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# Liquefaction and Deformation of Soils and Foundations Under Seismic Conditions

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# STATE OF THE ART (SOA2)
# Liquefaction and Deformation of Soils and Foundations Under Seismic Conditions

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SYNOPSIS A summary of significant developments in seismic liquefaction research and applications is presented for the period 1985–1995. It is concluded that rapid progress is being made, especially in evaluating ground deformation and straining and their effects on constructed facilities. Four topics illustrating these developments are selected and discussed in more detail.

## INTRODUCTION

Liquefaction of loose, saturated granular soil during earthquakes has been and continues to be a major cause of destruction of constructed facilities. This is shown by simply remembering some seismic events of the last decade where liquefaction-related damage was paramount: Chile, 1985; Loma Prieta, California, 1989; Philippines, 1990; Costa Rica, 1991; and of course, Kobe, Japan, 1995. At the time of writing of this paper, damage estimates for this last earthquake ranged between thirty billion and more than one hundred billion US dollars, of which a significant fraction was related to liquefaction effects on the port and other facilities.

While some main aspects of liquefaction are now well understood and useful engineering tools are available for their evaluation, others remain either mysterious and controversial or are understood only at a qualitative level. Despite several decades of work on the subject, liquefaction continues to be the focus of extensive research in several countries. In fact, the pace of the effort has even accelerated in the last decade, with half a dozen centers and government organizations in Canada, Japan and the US supporting comprehensive and systematic liquefaction-related efforts.

In addition to the practical importance of the problem, there are some clear reasons for this continued interest. The first is that ground liquefaction, with or without the presence of structures, is a very complex phenomenon. In fact, it may be advantageous to visualize it as a *family* of phenomena, with this family having as common denominator a significant buildup of excess pore water pressures due to the earthquake excitation. These excess pore pressures constitute a *necessary but not sufficient* condition for liquefaction and related damage to occur; also, the level of excess pore pressure needed to trigger liquefaction may be different in a slope than in level ground. But even when this level of pore pressure is reached, the appearance of significant engineering consequences depends on a number of other factors whose combined effect is still poorly understood, such as soil density, layer thickness, permeability, layering, soil-structure interaction aspects, etc. A second reason for the continued interest in liquefaction is that for many years the research focused on pore pressure buildup and liquefaction triggering, with this focus switching only recently to ground deformations and liquefaction effects on constructed facilities. And finally, a third reason is the increasing importance of retrofitting and ground remediation of existing facilities (as compared to the more traditional seismic design of new structures), for which conservative assumptions can be very costly and thus require more precise scenarios and predictions of engineering effects.

This expanded interest in liquefaction and its effects is reflected in the number of State-of-the-Art (SOA) and Special Presentation papers in these proceedings that deal with the subject. In addition to this article, they include: Finn et al. (1995); Youd (1995); Kutter (1995); O'Rourke and Pease (1995); Robertson et al. (1995); and Arulanandan et al. (1995). Therefore, the author decided that it was not necessary—or possible—to write a SOA paper covering all aspects of what has become a vast and expanding field. For further information, the reader is directed to the references listed above and in Table 1 (Item 1), as well as to the SOA papers presented in the two previous conferences by Finn (1981, 1991).

Two things are done in the remainder of this paper. Following the tradition established by Prof. Finn in the previous conferences, important recent developments are first identified and the corresponding references are provided for the last decade. Then, four topics of special interest to the author are selected and discussed in more detail.

## RECENT DEVELOPMENTS

Table 1 summarizes what the author considers to be the fifteen most important developments of the last decade, including a list of selected publications attached to each item. The table starts, under Item 1, with the seminal publication developed at the 1985 workshop on liquefaction sponsored by the US National Research Council, and organized by Prof. Whitman (NRC, 1985), which provides a natural initiation to the period covered by the table. Like any attempt of this type, the exact organization of the table and the selection and wording of the fifteen items are quite subjective, and different people could certainly arrive at different versions of Table 1. In particular, the table does not imply any order of importance (Item 1 is not necessarily more important than Item 15!). Also, there is considerable (and probably unavoidable) overlap between different items (e.g., compare Items 3, 4 and 10).

Still, the table provides useful information and a general perspective. While a few of the issues listed are covered in more detail in the rest of this paper, an inspection of the table suggests the following thoughts about current trends in liquefaction research and applications:

- In the last decade, research incorporating case histories, field measurements, and more generally in situ work, has become extremely important, especially when compared with previous decades, which were dominated by laboratory research.
- Ground deformation evaluation in free field studies, and engineering effects of liquefaction on structures and lifelines, are receiving increasing attention, as compared to the past emphasis on pore water pressure buildup. There has been a quantum jump in the last decade in our understanding of these issues, which continues to develop at a fast pace.
- There is a rapid emergence of centrifuge model testing as a main, cost-effective tool to clarify the mechanics of liquefaction phenomena and provide quantitative evaluations of ground deformation and engineering effects on different systems. There is also a parallel development of sophisticated numerical techniques, mostly still in the research stage, which offer great promise of becoming extremely useful engineering tools in the near future.
- International cooperation, organized team efforts both within and between countries, and the leadership and support of national centers and government organizations have been critical to the success of a number of the developments listed in Table 1.

Expanded discussions of four topics included in Table 1 are presented in the following sections. These topics range from silty sand behavior to the effects of permanent ground straining on foundations and structures, and they illustrate useful new developments for the evaluation of liquefaction-induced ground deformation and associated engineering damage.

## THE RATIO $S_r / \sigma'_{v0}$ IN SILTY SANDS AND THE WATER-SEDIMENTATION TECHNIQUE

A key question when evaluating the potential for post-liquefaction large ground deformation and flow sliding is the determination of the shear strength characteristics of the liquefied soil (Finn, 1991). Over the years, limiting equilibrium analyses have been developed which assume the existence of well defined failure block(s), both for post-shaking static evaluations of flow sliding (Fig. 1; Castro et al, 1982; Seed, 1987), and for dynamic evaluation of lateral spreading during shaking (Fig. 2; Castro, 1987; Dobry and Baziar, 1992). These limiting equilibrium analyses assume that the liquefied soil has a well defined shear strength which is constant over a wide range of shear strains; this strength has been variously identified with the undrained steady-state shear strength, $S_{us}$, obtained in the laboratory (Castro et al., 1982), and with the residual shear strength, $S_r$, backfigured from case histories (Seed, 1987). A number of authors have backfigured the average residual shear strength, $S_r$, of liquefied loose sands, silts, and gravels from case histories of lateral spreading and flow failure, and have correlated $S_r$ to in situ penetration resistance. Relevant references are listed in Table 1, Item 10. For liquefied silty sands and sandy silts, these case histories show a consistent increase of $S_r$ with average vertical effective confining stress, $\sigma'_{v0}$, as illustrated by Fig. 3(b). (Relevant references which have pointed out this influence of $\sigma'_{v0}$ or of depth on $S_r$ are listed in Table 1, Item 12.) As shown by Fig. 3(b), the ratio $S_r / \sigma'_{v0}$ obtained from the case histories ranges from about 0.04 to 0.2. Laboratory tests using the water-sedimented technique developed by Vásquez-Herrera et al. (1990) and Baziar and Dobry (1995) for silty sands have helped explain this increase of $S_r$ with $\sigma'_{v0}$ in terms of the high compressibility of the soil.

Many loose saturated silty sand deposits have been sedimented in water and contain sequences of finely divided thin layers composed of soils of different gradations. This microlayering is found in natural sediments and hydraulic fills and has also been reported in clean sands (Ishihara, 1990; Baziar and Dobry, 1995). The slower fall velocities in water of the finer grains, which cause the coarser soil to sediment first followed by the finer sand or silt, is a main reason explaining this type of fabric. For example, the hydraulic fill of the Lower San Fernando Dam (LSFD), which experienced a flow slide during an earthquake in 1971 (see Fig. 1), was found to be intensely stratified by microlayers from about 0.05 to 0.20 inches thick (Castro et al., 1989).

**Table I. Recent Developments in Liquefaction Research and Applications: 1985–1995**

| Item | Explanation | Selected References |
|---|---|---|
| 1. SOA Reviews | Publication of several comprehensive state-of-the-art documents | NRC (1985)<br>Castro (1987)<br>Seed and Harder (1990)<br>Finn (1991)<br>Marcuson et al. (1992)<br>Ishihara (1993)<br>Finn et al. (1995)<br>Youd (1995) |
| 2. Lower San Fernando Dam, California | Re-visiting of 1971 flow slide in upstream slope of Lower San Fernando Dam due to San Fernando earthquake. New field and laboratory studies and re-evaluation of slide. | Marcuson et al. (1990)<br>Vásquez-Herrera et al. (1990)<br>Castro et al. (1992, 1993)<br>Gu et al. (1993)<br>Baziar and Dobry (1995) |
| 3. Hamada's Air Photo Technique | Development of Hamada's air photo surveying technique to study post-liquefaction ground deformation values and spatial patterns. Application to several Japanese and U.S. earthquakes. | Hamada et al. (1986)<br>Hamada and O'Rourke (1992)<br>O'Rourke and Hamada (1992) |
| 4. Cooperative Research | Extensive U.S.-Japan cooperative research, joint workshops, and publication of volumes documenting case histories of liquefaction-induced ground deformation and effects on lifelines for 10 earthquakes in Japan, the Philippines and the U.S. | Hamada and O'Rourke (1988, 1992, 1992a)<br>O'Rourke and Hamada (1989, 1991, 1992) |
| 5. In Situ Recording of 100% Pore Pressure in Earthquakes | First in situ recording of initial liquefaction (pore pressure ratio, $r_u \approx 1.0$) at Wildlife site in California, during M = 6.6 Superstition Hills earthquake, and in-depth studies and discussions of pore pressure and acceleration records. Development of Elgamal-Zeghal system identification technique to extract and image average stress-strain response of soil from records. | Holzer et al. (1989)<br>Dobry et al. (1989)<br>Hushmand et al. (1991, 1992)<br>Zeghal and Elgamal (1994)<br>Youd and Holzer (1994)<br>Zorapapel and Vucetic (1994) |

Laboratory evaluations of the in situ undrained steady-state shear strength, $S_{us}$, of such soils is very difficult. Castro et al. (1982) and Poulos et al. (1985) note that even high quality "undisturbed" samples are subjected to inevitable densification, which often transforms an originally contractive sand to a dilative state when reconsolidated to the in situ pressure. They reconsolidate the undisturbed triaxial sample to a much higher pressure than that in situ so that the soil behaves contractively again, and then correct the measured $S_{us}$ back to the in situ void ratio (Castro-Poulos-France method). The procedure recognizes the importance of preserving the original microlayered fabric of the soil, as compared with the option of ignoring the effect of microlayering by testing remolded homogeneous specimens having the in situ void ratio. The Castro-Poulos-France method was applied by Castro et al. (1992) to the re-evaluation of the 1971 flow slide of the LSFD, which is further discussed in the next section.

**Table I con. Recent Developments in Liquefaction Research and Applications: 1985–1995**

| Item | Explanation | Selected References |
|---|---|---|
| 6. 1g and Centrifuge Model Tests and VELACS | Extensive use of base shaking of instrumented soil and soil-structure models, including especially centrifuge models shaken in flight. Rapid emergence of centrifuge modeling as main tool to evaluate mechanics and effects of liquefaction and to calibrate numerical codes. Cooperative research project VELACS (VErification of Liquefaction Analysis by Centrifuge Studies) involving eight universities in the USA and United Kingdom. | Schofield and Steedman (1988)<br>Ishihara and Takeuchi (1991)<br>Sasaki et al. (1991)<br>Steedman (1991)<br>Dobry and Liu (1992)<br>Arulanandan and Scott (1993, 1994)<br>Dobry and Taboada (1994)<br>Kimura et al. (1994)<br>Ko (1994)<br>Whitman and Ting (1994)<br>Zeng (1994)<br>Arulanandan et al. (1995)<br>Elgamal et al. (1995)<br>Kutter (1995) |
| 7. Canadian Liquefaction Experiment | Cooperative Canadian liquefaction experiment (CANLEX) involving industry, consultants and universities. It includes high quality in situ sampling at two sites, field and laboratory testing, inducing of liquefaction event at one of the sites, and analytical studies. | Robertson (1993)<br>Robertson et al. (1995) |

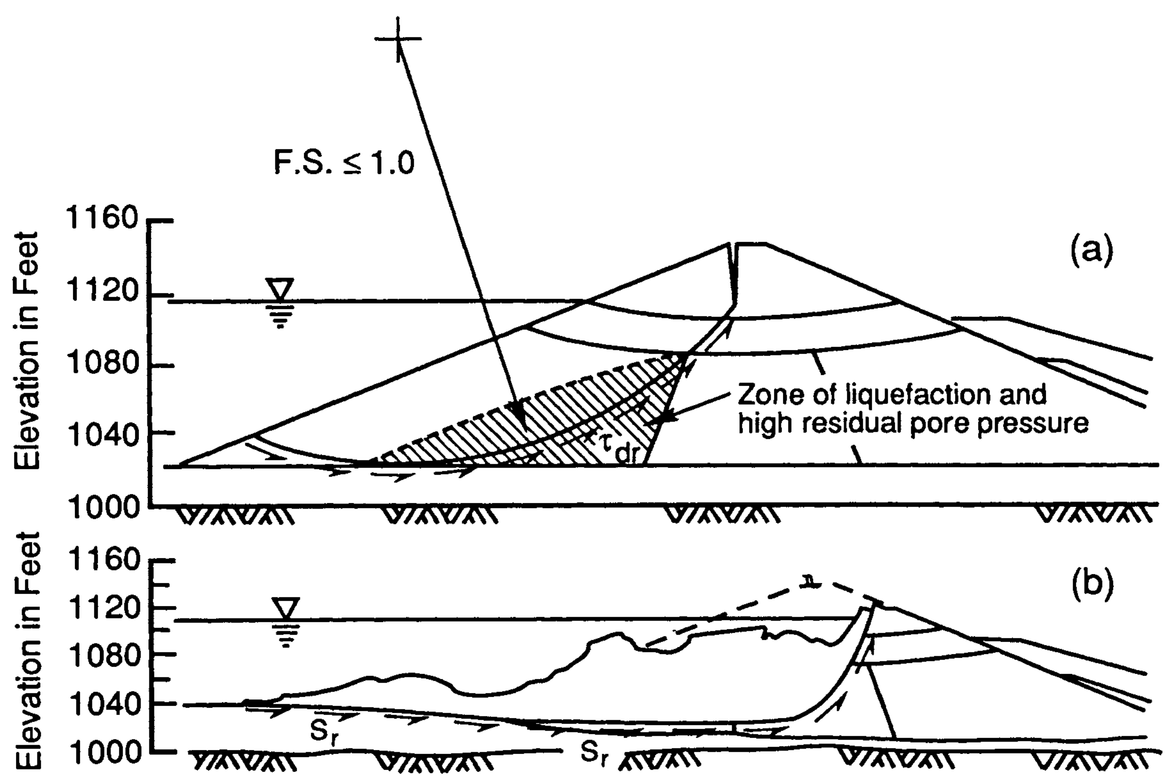


Figure 1. Flow slide of upstream shell of the Lower San Fernando Dam caused by the 1971 San Fernando Earthquake: (a) initial configuration, and (b) final configuration (Baziar and Dobry, 1995, modified after Castro et al., 1992).

**Table I con. Recent Developments in Liquefaction Research and Applications: 1985–1995**

| Item | Explanation | Selected References |
|---|---|---|
| 8. Flow Failure Triggering | Development of accumulated strain and pore pressure criteria for triggering of flow failure in embankments and slopes. Recognition that triggering occurs at $r_u < 1.0$ in the presence of a static driving shear stress. | Dobry et al. (1985)<br>Sladen et al. (1985)<br>Vaid and Chern (1985)<br>Castro et al. (1989, 1993)<br>Seed and Harder (1990)<br>Vásquez-Herrera et al. (1990)<br>Finn (1991) |
| 9. Post-liquefaction Settlement | Development of criteria and simplified engineering procedures for evaluating post-liquefaction ground surface settlement due to compaction. | Tokimatsu and Seed (1987)<br>Nagase and Ishihara (1988)<br>Ishihara and Yoshimine (1991)<br>O'Rourke et al. (1992a)<br>Ishihara (1993) |
| 10. Case Histories of Lateral Spreading and Flow Failure and Their Use | Documentation and publication of an increasing number of case histories of flow failure and lateral spreading, including in situ penetration resistance of soil, and use of these case histories to develop: (i) empirical correlations for evaluating lateral ground deformation, (ii) screening procedures to determine if soil in situ can develop large deformation, (iii) corre-lations to evaluate residual shear strength of soil for limiting equilibrium calculations of flow failure and lateral spreading, and (iv) use of the sliding block analysis technique for evaluation of horizontal ground deformation in lateral spreads. | De Alba et al. (1987)<br>Castro (1987)<br>Seed (1987)<br>Youd and Perkins (1987)<br>Davis et al. (1988)<br>Sladen and Hewitt (1989)<br>Seed and Harder (1990)<br>Baziar and Dobry (1991a, 1995)<br>Dobry and Baziar (1992)<br>Robertson et al. (1992)<br>Stark and Mesri (1992)<br>Bartlett and Youd (1992, 1995)<br>O'Rourke and Hamada (1992)<br>Hamada and O'Rourke (1992)<br>Ishihara (1993)<br>O'Rourke and Pease (1995)<br>Youd (1995) |
| 11. New Laboratory-Based Developments | A number of laboratory-based findings and concepts have been proposed as a basis for practical evaluations involving the undrained response of liquefied soil. They include, among others: (i) collapse surface, state parameter and similar concepts providing unified pictures of the responses of different sands, (ii) quasi-steady state strength for soils exhibiting contractive behavior at intermediate shear strains and dilative behavior at larger strains, and (iii) use of water-sedimentation technique simulating the field depositional process for the testing of silty sands and sandy silts. | Been and Jefferies (1985)<br>Sladen et al. (1985)<br>Been et al. (1986, 1991)<br>Mohamad and Dobry (1986)<br>Alarcon-Guzman et al. (1988)<br>Kuerbis and Vaid (1989)<br>Vaid et al. (1989)<br>Vásquez-Herrera and Dobry (1989)<br>Verdugo (1992)<br>Ishihara (1993)<br>Baziar and Dobry (1995) |

**Table I con. Recent Developments in Liquefaction Research and Applications: 1985–1995**

| Item | Explanation | Selected References |
|---|---|---|
| 12. Residual Strength of Silty Sands and Confining Pressure | Recognition that the residual strength of many loose silty sands, sandy silts, and tailings, increases with confining pressure, and development of $s_r / \sigma'_{v0}$ parameter for engineering applications. | Castro and Troncoso (1989)<br>Ishihara et al. (1990)<br>Wightam and Jefferies (1991)<br>Baziar and Dobry (1991a)<br>Lo et al. (1991)<br>McLeod et al. (1991)<br>Stark and Mesri (1992)<br>Ishihara (1993)<br>Baziar and Dobry (1995) |
| 13. Mechanics of Flow Failure and Lateral Spreading | Vigorous discussions on detailed mechanics of large ground deformation, and especially of lateral spreads, and extensive research to clarify issues still pending, such as: (i ) purely undrained character of flow failure versus partial drainage, void ratio redistribution, pore pressure migration/soil cracking, and water interlayer phenomena, (ii) role of water interlayer under cohesive stratum in lateral spreads, (iii) role of distributed shear strains within liquefied layer in causing lateral ground surface deformation in lateral spreads, versus strain concentration(s) at failure surface(s) and validity of sliding block analyses, (iv) relative importance of gravity and inertia forces in lateral spreads, importance of delayed deformations occurring after the end of shaking and possible mechanism(s) of these delayed deformations; and (v) nature and properties of liquefied soil, and especially role of dilative behavior in limiting soil deformation. | NRC (1985)<br>Castro (1987)<br>Seed (1987)<br>Dobry (1989)<br>Towhata (1991, 1993)<br>Yasuda et al. (1991)<br>Sasaki et al. (1991)<br>Ishihara and Takeuchi (1991)<br>Dobry and Baziar (1992)<br>Dobry and Liu (1992)<br>Arulanandan and Zeng (1994)<br>O'Rourke (1994)<br>Fiegel and Kutter (1994)<br>Zeghal and Elgamal (1994) |
| 14. Effects of Large Ground Deformation on Constructed Facilities and Soil-Structure Interaction Aspects | Studies of spatial patterns of large ground deformation and ground strains, as well as their effects on buried lifelines, foundations and other constructed facilities using case histories, analyses and model tests. | Hamada and O'Rourke (1988, 1992, 1992a)<br>O'Rourke and Hamada (1989, 1991, 1992)<br>Youd (1989)<br>Susuki and Masuda (1991)<br>Meyersohn et al. (1992)<br>Dobry (1994)<br>O'Rourke and Pease (1995)<br>O'Rourke et al. (1995) |

**Table I con. Recent Developments in Liquefaction Research and Applications: 1985–1995**

| Item | Explanation | Selected References |
|---|---|---|
| 15. Numerical Techniques to Evaluate Liquefaction and Large Ground Deformation | Rapid development of numerical techniques aimed at providing realistic modeling of liquefaction, large ground deformations and nonlinear soil-structure interaction. Increased use of partially and fully coupled effective stress computational methods; verification and calibration by case histories and laboratory/centrifuge test results. | Finn et al. (1986)<br>Bardet (1987)<br>Chan (1988)<br>Yashima et al. (1988)<br>Finn and Yogendrakumar (1989)<br>Zienkiewicz et al. (1990, 1994)<br>Muraleetharan et al. (1991, 1994)<br>Iai et al. (1992)<br>Li et al. (1992)<br>Towhata et al. (1992)<br>Arulanandan and Scott (1993, 1994)<br>Ishihara et al (1993)<br>Lacy (1993)<br>Popescu and Prevost (1993)<br>Bouckovalas et al. (1994)<br>Dafalias (1994)<br>Darve (1994)<br>Smith (1994) |

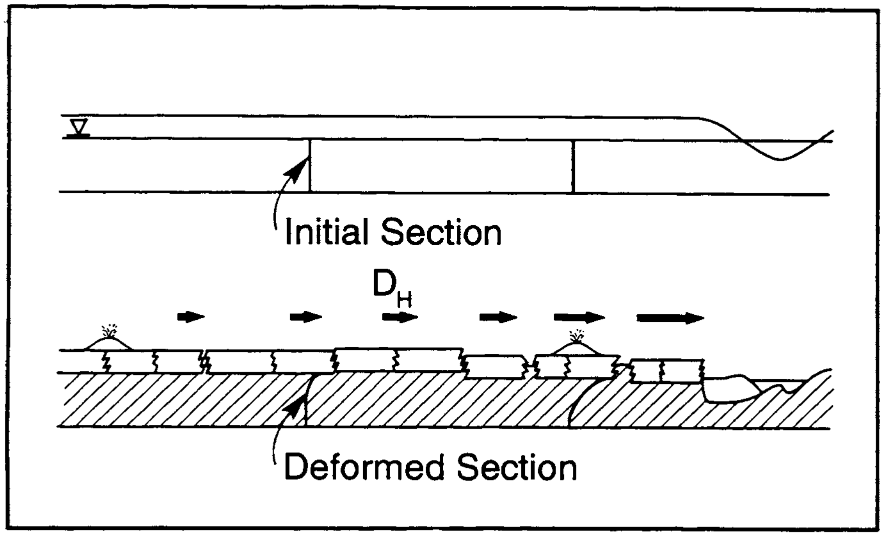


Figure 2. Sketch of lateral spread before and after ground deformation; liquefaction occurs in the cross-hatched zone (Youd, 1984). $D_H$ is the horizontal deformation of the ground surface.

The alternative water-sedimentation approach was proposed by Vásquez-Herrera et al. (1990) as part of the same LSFD study. Remolded layered (microlayered) triaxial soil specimens are formed by pluviating equal weights of the sand-silt mixture sampled from the site into the triaxial preparation mold previously filled with water and then waiting enough time for full sedimentation to occur before pouring the next layer. Figure 4 sketches a typical segregated layered triaxial specimen formed this way, which attempts to simulate the in situ fabric by, in effect, mimicking the sedimentation history of the deposit. This method provides void ratios and values of $S_{us}$ similar to those in the field for very loose, natural or artificial silty sands and sandy silts. In the rest of this section and in the next section, results obtained on remolded layered LSFD silty sand are discussed and compared with other relevant in situ and laboratory data summarized by Castro et al. (1992), as well as with the average residual strength $S_r$ exhibited by the LSFD in the 1971 slide.

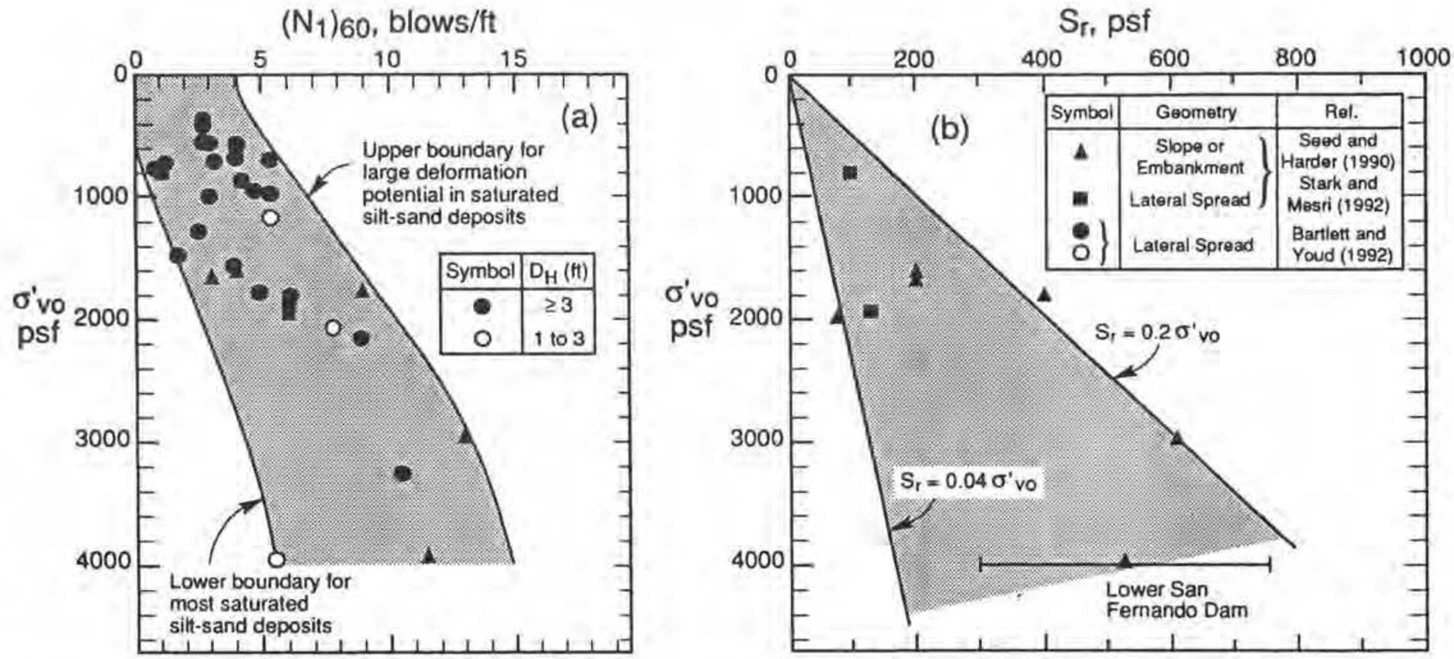


Figure 3. Charts relating: (a) normalized Standard Penetration Resistance, $(N_1)_{60}$, and (b) residual shear strength, $S_r$, to vertical effective overburden pressure, $\sigma'_{v0}$, for low plasticity, saturated nongravelly silt-sand deposits with fines contents greater than 10%, that have experienced large deformations. All data points correspond to case histories obtained from Stark and Mesri (1992) and Bartlett and Youd (1992) (modified from Baziar and Dobry. 1995).

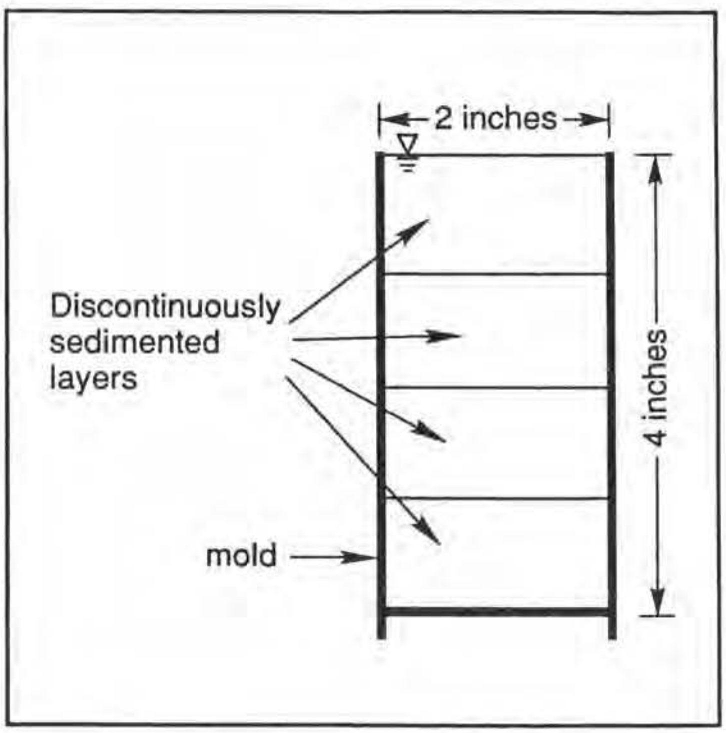


Figure 4. Remolded layered triaxial specimen of silty sand prepared by water sedimentation. Technique proposed by Vásquez-Herrera and Dobry (1989) to simulate observed microlayering of hydraulic fill in Lower San Fernando Dam.

The results of fourteen tests performed by Vásquez-Herrera et al. (1990) and Baziar and Dobry (1995) are summarized here in Figs. 5-7. In most of the experiments, the specimen was composed of four 1-inch layers (Fig. 4). In all cases, the layered triaxial specimen was first consolidated under effective vertical $(\sigma'_{1c})$ and horizontal $(\sigma'_{3c})$ stresses, either isotropically $(K_c = \sigma'_{1c} / \sigma'_{3c} = 1)$ or anisotropically $(K_c > 1)$. Then undrained monotonic triaxial or cyclic torsional loading was applied to failure. The observed steady-state strength response was the same in both monotonic and cyclic tests. The deposition method produced a very loose soil with a void ratio, e, after consolidation ranging from 0.66 to more than 0.8. All specimens exhibited contractive behavior and experienced flow failure at large shear strains, even under consolidation pressures as low as 0.2 tsf, representing a depth of soil of only a few feet in the field. Figure 5 displays the stress-strain curve from one of the monotonic tests and illustrates the determination of the steady-state shear strength.

These tests are plotted together in Figs. 6 and 7, irrespective of their being monotonic or cyclic. Figure 6 shows that:

(1) the soil is very compressible, with the void ratio e decreasing and $S_{us}$ increasing rapidly as the vertical pressure $\sigma'_{1c}$ increases;

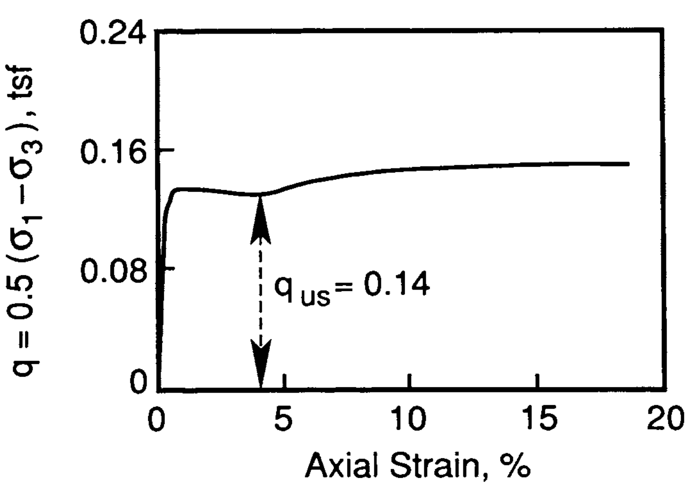


Figure 5. Typical stress-strain curve from monotonic undrained triaxial test on isotropically consolidated, remolded layered specimen of silty sand prepared using water sedimentation. Batch Mix 7, Lower San Fernando Dam ($\sigma'_{1c} = \sigma'_{3c} = 0.9$ tsf, e = 0.76) (Baziar and Dobry, 1995).

(2) the relation between e and $\sigma'_{1c}$ is unique and independent of $K_c$;
(3) for a given $K_c$ the ratio $S_{us} / \sigma'_{1c}$ is nearly constant; and
(4) this ratio $S_{us} / \sigma'_{1c}$ increases as $K_c$ increases.

Conclusions (1) and (3) are reminiscent of the undrained static strength behavior of normally consolidated clays and of the use of similar "c/p ratios" for static loading evaluations in clays (e.g., see Ladd, 1991). The use of an $S_{us} / \sigma'_{1c}$ ratio was first proposed by Castro and Troncoso (1989) for tailings dams, and the range of $S_{us} / \sigma'_{1c} \approx 0.12$ to 0.19 in Fig. 6 is generally consistent with laboratory results presented by Castro and Troncoso (1989), Castro (1991) and Ishihara (1993). Both relations of e and $S_{us}$ with $\sigma'_{1c}$ in Figs. 6(a) and (b) are very useful, as $\sigma'_{1c}$ can be readily interpreted in field studies as the vertical effective overburden pressure, $\sigma'_{v0}$. Furthermore, this ratio $S_{us} / \sigma'_{1c} \approx 0.12$ to 0.19 is included within, and covers most of the range $S_r / \sigma'_{v0} \approx 0.04$ to 0.2 from case histories already discussed and plotted in Fig. 3(b).

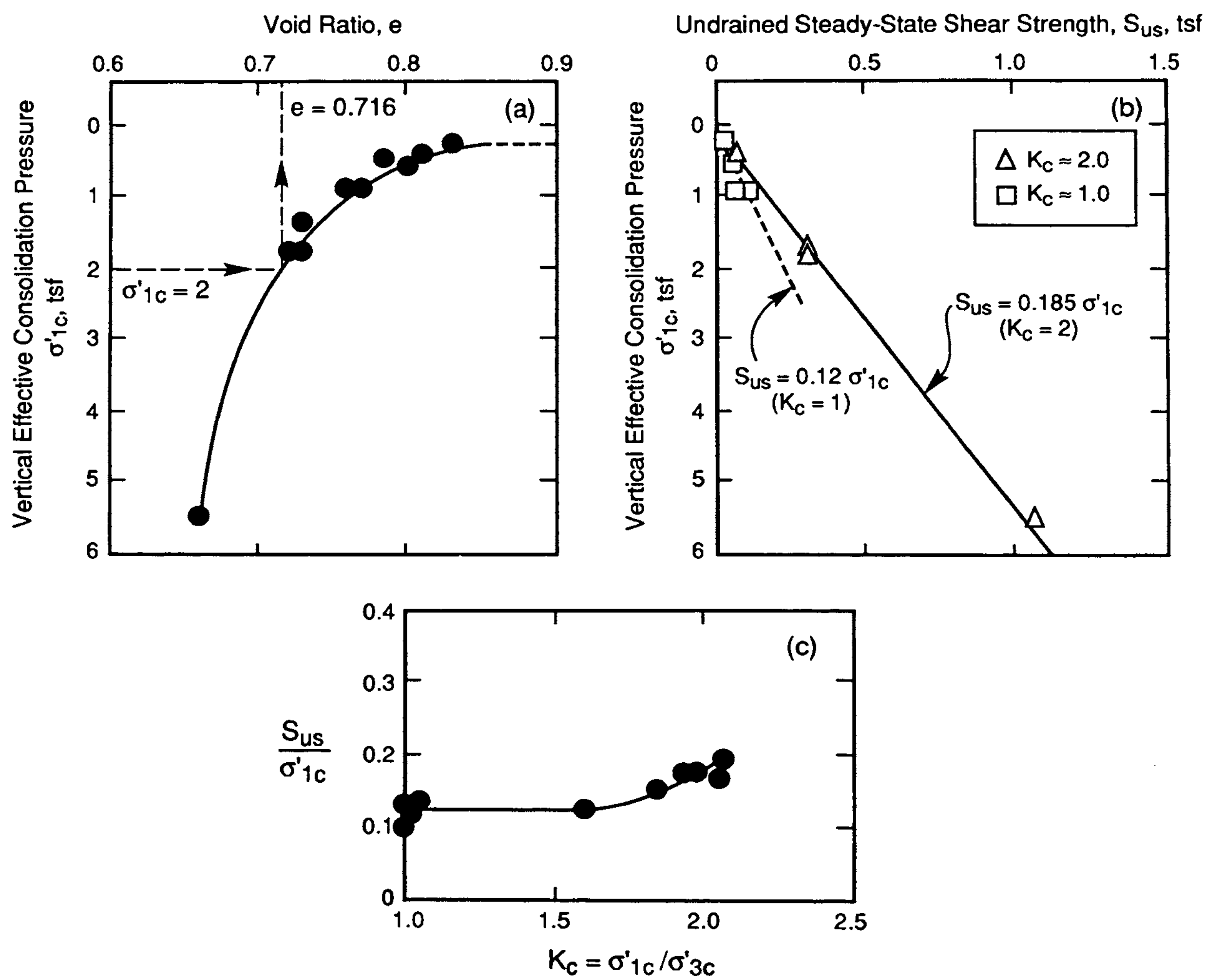


Figure 6. Relations obtained from ten monotonic and cyclic undrained tests on remolded layered specimens of silty sand, Batch Mix 7, Lower San Fernando Dam (Baziar et al., 1992, Baziar and Dobry, 1995).

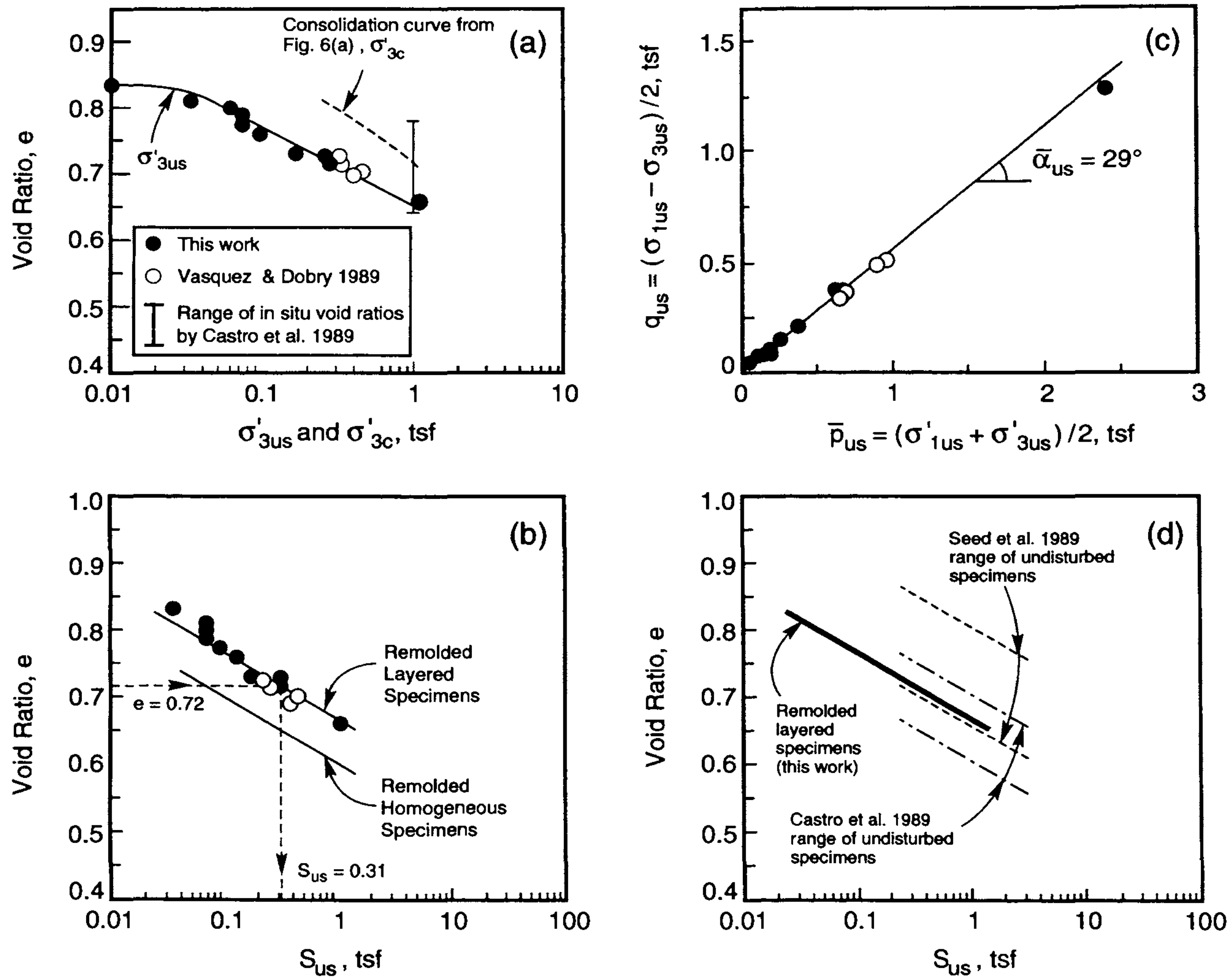


Figure 7. Steady-state relations for the same tests on remolded layered soil of Fig. 6, supplemented by four tests reported by Vásquez-Herrera and Dobry (1989), Batch Mix 7, Lower San Fernando Dam (modified from Baziar and Dobry, 1995).

The same monotonic and cyclic tests are presented in Fig. 7, where unique steady-state lines are obtained for these remolded layered specimens. Figure 7(a) also includes the consolidation curve from Fig. 6(a) for the case of $K_c = 2$. For the range of pressures of interest, the consolidation curve is located above the SSL ($\sigma'_{3us}$ versus e), consistent with the contractive behavior observed in the tests. Figure 7(b) includes a comparison with the $S_{us}$ steady-state line of remolded homogeneous specimens of the same soil, obtained from tests conducted at four organizations: GEI Consultants, Stanford University, US Army Corps of Engineers Waterways Experiment Station, and Rensselaer Polytechnic Institute (Marcuson et al., 1990; Castro et al., 1992). While the two steady-state lines in Fig. 7(b) are parallel, the one for layered soil is significantly higher, with $S_{us}$ of remolded layered soil being about four times larger than the $S_{us}$ of remolded homogeneous soil having the same void ratio. Figure 7(d) is discussed in the next section.

## APPLICATION TO LOWER SAN FERNANDO DAM

The 1971 upstream flow slide of the LSFD shortly after the end of the ground shaking caused by the San Fernando earthquake has been extensively studied. Based on field trenching and other investigations, Seed et al. (1973, 1975) identified the part of the upstream liquefied hydraulic fill that had flowed into the reservoir (cross-hatched zone in Fig 1(a)). A second effort was conducted in 1985-1989, sponsored by the US Army Corps of Engineers Waterways Experiment Station (WES), including in situ density and standard penetration tests as well as undisturbed sampling in the still intact downstream side, undisturbed and remolded laboratory testing, and re-evaluation of the 1971 failure. The 1985-1989 investigations focused on a location downstream which is about the mirror image of the 1971 failure zone in the upstream shell; therefore, the soil conditions investigated correspond reasonable well to those in the liquefied soil upstream (Castro et al., 1992). The author participated in this re-evaluation effort as part of the RPI group (Vásquez-Herrera and Dobry, 1989), together with WES, GEI Consultants (Castro et al., 1989) and the Berkeley-Stanford University group (Seed et al., 1989). The results of the 1985-1989 effort have been summarized by Marcuson et al. (1990), Castro et al. (1992, 1993), and Baziar and Dobry (1995). Both Castro and Seed used for their analyses values of $S_{us}$ based on the Castro-Poulos-France method and on their best estimates of the void ratios of the failed soil upstream prior to the 1971 slide. They also backfigured average values of the residual shear strength $S_r$ from analyses of the failure itself.

All tests on remolded layered water-sedimented specimens presented in Figs. 5-7 were done on a representative batch of soil obtained by GEI Consultants downstream, and distributed and used by all groups participating in the 1985-1989 effort. Therefore, a unique opportunity arises to verify the validity of the remolded layered specimen testing approach, by comparing these remolded layered results on water-sedimented specimens, both to the laboratory data and interpretations produced with the Castro-Poulos-France method, and to the best estimates of the state of the soil upstream before the 1971 slide including in situ void ratios and backfigured values of $S_r$. These comparisons, already presented and discussed by Baziar and Dobry (1995), are reproduced in the rest of this section and are summarized in Fig. 7 and Table 2. In all cases, average representative values $\sigma'_{1c} \approx 2$ tsf, $\sigma'_{3c} \approx 1$ tsf, and $K_c = \sigma'_{1c} / \sigma'_{3c} \approx 2$ are used for the upstream hydraulic fill along the failure surface shown in Fig. 1(a). These values were obtained from static finite element and stability analyses (Vásquez-Herrera and Dobry, 1989; Castro et al., 1992).

The first comparisons relate to the *in situ void ratios*. The band of void ratios estimated in Castro et al. (1989) for the critical hydraulic fill upstream in 1971, e = 0.64 to 0.78, has been plotted at $\sigma'_{3c} \approx 1$ tsf in Fig. 7(a). This range was obtained in that publication from 22 in situ density measurements made downstream, after Castro et al. corrected them for the different confining stresses between upstream and downstream and for densification after 1971. The band is located above the steady-state line in Fig. 7(a), and thus the water-sedimentation procedure predicts that the hydraulic fill upstream was contractive and susceptible to flow failure under undrained loading. The laboratory consolidation curve for the remolded layered soil, obtained from Fig. 6(a) and plotted in Fig. 7(a) for the relevant case $K_c = 2$, predicts e = 0.72 for $\sigma'_{3c} \approx 1$ tsf, essentially identical to the average in situ void ratio upstream in 1971 determined from the same 22 data points.

Another interesting comparison is between the $S_{us}$ *steady-state line (SSL)* for remolded layered soil of Fig. 7(b) and the SSLs obtained from the undisturbed layered specimens of the hydraulic fill tested as part of the Castro-Poulos-France method. This is done in Fig. 7(d), where the remolded layered SSL of Fig. 7(b) is repeated. Two ranges are included in Fig 7(d), corresponding to tests on undisturbed samples performed by Castro et al. (1989), and Seed et al. (1989), respectively. The remolded layered SSL is within the two ranges and close to the middle of the whole band. Therefore, the SSL obtained with the remolded layering water-sedimented technique agrees well with the range of SSLs determined by the Castro-Poulos-France procedure.

Finally, it is most useful to compare the *average undrained steady-state strength*, $S_{us}$, predicted along the failure surface in Fig. 1(a) from the remolded layered, water-sedimented soil tests, with both: (i) the corresponding average $S_{us}$ predicted by the Castro-Poulos-France method, and (ii) the average residual shear strength $S_r$ backfigured from the 1971 slide. Table 2 summarizes the corresponding information. The water-sedimentation laboratory technique predicts $S_{us} = 0.31$ tsf from $\sigma'_{1c} = 2$ tsf and the corresponding e = 0.72 as shown in Figs. 6(a) and 7(b); and $S_{us} = 0.37$ tsf from $S_{us} / \sigma'_{1c} = 0.185$ corresponding to $K_c = 2$ in Fig. 6(b). These two values compare favorably in Table 2 with the average $S_{us} = 0.305$ to $0.405$ tsf determined using the Castro-Poulos-France procedure. That is, both the Castro-Poulos-France method, using undisturbed layered specimens, and the remolded layered, water-sedimented soil approach predict an average $S_{us} \approx 0.3$ to $0.4$ tsf along the failure surface of Fig. 1(a).

Table 2 also includes various estimates of *residual strength* $S_r$ backfigured from analyses of the initial slope configuration in Fig. 1(a), of the configuration after failure in Fig 1(b), or of a combination of both. The average driving static shear stress in the hydraulic fill ($\tau_{dr}$ in Fig. 1(a)), obtained from slope stability analyses, was $\tau_{dr} \approx 0.43$ to $0.53$ tsf (Castro et al., 1992; see also Gu et al., 1993).

The original estimate made by Seed (1987) of $S_r = 0.375$ tsf for the start of the sliding is close to $\tau_{dr}$, and he suggested that this value of $S_r$ may have decreased as the failure progressed. Confirming this hypothesis of Seed, significantly lower values ($S_r = 0.15$ to $0.25$ tsf) are obtained from analyzing the failed configuration of Fig. 1(b). It is interesting that this original estimate at the outset of the sliding, $S_r = 0.375$ tsf, as well as the upper part of the range estimated by Castro and Davis, are all within the band $S_{us} = 0.3$ to $0.4$ tsf predicted from the tests on both undisturbed and remolded *layered* soil done at three different laboratories.

**Table 2. Estimates of Residual Shear Strength, Upstream Shell of Lower San Fernando Dam, 1971 Earthquake (modified after Baziar and Dobry, 1995)**

| General Approach (1) | Reference(s) (2) | Average Undrained Steady-State Shear Strength $S_{us}$ (tsf) (3) | Average Residual Shear Strength $S_r$ (tsf) (4) | Comments (5) |
|---|---|---|---|---|
| Undrained Laboratory Testing | Castro et al. (1989) Castro et al. (1992) | 0.305 | — | Method A[a,b] |
| | Seed et al. (1989) Castro et al. (1992) | 0.405 | — | Method B[a,c] |
| | Vásquez-Herrera & Dobry (1989) | 0.35 | — | Remolded layered specimens and in situ void ratios |
| | Baziar and Dobry (1995) | 0.37 | — | Remolded layered specimens: from $S_{us} / \sigma'_{1c} = 0.185$ and $\sigma'_{1c} = 2$ tsf (Fig. 6(b)) |
| | | 0.31 | — | Remolded layered specimens: from e = 0.72 (Fig. 6(a)) and steady-state line (Fig. 7(b)) |
| Backfigured from 1971 Flow Slide | Seed (1987) | — | 0.375 | $S_r$ at start of sliding (Fig.1(a)) |
| | Seed et al. (1989) Seed & Harder (1990) Castro et al. (1993) | — | 0.15 to 0.25 | $S_r$ at end of sliding (Fig. 1 (b)) |
| | Davis et al. (1988) Castro et al. (1993) | — | 0.22 to 0.32 | Representative average of whole failure process (Figs. 1(a) and 1(b)) |

[a] Castro-Poulos-France (1982) procedure applied to the dam. Included steady-state strength determinations on remolded homogeneous specimens and undisturbed specimens; field density tests; in situ void ratio estimates from tube samples including corrections for changes during excavation and sampling; void ratio corrections for changes between 1971 and 1985 (year of field exploration); and statistical analyses of results to obtain average $S_{us}$.

[b] Method A: Change of in situ void ratios between 1971 and 1985 estimated by Castro et al. (1989).

[c] Method B: Change of in situ void ratios between 1971 and 1985 estimated by Seed et al. (1989).

On the other hand, the value of $S_r$ estimated at the end of the flow failure in Table 2 is significantly lower, having decreased by a factor of about 1.5 or 2. A possible reason for this reduced $S_r$ may have been the severe remolding of the liquefied soil originally in the cross-hatched triangle of Fig. 1(a) that took place during the flow slide. The field investigation after the earthquake revealed that this soil had lost its original shape and was spread over a large distance throughout the slide zone, with part of it having been extruded between blocks of undisturbed material originated from outside the triangle and with significant mixing of layers (Seed et al, 1973, 1975). Therefore, it is possible that during this process the hydraulic fill may have lost part of its original microlayering, approaching the state represented by the remolded homogeneous SSL in Fig. 7(b) and decreasing its $S_{us}$ from somewhere in the range 0.3 to 0.4 tsf to the final value $S_{us} \approx 0.2$ tsf. A simple way to visualize this speculation is to look at Fig. 7(b); during the slide the liquefied soil would have moved to the left along the horizontal line of constant e ≈ 0.72 from the layered SSL ($S_{us} \approx 0.3$ tsf) toward the homogeneous SSL, coming to rest at $S_{us} \approx 0.15$ or 0.2 tsf. This discussion is important because the higher value of $S_{us} \approx 0.3$ to 0.4 tsf of the intact microlayered soil existing at the outset of the slide (which, under this hypothesis, would be correctly predicted by the laboratory tests) should be the undrained strength relevant for engineering flow failure stability evaluations, rather than the lower amount $S_{us} \approx 0.15$ or 0.2 tsf requiring large amounts of prior straining and remolding.

Therefore, the remolded layered water-sedimentation testing approach successfully predicts: the average in situ void ratio of the upstream silty sand hydraulic fill in the LSFD prior to the 1971 earthquake; the fact that the soil was contractive and thus susceptible to flow sliding; and also, seemingly, the in situ residual shear strength at the outset of the failure. In addition, the predictions based on the water-sedimentation technique are consistent with those of the Castro-Poulos-France method, and they also provide a possible explanation for the reported decrease in residual strength of the liquefied soil between the beginning and the end of the 1971 flow slide.

It is interesting to note that Ishihara (1993), using a different interpretation of the same RPI laboratory results on water-sedimented specimens presented in Figs. 6-7, predicts an in situ ratio $S_{us} / \sigma'_{v0} \approx 0.11$ for the LSFD and thus $S_{us} = (0.11)(2) = 0.22$ tsf, closer to the lower values of $S_r$ in Table 2. This illustrates the uncertainty in the prediction of the in situ $S_{us}$, even when the same laboratory data are used. As shown by Table 2 and reflected in the band for LSFD in Fig. 3(b), a similar uncertainty exists when backfiguring $S_r$ from the failure itself.

Based on this application to the LSFD case history, the use of remolded, water-sedimented laboratory specimens is clearly an alternative technique for estimating in situ void ratios and undrained residual shear strengths of microlayered, loose, recently sedimented, natural or artificial silty sand deposits.

## SCREENING TECHNIQUES TO EVALUATE LARGE GROUND DEFORMATION POTENTIAL

In many engineering applications, charts such as that proposed by Seed et al. (1984) and reproduced in Fig. 8, are used to evaluate liquefaction at level or almost level sites during earthquake shaking. The curve separating "liquefaction" from "no liquefaction" in Fig. 8 was obtained as the boundary between clean sand sites that liquefied or did not liquefy during earthquakes of magnitude M ≈ 7.5. While some of the liquefied sites exhibited large ground deformations or other manifestations of ground failure or damage to constructed facilities, other sites were considered to have liquefied based on observed sand boils at the ground surface. Therefore, the boundary curve in the figure has been associated with *initial liquefaction* of the soil, that is with an excess pore pressure ratio, $r_u \approx 1.0$. The chart is based on $(N_1)_{60}$ = Standard Penetration Resistance in blows/ft normalized both to $\sigma'_{v0} = 1$ tsf and to a rod energy ratio of 60%. Note that if the ground shaking is strong enough, sites with $(N_1)_{60}$ as high as 30 blows/ft are predicted to liquefy by Fig. 8 during an earthquake of M = 7.5. The value of $(N_1)_{60}$ has been correlated with relative density, $D_r$, in clean sands (Tokimatsu and Seed, 1987), with $(N_1)_{60}$ = 15 blows/ft corresponding to $D_r \approx 60\%$, and $(N_1)_{60} = 30$ corresponding to $D_r \approx 80\%$.

The same Fig. 8 gives other information based on undrained laboratory cyclic tests and shaking table tests, which shows that a saturated clean sand in a level site with $(N_1)_{60} = 30$, even if it liquefies, will be able to develop only up to a cyclic shear strain of 3% after liquefaction due to the *dilative response* of the sand at large strains. The same sand subjected to a driving static shear stress (as in a slope or under a foundation), will not be able to develop flow failure when loaded undrained due to this same dilative behavior. When $(N_1)_{60}$ is decreased in Fig. 8, the sand becomes able to develop larger and larger cyclic strains, and for $(N_1)_{60} < 10$ or 15 blows/ft it can strain up to 20% or more, eventually becoming *contractive* and thus able to flow when under a static driving shear stress (see also Robertson et al., 1992).

A number of authors have further calibrated this concept with case histories, in attempts to develop reliable screening techniques to evaluate the *large ground deformation potential* of a site during an earthquake, rather than initial liquefaction. These attempts have utilized either

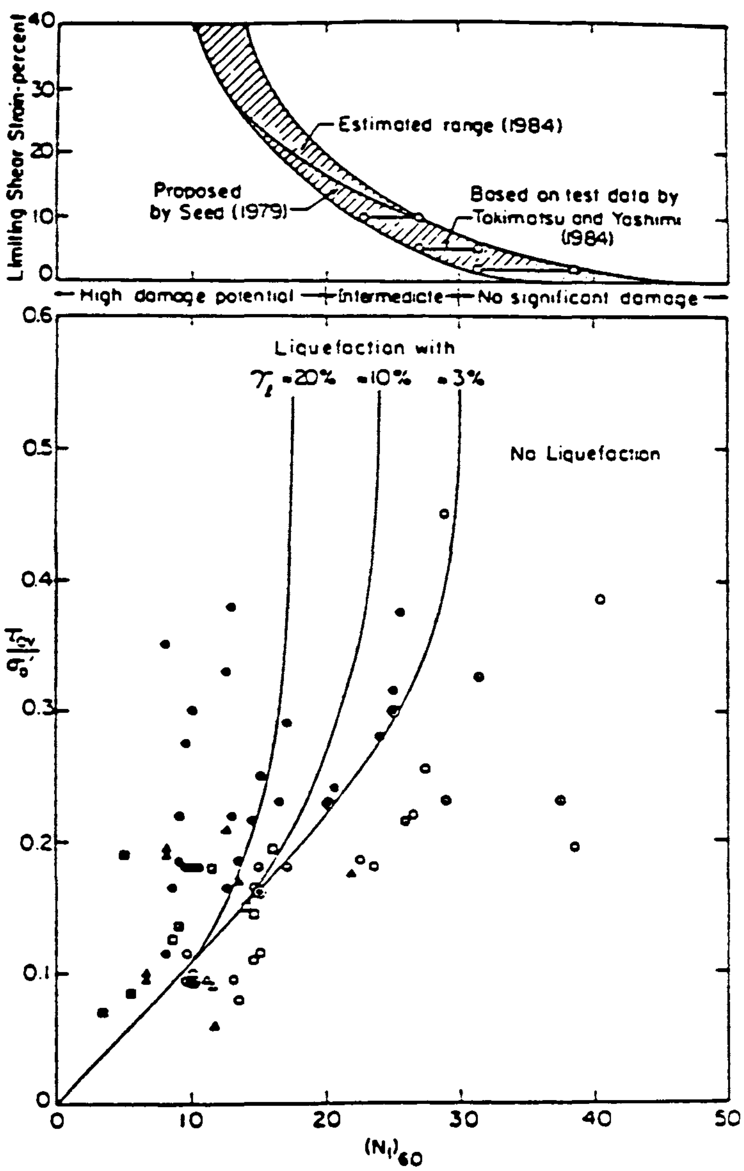


Figure 8. Evaluation of liquefaction and deformation due to earthquake loading using the SPT (from Seed et al., 1984 and Robertson et al., 1992).

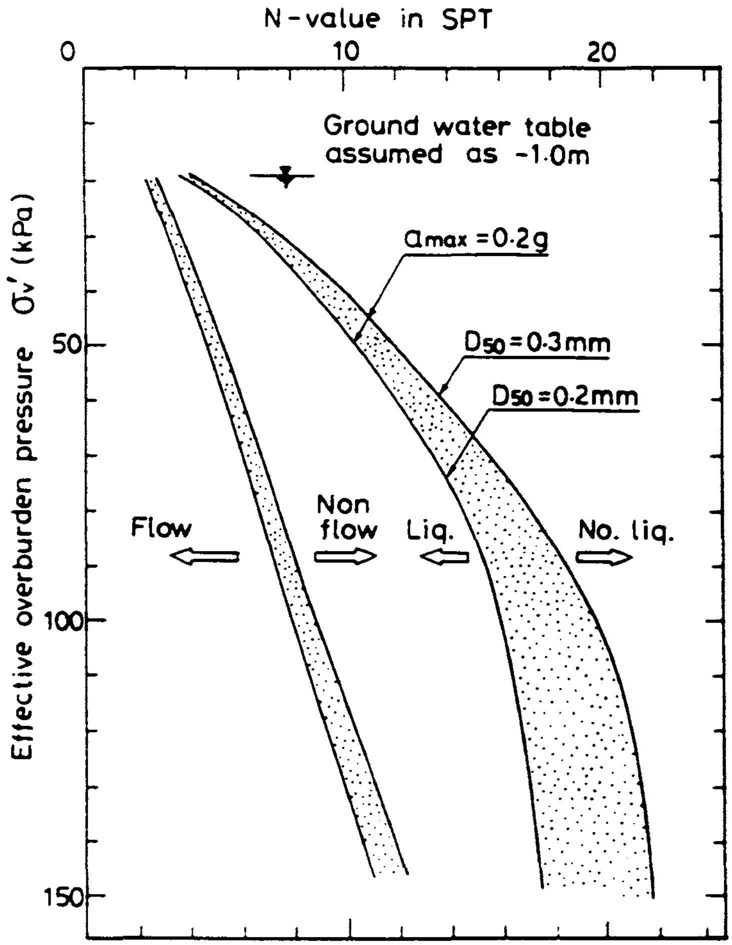


Figure 9. Two boundary curves in SPT N value identifying three classes of sand deposit with different levels of damage due to liquefaction (Ishihara, 1993).

the same Standard Penetration Test (SPT) used in Fig. 8, or the static Cone Penetration Test (CPT). Publications addressing the issue include Sladen and Hewitt (1989), Robertson et al. (1992), Bartlett and Youd (1992, 1995), Ishihara (1993), and Baziar and Dobry (1995). After an extensive study of lateral spreads in Japan and the U.S., Bartlett and Youd found that no significant lateral ground displacement had occurred if $(N_1)_{60} > 15$ in nongravelly sands and silts during earthquakes of moment magnitude $M_W < 8$. Figure 3(a), applicable to nongravelly silty sand or sandy silt with fines contents between 10% and 80%, and to level sites as well as slopes, makes the boundary value of $(N_1)_{60}$ as small as 4 or 5 blows/ft near the ground surface, increasing to $(N_1)_{60} \approx 15$ at $\sigma'_{v0} \approx 4{,}000$ psf. Figure 3(a) was developed by Baziar and Dobry using the same data base for lateral spreads compiled by Bartlett and Youd (1992), plus cases of flow failure and lateral spreading compiled by Seed (1987), Davis et al. (1988), Seed and Harder (1990), and Stark and Mesri (1992). As values of lateral displacement $D_H$ were available from these case histories, the (upper) boundary curve in Fig. 3(a) is defined as giving the maximum value of $(N_1)_{60}$ of sites capable of developing more than $D_H$ = 1 to 3 ft. (See Fig. 2 for definition of $D_H$). Figures 9 and 10 present similar screening curves or bands presented by Ishihara (1993) and Robertson et al. (1992) for clean sands (up to 30% fines in the case of Ishihara's chart), using SPT and CPT, respectively.

Screening recommendations and charts such as these are obviously very useful in engineering practice. They help remove the conservatism associated with predicting

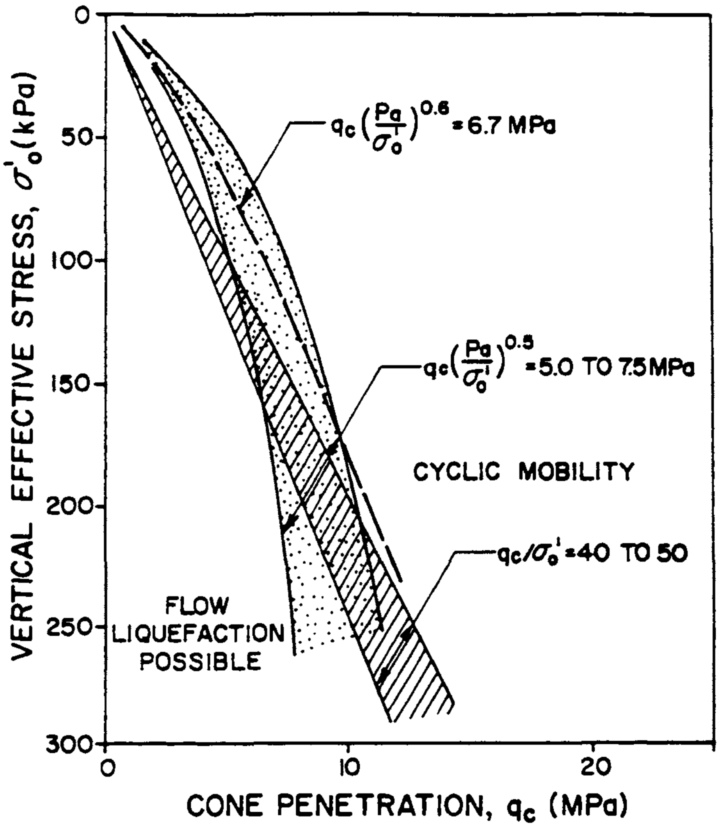


Figure 10. Comparison of CPT penetration profiles to define contractive state for clean sand (Robertson et al., 1992).

liquefaction only in terms of excess pore pressure, in soils which are not loose enough for these pore pressures to have serious engineering consequences. One particularly useful feature is that all these recommendations and charts are valid for a wide range of earthquake magnitudes and levels of ground shaking; that is, the boundaries for large ground deformation in Figs. 3(a), 9 and 10 *are not* associated with a specific earthquake magnitude or ground acceleration. In addition to classifying a saturated cohesionless site in terms of its ground deformation potential, these screening techniques may also be used to establish targets for cost-effective site remediation aimed at a significant reduction in the level of ground deformation in future earthquakes.

## EFFECTS OF GROUND DEFORMATION ON FOUNDATIONS AND STRUCTURES

Lateral and vertical ground deformations associated with liquefaction are an extremely significant cause of damage to foundations and structures during earthquakes. Compaction settlement, cyclic ground oscillations, and permanent lateral and vertical displacements due to lateral spreading are some main sources of the problem. Of these, the phenomenon of lateral spreading sketched in Fig. 2 is the most important, and most of the effects summarized in the case history volumes by Hamada and O'Rourke (1992) and O'Rourke and Hamada (1992) are associated with lateral spreads (Fig. 11). The rest of the discussion below on the effects of ground deformation is based on several of the references listed in Table 1, Item 14, and especially Dobry (1994).

Similar to the case of static settlements, the cause of earthquake damage to foundations and buildings is not so much the ground displacement itself, but the *ground straining*. For example, the destruction of the building on shallow foundations in Fig. 11 was caused by horizontal extension of the ground associated with a lateral spread. Therefore, it is useful to examine the values and spatial patterns of ground deformation associated with these liquefaction-related phenomena. In the case of *compaction settlement*, vertical deformations as much as 5% or more of the thickness of the loose sand layer have been reported. Differential settlements and associated vertical shear straining of the ground and of foundations placed on it can occur in areas where the thickness or density of the compacting soil changes rapidly over short distances (Tokimatsu and Seed, 1987; Ishihara and Yoshimine, 1991; O'Rourke et al, 1992a).

In the case of lateral spreads, horizontal displacements from a few centimeters to more than 10 m have been observed, with the phenomenon sometimes affecting a large area which moves, either downslope along a slope as small as 0.5%, or toward a free face. The amount of lateral displacement typically increases with slope and height of the free face and decreases with distance from the free face. Extensional ground straining including fissures, as well as vertical settlements, tend to occur at the head of the spread while compression and ground uplifting appear at the toe. Ground shear develops especially at the spread margins. Fig. 12 shows the pattern of lateral ground displacements for the 1971 San Fernando, California, earthquake, obtained mainly by comparison of air photos before and after the earthquake in a large area of more than 1 $km^2$. Fig. 13 presents a map of the corresponding surficial ground cracks. Although most of the lateral displacements were due to liquefaction and lateral spreading of a loose alluvium layer, they also included a tectonic (faulting) component. The average ground surface in the area was 1.5°, with a maximum slope through the Juvenile Hall of about 3° (Youd, 1973; Youd and Perkins, 1987; Bartlett and Youd, 1992; O'Rourke et al. 1992b).

Differential lateral displacements—such as associated with the variation with distance of the magnitudes of the vectors in Fig. 12—can produce horizontal extension, compression or shear, while differential vertical displacements cause vertical shearing of the ground. As noticed by Youd (1989), generally shallow foundations are most sensitive to ground extension and vertical shear, and somewhat less sensitive to horizontal shear and compression. A main cause of damage to pile foundations is the variation of lateral ground displacement with depth.

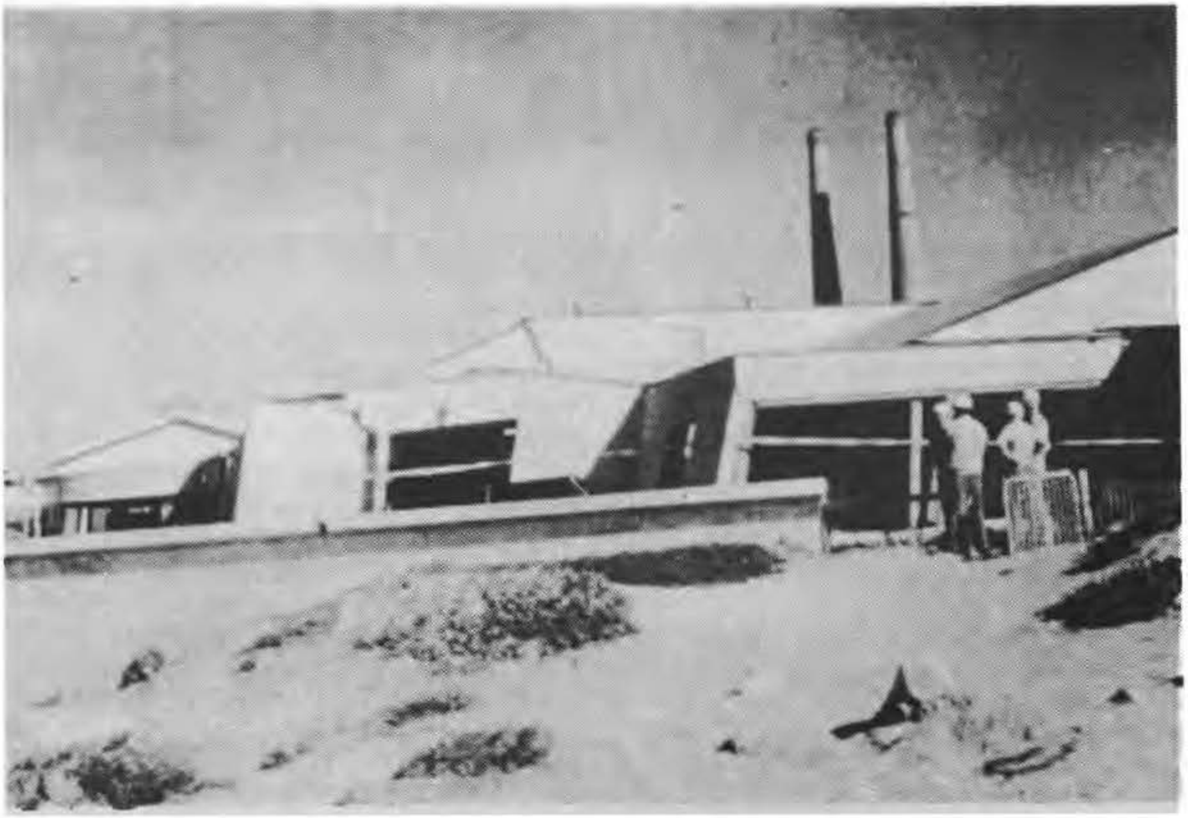

Figure 11. Lateral Spread Failure due to Liquefaction, Marine Sciences Laboratory at Moss Landing, CA, 1989 Loma Prieta Earthquake (Youd, Personal Communication; Photo Taken by G. Castro).

Therefore, any indication of the *type* of ground surface straining expected due to the design earthquake is useful to the engineer and should help his/her judgment when making design or retrofitting decisions for shallow foundations. A rational evaluation procedure for structural damage should include methods to predict the type and amount of ground strain in the free field, as well as the degree of foundation/building damage associated with such free field strain. Susuki and Masuda (1991) have studied the measured surface ground movements due to lateral spreads at two Japanese cities after earthquakes, and have attempted to model analytically the corresponding patterns of permanent ground straining. A similar attempt has been presented by Finn (1991), while Zeghal and Elgamal (1994) have backfigured from acceleration earthquake records the transient ground shear strains associated with post-liquefaction ground oscillations. O'Rourke and Pease (1995) and O'Rourke et al. (1995) have used estimated patterns of free field transient and permanent ground deformations and strains for damage evaluations of buried pipelines. Unfortunately, ground straining is very difficult to measure and even more difficult to predict. As a result, foundation and building damage have been generally correlated to *ground displacement* rather than to strain (Table 3 and Fig. 14). Again, the use of ground displacement as in Table 3 is similar to the standard static design procedure for shallow footings on sand, where an acceptable settlement of 2.5 cm (1 inch) is taken to imply that the differential settlements/vertical shear straining of ground foundation will also be small and acceptable.

There are a couple of cases for which the engineering evaluation of ground straining (as different from ground displacement) is more feasible. One of them is the vertical shear ground straining due to compaction settlement already mentioned. Another is the evaluation of the effect of a lateral spread on a pile foundation, once the lateral surface ground displacement $D_H$ at the site has been determined. As reasonable assumptions are possible for the distribution of lateral displacement with depth—based on the location and thickness of the liquefiable layer—the analysis of piles is generally more straightforward than that of shallow foundations. Fig. 15 shows the observed damage to reinforced concrete point bearing piles 350 mm in diameter produced by $D_H \approx 1.2$ m at the ground surface in the 1964 Niigata earthquake. Fig. 16 presents pile bending moments predicted using a numerical model developed by Miura and O'Rourke (1991) and Meyersohn et al. (1992). This model accounts for geometrical and material nonlinearities of both piles and soils. The flexural characteristic of the reinforced concrete piles are modeled by moment-curvature relationships, which are

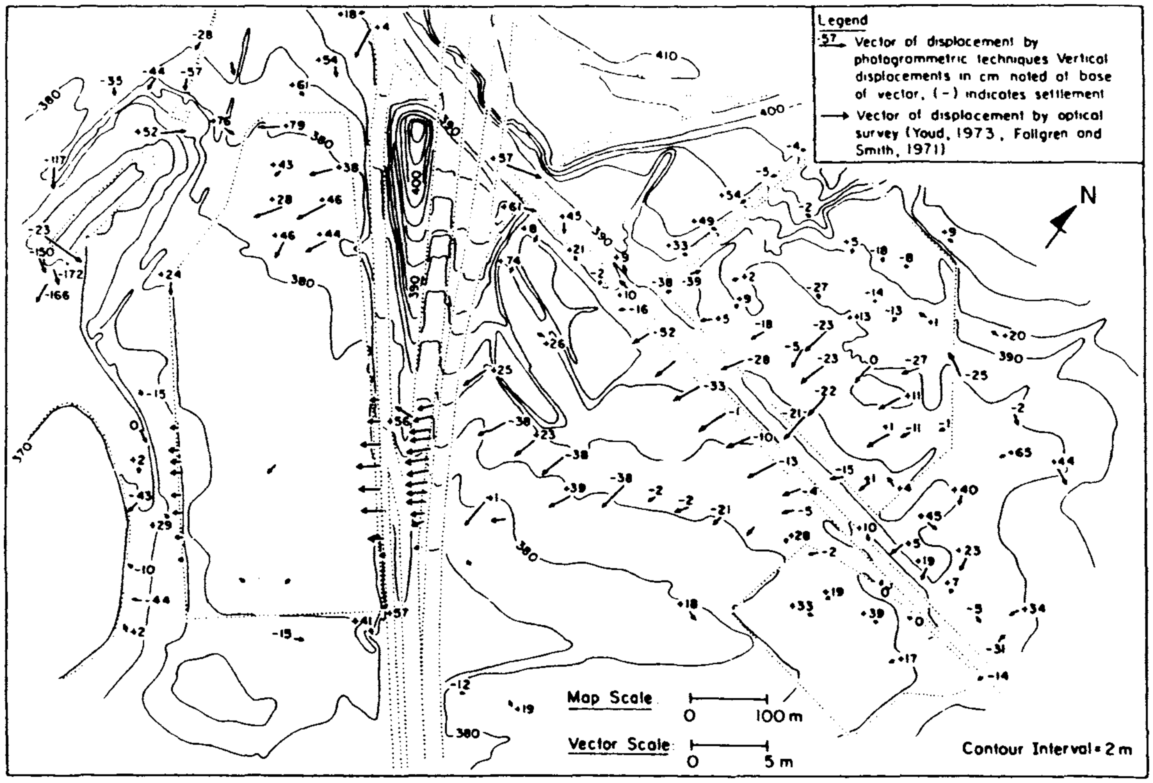


Figure 12. Lateral Displacement Vectors Obtained from Air Photo Analyses and Optical Surveys, Juvenile Hall and Nearby Areas, 1971 San Fernando, CA Earthquake (O'Rourke et al. 1992b).

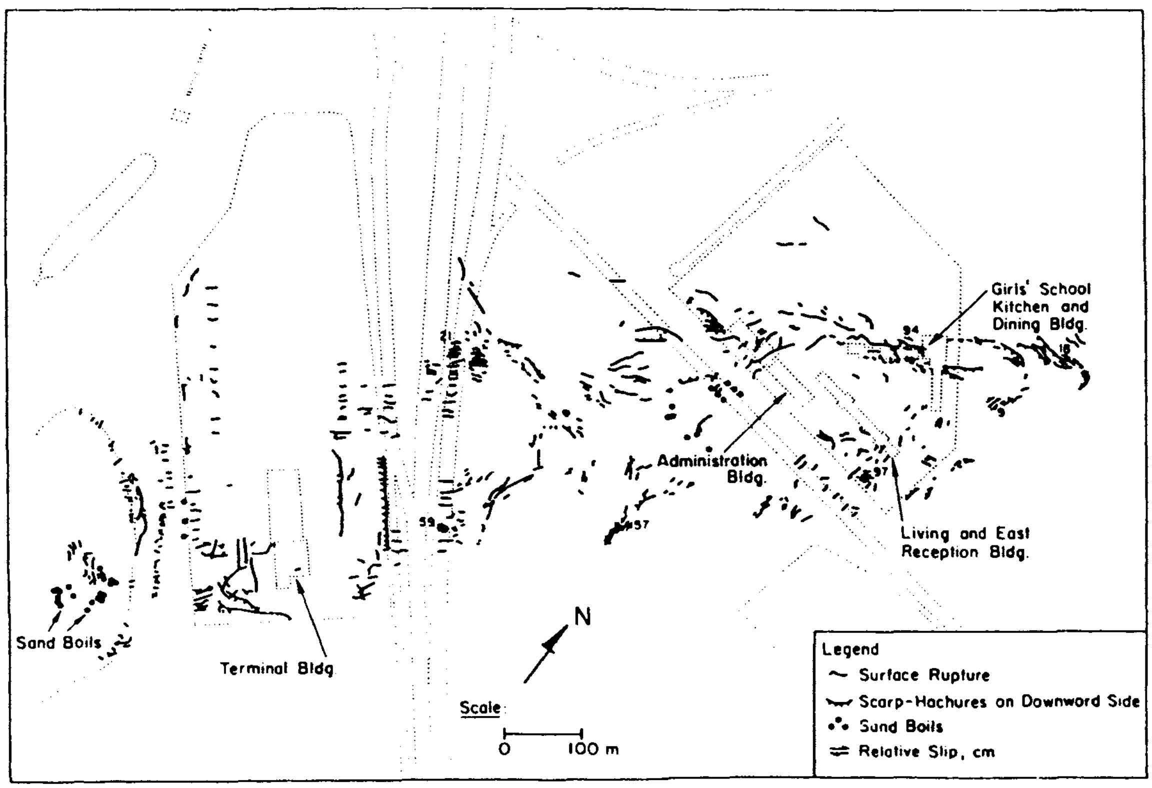


Figure 13. Map of Surficial Ground Cracks, Sand Boils, and Pressure Ridges for the Same Area of Fig. 6, 1971 San Fernando, CA Earthquake (O'Rourke et al. 1992b).

**Table 3. Approximate Amounts of Ground-Failure Displacement Required to Cause Repairable and Irreparable Damage (Youd 1989)**

| Type of Deformation | Foundation | Displacement Required to Cause | |
|---|---|---|---|
| | | Repairable Damage (m) | Irreparable Damage (m) |
| Shear | Poorly-Reinforced[1] | 0.1 | >0.3 |
| | Well-Reinforced[2] | >0.3 | ? |
| Extension | Poorly-Reinforced | <0.05 | >0.3 |
| | Well-Reinforced | >0.1 | ? |
| Compression | Poorly-Reinforced | <0.3 | >0.5 |
| | Well-Reinforced | >0.5 | ? |
| Compression with Vertical | Poorly-Reinforced | <0.2 | >0.2 |
| | Well-Reinforced | <0.3 | >0.3 |
| Vertical | Poorly-Reinforced | <0.05 | >0.2 |
| | Well-Reinforced | <0.1 | >0.3 |

[1]Foundations with minimal or no temperature reinforcing steel.
[2]Foundations with adequate reinforcing steel to provide considerable structural strength.

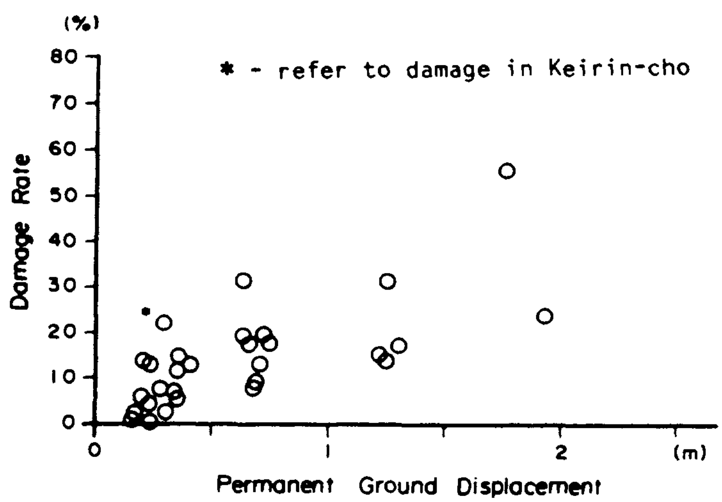


Figure 14. Relation between Damage Rate to Houses and Permanent Ground Displacements, 1983 Nihonkai-Chubu, Japan Earthquake (Hamada 1992).

obtained by appropriate selection of stress-strain curves of concrete under compressive and tensile stress (Meyersohn et al., 1992; Meyersohn, 1994). Simplified models of pile group performance have also been proposed. This analytical procedure for piles and pile groups subjected to lateral spreading has been calibrated by field case histories such as that of Fig. 15 and is currently being further refined with the help of centrifuge models (Abdoun and Dobry, 1995).

FINAL COMMENTS

We are clearly somewhere in the middle of a period of rapid progress in our understanding of the liquefaction phenomena and their engineering implications. Case histories, instrumented sites and soil-structure systems, field measurements, laboratory results, 1g and centrifuge earthquake model tests, calibrated numerical techniques, and team work and international cooperation, are the main tools we are using to advance the state-of-the-art. A main trend is the increasing importance which is being given to understanding and evaluating the effects of liquefaction, such as ground deformation and straining and their effects on constructed facilities.

This paper provided a general perspective of where we are in the process— through Table I—and discussed in more detail four selected topics related to the engineering evaluation of liquefaction-induced ground deformation and its effects on constructed facilities.

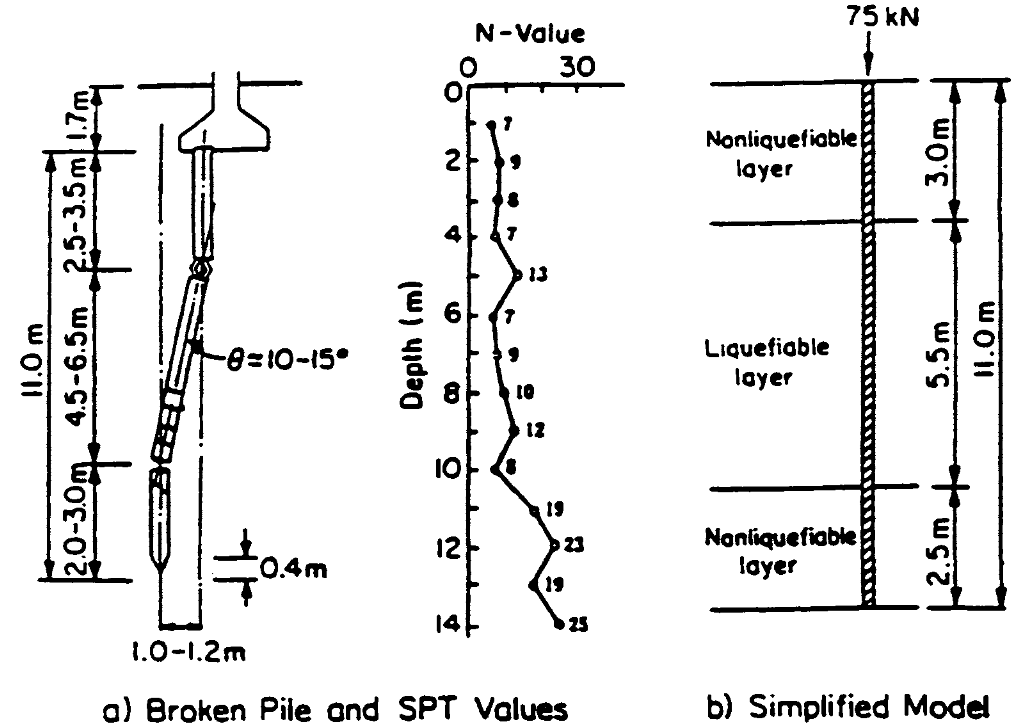


Figure 15. Observed Damage to Reinforced-Concrete Pile Foundation at NHK Building due to Liquefaction-Induced Lateral Spreading, 1964 Niigata, Japan Earthquake (Hamada et al. 1986; Meyersohn 1994).

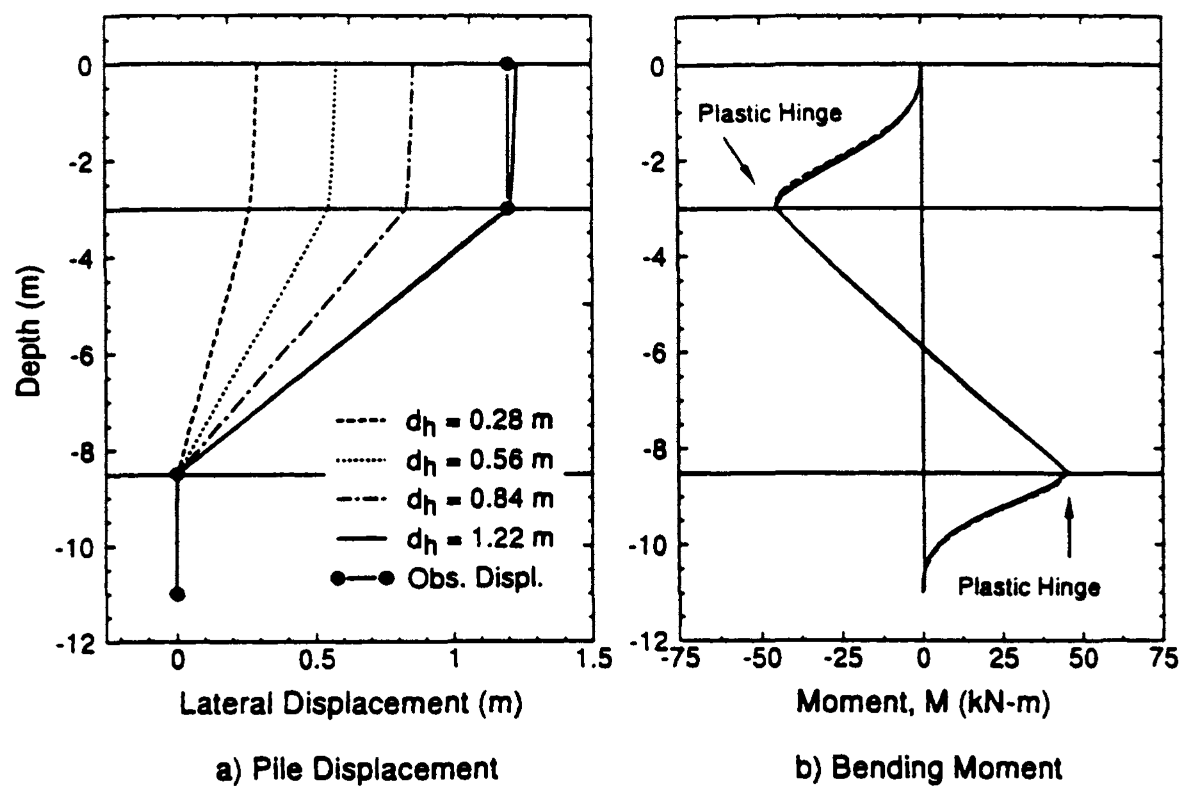


Figure 16. Analytical Results for the NHK Building Pile Foundation (Meyersohn et al. 1992; Meyersohn 1994).

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