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General Report, Loma Prieta Earthquake Sessions

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CLASSIFICATION OF PAPERS

The seventeen papers received for this session may be classified into five general topic areas. The number of papers in each topic area is shown in parentheses.

1.	Seismic Ground Motions	(4)
2.	Liquefaction and Lifelines	(6)
3.	Structural Damage	(3)
4.	Performance of Earth Dams	(1)
5.	Performance of Landfills	(3)

SEISMIC GROUND MOTIONS

Baba, Nishigaki and Inoue present a numerical technique to analyze the response of a basin with multiple layers and irregular boundaries on a rock base subjected to a far-field seismic disturbance, and applied this technique to compute the reponse of the San Francisco Bay area to the 1989 Loma Prieta event. The solution is performed in the frequency domain, and the materials are assumed to be elastic. In the wave propagation approach, the incident and scattered motions are separated. The scattered motion is further divided into scatter of motion from the one-dimensional problem with layers of infinite extent, and scatter caused by the irregular boundaries. The resulting parts are then combined to be consistent with the boundary conditions. The authors represented the key Bay geologic units of Bay Mud, alluvial layers, beach deposits, and the underlying Franciscan formation in four layers with piecewise linear boundaries. The resulting model simulated an area 26 km wide by 180 m deep. In the wave propagation results, the authors found that severe amplification and longer duration of motion occurred in the softer surface soil layers. The most severe amplification was calculated to occur at frequencies in excess of about 20 (radians per second).

Ohmachi, Nakamura, and Toshinawa measured predominant periods (F_p) and ratios of horizontal to vertical spectral acceleration components (A_p) in microtremor measurements with portable surface equipment at several sites in San Francisco and related these values to damage levels at the sites. Measurements were made at severely damaged sites as well as nearby undamaged sites for comparison. At each measurement point, the three-component (two horizontal and one vertical) ground motion was recorded. The processing of the records was as follows: the record was divided into three 10-second segments; the Fourier spectra were computed for each segment; the spectral amplitudes were averaged for the three segments; finally, the ratios of horizontal to vertical spectral amplitudes were calculated (A_p). In looking at the Embarcadero and Cypress viaduct sites, the authors found little difference between the sites and suggest that the difference in performance may be due to structural design differences. The authors found similar trends in F_p and A_p at the Marina, South of Market and Mission Creek districts where liquefaction occurred. As a practical rule of thumb, the authors conclude that sites with higher A_p and lower F_p generally were the sites with the most severe damage, given equal structural design. The authors state, however, that the theoretical basis for this is still left unclarified.

Lew examined 91 strong motion records generated by the 1989 Loma Prieta event and found that in the near field, the vertical peak ground acceleration (vpga) sometimes exceeds the horizontal peak ground acceleration (hpga). The author notes similar instances occurred in the 1979 Imperial Valley event. But, with increased distance from the rupture zone, the vpga values were typically less than the hpga values. The author concludes that the practice of assuming vpga is about two-thirds of hpga appears to be satisfactorily conservative for distances greater than about 25 km from the fault rupture zone, but at distances less than 25 km, a vpga to hpga ratio of two-thirds to unity would be more appropriate. The author further examined the Fourier and response spectra from 33 mostly free field recording stations located 100 km or less from the Loma Prieta epicenter. He found that the average ratio of vertical to horizontal 5-percent damped response spectra was about unity at periods of about 0.1 sec, decreased to about 0.3 between 0.3 and 1 sec, and increased again to near unity at about 10 sec. The author cautions generalizing from these observations because a limited number of records were analyzed, all records were from a single event, and the influence of local site conditions had not yet been assessed.

Mizuno and **Abe** alert us that the measured values of maximum horizontal acceleration in the 1989 Loma Prieta event severely exceeded the design values that would be computed with the Association of Bay Area Governments (ABAG) procedures. The ABAG procedures provide equations for computing (1) magnitude and epicentral intensity as a function of fault type and rupture length, and (2) site intensity and peak ground motions as a function of distance and site conditions. The authors showed that the computed magnitude and near field rock response values agreed fairly well with the actual values. However, the computed values for the more distant motions and soil site motions were dramatically lower than the actual values. The authors further investigated ground motion propagation using a one-dimensional solution with an equivalent linear Ramberg-Osgood constitutive model. They simulated the Foster City-Redwood Shore site which consists of 210 m of soft silty clay, and propagated a nearby rock record through the soil column. They used viscous damping for some cases and hyteretic damping for others, and found that the best match to the recorded accelerogram was obtained using a low level of viscous damping, significantly lower than the damping typically used in such analyses determined from laboratory tests. The authors suggest that current equivalent linear wave propagation solutions may be overdamped.

LIQUEFACTION AND LIFELINES

Seed, **Dickenson** and **Riemer** provided a comprehensive report on the damage caused by this earthquake, mostly due to liquefaction and ground motion amplification, and compared the 1989 performance of these sites to their performance during the 1906 earthquake. The authors note that there were few surprises amongst the geotechnical lessons learned -- zones that were expected to liquefy did, and were documented as having liquefied in 1906. The occurrences of liquefaction throughout the affected area are consistent with the widely used Standard Penetration Test (SPT) based correlation for liquefaction resistance. Regionally, the liquefaction occurrences can be characterized as follows: Central San Francisco Bay - uncompacted hydraulic sand fills underlain by soft clay with SPT blowcounts N_1 = 7-12 suffered widespread liquefaction, loosely dumped sand fills with N_1 = 10-20 suffered occasional liquefaction, and alluvium with N_1 = 15-25 did not liquefy; Southern San Francisco Bay and San Jose - no major uncompacted hydraulic fills are in this area, and natural alluvial deposits in the area did not liquefy in 1989 but did liquefy during the stronger, longer motions of the 1906 event; and Santa Cruz and East Monterey Bay - widespread liquefaction of alluvial channel deposits occured due to strong shaking. The authors conclude with several key points: (1) the Loma Prieta event provided a wealth of new strong motion records and an inventory of case histories for testing of other in situ or laboratory methods of liquefaction assessment; (2) our remediation techniques apparently are working because compacted hydraulic fills did not liquefy while adjacent uncompacted zones did, but we still lack the ability to design cost-effective retrofit of structures and the sites on which they are founded; (3) planners and geotechnical engineers need to work closely together because there is a tremendous aggregate risk of damage in residential areas, to vital transportation links at airports, in harbors and on the ground, and the loss of utilities combined with fire hazard. Many buildings, transportation facilities and lifeline utilities failed even during this moderate earthquake with an unusually short duration.

Tokimatsu, **Kuwayama**, **Abe**, **Nomura** and **Tamura** performed surface Rayleigh wave investigations at five sites in the Marina District using a newly developed portable system. They used applied motions at the surface for short wavelengths and measured microtremors for long wavelengths to determine shear wave velocity (V_s) profiles for the five sites. The measurement locations spanned the old 1892 coastline in the Marina district, with site 1 on the western boundary between the liquefied and nonliquefied zone, sites 2-4 in the liquefied fill zone, and site 5 to the east outside the liquefied area. They found that the foundation could be characterized with three layers: layer 1 - the sandy fill had V_s = 110-135 m/s in the liquefied area and greater than 140 m/s outside this area; layer 2 - the recent Bay Mud had V_s = 165-185 m/s in the liquefied zone and about 235 m/s outside this zone; and layer 3 - the old Bay Mud and dense sandy strata had V_s = about 300 m/s. As indicated by V_s methods to assess liquefaction, the materials with V_s less than 135 m/s with fill thicknesses up to about 10 m are likely to liquefy in moderate to strong earthquakes. The zone of maxiumum structural damage occurred where V_s of the fill was about 120 m/s. Also, the recent Bay Mud was softer and thicker in the zone of maximum structural damage. Using SHAKE, the authors computed ground amplification ratios of about 2, which did nor vary significantly from site to site, but the fundamental periods did. The authors estimated site fundamental periods of about 1 second in the zone with the most structural damage and observed that this period corresponds quite well with the fundamental period for structures about four stories tall with soft first stories.

Rosidi and **Wigginton** studied the liquefaction potential of three sandy materials in the Marina district and related peak ground acceleration (pga) to the thickness of material that would liquefy and the settlements that would occur. The deposits studied were the hydraulic fill, the artificial (dumped) fill, and the Strawberry Island modern beach deposits. The authors make the excellent point that both the thickness and proximity to the surface of a potentially liquefiable soil need to be considered in assessing the seismic performance of a site. The authors provide representative soil columns and energy corrected SPT blowcounts (N_{60}) for the sandy materials. They computed that a pga of about 0.1 g would be sufficient to initiate liquefaction in all three sandy deposits, but about 0.2 g would be needed to develop liquefaction throughout the full depth of the hydraulic fill, about 0.23 g in the artificial fill, and about 0.3 g in the Strawberry Island deposits. For the estimated 0.21 g that developed in the Marina district, the authors computed the following settlements would occur: 175 mm in the hydraulic fill, 50 mm in the artificial fill, and 80 mm in the Strawberry Island deposit. The computed values compare quite well with the observed displacements in these materials which were 96-143 mm in the hydraulic fill, 11 mm in the artificial fill and 25-30 mm in the Strawberry Island deposits. The authors suggest that the full thickness of the sandy materials in the Marina district may not have fully liquefied and that greater damaging effects can be expected during large future earthquakes.

Bardet and **Kapuskar** examined the occurrence and effects of sand boils in the Marina District. They observed that sand boils occurred primarily under or at the edges of buildings rather than in backyards or in the streets. They also observed that the worst structural damage was to buildings at the boundary of the zone of liquefied foundation soils. They attribute this to a shock absorber effect that the liquefied materials and sand boils provided in the central portion of the area and differential displacement and ground amplification effects at the boundary of the zone. The authors give an excellent description of the limited past work on sand boils which indicates that (1) sand boils make their way to the surface by a process of cavity and channel formation in the material above the liquefied layer, and (2) the greater the thickness of the overlying layer, the larger and fewer the boils. The authors concentrated on the portion of the Marina district which received the most damage, mainly the the old lagoon and peripheral landfill. A total of seventy-four sand boils were recorded in the liquefied area and ten of these had volumes that exceeded 1 m^3. The largest had a volume of about 3.5 m^3, but 42 percent had a volume less than 0.2 m^3. The total volume of material observed in the boils was 37 m^3. The largest boils were systematically located about 100 m from the edge of the lagoon fill. The collapsed buildings were located at the periphery of the old lagoon, about 100 m away from the largest boils.

O'Rourke, Gowdy, Stewart and **Pease** examined the patterns of liquefaction-induced ground deformations and damage to water and gas pipelines that resulted from the 1989 Loma Prieta event, and compared this performance during a major earthquake with 1906 great earthquake patterns. They found that such damage was concentrated in the Mission Creek, Foot of Market, South of Market and Marina districts during both the 1989 and 1906 events, but the 1906 damage was more extensive. More than 50 percent of the total pipeline damage was concentrated in these four districts in the 1906 event, whereas more than 70, percent was concentrated there in the 1989 event. Based on data from this and previous earthquakes, the authors observed that an exponential correlation exists between the rate of cast iron pipeline repair (number of repairs per km, r/km) and site Modified Mercalli Intensity (MMI): a one unit increase in MMI corresponds to a log cycle increase in repair rate (VI and 0.01 r/km to IX and 10 r/km). In a computer simulation of the hydraulic pipeline network, the authors estimated that the auxiliary water supply system (for fire fighting) would be out of water in 35-40 minutes due to hydrant and main breaks. This was consistent with the field observation that there was no water in the system 45 minutes after the main shock. The field experience and computer simulation emphasized that: (1) independent power supply is essential for isolation values; (2) hydrant breaks have a substantial effect on water lost fom the system; and (3) the City's newly implemented portable water supply system provided flexibility in these emergency circumstances. In addition to the excellent description of lifeline damages, the authors also provide a detailed description of the sandy materials and methods of deposition in the Marina district.

Tokimatsu, Midorikawa, Kuwayama, and Abe provide a review of the extensive damage resulting from the 1990 Phillipine earthquake (Ms = 7.8, epicentral Rossi-Forel Intensity = VIII) which killed at least 1600 and injured over 3400 people, destroyed more than 25,000 and partially damaged about 60,000 homes, and displaced about 44,000 people. Extensive damage was also caused to commercial buildings and transporation infrastructure. Damage resulted from strong shaking and inadequately designed structures, and by ground failures, particularly liquefaction and landslides. The fault trace extended over 110 km with up to 6 m of left lateral movement. In the epicentral region, buildings constructed across the fault were damaged, while those nearby but not on the fault were undamaged. Intensity VIII damage occurred over a 20,000 km^2 area (about 100 by 200 km) surrounding the rupture zone. Liquefaction was widespread and caused damage to roads, embankments, bridges, wooden houses, and reinforced concrete buildings. Liquefaction-induced subsidence submerged several villages below sea level. The Port of San Fernando was rendered useless. Areas of fine sandy fill underlain by clay suffered severe liquefaction in this event, similar to the Loma Prieta experience. In these areas, severely damaged buildings were typically 2 to 4 stories tall with shallow footings, indicating that no consideration was given to mitigate liquefaction hazards in the foundation design. This is somewhat surprising since much of the affected area suffered similar or worse damage in the Intensity XIII earthquake of 1892. The authors compared damage to reinforced concrete buildings in this event with damage in Niigata in 1964. Like Niigata, more than half of the reinforced concrete buildings tilted by more than one degree, but settlement was typically between 20 and 50 cm, much less than Niigata. Similar structure-structure interaction effects were observed in this event as in Niigata 1964 and Loma Prieta 1989. In liquefied areas, the presence of adjacent buildings of similar size and weight increased the apparent foundation width, which decreases the amount of liquefaction-induced displacement. Hence, buildings located at street corners were more likely to suffer severe damage. Structure-structure interaction also resulted in damage to lighter weight buildings adjacent to more massive structures.

STRUCTURAL DAMAGE

Mahin provided a comprehensive description of the strucutral damage resulting from the 1989 Loma Prieta event. He observed that the damage to structures during this event reinforced well known lessons about vulnerable building types and key transportation links. Like most authors, he pointed out that since the earthquake-induced ground motions during this moderate event were lower than what can occur, this was not a full-scale test of the necessary design level. The consequences of this event emphasized the need for economical methods to retrofit existing facilities, lifelines and transportation infrastructure. This was the first major earthquake to hit the Bay area in 20 years. FEMA estimated the total structural damage exceeded $6.7 billion. There were 67 fatalities, more than 3700 injuries, more than 18,000 dwellings damaged or destroyed, several thousand other facilities significantly damaged, and about 10,000 people displaced. Numerous buildings are still empty as studies continue to determine technical and economic feasibility of restoring structural integrity. The author points out that the distribution of severe damage was mainly related to local site conditions, emphasizing the need for site-specific response assessment or defensive structural design tolerant of large, inelastic displacements. Wooden buildings either shifted off their foundations or suffered collapsed cripple walls. Main damage was to buildings 3 to 4 stories tall and soft first stories. Like Niigata and the Phillipines, buildings on street corners suffered more than those sandwiched between adjacent structures. In unreinforced masonry structures, parapets and walls fell onto people and other buildings below; some of these buidlings had inadequate retrofit measures implemented, and others suffered from hammering of adjacent structures. Some older reinforced concrete structures with good or retrofit shear walls performed quite well, but new construction on fill over Bay Mud were severely damaged with spalling and shear wall failures. Steel buildings were also damaged with buckled braces, panels and columns, and gusset plate yielding. The Loma Prieta event was an opportunity to assess the effectiveness of retrofit schemes at a moderate level. The author found that only some retrofit designs performed well, many did not, adding a new class of hazard, namely "the inadequately retrofit structure." Steel and reinforced concrete bridges also had problems, most notably the Bay Bridge, Struve Slough Bridge, and the Embarcadero and Cypress viaducts. **One big departure from past practice that the author notes is the unacceptability of loss of function, rather than just life safety in the required design or retrofit, due to the** unacceptably expensive, time consuming repairs that become necessary. The author cites the Governor of California's proclamation which asks that buildings be designed or retrofit to remain operational after the design event.

Gould and Ahn present a numerical procedure to analyze the response of a pile supported structure subjected to strong ground motion. The authors interpret the soil-pile-structure system as a mechanism for lengthening the effective period of a structure. They consider the incoming rock motions to be rich in high frequencies, thus lengthening the period should reduce interia forces. They idealized the Clarion Hotel for their numerical studies. First, they determined the vibrational characteristics of the building alone on a rigid base. Then they constructed a Winkler model with non-linear springs and dashpots placed in parallel; the springs were used to simulate soil nonlinearity and the dashpots were used to simulate hysteretic and radiation damping. The pile was modeled with beam-column elements with material and geometric nonlinearities. The nonlinear shear modulus for the soil springs was defined using p-y curves for the horizontal direction, q-z curves for the vertical, and t-z curves for the pile tip springs. The site foundation consists of about 200 ft of Bay Mud with about 6 ft of sandy fill on top. The free field fundamental period was estimated to be 0.8 to 1 sec. The authors varied the stiffness of the rotational spring at the base of the building/top of the pile cap to simulate a range of base conditions. From their study, the authors concluded that pile foundations can be designed to function as base isolators to reduce the seismic stresses imparted to the superstructure.

Included in this session was a paper by **Kateziotis, Koukis, Tsiambaos, Sabatakakis, and Zervogiannis** dealing with the structural damage that occurred in the 31 August 1989 earthquake in the City of Patras, Greece's third largest city. The damage resulted from a magnitude M_S = 4.8 shallow earthquake (focal depth 1 km) whose epicenter was located 5 km from Patras. The earthquake caused surface rupture about 1500 m long along a preexisting normal fault. The rupture caused serious damage to mainly residential buildings located within a 50-m wide zone along the fault. Detailed site investigations were performed, including geological mapping, drilling, and in situ and laboratory testing, and monitoring of horizontal and vertical movements. Low level seismic activity (M_S = 4.5 to 5.2) continues in the area, with three shocks in 1989 alone, all within 10 km of Patras, with epicenters located in the Gulf of Patras. Shortly after the August 1989 event, the aperture of the rupture was about 1 cm and the vertical displacement on the downthrown block was about 5 mm. A few months later, these displacements had increased to 3 cm aperture and 2 cm vertical drop. The authors provide a very detailed description of the site conditions and the geodetic methods and procedures used to monitor the deformations. The presence of the fault had been obscured by recent residential development, but was observed in old aerial photographs of the area. The authors point out that had the fault been observed, the ACT and French regulations would have prevented siting of important buildings in the area. The authors conclude that this field experience validates the existing regulations which require that structures nearby active faults need special consideration.

PERFORMANCE OF EARTH DAMS

Only one paper in this session dealt with earth dam performance during and immediately after the Loma Prieta event. **Harder** gives an excellent, comprehensive description of earth dams shaken during the event, their performance and any remedial actions taken following the earthquake. There were 111 earth dams within 50 miles of the fault rupture. The majority are essentially homogeneous in section. The dam heights ranged from less than 10 ft to about 350 ft, and 21 of the dams were constructed prior to 1906. Peak ground accelerations at the dams were estimated to range from 0.05 to 0.6 g. At the time of the Loma Prieta event, most of the reserviors were relatively low, and some were essentially empty. The 21 dams which had been built prior to 1906 performed well during the 1906 event and also during the 1989 event. There were 5 hydraulic fill dams within 50 miles of the epicenter. All of them performed well. Minor longitudinal cracking was observed at the crest or upstream slope in two of the hydraulic fill dams. Since the reserviors were so low, this event was only a low level test of the seismic performance of the 5 hydraulic fill dams.

Harder points out that no dams composed completely of rockfill were located within 50 miles of the fault rupture, but many had large zones of rockfill. All of these dams performed safely. Longitudinal cracking occurred at Newell Dam, thought to be caused by settlement, and was repaired with compacted material. Harder provides a summary table of the performance of 35 dams and fairly detailed descriptions of 14 case histories including repairs that were implemented after the earthquake. Only two dams suffered even moderate damage, namely Austrian Dam and Soda Lake. Austrian Dam suffered extensive settlement and cracking and damage to the concrete spillway. Extensive repairs had to be implemented. Soda Lake is a complex of 4 embankments that retain granite tailings and sometimes water. The granite tailings liquefied as a result of the Loma Prieta event, and a wedge-shaped slump developed in one of the embankments thought to be caused by granite tailings erroneously left in place during construction. This dam was taken out of service rather than repaired.

Harder further described the ground motions and responses at the dams. Strong motions were recorded at 8 dams, which, as the author points out, provided an excellent opportunity to calibrate dynamic response techniques. Harder provides a plot of peak transverse crest acceleration versus peak transverse base acceleration for the 8 dams. The data indicate fairly high amplification at low base acceleration levels (typically between 2 to 4), and decreasing amplification at higher base motion levels (less than 2 for base motions greater than 0.2 g) possibly due to damping of the motions or yielding of the embankment materials. The author makes an excellent contribution by proposing an envelope to the embankment response data from this and previous earthquakes to serve as a guide for verifying the results of dynamic response computations.

PERFORMANCE OF LANDFILLS

Three papers dealt with the performance of landfills. **Johnson, Lew, Lundy, and Ray** provided an overview of the performance of ten landfills shaken during the Loma Prieta event. They found that in general all ten landfills performed very well, and what little damage occurred was in the form of minor cracks and settlement. The authors estimated that the pga at the landfills ranged from 0.04 to 0.5 g. No slope failures occurred, even with slopes as high as 250 ft and inclinations as steep as 2:1. The highest ground motions were estimated for two landfills nearest the epicenter. At these two landfills, minor cracking occurred at contacts between dissimilar materials or where there were changes in geometry, such as benches. The authors attribute the good performance of the landfills to the damping of the motions by the relatively loose trash, the reinforcement of the fill by the stronger materials in the variable trash, and the flexible nature of the slope face. The authors point out the large density contrast between the trash at 25 to 75 pcf and the soil and rock adjacent to it at 100 to 120 pcf. The authors state that such trash fill can absorb or dissipate the energy near the interface much as a base isolator, hence the slope face never experiences high vertical and lateral forces. The authors further draw an analogy between reinforced earth slopes and landfill trash slopes. The landfill trash slopes are known to contain an abundance of reinforcing material (lumber, branches, plastic, cloth, wire, and metal) interwoven with weaker material, and the authors thus expect similarities in performance.

Buranek and Prasad compared the observed and computed slope performance of 6 landfills shaken by the Loma Prieta earthquake. The 6 landfills were constructed on foundations that ranged from rock (3 sites) to alluvium (2 sites) and soft soil (1 site). The authors estimated average pga (average of the two horizontal components) at each site by considering a combination of recorded data and seven established attenuation relationships. They found that calculated pga values from established attenuation functions compared well with observed values for sites more than 20 km from the epicenter; however, the calculated values for sites less than 20 km away were significantly lower than the recorded values. Consequently, they determined motions at near field landfills primarily from recorded data, and far field motions were determined from a combination of calculated and recorded values. In general, all 6 landfills performed well; 4 had minor cracking. At 2 sites, ground motions were sufficient to displace on-site trailers off their foundations. The authors performed Newmark-sliding block type analyses for each of the 6 landfills. They estimated refuse strength parameters from results of backcalculations of past slides in these mixture materials. Yield accelerations were determined for circular surfaces applied seismic load (k_{max}) was estimated to be equal to the base acceleration. All 6 landfills were about 100 ft tall with slopes of about 3:1. In all cases, small to no Newmark-type permanent displacements were computed to result from the applied ground motions, which is consistent with the field observations. The authors conclude that this type of displacement analysis is an appropriate tool for assessing landfill stability, but considerable judgement is required to interpret the results.

Sharma and Goyal provided a detailed study of one particular landfill sited on reclaimed marshland in Richmond, CA. This landfill contains both hazardous and sanitary wastes, but in separate areas. The authors had extensive laboratory and field data about the site including the fill properties. They performed SHAKE analyses using the Yerba Buena record attenuated to 0.06 g as the input motion and computed 0.18 g as the pga at the top of the foundation soil column. They then performed sensitivity analyses applying this ground motion to the refuse fill and varying fill height, shear wave velocity and unit weight of the fill. The resulting pga in the fill did not change significantly over the range of parameters studied. The results indicated that the greater the refuse thickness, the lower the peak surface acceleration. The authors attribute the lower pga values to the inherent energy absorbtion mechanism of the refuse. The liquefaction analysis indicated that 1 to 3 ft thick silty sand layers in the foundation would liquefy due to the input motions, but that settlements would be less than 1.5 in. This is fairly consistent with the field observations. After the earthquake, vertical deformations observed in the fill were very small, and no trends were apparent. Newmark-type permanent deformation analyses indicated such deformations would be small, less than 3 in.; in the field, lateral deformations of up to 0.2 in. were observed in slope inclinometers.

COMMENTARY ON LOMA PRIETA PAPERS

The papers from this session demonstrate the great progess our profession has made towards assessing the potential for earthquake damage and the talent that abounds for finding practical, meaningful insight into actual field behavior from simplistic engineering models calibrated to actual field performance. As geotechnical engineers, we did well in this earthquake; damage occurred where it was expected to occur based on analysis and 1906 earthquake performance. The experiences in the 1989 Loma Prieta and 1990 Phillipines earthquakes emphasize that liquefaction is still our biggest problem because bearing failures, lateral spreading, differential displacement, subsidence, and lifeline and transportation disruption are incurred. The analyses presented consistently show that empirically-based procedures that relate SPT blowcounts to liquefaction resistance are the most reliable means for determining liquefaction potential. The overwhelming importance of these correlations were not fully perceived when they were proposed almost 20 years ago. At that time, the profession was still focusing on analytical methods and laboratory testing. Since then, simplified analytical procedures and improved correlations have been developed for assessing liquefaction potential.

Another area where we do well is the estimation of site specific response. Although there are gaps in our knowledge when it comes to seismic behavior of unusual materials such as refuse fills, the papers presented demonstrate that using simplistic one-dimensional equivalent linear wave propagation models, we can estimate with considerable accuracy the ground motions that will occur at a specific site. But, the shortcoming is that we need to know the input motion and site stratigraphy and properties to compute accurate response results. The biggest uncertainty that remains is the exact nature of the incoming ground motions our site will experience. The papers that discussed ground motions and response emphasize the limitations we still face in estimating peak horizontal and vertical vibrations in the near field and in the far field. These papers also demonstrated the ingenuity and creativity exercised to evaluate site stratigraphy and material properties (for example, response to microtremors and backcalculation of strength, stiffness and damping properties from field observations). The damage to dams, for example, came primarily from (1) a site characterization flaw -- unexpected material present in the foundation, and (2) settlement, differential displacement, and cracking at soil-concrete interfaces. Similar problems arose in the landfills at changes in materials and geometry. These mechanisms of behavior are beyond what we usually analyze and design against. This raises the question, are we designing for all the important mechanisms?

A distinct change in approach is the departure from simply assessing gross stability to estimating deformations and applying acceptable levels of deformation as the failure criteria. We seem to be able to avoid gross stability failure with current, albeit simplistic, stability analyses; however, there is now a shift from simply preserving life safety to also preserving operational functionality during and after the earthquake. Our analytical and empirical approaches need to become more finely tuned to the necessity for designing within tolerable deformation levels. The papers point out that basic gaps exist in our ability to truly represent (simply and practically) three dimensional problems; among them are response of basins like the Bay area and Mexico City, and on a smaller scale, the deformation and stress response of structure-pile-foundation systems.

The Loma Prieta event provided a unique opportunity to test at a moderate level the retrofit measures taken at various sites to improve foundation and structural performance. It was gratifying to observe that in situ improvement of hydraulic fills by densification was successful in preventing liquefaction, whereas adjacent, uncompacted material did liquefy. Methods of site improvement available to us now seem to be effective, but very costly. The performance of retrofit structures was more variable -- some performed well and some failed tragically, introducing "a new class of hazard, namely the inadequately retrofitted structure." Hopefully, with all we have learned from this and past earthquakes, we will have time to work more closely with structural engineers and planners to avert serious losses caused by future earthquakes.