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Case Histories of Geotechnical Earthquake Engineering

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INTRODUCTION

Geotechnical earthquake engineering has developed tremendously over the past 25 to 30 years. The field has reached the point where many phenomena are now fairly well understood, and where impressive analytical models are available for computation of the dynamic response of soils and their interaction with a variety of structures. With these developments, however, has come the need for data with which to verify the accuracy of theories and numerical models. The detailed study and interpretation of case histories is a necessary and extremely important step in the verification of these theories and models, and in the continuing maturation of the field.

Of the 16 papers submitted to this session 15 can be divided into the following five general categories:

1.	Observations of earthquake damage	5 papers
2.	Interpretation and measurement of dynamic response	3 papers
3.	Analysis of potential seismic hazards	4 papers
4.	Numerical modeling	2 papers
5.	Soil improvement	1 paper

The sixteenth paper concerns the mechanical response of nuclear power plant piping systems; geotechnical factors are not considered.

DAMAGE OBSERVATION

Five papers discuss observations of earthquake-related damage in the United States, Chile, Japan and Iran. The papers present descriptions of damage to fills, quay walls, earth dams, and natural slopes.

In their paper, "*Performance of Fill Soils during the Loma Prieta Earthquake*," **Frost, Reyna, Chameau,** and **Karanikolas** report the results of an investigation of the effects of the 1989 Loma Prieta earthquake on a hydraulically placed fill at the Hunter's Point Naval Station in San Francisco. The site, consisting of 2-3 m of gravel fill over 13-15 m of hydraulic fill, had been extensively tested by others in 1985 and 1986 as part of a FHWA research project. This "pre-earthquake" investigation featured CPT, SPT, DMT, PMT, stepped-blade, and cross-hole seismic testing. After the earthquake, the investigators performed CPT, SPT, DMT, and seismic cone tests at the same locations and compared the results with those from the pre-earthquake investigation. Averaged cone penetration resistances showed regular increases over the entire thickness of the hydraulic fill, suggesting that the relative density increased by amounts ranging from several percent in the upper 2 m to about 80 percent at depths of 7-9 m. Increases in SPT resistance were irregular, implying density increases above 7 m depth and below 11 m depth, and a possible decrease between 7 m and 11 m. The horizontal stress index from the DMT increased in a fairly regular manner throughout the thickness of the hydraulic fill. Shear wave velocities from the post-earthquake seismic cone test were also observed to be systematically larger than those obtained in the pre-earthquake cross-hole seismic tests. One-dimensional ground response analyses using pre-earthquake soil conditions produced peak ground accelerations of 0.12 g with an input motion scaled to represent that of the Loma Prieta earthquake. When the analysis was repeated with the stiffer, post-earthquake soil conditions, peak ground accelerations of 0.19 g were computed. The question of how these increased accelerations would affect pore pressure generation or damage in a repeat of the Loma Prieta earthquake was not addressed. The results of this investigation provide a useful indication of the relative effectiveness of various insitu tests for soil characterization. Though not discussed in the paper, they also raise interesting quesitons regarding the levei of consertavitsm inherent in empirical correlations of liquefaction resistance with the results of insitu tests performed after earthquakes.

Ortigosa, Retamal, and **Whitman** describe a variety of effects observed at the ports of Valparaiso and San Antonio after the March 3, 1985 Chilean earthquake (M_s = 7.8) in their paper "*Failures of Quay Walls During Chilean Earthquake of March 1985*." At the port of Valparaiso, several quay walls consisting of stacked concrete blocks without shear keys experienced horizontal sliding of up to 40 cm between concrete blocks at a depth of about 12 m. One of the walls, for which foundation conditions were less favorable, also underwent permanent, rigid body rotation at its base. Since no acceleration measurements were available, the investigators were forced to use a somewhat convoluted procedure to back-calculate the apparent peak acceleration from the observed permanent wall displacements. A liquefaction analysis showed that the average pre-earthquake SPT N-values were one-half to two-thirds of that needed to resist liquefaction for the back-calculated peak acceleration. The fact that liquefaction was not clearly observed either illustrates the conservative nature of the SPT-based liquefaction evaluation procedure, or indicates that the back-calculated peak acceleration was too high. The behavior of seven berths were observed at the Port of San Antonio; 2 were concrete block walls, 2 were anchored sheet-pile walls and 3 berths were pile-supported. One of the concrete block walls collapsed and the other rotated up to 25°. This performance was consistent with the low (0.08 g) yield acceleration computed from previous earthquake performance. One of the anchored bulkheads performed very well; the other moved by nearly 1 m apparently due to inadequate anchorage in liquefied soils. The pile-supported berths performed fairly well, though some fill settlements did occur and a rockfill slope beneath two of the berths experienced some sliding. Back-calculated rockfill friction angles were about 10° lower than the laboratory-determined values, the effect of vertical accelerations which was not considered in the back-calculation was suggested as a possible reason. This case history illustrates the difficult situation geotechnical earthquake engineers frequently face in attempting to interpret quantitative behavior from marginally documented events.

Observations of earthquake damage to embankment dams in Japan and elsewhere are described by **Tani** in *"Earthquake Damage to Fill Dams."* Of an estimated 250,000 such dams in Japan, 75% are more than 100 years old and 80% are less than 10 m high. The paper concentrates on dams greater than 15 m high. Of the 1,506 dams considered, only one, the 23 m high Manno Ike dam, suffered complete failure (in the 1854 M; = 8.4 Ansei-Nankai earthquake). Damage to other dams subjected to accelerations up to 600 gal was largely limited to crest settlement and cracking of the crest and slopes. No details regarding soil types, construction methods or foundation conditions are presented. A number of well-known U.S. dam case histories are also mentioned; most damage was associated with liquefaction. The authors conclude that large dams with modern designs "have excellent earthquake resistance and provide sufficient safety against earthquakes." While the relatively low damage rates are comforting in a general statistical sense, the lack of available details makes it difficult to interpret any specific performance trends.

In their paper, *"Liquefaction Case Histories from 1990 Manjil, Iran Earthquake,"* **Yegian, Nogole-Sadat, Ghahraman,** and **Darai** describe an investigation of several cases of liquefaction resulting from the M_S = 7.7 Manjil earthquake that occurred in northwestern Iran in June, 1990. The studies focused on two regions near the Caspian Sea: the Astaneh-Ashrafieh region where liquefaction occurred in levee deposits along the banks of the Sefid and Heshmat rivers, and the Leef-Shagerd region where the liquefied sands were of marine origin. Field investigations conducted about a year after the earthquake allowed characterization of the grain size distributions and standard penetration resistances. In the Astaneh-Ashrafieh region, the sands were poorly graded, fine sands with less than 10% silts; in Leef-Shaferd they were well-graded with a somewhat greater fines content. Normalized and corrected SPT results from 31 borings at 9 sites and cyclic stress ratios computed by the 1983 Seed, Idriss and Arango procedure were used to evaluate the applicability of the Seed, Idriss, and Arango liquefaction resistance curves. The data was in general agreement with the published curves for magnitudes between 7.5 and 8.5. The paper describes clear evidence of liquefaction of both clean and silty sands at sites ranging from 15 to 81 km from the source, with peak accelerations ranging from 0.11 to 0.40 g. Comments on the influence of fines content on observed liquefaction behavior would have been helpful.

Zaré presents a preliminary macrozonation of landslides caused by the 1990 Manjil earthquake (MSK > VII) in northwestern Iran in *"Macrozonation of Landslides for the Manjil, Iran, 1990 Earthquake."* Landslides were divided into seven classes: rockfalls and debris slides, soil falls, earth flows and debris flows, rotational and planar slides and block guides, debris avalanches, and fault-induced slides. Examples of each type of slide are described and illustrated. The large landslides were located throughout what is referred to as the "earthquake origin zone" which appears to coincide with the MSK VII isoseism; rockfalls were found in this area and the area bounded by the MSK VI isoseism. Though the landslide descriptions are qualitative in nature, they illustrate the variety of mechanisms by which slope failures can occur, and serve as a reminder of the need to consider different failure mechanisms in the evaluation of slope stability.

INTERPRETATION AND MEASUREMENT OF DYNAMIC RESPONSE

Three papers relate to the interpretation and measurement of dynamic response. One addresses aspects of soil-structure interaction for a pile-supported 30-story structure, another considers the measured seismic response of two dams. The third describes the development and deployment of a remote, automated monitoring system.

Kagawa and **Al-Khatib** describe their analysis of the response of a pile-supported, 30-story building to the 1989 Loma Prieta earthquake in *"Earthquake Response of 30-Story Building During the Loma Prieta Earthquake."* The building was a ductile, moment-resisting frame structure founded on a 5-ft-thick reinforced concrete mat supported by 828 14-in-square prestressed concrete piles. The piles were driven in groups of 10 to 29 piles at the locations of structural columns. The site is typically underlain by 10 to 20 feet of hydraulically-placed fill (densified by vibroflotation prior to construction) over several feet of soft Bay Mud underlain by a sequence of stiff to very stiff clays and dense to very dense sands. A detailed model of the structure was condensed into a much simpler, lumped mass model with two translational and one rotational degree of freedom at each floor level. Kinematic interaction between the mat and supporting soils was found to be modest by an approximate procedure; inertial interaction was evaluated by approximating the foundation by a circular mat and then again by considering all 828 piles. Four different input motions were used: the recorded free-field motion, a modified (for kinematic interaction) free-field motion, and motions developed from two rock outcrops at similar epicentral distances. Both foundation conditions predicted the observed natural frequencies well for all four input motions. Use of the modified free-field motions gave the best prediction of peak acceleration, but tended to overpredict spectral accelerations at long periods. The peak base shear prediction was sensitive to inertial interaction and to the characteristics of the input motion. The study showed that reasonable results could be obtained with a mat foundation approximation. The paper provides a useful illustration of the influence of kinematic and inertial interaction on structural response. The circular mat foundation approximation was interesting; an indication of how its dimensions were determined for this irregularly shaped structure would be useful.

In their paper, *"Response of Two Dams in the 1987 Whittier Narrows Earthquake,"* **Boulanger, Bray,** and **Seed** use ground motions from two dams shaken by the 1987 Whittier Narrows earthquake to evaluate the accuracy of existing analytical procedures and to gain insight into the dynamic properties of the dam materials. Puddingstone Dam (L/H ~ 4.5), constructed in 1926 and 1927, consisted of sandy clayey silt with weathered shale fragments. It was instrumented with 18 accelerometers, 6 of which were located on the crest and downstream face with the remainder on "near" rock conditions close to the dam. Located at a distance of about 16 miles from the Whittier Fault, the "near" rock sites experienced peak ground accelerations of 0.04 g to 0.08 g. Two-dimensional FEM analyses showed good agreement between predicted and measured peak accelerations, but only fair agreement between predicted and measured response spectra. Cogswell Dam was constructed of rockfill to a maximum height of 280 ft in a narrow canyon (L/H ~ 2.1) 20 miles north of Whittier, California. Nine accelerometers located on or near the dam measured peak accelerations ranging from 0.06 g on an abutment to 0.16 g at the center of the crest. Analytical modeling was complicated by uncertainty in the dynamic properties of the rockfill and three-dimensional effects. A series of two-dimensional analyses indicated that the mid-crest response was best predicted with rockfill $K_{2,max}$ values of 150 to 180. Further approximations indicated that $K_{2,max}$ values of 100 to 125 would be more representative of the actual three-dimensional response. The authors concluded that Cogswell Dam was not amenable to accurate analysis by two-dimensional methods.

An automated, remote monitoring system installed at Wappapello Dam in Missouri 50 km northeast of the New Madrid Seismic Zone is described by **Hempen, Keim**, and **Mayo** in their paper "*Earthquake-Induced Parameter Automation.*" The dam was designed in the 1930s and completed in the early 1940s, resting on very loose to medium dense Point-Bar sands. The monitoring system includes 12 rapid-response piezometers installed in the Point-Bar sands beneath the crest, mid-slope, and downstream toe and 3 triaxial force balance accelerometers at the same elevation within a few meters of the piezometers. The system is designed to allow remote (by telephone) manual polling of all transducers, and to trigger an automatic, high speed data acquisition sequence upon exceedance of a 0.01 g threshold acceleration by any of the accelerometers. The sequence acquires data at a rate of 100 samples/second/channel during the event, then at progressively slower rates in a series of increments. Recording of other instruments (piezometers in potentially non-liquefiable strata) is simultaneously triggered. User-friendly control, analysis, and graphical display software is described. The system is designed to provide engineers with virtually real-time data on pore pressure increases in the potentially liquefiable soils and also on ground motion characteristics, thereby allowing the dam's stability to be evaluated in a timely manner. The authors do not indicate whether any triggering events have occurred since the system was installed or what the possible consequences of liquefaction might be.

ANALYSIS OF POTENTIAL SEISMIC HAZARDS

Various topics relating to the analysis of seismic hazards are described in four papers. One describes an extensive investigation of the response of gravelly soils at a potential nuclear power plant site to cyclic loading. Two describe methods for evaluation of liquefaction potential, one under seismic and one under static loading conditions. The fourth paper proposes an uncommon seismic hazard as an explanation of observed damage.

In a very interesting paper "*Gravelly Soil Properties by Field and Laboratory Tests,*" **Konno, Suzuki, Tateishi, Ishihara, Akino,** and **Iizuka** describe an investigation of the response of gravelly soils to cyclic loading. The investigation, conducted to aid in the evaluation of site suitability for nuclear power plant development, included large-diameter undisturbed sampling (by insitu freezing), triaxial testing of undisturbed and reconstituted specimens, and cyclic testing of very large insitu soil columns. The insitu freezing and sampling process used to obtain 30 cm diameter undisturbed specimens is clearly described and illustrated. Laboratory testing included undrained cyclic triaxial tests, consolidated-drained triaxial tests, and isotropic triaxial consolidation (with rebound) tests. The cyclic triaxial tests showed that the shear moduli of the undisturbed specimens were approximately twice as high as those of the reconstituted specimens, though the shapes of the modulus reduction curves were quite similar. Damping of the undisturbed specimens was somewhat lower than exhibited by the reconstituted specimens. The consolidated-undrained tests showed $\phi' = 36°$ to $37°$ and c' ranging from 0.25 to 0.67 kPa, even though the fines contents were very low. The field tests were particularly noteworthy. Two soil columns, each 10 m in diameter with heights of 5 m and 9 m, were constructed by excavating circular trenches that were later filled with water-filled rubber bags. To produce the desired stress conditions, thick concrete cap blocks were cast on top of each column and the water bags were pressurized. Torsional dynamic loading was applied by means of vibrators placed at opposite sides of the top of each cap block and operated out of phase. Slow cyclic torsional loads were applied by actuators connected to a large reaction block. The columns were instrumented with pore pressure, tilt, and settlement gauges. The dynamic tests produced shear strains of 10^{-4} to 10^{-5} and resonant frequencies were observed to decrease as the amplitude of the torsional load increased. Pore pressure accumulation was not observed at shear strains less than 10^{-4}. The insitu cyclic torsional tests produced shear strains up to 10^{-3}. Pauses in the cyclic loading sequence, coupled with its low frequency, prevented pore pressure accumulation in the highly permeable gravel, but permanent settlement was observed suggesting that pore pressures would have developed under undrained conditions. Comparing the results of field and laboratory tests, the remarkable agreement between the large-scale field tests and the laboratory tests on undisturbed specimens led authors to conclude that laboratory tests on high-quality undisturbed specimens could be used to accurately evaluate the cyclic response behavior of gravelly soils. This paper shows the type of detailed, comprehensive investigation that can be performed when adequate resources are available.

Cowherd, Miller, Perlea, and **Prakash,** in the paper "*Evaluation of Liquefaction Potential of Coal Slurry,*" describe a method for the evaluation of liquefaction potential of coal refuse that relies upon downhole nuclear density and moisture probes for evaluation of insitu soil conditions. Aside from the use of the nuclear probe, the method is conventional, based on the Seed and Idriss simplified approach. It is applied to a case history involving expansion of a coal refuse disposal site in West Virginia. Nuclear probe measurements indicated dry unit weights of 50 to 70 pcf, which translated to relative densities of 40% to 60%. Stress-controlled triaxial testing of reconstituted specimens was used to generate liquefaction resistance curves (based on ±5% limiting strain), and the anticipated cyclic stresses were computed by the simplified procedure. Based on a comparison of the anticipated cyclic stresses with the cyclic stresses required for liquefaction in an equivalent number of uniform stress cycles, a factor of safety of 1.6 was computed. Pneumatic pore pressure devices installed in the fine coal refuse showed that pore pressures dissipated quickly during refuse placement. Static limit equilibrium stability computations using seismically-generated pore pressures indicated a minimum factor of safety of 1.6. Though the method of analysis was quite standard, the use of the nuclear probe is of interest. A discussion of disturbance due to the installation procedure, and details regarding any special calibration of the probe to account for the mineralogy of the coal slurry would be useful. It would also have been helpful if the authors had commented on the consistency between densities measured by the nuclear probe and those inferred from other insitu tests (e.g. SPT, CPT, DMT, etc.).

Stoutjesdijk describes an evaluation of the flow slide susceptibility of a submerged sand slope near the Zeeland Bridge in the Eastern Scheldt estuarium of the Netherlands in the paper "*Liquefaction Study Eastern Scheldt Foreshore.*" Empirical rules based on slope height, slope angle, and sand density historically used for flow slide evaluation in the region are reviewed. Because the sand at the site was an old marine sand, rather than the young marine sands involved in 99% of observed flow slides in the region, it was analyzed in detail even though the empirical rules clearly indicated that it was susceptible. A stability analysis based on a failure criteria expressed in terms of the "eigenvalue of a matrix system comprising the entire system of stress-strain relationships" was performed. The nature of the matrix system and of the stress-strain relations themselves are not described or referenced. For stability, the quantity $d\tau_{xy}/d\gamma$ was required to be non-negative. Application of the analysis to the slope showed that the unstable zone had a reasonable location and orientation, and that its size increased with increasing slope height and slope angle. Based on a statistical characterization of input parameter uncertainty, a probabilistic analysis was performed. Assuming that no drainage could occur, and computed instability at any point in the slope would lead to slope failure, fairly high probabilities of failure were computed. The author's approach provided no evidence that the old marine sand should be more stable than the young marine sand. Since the sand behavior is characterized by its response in triaxial tests of "undisturbed" specimens, the beneficial effects of aging were probably reduced, if not erased, by the sampling process. It would be interesting to compare the shear wave velocity or some other measure of low-strain characteristics, that could properly reflect aging effects between the young and old sand under identical stress conditions.

In "*Ground Waving and its Damaging Effect,*" **Wang** discusses the implications of permanent ground deformations observed following earthquakes in Tangshan and other locations in the People's Republic of China. The author hypothesizes that several instances of damage were caused by permanent deformations associated with very large surface wave amplitudes. A discussion of the rotational components of surface wave motions is presented. The author contends that this type of damage cannot be resisted, and recommends that sites that have exhibited such damage in the past be avoided in the future. While the existence of rotational components of strong ground motion is well-established, the author has provided no proof that they were responsible for the observed damage. Indeed, examination of the damage photographs suggest a variety of phenomena (e.g. buckling, local compaction or slope failure, and liquefaction) that could very easily be responsible for the observed damage. A critical discussion of all of the possible causes of each case of observed damage is needed before the author's hypothesis can be accepted.

NUMERICAL MODELING

Two papers described numerical modeling of seismic behavior; one with respect to retaining wall response, and the other with respect to source mechanisms.

Al-Homoud and **Whitman** used finite element analyses to predict dynamic response of a centrifuge model of a gravity retaining wall in "*Comparison Between Finite Element Predictions and Results from Dynamic Centrifuge Tests on Tilting Gravity Wall Retaining Dry Sand.*" The centrifuge testing, conducted in a previous investigation, used 14/25 Leighton Buzzard sand placed against a hinged, tilting wall apparatus. The rotational stiffness of the wall could be varied by substituting different supporting springs; three different values (soft, medium, and stiff) were used. Wall response was measured by force, displacement and acceleration transducers. Analytical modeling was accomplished with the FLEX finite element code incorporating a viscous cap constitutive model whose parameters were evaluated from the previously reported results of laboratory tests on 120/200 Leighton Buzzard sand. Interface elements were used on all wall/soil interfaces and a transmitting boundary was placed at the end of the mesh. The analytical model was able to predict most aspects of the observed response well, and some very well. The maximum earth pressure was observed, in both the centrifuge and analytical models, to occur when the wall was at its maximum displacement toward the backfill; the opposite was observed for the minimum earth pressure. Peak accelerations at the top of the wall and on the ground surface were observed to lag those at the base. The location of the earth pressure resultant varied, reaching its highest point at the time of maximum earth pressure. The average absolute error between the measured and predicted values of response, such as wall displacement, increase i earth pressure resultant forces, backfill accelerations, and earth pressu resultant location was 26%. Interestingly, the average error of th displacement and acceleration parameters was substantially lower (b over 40%) than the force-related parameters. Additional parametri analyses emphasized the importance of tilting deformations of gravit retaining walls. The limitations of the analytical model were als discussed.

In "*Numerical Estimate of Tangshan Earthquake Damage,* **Loo** discusses the results of three-dimensional, dynamic, finite elemer analyses of fault rupture. To simulate permanent deformation an ground motion form the 1976 Tang shan (M=7.8) earthquake, a 50 kı x 50 km area 30 km deep was discretized into 594 elements. A 25-km long, 20-km-deep fault was simulated by planar joint elements. Ruptur was assumed to occur across the entire fault over a period of 0.3 sec For pure strike-slip motion of a vertical fault, displacements wer consistent with those computed by a dislocation model. For norma faulting on an oblique plane, the computed settlement of the hanging wall was greater than the computed uplift of the foot wall. Peal accelerations also attenuated faster on the footwall side of the fault partially explaining the greater damage that was observed on th hanging wall side of the fault. The author suggests that a combinatior of the two analyses would describe the observed fault displacemen which included both strike-slip and dip-slip components. The model car only be regarded as a very simple approximation to the Tangshan area, and the results can only be compared with the observed behavior in a qualitative sense. Comments on the advantages of the author's computational model relative to more recently developed and commonly used models (e.g. Green's function modeling, etc.) would be helpful.

SOIL IMPROVEMENT

Tsai, Chou, Chang, and **Wang** describe efforts at soil improvement of an 18 m thick layer of loose to medium dense silty sand at the site of a proposed station of the Taipei Metropolitan Rapid Transit System in their paper "*Jet Grouting to Reduce Liquefaction Potential.*" After considering a variety of soil improvement methods, the use of compaction sand piles was selected on the basis of economic considerations. Construction was suspended after installation of the first few piles due to complaints of excessive vibrations from a densely populated residential area near the site. A jet-grouting approach, based on 14-m-deep columns at 2 m spacing, was begun. After the initial group of jet grout columns was installed, no trace of cement grout was detected in verification borings. After adjusting the grouting pressure for each soil type, adding sodium silicate to accelerate the hardening process, adjusting the rate of grout pipe withdrawal, and predrilling through an overlying dense, cobbly layer, satisfactory results were achieved. The importance of verification testing and flexibility in construction procedures was clearly illustrated.

CONCLUSIONS

The papers presented in this session cover a wide variety of important topics in geotechnical earthquake engineering. They illustrate many of the impressive advances that have been made in this relatively young field. They also, however, illustrate many of the difficulties and challenges that still remain.

Geotechnical earthquake engineers are faced with problems involving extremely complex materials whose properties can vary spatially and, in many cases, temporally. The earthquake-induced loading applied to these materials is also complex and variable. There is little hope that all of the details of any specific geotechnical earthquake engineering problem can be explicitly modeled; some degree of empiricism, or at least stochasticism, is inevitable. Advanced theories and computational models can go far toward minimizing the empiricism required in analysis and design, but there is a pressing need for additional well-instrumented, well-documented case histories with which they can be calibrated.

ISSUES FOR DISCUSSION

True case histories in geotechnical earthquake engineering, i.e. case histories that involve actual earthquakes and actual soil/structure systems, can be divided into two main types:

1. Those that address the *performance* of a soil/structure system based on observations made *after* an earthquake, and

2. Those that address the *response* of such a system based on measurements made *during* an earthquake.

Both types of case histories have great value; indeed, case histories of the first type from the 1964 Nigaata and Alaska earthquakes provided much of the impetus that led to the establishment of geotechnical earthquake engineering as a discipline. As quantitative methods of analysis have developed, case histories of the second type have taken on increasing importance. With both types, however, a number of questions may be asked.

With regard to the first type of case history, we might ask:

1. Which phenomena should be emphasized in studying case histories of performance? Where are the most glaring "knowledge gaps" that case histories of this type can address?

2. Which phenomena, if any, are documented sufficiently that the marginal benefit of additional case histories of performance is low?

3. Do typical case histories of this type define terms accurately enough? Papers in this section refer to observations of "liquefaction" ranging from sand boils to flow slides.

With regard to case histories of response, it is clear that our ability to acquire data has increased dramatically in the past decade, and it appears likely that it will continue to increase in the near future. The development of digital instruments, automatic data acquisition systems, and remote monitoring systems provide unprecedented capabilities for gaining the type of information on which detailed case history investigations should be based. With this in mind, we might ask:

1. How can geotechnical engineers promote the use of such advanced data gathering systems?

2. What parameters should be measured for various problems?

3. How do we rationally and objectively process and interpret the massive amounts of data that are likely to be generated in the future?