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INTRODUCTION

Eleven papers were submitted to this session. These papers covered a wide range of interesting topics, and may be divided into the following general categories:

Pile foundations	2 papers
Lateral resistance plate load tests	
Dynamic tests on friction piles	
Demolition-induced ground vibrations	1 paper
Earthquake site investigation	3 papers
Study of a school site	
Seismically-induced landforms in New Madrid	
Foundation renovation for old building	
Liquefaction	5 papers
Earthquake reconnaissance in Nepal-Bihar	
Damage to small earth embankments	
Dilation and cyclic behavior of sand	
Caisson sea-wall movement during earthquakes	
Effect of lateral spreading on buried pipes	

The above papers include the results of site testing and observation, full-scale testing and measurements, centrifuge model studies, and literature survey investigations. Each paper will be briefly summarized in the following sections.

PILE FOUNDATIONS

Sakata, Matsui, Maeda, and Ochiai, investigated the influence of pile diameter on lateral soil stiffness in the paper "*Lateral Loading Tests in the Pit for a Large-Diameter Deep Pile*". In pile design, lateral soil resistance is often represented by a ground reaction coefficient that depends on foundation width. The conducted experimental investigation was intended to evaluate the dependence of lateral resistance on pile diameter. The tests were executed at the location where large diameter piles were to be placed as a bridge abutment foundation. Lateral resistance was investigated by performing lateral plate load tests near the base of a 14m deep cased pile shaft. The pile shaft diameter was 2.5m. At the test location (13m depth), the casing was removed, and a hydraulic jack was placed to push a loading plate towards the exposed lateral soil surface. The jack transmits its force by reacting against an anchor plate placed along the shaft wall diagonally across from the load plate. Four tests were conducted with plate lengths of 0.5m, 0.8m, 1.5m and 2.0m (along the shaft circumference). The load-plates were curved to conform to the shaft curvature, and were all of the same aspect ratio of 2:1 (plate length to plate height). In each test, the plate was subjected to a number of loading-unloading cycles of increasing amplitude up to the yielded state (low tangential relation between load and displacement). For the largest plate (2.0m by 1.0m), the yield load was 350 tons. Based on the recorded response, a relationship for dependence of lateral sub-grade reaction coefficient and a measure of pile diameter was derived. This relationship showed a decrease in sub-grade reaction with the increase in pile diameter. However, the observed decrease was significantly milder as compared to the current Highway bridge specifications in Japan (the latter being in-line with elastic theory predictions). The results were found useful in estimating expected lateral bridge deflection during construction in a cantilever configuration.

Sevilla, Gilroy, and Jenkins present a paper entitled "*Results of Dynamic Testing on Friction H-Piles*," that is not related to geotechnical earthquake engineering, but deals with a dynamic testing program in the evaluation of allowable pile capacities and soil setup. The paper reports on the design and installation of pile foundations for support of two taxiway bridges at the Mid-America Airport, near St. Louis, Missouri. Typical practice in the area is to use H-piles or closed-ended pipe piles driven to bedrock for end bearing. Nevertheless, the pile foundations for the taxiway bridges (designed to support a 1.25 million pounds aircraft) included heavy HP steel piles designed as friction piles embedded in stiff clayey silt till. During the preliminary design, it was determined that the wide, heavy bridges would require a large number of foundation elements for the bridge piers and abutments. It was also determined that high capacity piles or drilled shafts would be economical because of the heavy live loads and the presence of high lateral loads due to high abutments and seismic loading. Drilled shafts were eliminated from consideration because of water-bearing layers of sand within the glacial deposits. A pile drivability study was conducted as part of the preliminary design using the program WEAP. The analyses also allowed a comparison between the axial load carrying capacities using static methods and the ultimate capacities generated by WEAP. HP pile sections were selected over other pile types due to difficulties in driving sections with larger cross-sectional areas or open- and closed-ended pipe piles. The final design was to use HP piles driven to about 15 feet above the top of rock. The choice of friction piles bearing in the stiff glacial till rather than end bearing in the rock resulted in significant cost and times savings. A Pile Driving Analyzer (PDA) was used and the data was analyzed with CAPWAP. The HP sections were well suited for this project due to lower resistance during driving and high soil setups. Restrikes of the piles after setup obtained after a waiting period of only 48 hours gave final capacities ranging from 132 to 180 percent greater than the capacities measured during initial driving, thus reducing the need for pile splicing. The use of test piles can reduce the uncertainties where variation in penetration resistance is expected and gives flexibility in managing the pile driving and may result in lower costs.

DEMOLITION-INDUCED GROUND VIBRATION

Joshi and Lee, in their paper "*Ground Motion Measurements from the Demolition of Steel Towers*," present the results of ground motion predictions and measurements from the felling of some twenty-four 125-foot-high steel extraction towers. This paper, although not on the topic of geotechnical earthquake engineering, does presents some insights on demolition, testing, and instrumentation. The site of the towers is within the Savannah River Site in South Carolina at a decommissioned heavy water plant. The towers were in an area where buildings, an above ground steam line, and buried piping were within 50 to 200 feet of the towers. Demolition of the towers by felling in a cantilever mode was found to be more economical than removing sections of the tower cylinders in a piecemeal fashion. To determine the impact of the felling on the neighboring commodities, weight impact data collected from soil compaction experiments were evaluated. Weights were dropped from a crane on virgin soil and the ground velocities in three orthogonal components were recorded at forty-three locations. The energy input was an order of magnitude less than that for the towers. Peak particle velocity was computed at each station and the peak ground acceleration was also estimated. The peak particle velocity as a function of distance from the drop location was used as the basis for the ground motion predictions. The test drops showed variability of the peak particle velocity with distance from test to test. The recordings indicated that the vertical and radial peak ground velocities were about the same; both the vertical and radial components were much larger than the transverse component. The upper bound of the attenuation of the peak component velocity scaled well with $1/R^{1.3}$. The lower bound of the attenuation scaled well with 1/R. For determining the energy scaling for the actual towers, it was assumed that the peak particle velocity scales as the square root of the applied energy. For prediction purposes, the kinetic energy was taken at a point source and equal to the total potential energy. Scaling factors were determined for the extraction towers relative to the dynamic compaction test results and predictions of the peak component velocity were made. The peak vertical and radial components were assumed to be equal, and the transverse component was taken to be about one-half of the vertical component. The ground motion predictions for the felling of the towers indicated that the neighboring commodities should not be adversely affected. Ground motions were measured during the felling of two of the extraction towers for validation and calibration of the predictions. The measurements from the felling of the towers had reasonable agreement with the predictions when one-fourth of the total potential energy of the towers was used; this corresponds to a one-half decrease in the predicted ground motions. Video monitoring of the two tower drops showed evidence that tower support pedestals absorbed significant energy before the towers began to fall. There was no noticeable structural damage for any of the neighboring commodities near the drop sites. This paper illustrates clearly that the use of the total potential energy is a conservative approach; applications to other forms of demolition by felling will depend on the mechanisms available to absorb energy. A testing program using smaller scale weights was shown to be very useful.

EARTHQUAKE SITE INVESTIGATION

Kasman, Lew and Elliott describe the use of various geotechnical and geophysical investigation techniques in "*Investigation of Ground Cracking at the Van Gogh Street School, Granada Hills, California.*" The school suffered significant cracking in slabs on grade and walls of the school buildings and the surrounding paved areas during the January 17, 1994 Northridge earthquake. The school had suffered similar damage during the San Fernando earthquake, in 1971. An investigation was conducted to evaluate the surface cracking and to develop an opinion as to the cause of distress to the school buildings from a geologic and geotechnical perspective. The school site is located in a narrow valley on a sloping alluvial fan at the mouth of a canyon. The site is underlain by a thin veneer of artificial fill, which are underlain by Holocene and Pleistocene alluvial sediments. At depth, the site is underlain by a Plio-Pleistocene formation. Within the canyon, the major geologic structural feature is a syncline where the beds within the Plio-Pleistocene age formation have been folded in a broad syncline. The syncline has been mapped as traversing the southern portion of the school site; another mapping places the syncline in the central portion of the school. The syncline is roughly parallel to an active fault zone located about one-half mile away. The exploratory program was developed to evaluate the potential fault rupture or other fault-related hazards at the school site. The investigation included a variety of exploratory tools because any single method would not be sufficient to provide an adequate evaluation by itself. Detailed surface mapping of the ground cracks and shallow trenching of the upper subsurface materials was performed to evaluate whether the observed surface cracking occurred as a result of fault rupture. The location of the cracks was mapped and several zones of cracks were identified. Trenching of the upper soils revealed that the cracking did not extend into the soils and evidence of surface faulting was not found. Closely spaced cone penetration tests in several lines extending to depths of about 80 feet were performed in combination with exploratory borings where soil samples were obtained to determine general stratigraphic correlation and verification; from these results, soil profiles were developed. Geophysical surveys consisting of a combination of refraction and high-resolution seismic reflection surveys were conducted to attempt to identify faults and the deeper geologic structure below the depths explored by the CPTs and borings. The interpretation of the geophysical survey identified several suspected faults under the site within the bedrock materials and the syncline beneath the site was located with the survey. Interpretation of the results from the combination of investigation techniques provided the conclusion that although several suspected faults existed within the bedrock features from the deep geophysical surveys, the closely spaced CPTs and borings showed that faults did not disrupt the Holocene age deposits. Furthermore, the contact between the Holocene and Pleistocene alluvial deposits did not appear to be offset. The surface trenching had already shown that the observed surface cracking was not a result of faulting. From these results, it was further concluded that the observed surface cracking might have been due to tectonic uplift in the vicinity of the site and the folding and faulting at depth causing deformation of the upper surficial sediments. It was also concluded that although similar surface cracking could occur again in the future with other events that may be similar, the potential for fault rupture at the site was remote. This paper illustrates the utility of combining various investigative tools to evaluate sites with deeper geologic structures where a single investigative tool is not sufficient.

Knox and Stewart revisited the New Madrid seismic zone and identify a number of earthquake-induced land formations in the paper "*Morphoseismic Features in the New Madrid Seismic Zone (Central USA) and Their Implications for Geotechnical Engineering*". They define Morphoseismic as new landforms produced by earthquakes or pre-existing landforms modified by their action. Such landforms are affected due to a combination of liquefaction, secondary deformation, and slope failures. Most of these landforms evolved due to the series of earthquakes that occurred during 1811-1812, with modifications due to erosion, sedimentation, differential compaction and farming practices. In the paper, a detailed comprehensive diagram is presented showing the inter-relation between cause and effect leading to the various kinds of morphoseismic landforms observed today. Emphasis is placed on the significance of such landforms from the geotechnical-engineering viewpoint. An awareness of the existence of a particular morphoseismic landform and an understanding of its characteristics is necessary. Certain existing ground deformation mechanisms will be possibly reactivated due to future earthquakes in this area.

LIQUEFACTION

In the paper entitled "*Case History on the Foundation and Site Liquefaction Potential Evaluation for the Renovation of a One-Hundred Year Old Building*," **Magginas** describes the geotechnical investigation and evaluation for an existing four-story building with one basement, built of thick masonry walls and supported on shallow foundations. The paper deals with the foundation rehabilitation and seismic retrofit of an old and historic building subject to local architectural and historic preservation laws, and with restrictions to investigation and rehabilitation work conditions with minimal effects, such as vibration on neighboring structures. The case history is presented for a building located in Ljubljana, Slovenia. The overall condition of the building is described as being good with no signs of structural damage, significant cracks, or foundation-related problems. Originally designed as a residential structure, the usage of the building has changed to an office building and a partial library. The building is located in an UBC seismic zone 3. The proposed renovation consisted of upgrading the several structural components, adding a new stair tower, and retrofitting the building seismically, including new reinforced concrete shear walls. The geotechnical investigation consisted of two borings to 20 meters depth and four dynamic cone penetration tests to depths of 15 meters. Four pits were also excavated to the foundation level both inside and outside of the building. Young alluvial deposits consisting of inter-bedded layers of silty clay, silty sand and clayey gravel underlie the site to a depth of 20 meters. The young deposits overlie older alluvial gravel extending to about 40m, which in turn overlie permocarbonic slate. Groundwater was found at a depth of 12 meters below the ground surface with a perched water table at a depth of about 2.5 meters. Laboratory testing of "undisturbed" samples consisted of Atterberg Limits, moisture content, grain size analyses, direct shear and consolidation tests. An analysis of the bearing capacity of the foundation soils showed that the factor of safety on bearing capacity for existing foundations ranged from 1.7 to 2.4. After the renovation, the factor of safety on bearing capacity is reduced to range from 1.5 to 2.1. The new shear walls for the seismic retrofit were proposed to be placed and doweled into the existing bearing walls at the same elevation as the existing footings. A settlement evaluation based on the renovation loads and the new shear walls estimated that the total long term additional settlement of the building would range from about 1.0 to 1.6 cm. The new stair tower addition was supported on new drilled piers having a diameter of 60 cm and length of 20 meters. The choice of drilled piers rather than new footings included the following reasons: reduction of differential settlements between existing and new structures; large overturning moments from the seismic analysis; bearing pressures for shallow footings exceeded the design soil bearing capacity; and minimizing disturbance to existing building at the foundation level. An evaluation of the liquefaction potential indicated the possibility of overall site liquefaction (at significant depth) is unlikely to occur; however, a medium liquefaction potential exists in a shallow sandy clay layer if there is perched ground water. The paper describes the case history of a typical historic building retrofit, however, the issue of liquefaction potential does not appear to have been addressed in the seismic retrofit. Although the potential for liquefaction was determined, estimates of the liquefaction-induced settlement were not presented and a description of the possible consequences to the existing building and new addition were not presented. Also, mitigation strategies against liquefaction were not given.

Mukerjee and Lavania have performed an analysis of the influence of acceleration level, relative density, particle size and depth of soil on the liquefiability of soils using the three most commonly used approaches in "*Soil Liquefaction in Nepal-Bihar Earthquake of August 21, 1988.*" The Richter magnitude 6.6 event in 1988 caused widespread damage to residential houses and other structures in the border region of Nepal and Bihar State of India. In addition, there was liquefaction of alluvial deposits in large areas and landslides in the hilly areas of the region. This earthquake occurred in a well recognized region of high seismic activity where much of the activity follows the trends of the Main Boundary Thrust and the Main Central Thrust, the two tectonic features of the Himalayan range. The earthquake occurred in the same area of the estimated magnitude 8.4 Nepal-Bihar earthquake of January 14, 1934; a total area of about 46,600 square kilometers is reported to have been affected by liquefaction. The authors report on the analysis of the liquefaction using the 1986 approach of Iwasaki, the 1971 simplified Seed and Idriss approach, and the 1974 Chinese Building Code approach to determine the influences of the various factors. Since ground motion measurements are not available from the earthquake, it was assumed that the maximum ground surface acceleration at sites in Madhubani and Darbhanga, 60 and 90 km from the epicenter, respectively, were between 100 to 150 gals. These values are somewhat higher than would be predicted by a typical ground motion attenuation relationship. The results of the analyses using the three different approaches show that there is occurrence of liquefaction for the acceleration levels assumed, however, there was found appreciable difference with regard to the influence of relative density, mean grain size, and depth on the potential for liquefaction. The authors state that there is a need to carry out further research to narrow the differences and bring out a more unified approach. The authors have used liquefaction analysis approaches that are rather dated in the light of new developments in the evaluation of liquefaction. The use of standard penetration test data has been refined during the last three decades and there are many variables that must be accounted and corrected for. For the most recent consensus thinking on this matter, the proceedings of the NCEER Workshop on evaluation of liquefaction resistance of soils (Youd and Idriss, 1998) provides much valuable information and should be referred to. It also seems that the influence of the size or magnitude of earthquake may not have been accounted for in the analyses by Mukerjee and Lavania. The 1971 Seed and Idriss approach used was based on triaxial test data causing liquefaction in 30 cycles. It is commonly believed that this number of cycles corresponds to a magnitude 7.5 earthquake event; consequently, unless a correction is made to determine the equivalent cycles for a magnitude 6.6 event, the results may be may not be totally reliable. Similarly, the Chinese Building Code may also be calibrated to a magnitude 7.5 event and may not be directly applicable to the magnitude 6.6 liquefaction evaluation.

Iwashita, Matsumoto, Nakamura, Yoshida, and Yokoyama in the paper "*Factor Analysis of Damage to Small Earth Dams Due to the 1995 Hyogoken-Nambu Earthquake,*" present a comprehensive survey. This survey is focused on small earth dams, or dams that are less than 20m high. A total of 266 dams were investigated within about 50 km from the source fault, and/or in the identified high seismic intensity zones. Damage was classified into five grades ranging from no damage to severe damage such as complete failure. Assessment of damage level was based on extent of longitudinal and transverse cracks, settlement deformation, damage to appurtenant works, and leakage. The damage levels were investigated in relation to factors such as distance from fault, topography and geology, intensity of earthquake motion, direction of dam axis relative to the fault direction, year of completion, and height of dam. Consequently, this study is deemed to be comprehensive and noteworthy.

The investigators observed that overall damage decreased with the increase of distance from the fault. Longitudinal cracking was the most common form of damage. Earthquake intensity judged by tombstone overturning was positively correlated to level of damage in these embankments. In addition, dams on soft soils were observed to suffer more damage. Nevertheless, damage was mostly inflicted on old dams constructed before 1925. These dams were generally poorly engineered and constructed with steep embankment slopes. Most of these old structures were under 10m in height.

Finally, it was noted that dams constructed after 1950 were found to suffer minimal damage in all cases. This favorable finding attests to the resilience of modern small earth embankments under conditions of severe earthquake excitation.

Elgamal, Dobry, Parra and Yang presented a comprehensive literature survey of data on cyclic loading characteristics of liquefied clean sands and silts in the paper "*Soil Dilation and Shear Deformations During Liquefaction*". The data presented demonstrates the important role of soil dilation in controlling and restraining the magnitude of shear deformations during liquefaction of clean sands and clean non-plastic silts. Typical cyclic stress-strain results from triaxial and shear tests are shown to clearly manifest this dilative response. In addition evidence of this response is presented from shake-table, centrifuge, and actual earthquake recorded responses. Computational models developed to simulate the involved patterns of cyclic soil response are also surveyed. Finally, the framework of a newly developed computational model is presented. This new model is capable of simulating the important response cases associated with biased accumulation of shear strains due to the presence of an imposed locked-in driving shear stress (e.g., building foundation, near and below slopes and retaining structures, lateral spreading situations). Under such conditions, the relatively large biased cyclic shear strain excursions are apt to invoke a tendency for soil dilation and an associated instantaneous increase in soil stiffness and strength, which can effectively restrain the magnitude of cyclic shear deformations.

Sato, Watanabe, and Katayama in the paper "*Study on Mechanism of Caisson Type Sea Wall Movement During Earthquakes*," present the results of shake-table and centrifuge testing programs. The conducted shake-table and centrifuge tests were designed to investigate the Caisson accumulated displacement due to liquefaction of backfill and foundation soils. During the 1995 Hyogo-ken Nambu, Kobe earthquake, caisson-type sea walls were observed to undergo large displacements due to liquefaction below and behind the walls. This liquefaction was widespread in the reclaimed ground supported by these caisson-type retaining structures. Two different soils were employed in the conducted tests. Some tests were conducted using Toyoura sand, which is widely used for testing applications in Japan. Other tests under similar loading conditions were conducted using a mixture of 80% Toyoura sand and 20 "DL" clay. This soil mixture exhibits high liquefaction susceptibility and a lower tendency for dilation under shear loading conditions. In addition, the mixture is lower in permeability than Toyoura sand, an aspect that prolongs the pore-pressure dissipation phase after liquefaction. These mixture characteristics make it a more appropriate soil for shake table and centrifuge tests, by providing a more accurate simulation of the prototype soil characteristics. In the conducted tests, the caisson rested on: a) a stiff base (model A series), b) a loose liquefiable layer (model B series). The instrumentation included accelerometers, pore-pressure transducers, earth-pressure transducers LVDTs and gap sensors. A set of harmonic input excitations were used ranging from 0.05g - 0.4g in peak prototype amplitude. In all tests, recorded displacements were observed to occur during the shaking phase only (no additional displacement after shaking). Thus it was concluded that inertial effect of the caisson and supported backfill must both be accounted for in analysis procedures. The results also revealed an interesting pattern in the recorded earth pressure and caisson lateral acceleration. In this regard, the correlation between earth pressure behind the wall and caisson acceleration was observed to change phase with the increase in input acceleration and associated increase in excess-pore water pressure. This change in correlation was also associated with the caisson movement away from the supported backfill. Further investigations of this important problem are needed, especially for clarification of the possible evolution of large caisson deformations after the shaking phase.

Ozeki and Katayama studied the important problem of lateral spreading effects on buried pipelines in the paper "*On the Behavior of Longitudinal Strain of a Buried Pipe Subjected to Ground Spread Caused by Liquefaction Observed in Centrifuge Model Tests*". In a series of centrifuge tests, smooth and rough surfaced pipes were placed in saturated soil supported by a retaining structure near one of the pipe free ends. At the other end, the pipes were adjacent to the container rigid boundary, and were either free or else hinged to this boundary. A sand-clay mixture similar to that used in the centrifuge program described above was employed to increase liquefaction susceptibility, decrease the dilative soil tendency, and decrease effective permeability. The centrifuge container was about 1m in length (model scale). Cyclic dynamic excitation was imparted to the models, and the response was recorded by accelerometers, pore-pressure transducers, displacement sensors and soil pressure sensors.

During dynamic excitation, the soil was observed to liquefy and move outwards along with the supporting retaining structure. Movement of the retaining structure and nearby soil constituted a lateral spreading process that acted on the buried pipes. Pipe surface-texture was observed to have a significant impact on the resulting longitudinal strains. The smooth surface pipes experienced a much smaller level of induced longitudinal strain. Longitudinal strains in the rough-surface pipes were about 3 times as much. Pipe strains were observed to occur during the lateral spreading process, with peak values associated with its initial phase. During this phase the displacement accumulated quickly, with a gradual slow down in lateral spreading thereafter. This observation suggested a correlation between the rate of soil movement, and the level of induced strain in the pipe. In this paper, the investigators proposed a linear relationship between longitudinal strain in the pipe, and the square of ground-surface lateral-spreading velocity. Such a relationship might indeed to applicable within specific ranges of this phenomenon, and more research is needed to establish its overall applicability. It may be concluded that this study has addressed an important problem in a well thought-out fashion, and has produced valuable results.

REFERENCES

Youd, T.L., and I.M. Idriss [1998], "Proceedings, NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," National Center for Earthquake Engineering Research Technical Report (in press), Buffalo, New York.