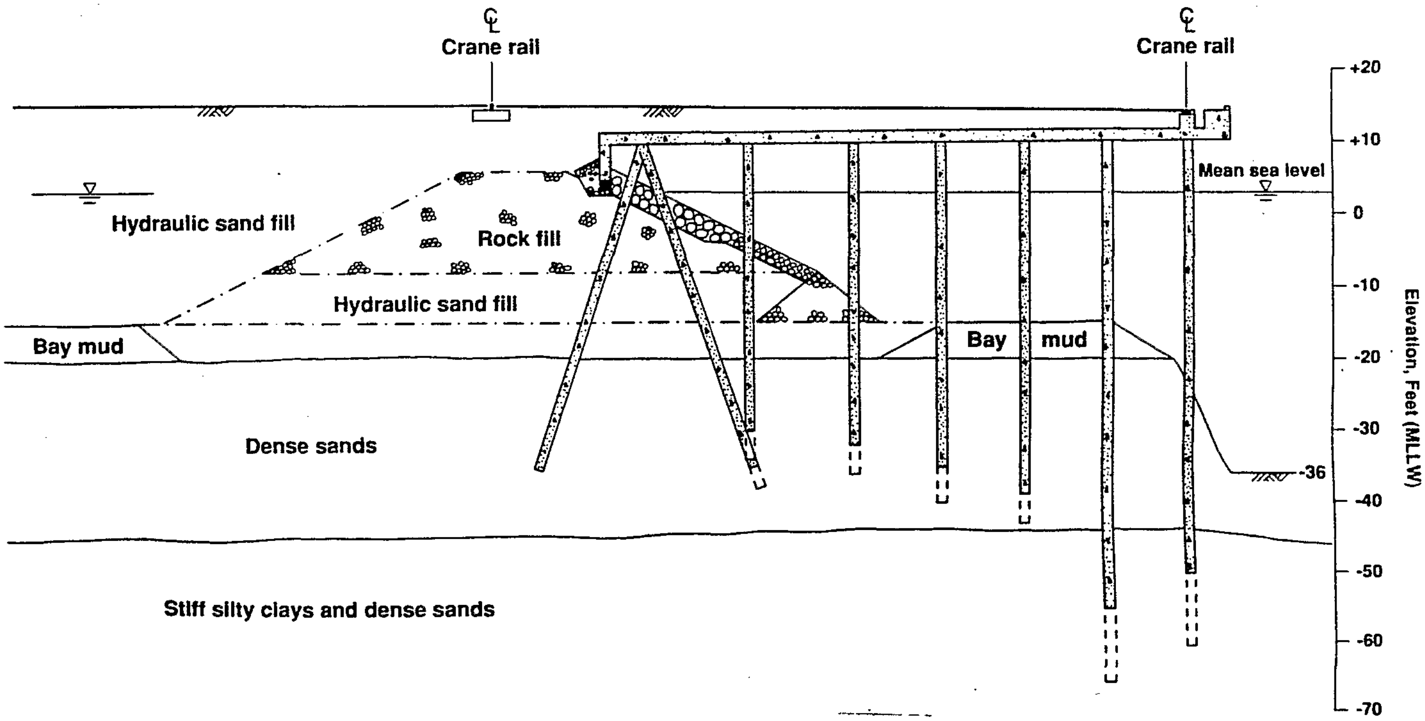

Fig. 12. 7th Street Terminal Wharf, Port of Oakland, damaged by lateral spreading in 1989, Loma Prieta, California earthquake (Benuzca, 1990).

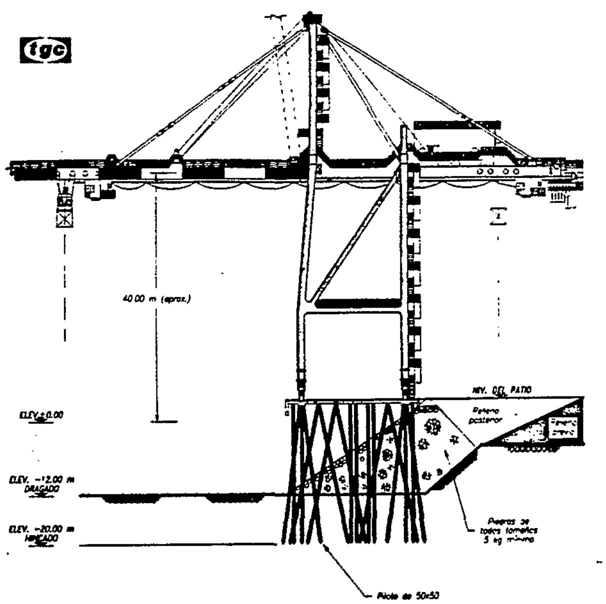

Fig. 13. Container quay and loading area, Puerto Manzanillo, damaged in 1995 Puerto Manzanilo, Mexico earthquake (TGC, 1995).

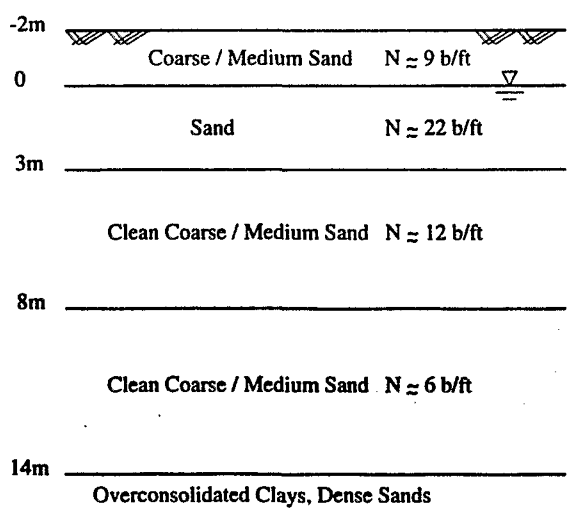

Fig. 14. Soil profile in container area of Puerto Manzanillo (from data in TGC, 1995).

While much of the damage to deep foundations due to liquefaction is associated with lateral spreading of the ground, which imposes a pseudostatic loading on the foundation only indirectly related to the earthquake shaking, the dynamic effects during shaking are also important. These effects include the inertial forces and overturning moments transmitted by the superstructure to the foundation, and the kinematic forces applied by the ground to the foundation due to earthquake wave propagation. However, these kinematic effects can often be neglected before the soil liquefies. Figure 15 is a conceptual sketch of the evolution of soil-pile-structure interaction in a laterally spreading soil. Prior to the development of high pore water pressures, the loading due to the inertia of the superstructure is paramount (Phase I). After high pore pressures and liquefaction, if the shaking continues, the inertial forces continue playing a paramount role, with larger cyclic foundation and ground deformations due to the weaker soil, now combined with kinematic forces on the foundation arising from large cyclic ground deformations in the free field (Phase II). Toward the end of shaking, while often the inertial loading decreases both because of ending of the earthquake and because of the isolating effect of the liquefied soil, the permanent ground deformations increase monotonically thus increasing the static lateral loading on the foundation (Phase III). (Tokimatsu, 1999.)

Figures 16-17 presents the evaluation of the Shin-Shukugawa Bridge that was subjected to lateral spreading during the 1995 Kobe earthquake. This is one of the bridges used to calibrate the limit equilibrium method for design of bridge foundations against lateral spreading adopted by the Japan Road Association, shown in Fig. 18 (Japan Road Association, 1996; Yokoyama et al., 1997). The Shin-Shukugawa Bridge, part of the Hanshin Expressway, is a 3-span cantilever box girder bridge that crosses a watercourse between reclaimed lands. The width of the bridge varies from 28 to 69 m due to the presence of a toll gate on Pier P-134 (Fig. 16). The bridge is supported by movable supports at both ends and by fixed supports on the intermediate piers. The foundations are different under different piers: Pier P-131 is a 2-column reinforced concrete rigid frame on two caisson foundations, 6 m in diameter; while Pier P-134 is a 4-column steel rigid frame supported by cast-in-place concrete piles, 1.5 m in diameter. Extensive liquefaction occurred around the bridge, with free field lateral spreading of about 2 m close to the waterfront. Bridge pier displacements as much as 0.9 m were measured, with damage as indicated in Fig. 16.

Figure 17 summarizes the limit equilibrium approach used to analyze Pier P-216 (not included in Fig. 16), which experienced a permanent lateral displacement of 0.9 m. This pier is a 2-story steel rigid frame supported by cast-in-place concrete piles 1.5 m in diameter. The bearings on this pier are fixed for the water-side girder and movable for the inland-side girder. In Fig. 17, it was assumed that the nonliquefied layer (between the ground surface and a depth of 3.3 m) applied passive pressure on the vertical areas of column and pile cap exposed to this layer. Between the depths of 3.3 m and 19 m liquefaction of the soil was assumed, with a lateral pressure on the pile cap and piles assumed to be a percentage of the total overburden pressure; this percentage was backfigured to be 32% in order to predict the 0.9 m lateral displacement of the foundation. Similar analyses conducted in other bridges of the Hanshin Expressway subjected to lateral spreading provided a similar factor of 0.3 for the liquefied soil pressure. Finally, the limit equilibrium procedure of Fig. 18 was adopted for design of bridge foundations subjected to lateral spreading and incorporated into the revised Japanese Specifications for Highway Bridges (Japan Road Association, 1996). The expression for q_L = 30% of the overburden pressure for the liquefied layer in the figure corresponds to full liquefaction and large free-field ground displacements, stated in the specifications to correspond to a distance of less than 50 m from the waterfront. For less than full liquefaction and greater distances to the waterfront, reduction coefficients are introduced and q_L is less than 30% of the overburden pressure. (Japan Road Association, 1996; Yokoyama et al., 1997.)

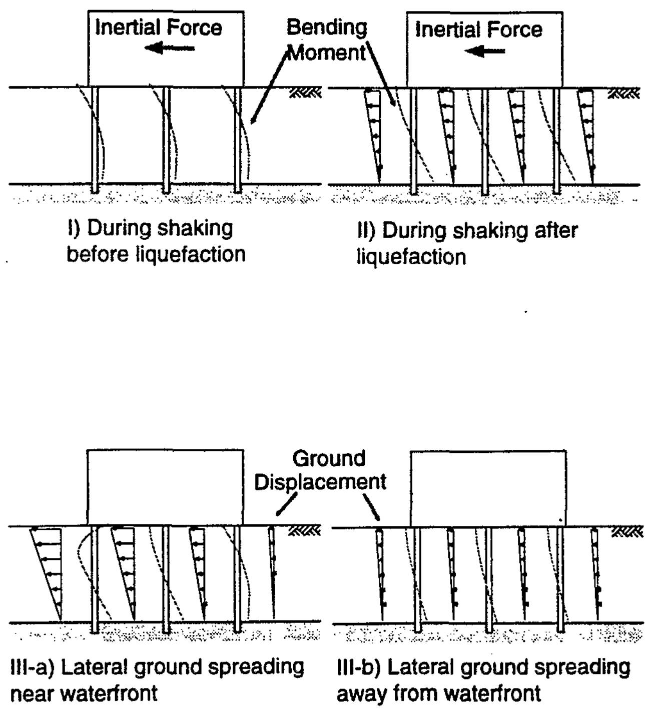

Fig. 15. Schematic figures showing soil-pile-structure interaction in lateral spreading soil (Tokimatsu, 1999).

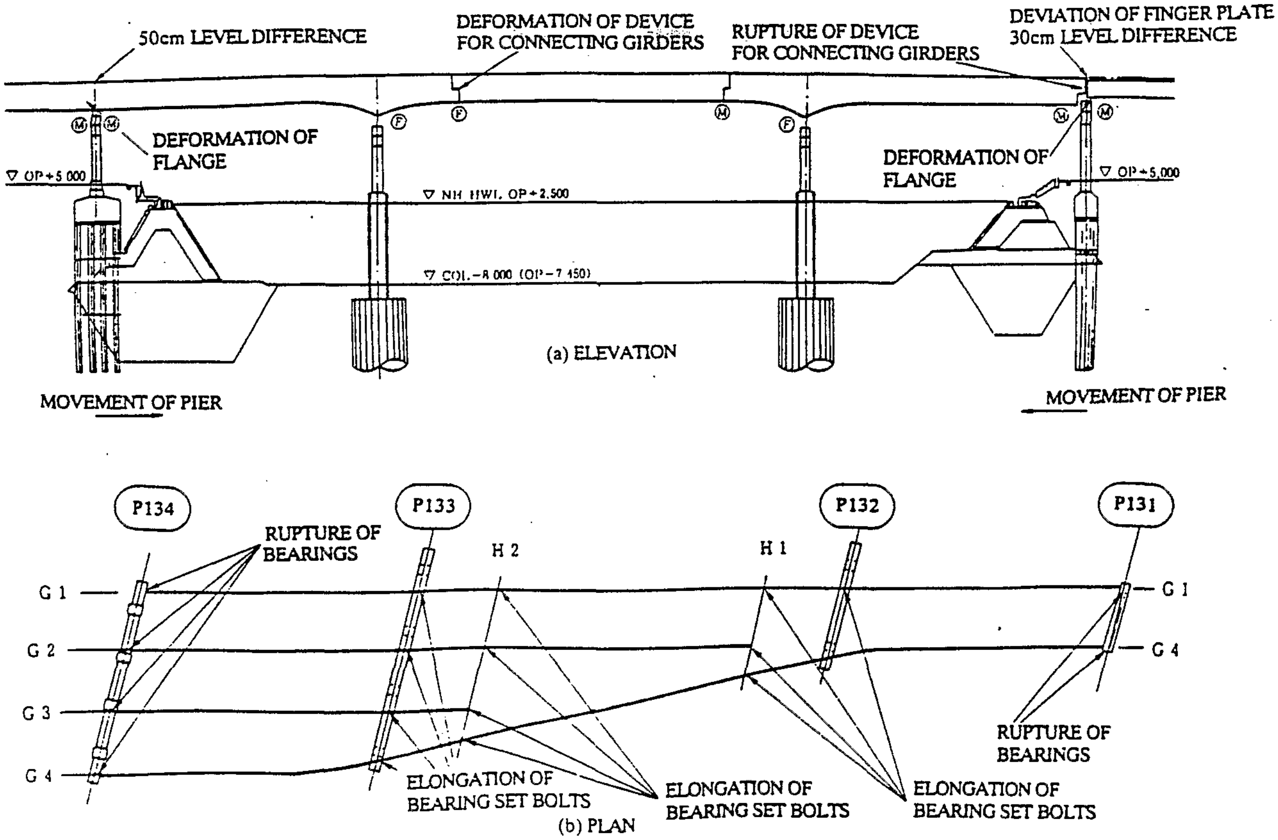

Fig. 16. Overview of damage to Shin-Shukugawa Bridge, 1995 Kobe, Japan earthquake (Yokoyama et al., 1997).

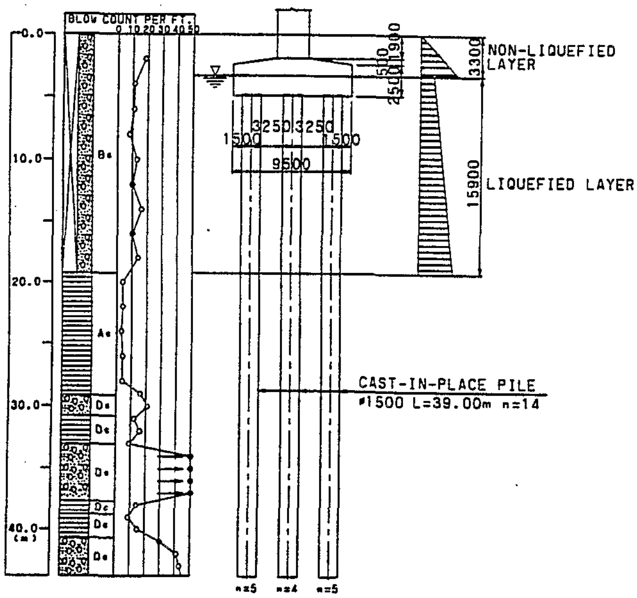

Fig. 17. Foundation of Shin-Shukugawa Bridge in 1995 Kobe, Japan earthquake, analyzed to develop limit equilibrium method of Fig.18 (Yokoyama et al., 1997).

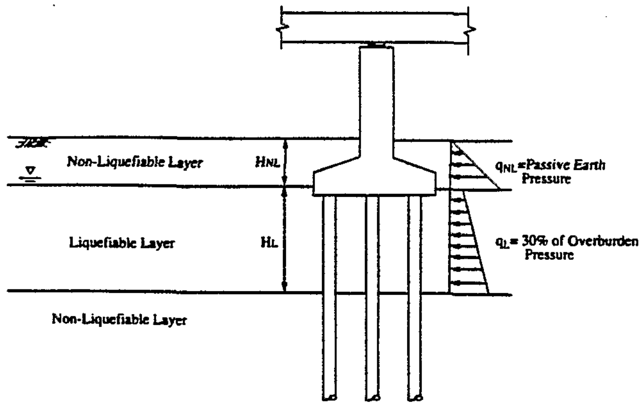

Fig. 18. Limit equilibrium method for design of bridge deep foundations subjected to lateral spreading (Japan Road Association, 1996; Yokoyama et al., 1997).

MAIN LESSONS LEARNED FROM CASE HISTORIES

Clearly the effects of lateral spreading on deep foundations are very significant and the associated soil-foundation-structure interaction phenomenon is rather complex. It corresponds essentially to a kinematic rather than to inertial loading, with the deep foundation and superstructure responding pseudostatically to the lateral permanent displacement of the ground. To this picture must be added the effects of the superstructural inertia forces and of the cyclic foundation and soil deformations during shaking, as indicated in Fig. 15. While the summaries of case histories by Youd and Tokimatsu reproduced at the beginning of the previous section point out clearly to some of the factors and lessons learned from the field experience, it is useful to revisit the situation once more, based on the discussion and figures of the previous section. This revisiting will also serve as a useful link between the field experience and the centrifuge modeling of deep foundations reported in the rest of the paper.

The examination and analysis of the case histories of lateral spreading point out to the great significance of several factors affecting the post-earthquake displacement and shape of the deep foundation, as well as any distress such as connection failure, or development of bending cracks/shear failures/plastic hinges in piles or caissons. These factors include:

- (i) free field permanent lateral ground displacement near the foundation (Figs. 9, 10);
- (ii) thicknesses and properties of soil strata - including soil shear strength and passive pressure - penetrated by the piles or caissons, including both he liquefied soil and soils above and below the liquefied layer (Figs. 2, 3, 8, 11, 17, 18);
- (iii) geometry and properties of the piles or caissons, including their bending stiffnesses as well as the areas of piles/caissons, pile cap and other structural elements exposed to the soil pressures generated by the lateral spread of the foundation (Figs. 10, 11, 17, 18);
- (iv) restraining stiffness and strength of the superstructure including strength of connections between piles and pile cap (Figs. 1, 5, 10, 11); and
- (v) presence of end-bearing pile groups or batter piles that both increase the lateral stiffness of the foundation and attract large loads to the battered pile heads (Figs. 11, 12, 13).

While in some cases the top of the foundation moves laterally a distance similar to that in the free field (Figs. 8, 10), in others it moves much less due to the constraining effect of the superstructure (Figs. 1, 2, 10, 11) or of the deep foundation's lateral stiffness including pile goups and batter piles (Figs. 1, 10, 11). Both very rigid and more deformable foundation-superstructure systems may be exposed to large lateral soil pressures, including especially passive pressures from the nonliquefied shallow soil layer riding on top of the liquefied soil (Figs. 8, 11, 12, 13, 17). The observed (or expected) damage and cracking to piles is often concentrated at the upper and lower boundaries of the liquefied soil layer where there is a sudden change in soil properties (Figs. 2, 3, 6, 8, 11, 17, 18), or at the connections between pile and pile cap (Figs. 1, 11). More damage tends to occur to piles when the lateral movement is forced by a strong nonliquefied shallow soil layer (end-bearing pile No. 2 in Fig. 8), than when the foundation is more free to move laterally and the forces acting on them are limited by the strength of the liquefied soil (floating Pile No. 1 in Fig. 8). The presence of batter piles may prove after the fact to have been beneficial in increasing the overall lateral stiffness and strength of the system (as in Fig. 11), or detrimental by attracting load concentrations to the batter pile heads that the foundation cannot resist (Figs. 12, 13). Finally, an attractive possibility for evaluation and design of deep foundations subjected to lateral spreading involving large free field deformations, is the use of a limit equilibrium approach, which assumes that the foundation is loaded by the maximum possible lateral soil pressures, including passive pressures when appropriate (Figs. 11, 17, 18).

CENTRIFUGE PHYSICAL MODELING

Centrifuge physical modeling with in-flight shaking has emerged as a most valuable tool in liquefaction research, both for the free field and for the interaction between the soil, foundation and structure. Its potential has been demonstrated by many studies, starting in the 1980's with the results reported by Whitman et al. (1981), Schofield (1981), Arulanandan et al. (1983), Steedman (1984), Coe et al. (1985) Hushmand et al. (1988) and Ketcham et al. (1991). Further important new developments took place under Project VELACS (VErification of Liquefaction Analysis using Centrifuge Studies), supported by the National Science Foundation and centered around a cooperative centrifuge liquefaction study involving seven universities (Arulanandan and Scott, 1993, 1994; Arulanandan, 1994; Scott, 1994).

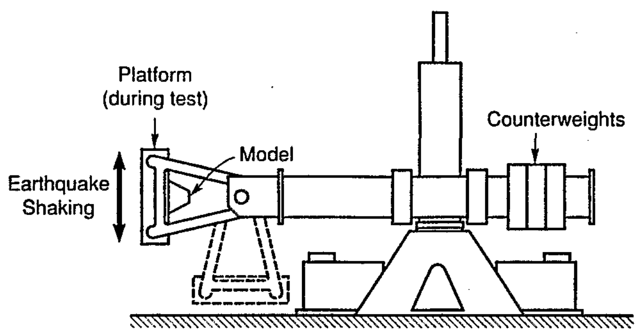

Fig. 19. Sketch of Rensselaer Polytechnic Institute (RPI) 100 g-ton geotechnical centrifuge with in-flight earthquake base shaking capability.

A centrifuge is any device that spins and generate centrifugal forces to achieve some practical purpose. It produces what is essentially an artificial gravitational field that is higher than the earth's 1 g field. In a geotechnical centrifuge like the one sketched in Fig. 19, a small-scale soil, soil-foundation or soil-structure system is subjected to a centrifugal acceleration typically somewhere between 30 g and 120 g. When the arm spins around the axis, the platform and model gradually rotate about 90° around the hinges during the flight. In Fig. 19, an earth embankment model is sketched, but it could be any soil or soil-structure system. If this model is spun for a few minutes or hours, the action of the earth's gravitational field on a real embankment many times bigger is simulated. Then, is a horizontal accelerogram is applied to the base of the model while everything is still in flight - as also sketched in Fig. 19 - this tests the seismic response of the model and provides answers that are applicable to the actual prototype system.

The need for small-scale model testing of liquefaction, lateral spreading and their effects on deep foundations arises due to the complexity of the phenomena involved. The case histories and field observations discussed in previous sections are very useful, but they do not provide all the information needed for analysis and design engineering applications. However, if a small-scale model of, say, a pile foundation embedded in a liquefying soil profile is tested on a regular shaking table in a 1 g environment, the total and effective confining stresses will be too small in the model soil, which will behave very differently from the prototype soil in the field. To get the stresses back up to their correct values, the g-level is increased by placing the model in a centrifuge. We expect - based on the relevant scaling relations listed in Table 2 - that the resulting model soil strains and deformations will be close to those in the field. Therefore, a basic fact of centrifuge testing is this expectation that the stresses and strains are the same for corresponding points of model and prototype, and that the displacements are n times bigger in the prototype than in the model. That is, if the centrifuge is spun at 30 g, and thus n = 30, 1 cm in the model represents 30 cm in the prototype. (These statements as well as the rest of the discussion below, are based on the assumption that the model soil and prototype soil are the same, that is there is no attempt to model the soil itself.)

The scaling of the time is especially important in liquefaction applications. From the viewpoint of the dynamic response, the factor is n; that is, for n = 30, 1 second in the model corresponds to 30 seconds in the prototype. However, the time scaling factor for diffusion and consolidation phenomena where the soil permeability plays a role, is different, n^2 (see Table 2); hence for n = 30 the scaling factor is 900, and 1 second in the model corresponds to 900 seconds in the prototype. This assumes that both soil and pore fluid (typically water) are the same in the soil and prototype. As the liquefaction centrifuge simulations of interest here involve base shaking and thus dynamic response, as well as diffusion and consolidation phenomena during and after shaking, this is a serious issue. It means, for example, that if a water-saturated fine sand is used in a centrifuge model spun at 30 or 50 g, it is simulating a water-saturated coarse sand of otherwise similar characteristics in the field having a prototype permeability 30 or 50 times higher. While the study of coarse sand liquefaction is useful, and thus water-saturated centrifuge liquefaction models do have a role, to bring the model fine sand down to the necessary permeability 30 or 50 times smaller than that of the prototype fine sand in the field, very often a viscous pore fluid is used having 30 or 50 times the viscosity of water. In this way, both the dynamic and diffusion scaling relations listed in table 2 are simultaneously satisfied. A number of the centrifuge tests discussed in the rest of this paper have used a viscous pore fluid, while others have used water.

Table 2 Partial List of Scaling Relations Used for Centrifuge Modeling (n = centrifuge acceleration in g)

Parameter	Model Units	Prototype Units
Length	1	n
Velocity	1	1
Acceleration	1	1/n
Stress	1	1
Strain	1	1
Time:		
Dynamic	1	n
Diffusion	1	n^2
Frequency	1	1/n

THE RPI CENTRIFUGE FACILITY

Most of the centrifuge model testing results presented in this paper were conducted at the Rensselaer Polytechnic Institute (RPI)'s geotechnical centrifuge sketched in Fig. 19. This machine has a capacity of 100 g-ton; that is, it can spin 1 ton at 100 g or 0.5 ton at 200 g; the arm has a total length to the platform in flight of 3 m.

The centrifuge experiments which simulate liquefaction-induced lateral spreading and its effect on pile foundations were performed using the rectangular, flexible-wall laminar container shown in Fig. 20. This RPI laminar box was designed for use with in-flight earthquake shakers available at the facility, for improved modeling of seismic phenomena in soil and soil-structure systems. As discussed later herein, in the lateral spreading experiments, both the laminar box and the shaker under it are inclined a few degrees to simulate an infinite mild slope and provide the shear stress bias needed for a lateral spread. This laminar box is comprised of a stack of up of 39 rectangular aluminum rings separated by linear roller bearings, arranged to permit relative movement between rings with minimal friction. Relative displacements of up to 6.35 mm between adjacent rings are possible, and the design permits overall shear strains of up to 20%. These large values were provided to accommodate large permanent deformations and strains - including strains concentrations - expected during simulation of earthquake-induced simulation and lateral spreading.

Additional details about the RPI centrifuge facility and its use for lateral spreading experiments can be found in Elgamal et al. (1991), Van Laak et al. (1994a,b, 1998), Dobry et al. (1995, 1997), and Taboada and Dobry (1998), as well as at the facility's Web site: www.ce.rpi.edu/centrifuge/. A number of centrifuges around the world - especially in the U.S. and Japan - are also equipped to conduct in-flight earthquake simulations including use of flexible-wall box containers; some of them have also been used to model liquefaction, lateral spreading and its effects on deep foundations. Several of these facilities are described in some of the papers listed under Item 1 in Table 3; other facilities can either be found as links through the RPI Web site mentioned above, or are described in papers in one of the international geotechnical centrifuge conferences which have taken place since 1985 (Craig, 1985; Ko and McLean, 1991; Leung et al., 1994; Kimura et al., 1998).

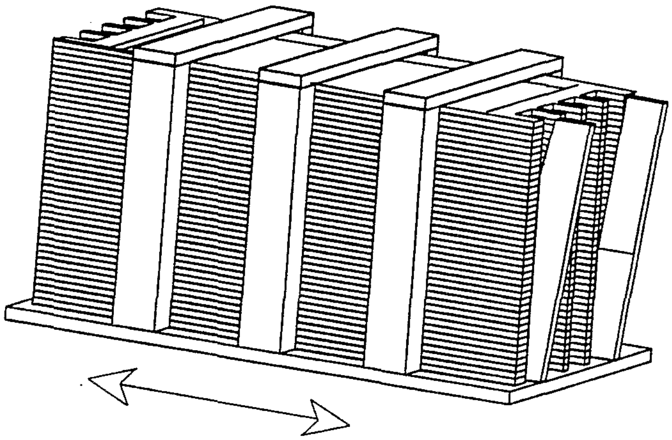

Fig. 20. Schematics of RPI laminar box container (Van Laak et al., 1994b).

CENTRIFUGE MODELING OF DEEP FOUNDATION RESPONSE

Table 3 summarizes recent research on centrifuge and 1g physical modeling of liquefaction and its effects on deep foundations, with appropriate references. Item 1 of the table includes a number of state-of-the-art reviews, general discussions and compilations of physical model experiments relevant to liquefaction, seismic centrifuge testing, and effects of liquefaction on foundations. While most of the publications listed in Item 1 focus on liquefaction and seismic centrifuge testing rather than on foundations - and thus serve as a general introduction to the subject - all references listed under Items 2 and 3 of the table include results of physical model tests specifically simulating the effect of liquefaction on deep foundations. While most of these model experiments were done in centrifuges, some of them were conducted at 1 g, typically using shaking tables; this is indicated at the side of the corresponding reference by either the symbols C (for centrifuge) or 1g. It is interesting that, without exception, all references listed in Items 2 and 3 correspond to model tests done either in the U.S. or Japan, reflecting the interest on the subject and rapidly increasing use of the centrifuge in these two countries. Item 2 relates to liquefaction and piles in level ground, while Item 3 relates to liquefaction and piles, either in sloping ground or behind a retaining structure, thus modeling the important effect of lateral spreading on the foundations. The experiments including a retaining structure (typically a quay wall) with a liquefiable sand and pile foundation behind it have been identified in Table 3 by the letters RS at the side of the corresponding reference; all of them are Japanese articles published after 1995 which reflect the need to understand better what happened at the port facilities in Kobe after that year's earthquake. While most of the 1g or centrifuge experiments listed under Items 2 and 3 were conducted on single piles, in some cases pile groups were also modeled; these have been identified by the letters PG at the side of the corresponding reference.

A comprehensive review of all results and conclusions presented in the references of Table 3 about this complex phenomenon is beyond the scope of this state-of-the-art paper. The interested reader is directed to the references themselves. The rest of this paper discusses selectively some of the centrifuge and 1g results reported in Items 2 and 3 of the table, with the help of Figs. 21-50, starting with piles embedded in liquefiable level ground and continuing with lateral spreading effects on piles embedded in sloping ground.

Piles in Level Ground

The response of piles and pile groups with a superstructural mass embedded in liquefiable level soil, having a mass above ground, and subjected to horizontal base shaking has been studied systematically at the University of California, Davis, centrifuge, through experiments such as shown in Fig. 21. The results and interpretations of these tests have been reported by Boulanger et al. (1997), Wilson (1998) and Wilson et al. (1998, 1999, 2000). Setups and results of the tests are reproduced in Figs. 21-24. As shown in Figs. 21-22, a flexible shear beam container box was used having the same general purpose of the RPI laminar box of Fig. 20: to allow the soil in the free field to deform in shear during and after shaking thus approximating the actual prototype field conditions during an earthquake. The large container (1.72-m long by 0.685-m wide by 0.70-m deep) used in this large centrifuge allowed testing three different pile foundation-mass systems at the same time, embedded in the same soil and subjected to the same base shaking and free field deformations: a single pile, a 2x2 pile group and 3x3 pile group (Fig. 21). The soil profile consisted

Table 3 Recent Developments on Physical and Centrifuge Modeling of Liquefaction, Lateral Spreading and Their Effects on Deep Foundations

Item	Explanation	References
1. SOA Reviews	Publication of several SOA reviews, general discussions and physical model test compilations relevant to liquefaction, seismic centrifuge testing, and effects of liquefaction on foundations	Schofield and Steedman (1988) Steedman (1991) Finn (1991) Ishihara and Takeuchi (1991) Dobry and Liu (1992) Ko (1994) Dobry (1995) Kutter (1995) Dobry et al. (1995) Dobry and Abdoun (1998) Ishihara and Cubrinovski (1998) Kutter and Balakrishnan (1998) Ko and Dewoolkar (1999)
2. Piles in Level Ground	Centrifuge and 1g shaking table model tests of instrumented deep foundations in level soil profiles including liquefiable soil (C = centrifuge tests; 1g = shaking table or other tests at 1g; PG = tests included pile groups)	Sato et al. (1995) C, PG Liu and Dobry (1995, 2001) C Kagawa et al. (1995) 1g, PG Sakajo et al. (1995) 1g, PG Boulanger et al. (1997) C, PG Wilson (1998) C, PG Wilson et al. (1998, 1999, 2000) C Koseki et al. (1998) C Taji et al. (1998) C, PG Adachi et al. (1998) C, PG Cubrinovski et al. (1999) 1g, PG
3. Piles in Sloping Ground or Behind Retaining Structure (effect of lateral spreading on deep foundation)	Centrifuge and 1g (mostly shaking table) model tests of instrumented deep foundations in sloping ground or behind retaining structure, with soil profile including liquefiable soil (C = centrifuge tests; 1g = shaking table or other tests at 1g; RS = tests included retaining structure; PG = tests included pile groups)	Tokida et al. (1992) 1g, PG Towhata (1995) 1g Abdoun et al. (1996) C Dobry et al. (1996) C Yasuda et al. (1996) 1g, RS Ohtomo (1996) 1g, PG Abdoun (1997) C, PG Tamura and Azuma (1997) 1g, RS Abdoun and Dobry (1998) C Satoh et al. (1998) C, RS Fujiwara et al. (1998) C, RS Takahashi et al. (1998) C, RS, PG Sato and Zhang (1998) C, RS, PG Horikoshi et al. (1998a,b) C, RS Ramos (1999) C Ramos et al. (1999, 2000) C Hamada (2000) 1g

of two layers of fine Nevada sand saturated with a viscous fluid having ten times the viscosity of water. All tests were conducted at a centrifugal acceleration of 30 g (n = 30). The bottom, dense layer consisted of 11.4 m thick (prototype) sand with D_r = 80%, while the top, loose liquefiable layer consisted of 9.1 m thick sand with D_r = 35% or 55%. Each model pile simulated a steel pipe pile 0.67 m in prototype diameter and a wall thickness of 19 mm, 14 to 17 m long and supporting a superstructure vertical load of 450 to 500 kN. The soil was instrumented with acceleration and pore pressure transducers at different depths as well as vertical LVDTs to measure the ground surface settlement. The piles were instrumented with strain gages at various elevations to measure bending moments as well as with horizontal LVDTs and accelerometers at and above the ground surface. In each centrifuge test of the type shown in Fig. 21, the box was subjected to a series of in-flight horizontal base shaking events, beginning with low-level shaking and progressing through strong motions with prototype peak accelerations of up to 0.6 g. Full liquefaction was induced in the upper soil layer by the stronger shaking events, with both the degree of liquefaction of this upper layer and its relative density (D_r = 35% or 55%) significantly influencing pile response.

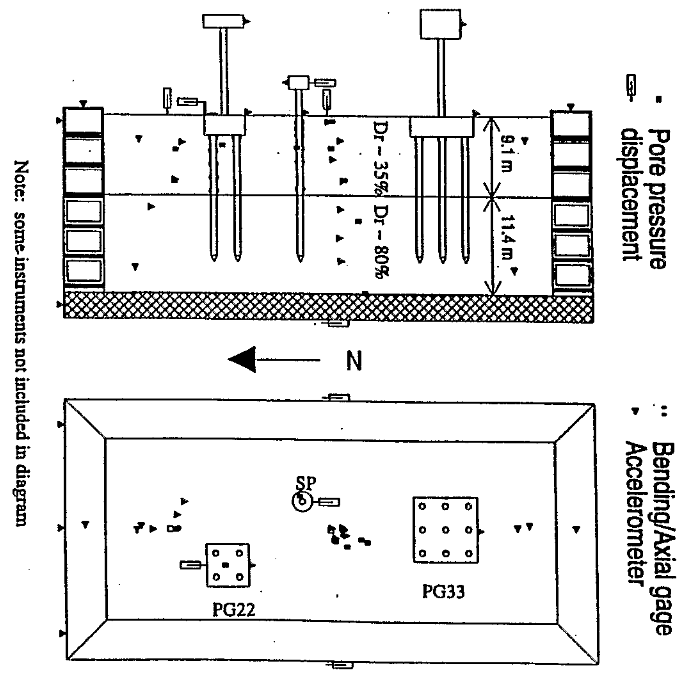

Fig. 21. Model layout for single pile and pile groups with mass on top, in level liquefiable sand, tested in U. of California, Davis flexible shear beam container (Boulanger et al., 1997).

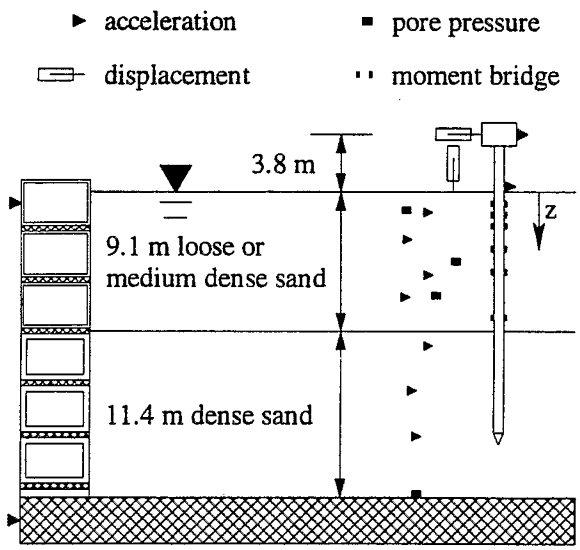

Fig. 22. Schematic of layout and instrumentation of centrifuge modeling of single pile with mass on top in level liquefiable soil subjected to baes shaking (Wilson et al., 2000).

Figures 23-24 present some results and analyses for the response of the single pile, for the test in which the upper sand layer had a D_r = 55%. A main desired result of these and other centrifuge tests is the evaluation of pore pressure buildup and liquefaction on the p-y curves characterizing the soil-pile load-deformation interaction at various depths, where p = horizontal load between soil and pile per unit pile length, and y = relative horizontal displacement between pile and soil. The use of nonlinear p-y curves, essentially characterizing the lateral resistance of the soil to pile movements by nonlinear Winkler soil springs, has been widely used for the analysis of offshore and inland structures on piles subjected to either ocean wave storms or earthquakes. The use of p-y curves was proposed by Reese et al. (1974) and Reese and Wang (1993), and has been extended to liquefied and lateral spreading soils by Meyersohn et al. (1992), Meyersohn (1994), O'Rourke et al. (1994), Liu and Dobry (1995), Debanik (1997), Wang and Reese (1998), Ramos (1999) and Goh (2001). Figure 24 includes the monotonic p-y curves at various depths for the upper sand layer in the centrifuge experiment of Fig. 22, calculated assuming drained loading and thus no pore pressure buildup using the recommendations of the American Petroleum Institute (API, 1993). A main purpose of the centrifuge experiment of Fig. 22 was to evaluate the degradation of the p-y curves as the pore pressures built up and the soil liquefied. Figures 23-24 illustrate the results of such analysis conducted by the researchers at the U. of California, Davis. In Fig. 23, the load p at a given time during the shaking was back-calculated by double differentiation with respect to depth of the recorded bending moments along the pile, while the displacement of the pile, y_{pile}, was obtained by double integration of the same bending moments. The simultaneous displacement of the soil, y_{soil}, in the free field at the same depth was obtained by double integration of the accelerations measured in the soil, and $y = y_{soil} - y_{pile}$. This creative application of System Identification techniques resulted in the profiles of p, y_{soil} and y_{pile} shown in Fig. 23 and in the backfigured hysteresis p-y loops depicted in Fig. 24 (for D_r = 55% in the upper sand layer). Figure 24 illustrates both the degradation of the p-y curves with pore pressure buildup, and their stiffening when the value of y exceeds a certain value due to the undrained dilative response of the sand in shear.

Figure 25 shows recorded bending moments time histories along a single pile in a similar test with a mass on top conducted at the RPI centrifuge by Liu and Dobry (2001). In this test the pile was fixed at the bottom of the box and the whole sand layer was liquefiable. The most interesting result in the figure is that while early in the shaking, before liquefaction (t = 1 to 2 sec) the maximum moment was small and occurred at a shallow depth, later in the shaking and after the whole soil had liquefied (t = 14 sec), a much larger maximum moment developed, which is now concentrated at the bottom of the pile.

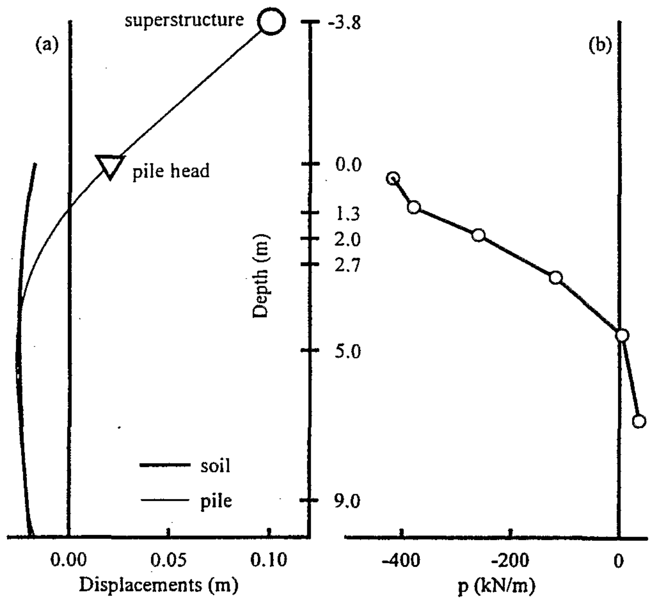

Fig. 23. Soil (y_{soil}) and pile (y_{pile}) instantaneous prototype displaced shapes, and simultaneous lateral soil force per unit pile length, p, measured or back-calculated at time t = 6.2 sec in one of the centrifuge tests of Fig. 22 (Wilson et al., 2000).

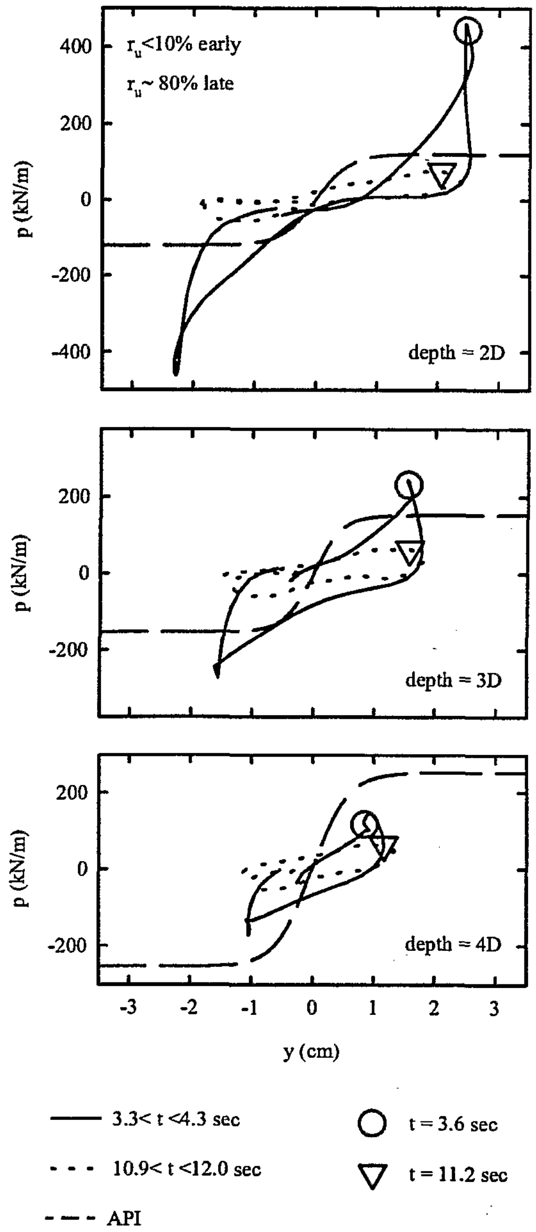

Fig. 24. Observed prototype p-y behavior in one of the centrifuge tests of Fig. 22 ($y = y_{soil} - y_{pile}$), relative density D_r = 55-60% for upper sand layer (Wilson et al., 1998).

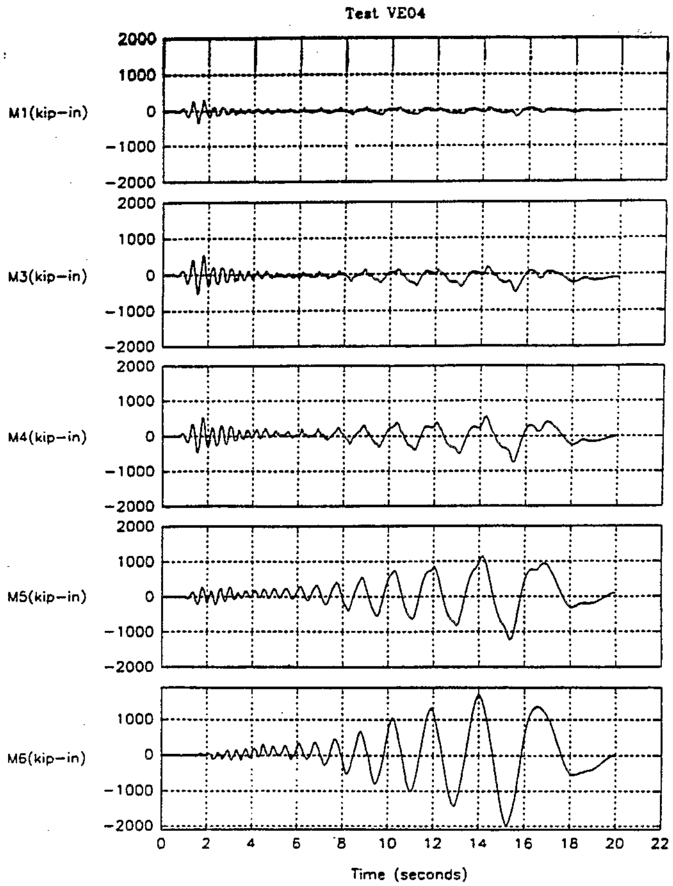

Fig. 25. Prototype bending moments versus time in centrifuge model pile fixed at the bottom in liquefiable level sand layer subjected to base shaking (Liu and Dobry, 2001).

Figures 26-27 summarize a study conducted by Liu and Dobry (1995, 2001) at RPI to obtain a correlation between p-y curve degradation and pore pressure buildup for Nevada sand of D_r = 60%, by conducting static cyclic lateral load testing of the pile in flight after the shaking had ended. In these tests, there was no mass above ground, and the top and bottom of the pile remained fixed during the shaking and associated pore pressure buildup in the soil. Immediately after shaking the top of the pile was released and the lateral static cyclic loading was conducted with the horizontal actuator shown in Fig. 26. A pore fluid ten times more viscous than water was used in these experiments, which were done in the rigid box container of Fig. 26 rather than in a laminar box. Figure 27 summarizes the correlation obtained between the degradation coefficient C_u and the pore pressure ratio in the sand at the same depth, r_u, where $C_u = p_{ru} / p_o$ is the ratio between the measured value of $p = p_{ru}$ and the same value of $p = p_o$ before pore pressure buildup, at a value of y = 2 inches. Figure 27 clearly illustrates the decrease in C_u and thus in the value of p for a given y, as r_u increases, with C_u being about 0.10 to 0.15 at full liquefaction. The approach taken in Fig. 27 assumes that C_u is the same at all values of the displacement y, which was a reasonable conclusion for these tests in the range of displacements used in the static loading tests, up to y = 2 inches. However, separate dynamic tests with a mass on top such as that of Fig. 25 showed that at the much larger displacements induced after liquefaction by the superstructure's inertial loading, dilative undrained stiffening of the p-y curves was observed, similar to that found in the Davis experiments (Fig. 24).

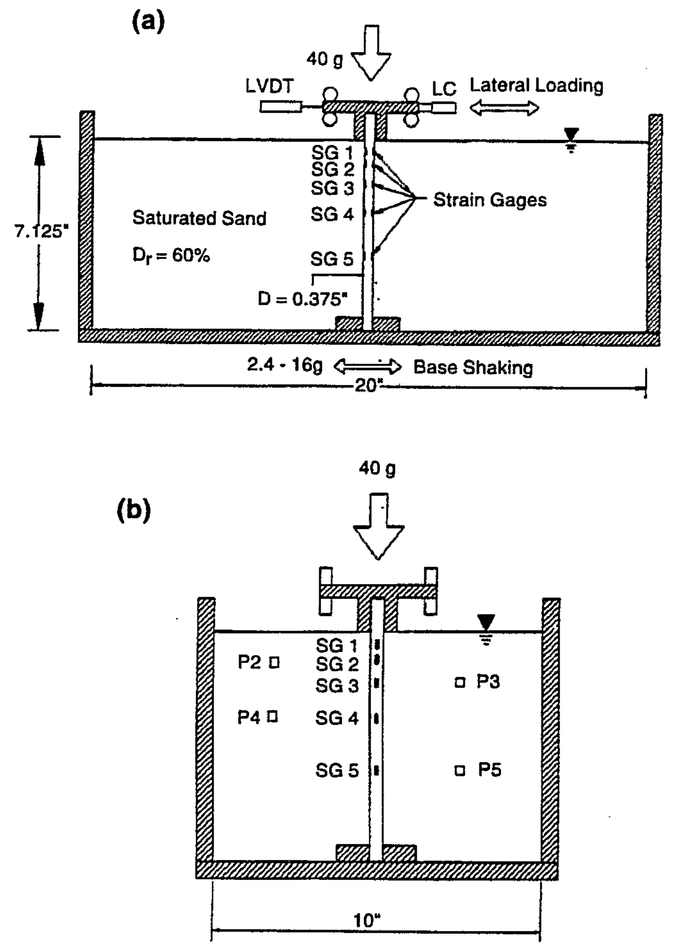

Fig. 26. Setup of centrifuge model tests of single pile in level liquefiable layer, consisting of base shaking followed by static lateral load test of pile for p-y curve determination (Liu and Dobry, 1995, 2001).

<u>Piles in Sloping Ground</u>

Item 3 in Table 3 lists a number of model studies focusing on the effect of permanent ground deformations associated with lateral spreading on the deep foundations, which as shown before in this paper is a major cause of damage to piles. Several of them are centrifuge studies involving a pile foundation behind a quay wall or other waterfront retaining structure, as illustrated by Fig. 28, while in other studies the lateral spreading is caused by mildly sloping ground (Fig. 29).

Both in the field (i.e., 1995 Kobe earthquake) and in the centrifuge studies of piles behind quay walls, large permanent lateral ground deformations, typically of the order of meters, have developed immediately behind the wall as the wall rotates and translates toward the ocean side. While these ground deformations decrease rapidly inland, they can be significant for a distance of a few hundreds of meters from the wall, and the distress to the pile foundations is directly related to the amount of free field ground deformation in the neighborhood of the foundation. This, of course, is true also when the lateral spreading is caused by sloping ground without a retaining structure. Therefore, much can be learned about the general aspects of the response of pile foundations to lateral spreading, by studying the behavior of piles to the free field ground deformations in the context of much simpler sloping ground centrifuge tests such as those of Fig. 29.

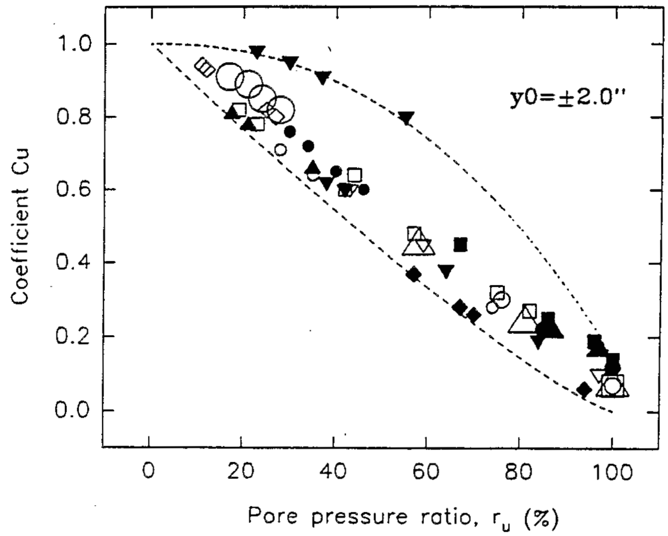

Fig. 27. Degradation coefficient, $C_u = p_{ru}/p_o$, versus pore pressure ratio, r_u, from static lateral load tests of Fig. 26 (Liu and Dobry, 2001).

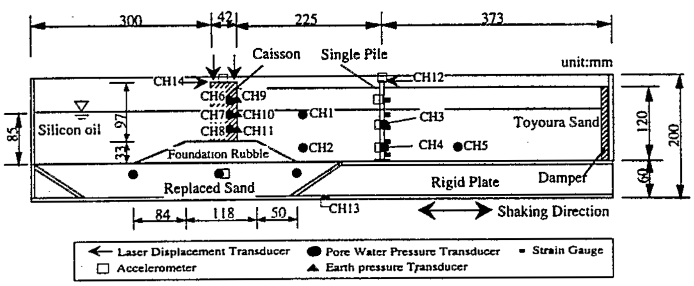

Fig. 28. Lateral spreading centrifuge modeling of pile behind quay wall (Fugiwara et al., 1998).

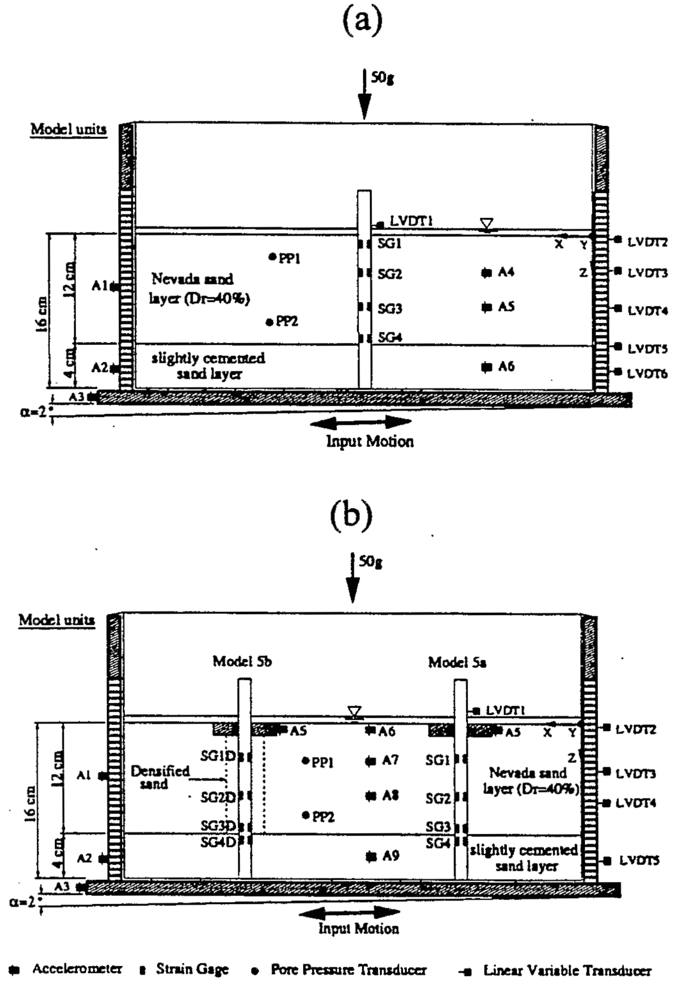

Fig. 29. Lateral spreading centrifuge setup of pile in inclined 2-layer soil profile simulating mild infinite slope, in model units (Abdoun and Dobry, 1998).

The rest of this section discusses the response of piles to lateral spreading in model tests of sloping ground, typically without a mass above ground, for single piles and pile groups in different soil profiles, and with the piles either unconstrained at the top or constrained by springs above ground to simulate the effect of the superstructure's stiffness. Most of these tests were conducted at the RPI centrifuge (Fig. 19) using the RPI laminar box container (Figs. 20 and 29). At the end of the section, one set of results is presented illustrating the combined effect of lateral spreading and a mass above ground.

Bending Response Controlled by Liquefied Soil. Figure 29 presents the setups of two centrifuge experiments including three pile models, labeled respectively Models 3, 5a and 5b. In both cases, the soil profile consisted of two layers of fine Nevada sand saturated with water: a top liquefiable layer of D_r

= 40% and 6 m protoype thickness and a bottom slightly cemented nonliquefiable sand layer having a thickness of 2 m. The prototype single pile simulated in the three models was 0.6 m in diameter, 8 m in length, had a bending stiffness, EI = 8000 kN-m^2, and was free at the top. The pile models were instrumented with strain gages SG1 to SG4 to measure bending moments along the pile, and LVDT1 to measure the lateral pile head displacement. The soil was instrumented with piezometers (PP1 and PP2) and accelerometers (A4 to A6), as well as with lateral LVDTs mounted on the rings of the flexible wall to measure soil deformations in the free field (LVDT2 to LVDT6). The whole model was slightly inclined to the horizontal to induce lateral spreading when base shaking was applied in flight. A prototype input accelerogram consisting of 40 sinusoidal cycles of a peak acceleration of 0.3 g was applied to the base, which liquefied the whole top layer in a couple of cycles and induced a permanent lateral ground displacement in the free field of about 0.8 m. (Abdoun, 1997; Abdoun and Dobry, 1998; Dobry and Abdoun, 1998).

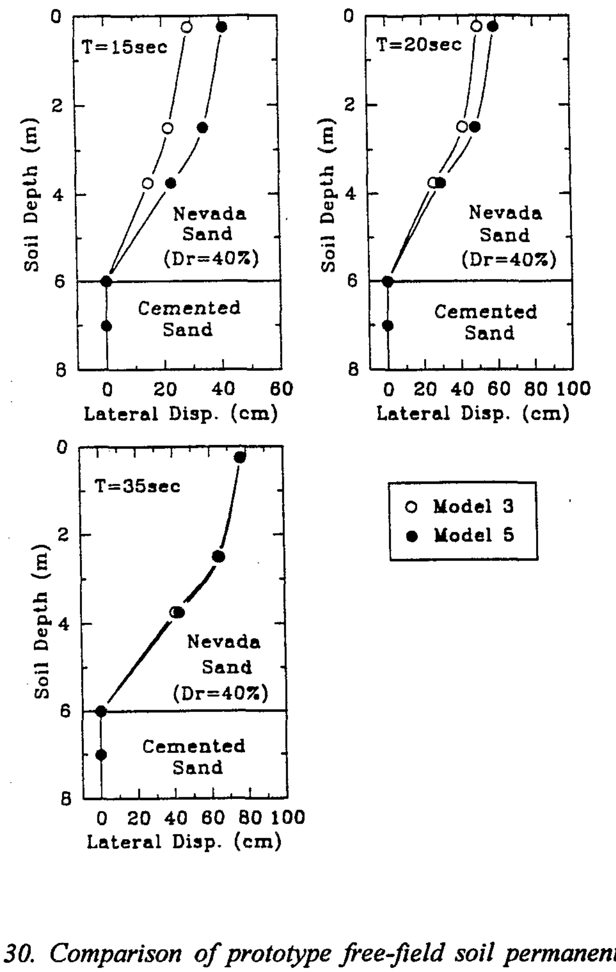

Fig. 30. Comparison of prototype free-field soil permanent displacement profiles in centrifuge Models 3 and 5 of Fig. 29 (Abdoun and Dobry, 1998).

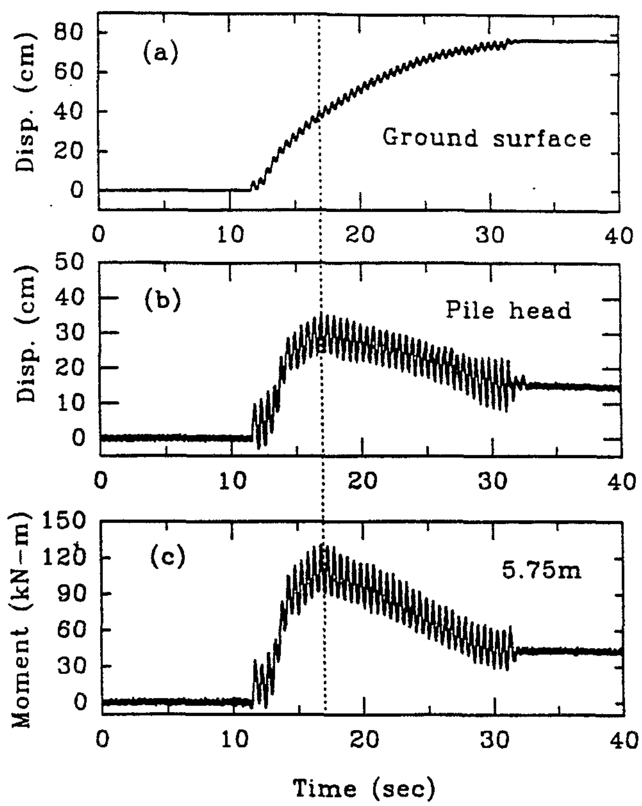

Fig. 31. Prototype lateral displacement of soil and pile at ground surface, and pile bending moment at a depth, z = 5.75 m. in centrifuge Model 3, see Fig. 29 (Abdoun and Dobry, 1998).

Results of the two experiments are shown in Figs. 30-31. As soon as the top sand layer liquefied at the beginning of shaking, it started moving laterally downslope throughout the shaking, with the maximum displacement at all times measured at the ground surface, and with this ground displacement increasing monotonically with time to its final value D_H = 0.8 m at the end of shaking (Figs. 30 and 31a). The maximum bending moment along the pile at any given time occurred at the interface between the two soil layers, that is at a depth of about 6 m (in prototype units). Figure 31c shows the time history of this prototype bending moment for Model 3, measured at z = 5.75 m; the plot reveals that the moment increased to a maximum M_{max} = 110 kN-m at a time, t = 17.5 sec, with the moment decreasing afterwards despite the continuation of shaking and the continuous increase of the soil deformation in the free field. The pile head displacement (Fig. 31b) also reached a maximum at 17.5 sec and decreased afterwards. Clearly at this time the liquefied soil reached its maximum strength and applied a maximum lateral pressure to the pile, with the soil flowing around the pile, exhibiting a smaller strength and applying a smaller pressure afterwards; as a result, the model pile bounced back and the bending moments decreased.

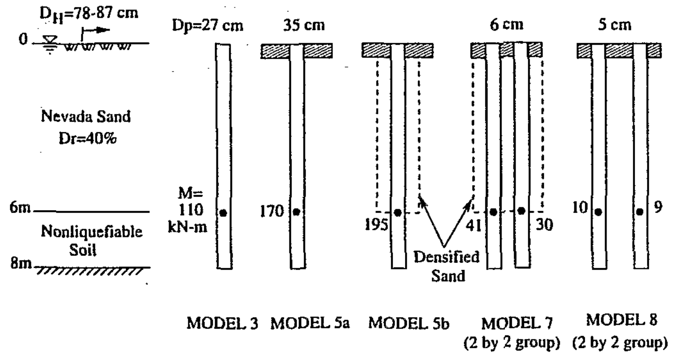

Fig. 32. Prototype maximum moments, $M = M_{max}$ measured at $z \approx 6$ m in several centrifuge pile models in 2-layer soil profile using the setup of Fig. 29 (Abdoun, 1997).

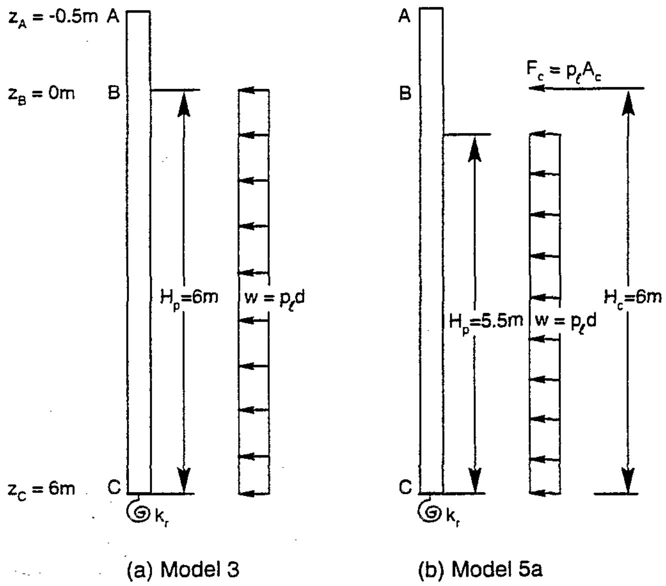

Fig. 33. Prototype free body diagrams for limit equilibrium evaluation of M_{max} and of maximum pile head lateral displacement of centrifuge Models 3 and 5a in Fig. 29

Figure 32 summarizes the values of maximum moments M_{max} measured at z = 6 m in the same three models, as well as in Models 7 and 8 simulating two different 2x2 pile groups composed of the same individual piles, embedded in the same soil profile and subjected to the same base shaking, with D_H = 0.78 to 0.87 m in the free field at the end of shaking in all these experiments. The most interest aspects of Fig. 32 are: (i) the increase of M_{max} in the individual piles when the area exposed to the lateral pressure of the liquefied soil increases, first by adding a pile cap (Model 5a) and then by densifying the sand along the pile to simulate the effect of pile driving (Model 5b); and (ii) the significant decrease of M_{max} in the pile groups as compared to the single piles. This last effect is clearly related to the frame effect of the group, which allows the bending moments to be reduced by the contribution to the total moment of axial forces in the piles. This last effect of bending moment reduction in pile groups is closely linked to the end-bearing character of the pile groups tested, which allowed the corresponding axial forces to be taken by the soil. That is, this bending moment reduction probably would not have taken place if these had been floating pile groups supported by the liquefied soil.

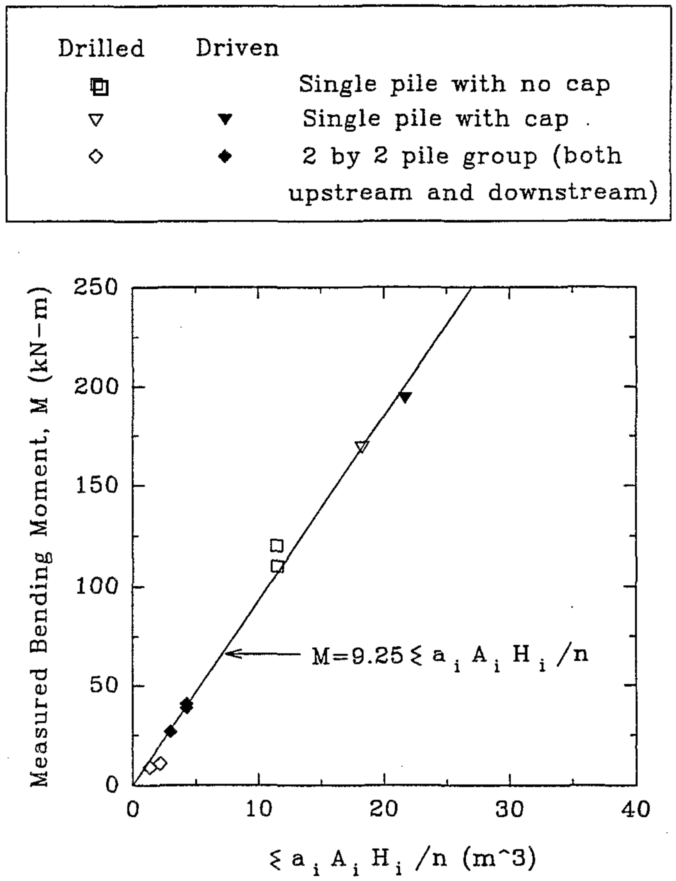

Fig. 34. Comparison between measured centrifuge bending moments, M_{max} (see Fig. 32), and those calculated using limit equilibrium method and p_l = 9.25 kPa (Abdoun, 1997).

The authors have used these centrifuge results in conjunction with analytical models of the piles such as those in Fig. 33 - with a constant assumed maximum pressure of the liquefied soil on the pile, p_l - to calibrate a Limit Equilibrium method for analysis and design of deep foundations. While the sketches in Fig. 33 correspond to single piles, similar calculations have been done for the pile groups. The results are summarized in Fig. 34, which shows that a liquefied soil pressure of the order of 9 to 10 kPa (p_l = 9.25 kPa in the figure) explains very well all measured trends and values of M_{max} for the centrifuge tests of Fig. 32. It is interesting to note that the procedure adopted by the Japan Road Association based on back-calculations from deep foundation response in the Kobe earthquake (Figs. 17-18), provide lateral pressures of the liquefied soil acting on the piles which are of the same order of the value of p_l obtained above from the centrifuge results.

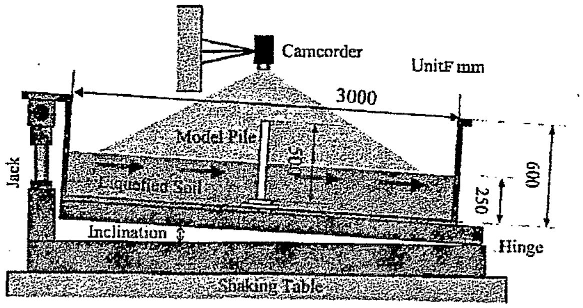

Fig. 35. 1g shaking table model of lateral spreading and lateral pressure on single pile (Hamada, 2000).

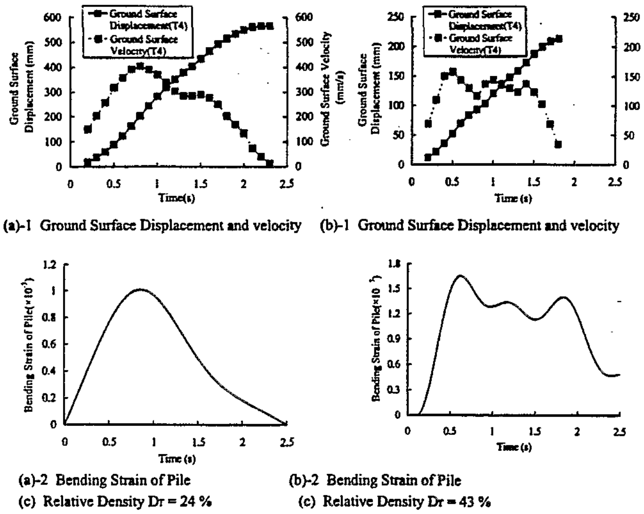

Fig. 36. Results of 1g test of Fig. 35 (Hamada, 2000).

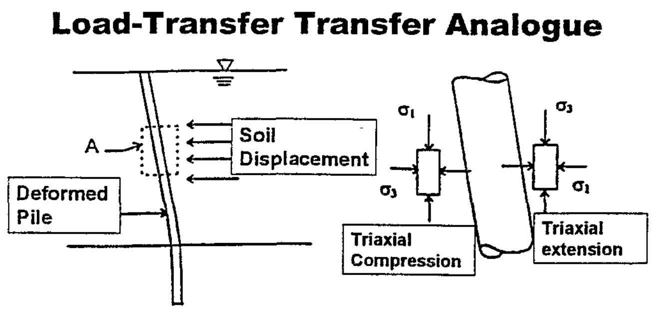

Fig. 37. Conceptual justification for using results of undrained triaxial extension tests to evaluate soil pressure on pile during lateral spreading (Goh, 2001).

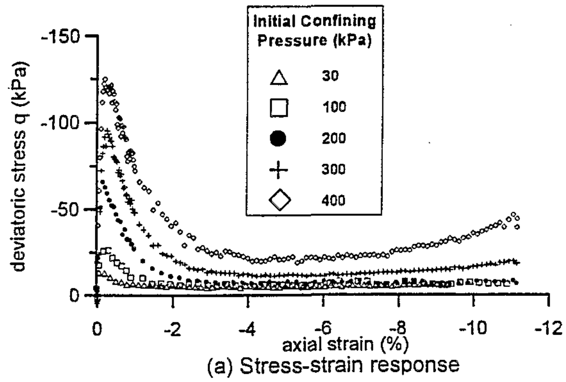

Fig. 38. Selected results from triaxial extension tests on Nevada sand, D_r = 40% (Goh, 2001).

There are still significant differences of opinion between researchers about the physical origin and mechanisms controlling the behavior of liquefied sand including the factors controlling p_l and M_{max}. In one school of thought, the liquefied soil behaves as a viscous fluid, with the value of p_l reflecting the viscous drag of this fluid on the pile, and with the maximum pressure corresponding to the time of maximum relative velocity between soil and pile (Hamada, 2000; see Figs. 35-36). In a second school of thought, the values of p_l and M_{max} are velocity-independent and are controlled by the peak undrained shear strength of the saturated sand loaded in the extension mode (Goh, 2001; see Figs. 37-39). While the authors tend to think that p_l and M_{max} are essentially velocity independent, thus siding with the second hypothesis, clearly the basic phenomenon is complicated and further research is needed.

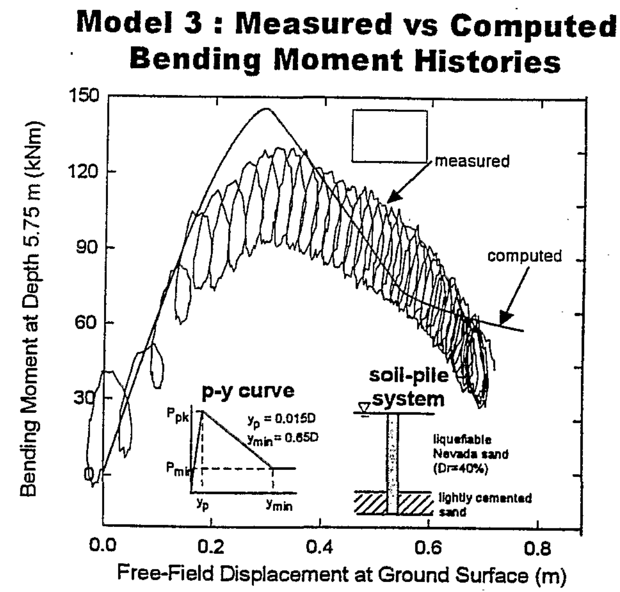

Fig. 39. Comparison between predicted and measured prototype pile bending moment at z = 5.75 m for centrifuge Model 3 (Fig. 29) using triaxial extension results of Fig. 38 (Goh, 2001).

Bending Response Controlled by Nonliquefied Soil. As mentioned in a previous section from examination of actual earthquake case histories, the presence of a strong shallow nonliquefied soil layer riding on top of the liquefied soil both changes the character and increases the bending response of pile foundations to lateral spreading. This was studied by the authors in the centrifuge by using a three-layer soil profile, see Fig. 40 (Abdoun et al., 1996; Abdoun, 1997). Figure 40 shows the basic setup used, which is very similar to that previously discussed except for the addition of a 2-m thick top layer of free draining, slightly cemented sand. That is, the total depth of the soil profile and length of the pile was now 10 m, and correspondingly more instrumentation was added; the rest of the characteristics of the pile, soil, box, base shaking and type of instrumentation were the same as before.

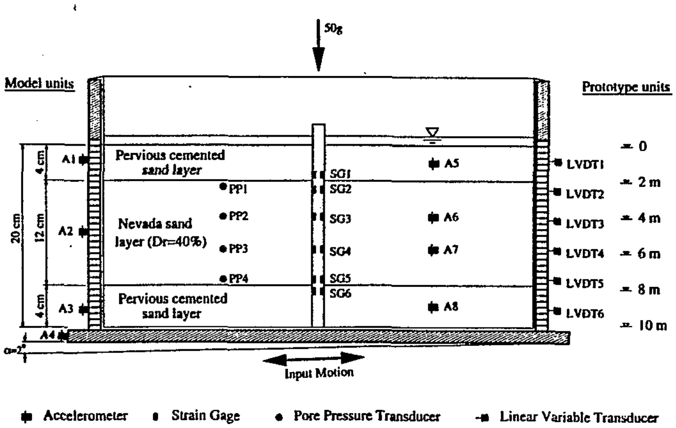

Fig. 40. Lateral spreading centrifuge setup of pile Model 1 in inclined 3-layer soil profile modeling mild infinite slope (Abdoun et al., 1996).

Figures 41-44 summarize the results for centrifuge Model 1 sketched in Fig. 40, which did not have a pile cap. Figure 45 presents the values of maximum bending moments measured in three models: Model 1, Model 2 (with a pile cap added) and Model 4 (short floating pile that did not reach the bottom nonliquefiable layer). The character of the data in Figs. 41-44, which was measured in Model 1, is also representative of Model 2 having a pile cap, except that M_{max} in Model 2 was higher at 2 m depth (Fig. 45). Again, the liquefiable, 6-m thick sand layer between the depths of 2 and 8 m liquefied early in the shaking after which the lateral spreading increased monotonically, reaching a value $D_H = 0.8$ m at the end of shaking (Fig. 41). The pile bending moments in the top 2 m first increased with time of shaking and then decreased after passive failure of the top nonliquefiable layer against the pile (Fig. 43); while the bending moments near the bottom increased monotonically and never decreased, as the bottom nonliquefiable layer did not fail. The shapes of the bending moment profiles at various times are presented in Fig. 44; they indicate that the deformed shape of the pile exhibited a double curvature caused by the top and bottom soil layers loading the pile in opposite directions. This double curvature was confirmed by the fact that when the top soil layer failed, the pile head "snapped" in the downslope direction (Fig. 42), showing that at very shallow depths the pile was pushing the soil rather than the other way around. Both the passive failure of the top layer and the moment concentrations at the top and bottom boundaries of the liquefied layer indicated by the figures are completely consistent with the experience from earthquake case histories as discussed in a previous section of this paper. Furthermore, the moment concentrations at the boundaries of the liquefied layer are predicted by theory and have also been observed in other centrifuge model studies (Sato et al., 1995). Another interesting aspect of Fig. 44 is that the bending moments vary linearly within the liquefied layer, suggesting that they are essentially controlled by the loading of the top and bottom layers, with the pressure of the liquefied soil having a negligible effect. Figure 45 shows that: (i) the values of M_{max} at z = 2 m and z = 8 m are higher than the corresponding values of M_{max} at z = 6 m in Fig. 32, which were controlled by the strength of the weaker liquefied soil; (ii) the value of M_{max} at z = 2 m increases when a pile cap is added due to the increase of foundation area exposed to the passive soil pressure, but the higher value of M_{max} at z = 8 m is unaffected by the presence of the pile cap; and (iii) the value of M_{max} at z = 2 m (125 kN-m) in Model 4, corresponding to the floating pile and thus again controlled by the strength of the liquefied soil, is much less than that of Model 1 (which is controlled by the much stronger top nonliquefiable layer), and in fact is similar to M_{max} of Model 3 in Fig. 32 (110 kN-m).

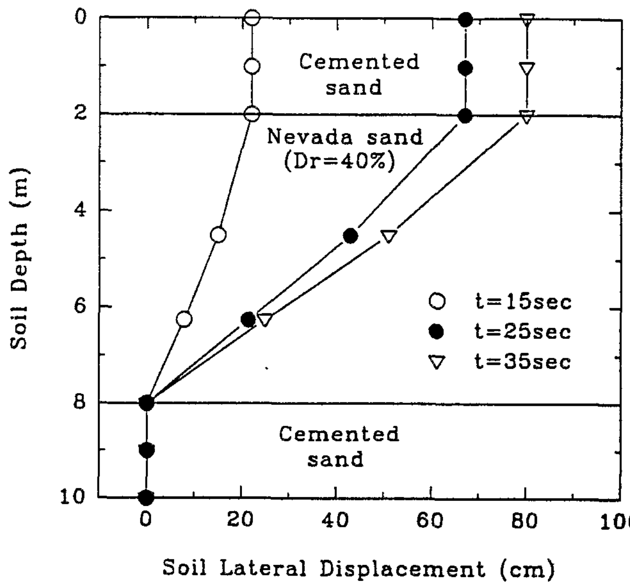

Fig. 41. Prototype free field soil permanent displacement profiles in centrifuge Model 1 of Fig. 40 (Abdoun et al., 1996).

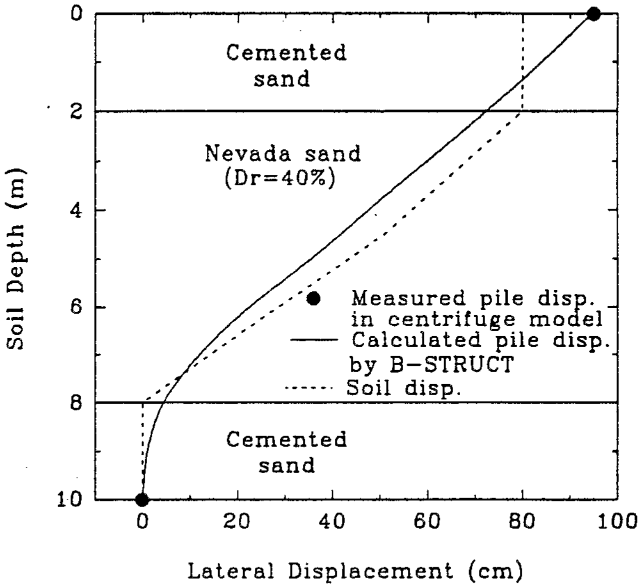

Fig. 42. Measured and calculated prototype pile displacements in centrifuge Model 1 of Fig. 40 (Abdoun et al., 1996).

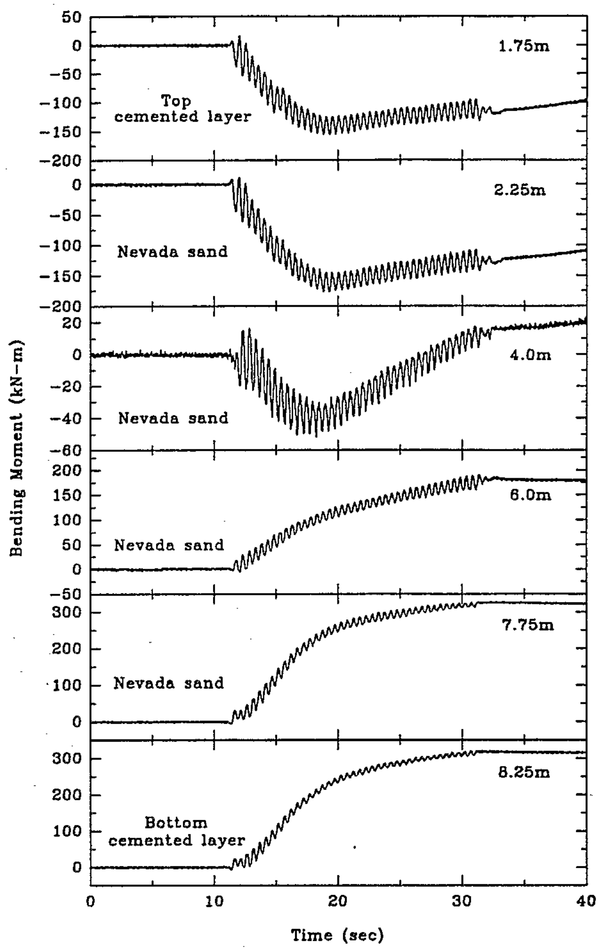

Fig. 43. Measured prototype bending moment time histories along pile centrifuge Model 1 of Fig. 40 (Abdoun et al., 1996).

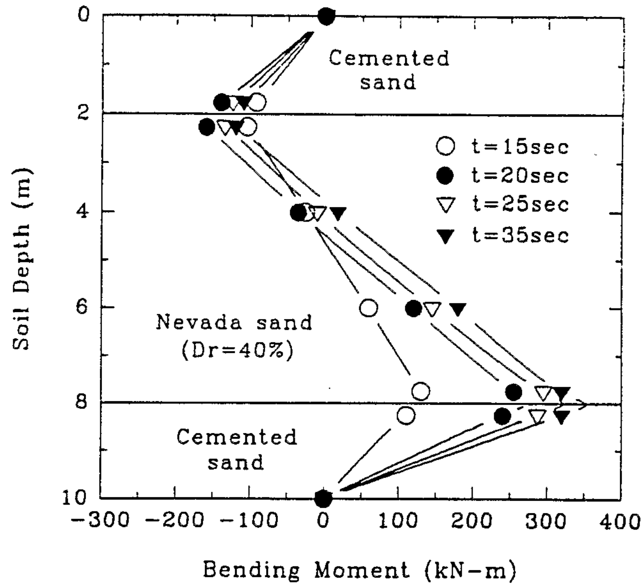

Fig. 44. Measured prototype pile bending moment profiles, centrifuge Model 1 of Fig. 40 (Abdoun et al., 1996).

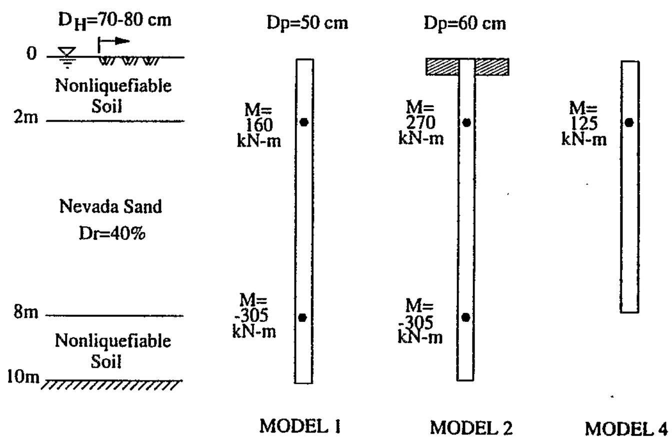

Fig. 45. Prototype maximum bending moments, $M = M_{max}$, measured at z = 2 m and 8 m, in several centrifuge pile models in 3-layer soil profile using setup of Fig. 40 (Abdoun, 1997).

<u>Effect of Superstructure Stiffness</u>. The effect of the superstructure stiffness on bending response to lateral spreading, which both theory and field experience suggests is very significant, was studied by Ramos (1999) at the RPI centrifuge using the setup of Fig. 46. This centrifuge model is essentially identical to Model 3 for the two-layer profile controlled by the strength of the liquefied soil (Fig. 29a), with the only change being the addition of a lateral spring k at the pile head in Fig. 46 to simulate the stiffness of the superstructure. Several centrifuge experiments were conducted with different values of k. The corresponding measured bending moment profiles at the time in which $M = M_{max}$ at z = 6 m are shown in Fig. 47 as data points. Both the value of M_{max} and the maximum pile displacement decreased as the value of k increased; however, increasing negative

moments appeared at shallow depths. The lines in Fig. 47 correspond to the bending moments calculated using Limit Equilibrium and the pile model and liquefied soil pressure distributions of Fig. 48 (Ramos, 1999; Ramos et al., 1999, 2000).

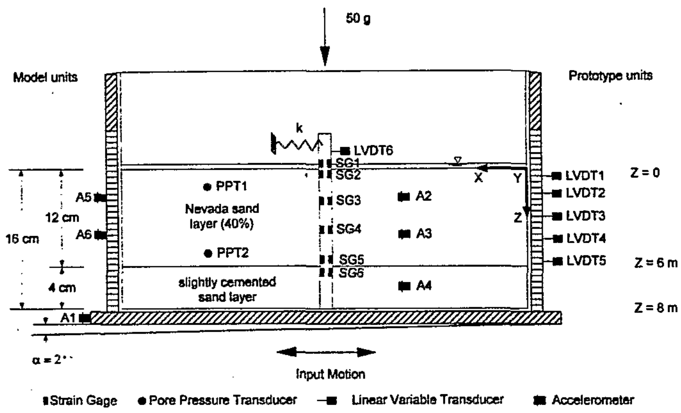

Fig. 46. Centrifuge model tests of single pile subjected to lateral spreading in inclined 2-layer soil profile, with lateral spring, k, above surface to simulate effect of superstructure's stiffness (Ramos et al., 2000).

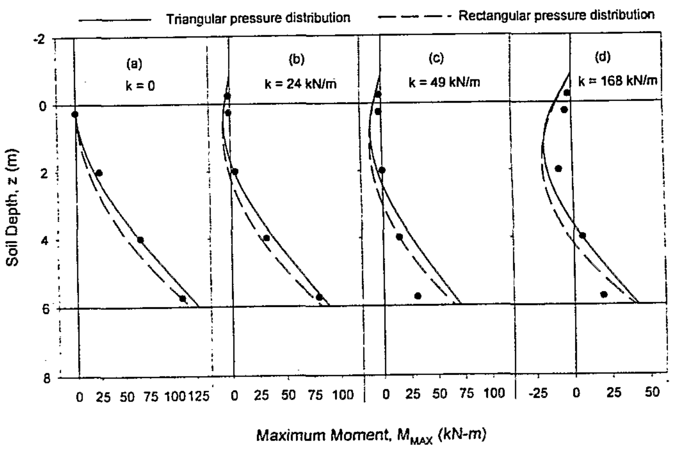

Fig. 47. Measured and computed prototype maximum bending moment distributions for centrifuge tests of Fig. 46 and several values of k (Ramos et al., 1999).

Combined Effects of Lateral Spreading and Superstructural Mass. Recently, Wang (2001) conducted the centrifuge test sketched in Fig. 49, labeled Model 2m, which is essentially a repeat of lateral spreading Model 2 (Figs. 40 and 45), but now with a mass added above ground. This was done in an effort to capture the combined effects on the pile of pseudostatic lateral spreading and dynamic inertial loading, that may be present in the field during shaking (Fig. 15). Figure 50 compares the bending moment profiles measured in Model 2 (no mass) and Model 2m (mass added), at various times during shaking as characterized by the increasing values of surface ground displacement D_H in the free field. The figure indicates that for depths greater than 2 or 3 m, the effect of lateral spreading predominates and the inertial loading due to the mass can be ignored, with the bending moments for the two models being essentially identical for a given value of D_H. However, at shallow depths of less than 2 m, that is in the top nonliquefiable layer, the bending moments of the two models are different, with those of Model 2m changing rapidly with time due to the combined effect of the inertial loading and the lateral spreading. However, even in Model 2m the maximum moments are still concentrated at the upper and lower boundaries of the liquefied layer, except when D_H is very small.

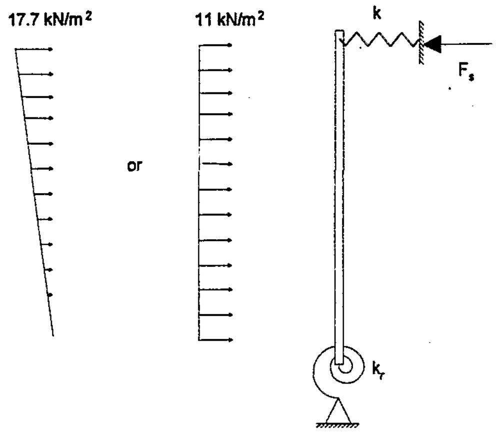

Fig. 48. Limit equilibrium model used to calculate bending moments in Fig. 47 (Ramos et al., 1999).

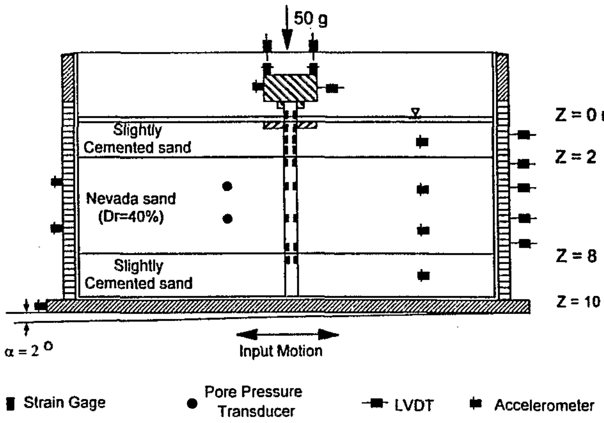

Fig. 49. Centrifuge Model 2m of single pile in slightly inclined 3-layer soil profile, simulating the combined effect of lateral spreading and inertial loading of superstructure (Wang, 2001).

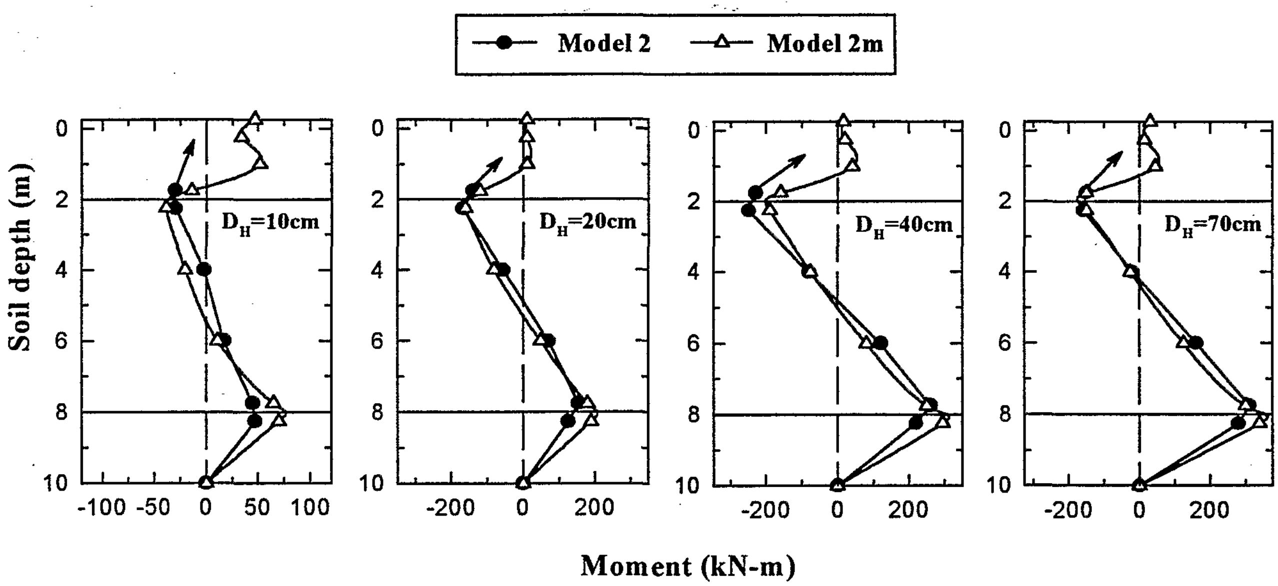

Fig. 50. Measured prototype bending moments in lateral spreading centrifuge Model 2 (no mass above ground), and in Model 2m (mass above ground) (Wang, 2001).

<u>Lessons Learned from Centrifuge Tests</u>. A number of lessons have been learned about the effect of lateral spreading on piles from these and other centrifuge experiments at RPI and their interpretations. These lessons are in good agreement with the field experience summarized in an earlier section herein. The main lessons are:

1. At least in the cases studied in the centrifuge, a free field ground displacement of the order of 1 m is generally more than enough to induce the maximum bending moment for a range of shallow depths, as well as to induce the maximum pile head displacement. By the time such free field displacement is reached, very often either the soil or the pile foundation will have failed. (In the RPI centrifuge experiments the pile models were very strong and hence the soil always failed first, but this is not always necessarily the case in the field, see Fig. 8).
2. The maximum lateral pressures exerted by the fully liquefied soil on the exposed areas of the piles and pile cap are low (of the order of 10-20 kPa). The total lateral forces are thus generally low, except when the exposed area is very large (closely spaced pile group with densified soil in between piles due to pile driving). As a result, the pile displacement and bending moments caused by the liquefied soil pressure tend to be low.
3. The maximum lateral pressures exerted by a nonliquefied layer riding on top of the liquefied soil are greater due to the greater shear strength of the soil. These lateral pressures are associated with passive failure of the soil against the foundation, and thus they depend strongly on the actual shear strength of the nonliquefied layer. The corresponding maximum lateral forces and pile head displacement and bending moments increase with this soil shear strength and with the area of pile and pile cap exposed to the soil pressure. The pile cap is especially critical in increasing these forces and bending response due to its typically large exposed area and shallow location which translates into a large moment arm for the bending moments of the pile below.
4. Therefore, the most critical cases arise in the case of end-bearing piles embedded in a stiff soil under the liquefied layer, and with the shallow part of the piles and the pile cap embedded in a strong, not too thin shallow soil layer or crust above the liquefied soil. In these cases the lateral forces imposed by this shallow crust on pile and pile cap will increase until either the pile foundation or the soil fails.
5. The effect of the shallow crust described above is in principle less critical if the pile is a floating rather than an end-bearing pile, with the tip of the pile supported by the liquefied soil rather than by the nonliquefied soil below, as in this case the bending moments tend to be controlled again by the pressure of the liquefied soil. On the other hand, and as discussed below, in pile groups the frame effect, which decreases both the bending moment and pile head displacement and rotation, depends greatly on the point resistance of the piles which tends to disappear when the piles float in the liquefied soil.
6. End-bearing pile groups tend to have significantly lower bending moments and pile head displacements due to the frame effect where axial pile forces contribute to resist the moments induced by the lateral soil pressures of both liquefied and nonliquefied soil layers. While this bending moment reduction has been verified in the centrifuge only for pile groups in two-layer soil profiles (that is without the shallow nonliquefiable soil layer), the beneficial frame effect should also be valid in decreasing the moments when a strong nonliquefiable top soil layer is present. As already mentioned, this beneficial frame effect of the pile

grouping assumes that the point bearing stiffnesses and capacities of the individual piles are large enough to provide the necessary axial reactions without a large rotation of the group.

7. The lateral and rotational constraints at the pile head level provided by the superstructure above affect profoundly the bending response of pile foundations to lateral spreading. They tend to increase the overall stiffness of the foundation, thus reducing the lateral displacement and rotation of the foundation, and the bending moments at depth also decrease; however, large negative bending moments may appear near the pile head.
8. Analyses using either a limit equilibrium approach or nonlinear soil springs (p-y curves), appropriately calibrated with centrifuge results and case histories can be very helpful. They provide deeper insights into the pile-soil interaction during lateral spreading and can also be used in engineering applications. Limit equilibrium methods are especially useful to evaluate maximum bending moments and other maximum response parameters of the foundation.

CONCLUSIONS

The discussion of case histories during earthquakes showed clearly the significance of liquefaction – and especially lateral spreading – in causing damage to deep foundations during earthquakes. The complexity of the problem – arising both from the complexity of the soil liquefaction/lateral spreading phenomenon itself and from the complexities of the resulting soil-pile-structure interaction, makes it necessary the use of centrifuge physical modeling to clarify mechanisms, quantify relations and calibrate analysis and design procedures. As a result, an increasing body of research has recently focused on centrifuge modeling of the effects of liquefaction and lateral spreading on pile foundations, as shown by the references listed in Table 3. The results so far have clarified important aspects of the deep foundation response, have shown significant agreement between centrifuge results and field experience, and are being used to calibrate limit equilibrium and p-y analysis methods. Much more work is needed along these lines, combining centrifuge model experiments, case histories and theory, to improve our understanding as well as the practice of seismic design and retrofitting of deep foundations against liquefaction.

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