General Report — Session 9: Model and Full Scale Tests of Geotechnical Structures Including Centrifuge Tests

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GENERAL REPORT ON SESSION 9: MODEL AND FULL SCALE TESTS OF GEOTECHNICAL STRUCTURES INCLUDING CENTRIFUGE TESTS

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1 Introduction

The importance of physical modelling of dynamic events is borne out very well in this session. There were 19 papers that are presented in this session, of which 9 papers were based on dynamic centrifuge model tests. Shaking tables formed the basis for 3 papers and 3 papers used field scale tests. Other papers dealt with analysis of earthquake input motion and other aspects of dynamic analysis.

A wide variety of problems were tackled in this session. These include lateral spreading of soil following liquefaction and their effects on pile foundations (two papers), seismic behaviour of water front structures (three papers), seismic bearing capacity of shallow foundations (three papers) and dynamic soil-structure interaction (three papers). Earthquake data analysis formed the basis of two papers. Also the boundary effects in dynamic centrifuge modelling were investigated in one paper.

Overall a wide range of problems were tackled in this session and the quality of papers is very high. In the following sections we attempt an objective summary of selected interesting aspects of each of the papers in this session. To maintain objectivity, any paper associated with one of the session reporters was summarized by the other reporter

2 Paper 9.06 Haigh, S.K., Madabhushi, S.P.G., Soga, K., Taji, Y. and Shamoto, Y.,: *Newmarkian Analysis of Liquefied Flow in Centrifuge Model Earthquakes*

This paper describes six centrifuge tests on sloping 20% relative density sand. The slope angle was varied between 3, 6 and 12 degrees. The authors used a Newmark sliding block model to predict the slope displacements during shaking. The authors used the measured pore pressure time history at some point in the soil layer to calculate a yield acceleration function that varied due to pore pressure generation and dissipation. Using the measured base accelerations, they predicted surface displacements observed in the centrifuge with an error less than about 33%. This was considered to be better than the predictions obtained with the "much more complex MLR method of Youd and his co-workers." It was noted that the MLR method gives predictions within a factor of two. Of course the MLR method is performed without knowledge of the pore pressure and base acceleration time history, and with considerably more uncertainty regarding initial soil properties.

The authors also present Fourier Spectra and wavelet transforms of the base and surface accelerations. The base motions in their experiments consisted of a 50 Hz motion with significant harmonics at 100, 150, and 250 Hz. During sliding the 50 Hz motion was amplified while the higher harmonics of 100 Hz, 150 Hz and 250 Hz are attenuated significantly. The modified Newmarkian analysis with variable threshold (or yield) acceleration based on pore pressure information, could predict the attenuation of higher harmonics observed in the experimental data. The authors conclude that Newmarkian theory is a powerful tool with some basis in true soil behavior.

3 Paper 9.31 by Teymur, B. and Madabhushi, S.P.G. : *CPT Assessment of Boundary Effects in Dynamic Centrifuge Modelling*

The authors point out that while ESB container was designed to be equivalent to an ideal, linear soil, non-linearity causes an imperfect match between the soil behavior and the container behavior. The ESB container includes rough "shear sheets" along the container end walls; these walls are intentionally rough to provide for development of dynamic complementary shear stresses on the end walls. A potential boundary effect noted by the authors is that the shear stresses on the container end walls could restrict settlement (and hence densification) near the boundaries.

Using a systematic series of tests including dense and loose, saturated and dry homogeneous sand layers, CPT soundings at varying distance from the end boundary were used to assess boundary effects before and after shaking the models on the centrifuge. Prior to shaking, the CPT resistance seemed reasonably independent of the distance between the CPT sounding and the end wall. Due to shaking on the centrifuge, however, the soil in the middle of the container appeared to densify while the soil near the end walls appeared to loosen. The authors attribute the boundary effect to friction on the end walls.

While this hypothesis sounds believable and correct, the authors did not systematically prove their assertion that the boundary effects are caused by non-uniform densification due to friction on the shear sheets. The cone resistance may be affected by lateral earth pressures that could also be non-uniform and could change during shaking. Furthermore, densification could be non-uniform due to inexact equivalence of the "Equivalent Shear Beam" container. This is an interesting, fact-filled, paper that convincingly quantifies the non-homogeneity of the soil that develops due boundary effects in the ESB container. More work is required to absolutely define the mechanism of development of the non-uniformity.

4 Paper 9.16 by Lin, G. : *Similarity Rule for Dynamic Model Tests of Geotechnical Structures*

An interesting and simple hypothesis put forward in this paper is that near failure, the dynamic amplification factor in earth and rock fill dams approaches 1, despite heterogeneity.

Safety evaluations of 80 thousand earthquake and rock fill dams in China is of great concern. Model test data could be a valuable tool to study earth dam behavior. The author defines the "similarity rule of elasticity" as: (time scale factor = length scale factor times the square root of (density scale factor / shear modulus scale factor): $\lambda_t = \lambda_L \sqrt{(\lambda_\rho / \lambda_G)}$. Without resorting to a centrifuge, it is difficult to simultaneously satisfy the similarity rule of elasticity and the "similarity rule of gravity" $\lambda_t = \sqrt{\lambda_L}$. The paper then presents a clever argument that 1-g model tests to investigate failure conditions do not necessarily need to satisfy the similarity rule of elasticity. This conclusion is based upon the observation in 1-g model tests, field recordings, centrifuge model tests and analyses, that resonant amplification of acceleration is negligible near failure. All of the cases cited indicate that near failure, the acceleration amplification factor for dams and embankments tends to one near failure, irrespective of heterogeneity.

The argument that dynamic amplification of peak acceleration approaches one near failure is convincing. The reporter was left wondering, however, if the same could be said for dynamic amplification of velocities and displacements. Furthermore, even if peak accelerations are uniform, the phase angle of accelerations may be distorted if the two similarity laws are not obeyed.

5 Paper 9.21 by Chazelas, J., Abraham, O and Semblat, J. : *Identification of Different Seismic Waves Generated by Foundation Vibration in the Centrifuge: Travel Time, Spectral And Numerical Investigations*

The authors conducted model tests in which they dropped a mass onto a footing and studied travel times of the ensuing waves. Rayleigh waves were recorded by a horizontal array of accelerometers along the ground surface; P-waves were monitored by a vertical array of accelerometers beneath the footing; and, another array of accelerometers at a 45 degree angle, was sensitive to a mixture of Rayleigh and P-waves.

Based on measured travel times and the elasticity theory, they were able to deduce a Poisson's ratio, $\nu = 0.25$. They also found by regression that the P-wave velocity increased with overburden pressure according to $V_p \propto (\sigma'_v)^b$, where b varies between 0.2 and 0.28. The authors also performed finite element analyses to characterize the container and these results confirmed the experimental results.

The authors presented some interesting details about signal processing and accounted for curved ray paths in their analyses. They presented a new way to view the data from pairs of orthogonal accelerometers by plotting the trajectory of dynamic displacement. For accelerometers along a diagonal (45 degrees off the footing centreline), as the P-wave arrived, the accelerometers moved nearly along the diagonal direction. After the P-wave, the S- and Rayleigh waves caused a transverse displacement of the accelerometers. By this technique they could clearly distinguish the travel times for P-waves and S-waves.

6 Paper 9.35 by Chazelas, J. : *Evaluation of the Shear Modulus in Models for Shallow Foundation Dynamics within the Elastic Domain"*

The aim of this paper is to perform model tests of a footing with impact loading to examine the relationship of the equivalent homogeneous shear moduli (used in impedance models) with the stresses under the footing. Rigorous procedures involving FEM or BEM analysis are relatively complex and hence analogues such as the cone model by Meeks and Wolf and the lumped parameter models by Wolf have advantages in simplicity and can provide insight. However, the simpler models require verification by model test data. The author uses impact loading in the experiments and argues that this is preferable to the use of continuous vibrations to excite the model foundation. This reduces the pollution of the response by wave reflections off the container boundaries. For the impact loading tests (which were in a large container) they found that treating the boundaries of the container with "Duxseal" had little effect on the noise in the data. Duxseal is a material that has been used by various other research groups to absorb wave energy at the container walls.

The author observed that tests in the container with Duxseal had a larger shear modulus, G, than tests in the container without Duxseal. The G value was determined by a least squares fit to the observed "mobility function". The author then attempted to correlate the back-calculated equivalent homogeneous G with $\sigma_o'^{0.5}$, and $\sigma_u'^{0.5}$, where σ'_o is the minimum mean effective stress beneath the footing centreline, and σ'_u is the average footing pressure. He found the correlation between G and σ'_o to be better than the correlation between G and σ'_u.

The author observes quite a large scatter in the correlations. One source of scatter could be the assumption in equation (6) that $K_o = \nu/(1-\nu)$ with $\nu = 0.25$. This relationship produces a calculation of K_o = 0.33 whereas K_o= 1-sinϕ produces a value closer to 0.5. Furthermore, it is possible that the Duxseal boundaries affect the K_o value. Because Duxseal is a visco-elastic material, it does not control the lateral strain $\varepsilon_3 = 0$; thus it may violate the conditions for 1-D compression. The relatively soft visco-elastic Duxseal could provide a lateral "fluid" stress at the boundaries as described by Campbell et al. (1991). This would cause a discrepancy with the assumption that $K_o = \nu/(1-\nu)$. Use of a correct and more certain value of K_o could improve the correlations presented in the paper.

7 Paper 9.33 Sato, M., Ogasawara, M. and Tazoh, T. : *Reproduction of Lateral Ground Displacements and Lateral-Flow Earth Pressures Acting on Pile Foundations Using Centrifuge Modeling*

Results of two model tests of sheet pile quay walls with liquefiable backfill are reported. One of the tests was performed to look at displacements of the walls and backfill due to liquefaction of a saturated layer of 50% relative density sand alone. The other test had similar ground and sheet pile wall but included a pile group foundation in the backfill.

In the experiments, particular attention was paid to assessment of the lateral pressures on the piles caused by the ground movements. Measurements included the bending moments at several locations on the piles and earth pressures on the pile cap and on the piles. The results compared well to each other, supporting the validity of earth pressure measurement, which is often considered unreliable. It would have been valuable if the details of earth pressure sensor were presented. It would have been useful to present the deformed shape of the piles (by integration of curvature distribution) and the lateral force on the piles (from differentiation of bending moment distribution), at least qualitatively, to show that these measurements are all consistent with each other.

A key conclusion of this work was that the measured earth pressure was smaller than that from the Japan Road Association's Specification for Highway Bridges. The specification assumes that passive pressures are developed during lateral flow.

8 Paper 9.30, Kitazume, M., Kikuchi, Y., Masuda, K. and Hashizume, H. : *Dynamic Centrifuge Tests on Sea Revetment with Multi-Anchors*

A common seawall design in Japan involves placement of heavy caissons on a gravel mound on the seabed. Due to several records of large earthquake-induced lateral deformations of these structures, a new design concept is studied. The new concept involves a slender, lighter caisson with anchor tiebacks. Centrifuge model tests were conducted to determine the effect of tieback length on 2-dimensional model retaining walls. In the tests reported, the backfill consisted of dense dry sand. Thus the models resembled a relatively slender gravity retaining wall with anchor tiebacks. In the future, the study will be extended to include saturated and liquefiable soils.

As expected, the performance improves as the tieback length increases. The authors measured and presented time histories of tieback force and earth pressure. They also compared the measured earth pressure to that based on Mononobe-Okabe theory. The authors observed that the maximum observed Mononobe-Okabe pressure falls between the measured dynamic maximum and minimum pressures. But, the Mononobe-Okabe theory predicts maximum pressure under active conditions and in the experiment, the maximum pressure occurs under passive conditions.

The authors point out that to maximize the anchor capacity, the anchor length should extend beyond the failure surface. And they found that only the shallow anchors were found to reach their pullout capacity. Pseudo-static stability analyses are presented which provide reasonable estimates of the earthquake acceleration observed to cause large deformations in the experiments.

The authors explain some earth pressure measurements by taking account of the wall friction on the sides of the container. The models were only 100 mm wide, but were 600 mm in length. The reporter recommends that wider models could be used to reduce the importance of arching on the model behavior. The observed failure mechanism due to earthquake loading bears significant resemblance to that observed in centrifuge model tests involving reinforced soil as reported by Howard and Kutter (1998).

9 Paper 9.18 Wei, X., Fan, L. and Wu, X., : *Shaking Table Tests of Seismic Pile-Soil-Pier-Structure Interaction*

In this paper, the Pile-Soil-Pier-Structure Interaction problem is investigated in a series of shaking table tests. The models were constructed in a large "rigid" wood box filled with loose sand. The box was 4 m deep and had a plan area of 3.3 m by 0.8 m. 40 cm thick styrofoam sheets were placed at each end of the box to prevent wave reflection from the sides of the box. The authors do not explain how the styrofoam was designed to permit the sand column to deform as a free field shear beam.

One series of tests involved a 3.5 m deep layer of sand with no piles or superstructure. They compared the accelerations at different depths in the experiment with those calculated using SHAKE and FLUSH. They found that the calculations agreed with the measurements and therefore concluded that the box effectively simulated free field conditions. Unfortunately, the authors did not clearly explain how they determined the soil properties (shear modulus and damping as a function of strain) for their analyses. Without this information, it seems likely that the measured amplification in the soil column is due to a combination of dynamic shear response of the soil and dynamic bending of the soil/container system.

Other test series included a superstructure supported on a column (pier) above the pile cap. The number of piles was varied, and the number of columns in the superstructure was varied (single pier or twin pier superstructure). The authors conclude that a double pier superstructure is superior to a single pier superstructure. This conclusion could change depending on the natural frequency of the superstructure relative to the frequency content of the ground motion. A double pier superstructure would be stiffer and would have a smaller natural period than a single pier superstructure. For a deep deposit of loose sand, the site period will be large, and hence it would be advantageous to install a stiff superstructure with a short natural period.

10 Paper No. 9.34 by Janoyan, K., Stewart, J.P. and Wallace, J.W., : *Preliminary Test Results for Full Scale Drilled Shaft Under Cyclic Lateral Loading*

This paper presents preliminary results from a field test of a full-scale, 6 foot diameter CIDH shaft/column that is similar to bridge supports used by the California Department of Transportation (Caltrans). Extensive instrument-ation was used to measure strains, curvature, soil pressure, and deflected shaft/column shape. Cables, attached to anchors at a distance of 80 ft from the column applied inclined lateral loads to the top of columns which extended 40 ft above ground. Cyclic lateral loading of different amplitudes was applied by the cable/anchor/ actuator mechanism which is nicely explained.

Yielding of shaft reinforcement occurred when the top of the column was subject to between 12 and 24 inch displacement cycles. Between 24 inch and 108 inch displacements, the stiffness gradually degraded as the amplitude of the cycles increased, but each cycle reached a peak load of about 310 kips. The second cycle at 108 inch displacement resulted in a significant loss of strength as hoop reinforcement fractured, and the longitudinal reinforcing bars buckled and fractured. Crushing of concrete was prominent four to five feet below the ground where a plastic hinge formed. The authors observed passive wedges forming in the soil around the base of the column. They summarize some observations obtained by inspection of the concrete shaft after excavation around the pile.

The authors plan, in the future, to use the data to check the parameters used in typical p-y analysis procedures. A concern in design practice is the degree to which commonly used p-y curves (obtained from tests on relatively small diameter piles) can be applied to large diameter CIDH shafts. This paper provides a tantalizing preview of some very interesting data that will become available as it is processed and analyzed.

11 Paper 9.23, Iiba, M., Tamori, S. and Kitagawa, Y., : *Shaking table test on effects of combination of soil and building properties on seismic response of building*

Shaking table tests were conducted at 1 g to study dynamic soil-structure interaction. The paper focuses on quantifying the resonant frequencies and amplification factors for buildings with different height, foundation types and soil types. Different building configurations represented 8, 11 and 15 story buildings, and different foundation types included spread footings and pile foundations.

The soil in the models was an artificial material "made of poly-acrylamide and bentonite, etc.". It appears that this combination of materials enabled the researchers to produce model soil deposits with shear wave velocities of about 14 to 25 m/s so that the natural frequency of the soil deposit could be adjusted according to the scaling laws for 1 g shaking table tests.

Data regarding acceleration amplification, dynamic earth pressures on the pile cap and bending moments in the piles is also presented. Almost all of the response data is presented in terms of ratios: (bending moment/base acceleration), (earth pressure/base acceleration), (foundation acceleration/base acceleration), etc.. The reported acceleration amplification factors ranged between 20 and 150: very large amplification factors. It was difficult to find any reference to absolute stress or acceleration; only the ratios were presented. Apparently, the models were all loaded using small amplitude excitation so that there was negligible non-linearity and therefore, the amplitude is irrelevant.

The authors present several nicely organized figures with a very high information content. These figures clearly illustrate mode shapes and natural frequencies, and amplification factors. The results also indicated that the bending moments in the piles were affected by the building response and by the kinematic loading of the soil.

12 Paper No: 9.10 Martinez-Carvajal, H., Taboada-Urtuzuastegui, V.M. and Romo, M.P., *Analysis of Some Downhole Acceleration Records from "Central De Abasto Oficinas" Site at Mexico City*

This paper deals with the analysis of surface and downhole accelerations obtained from the "Central De Abasto Oficinas (CAO)" site during the 09-10-1995 event. The paper describes an analytical method to obtain the soil stiffness as a function of shear strain amplitude.

The N-S components of the surface and downhole accelerations are first used to obtain the shear stresses at mid depth between the location of the accelerometers. The shear stress at any position is calculated using linear interpolation of accelerations recorded above and below the position. The next step is to double integrate the acceleration traces to obtain the absolute displacements. These absolute displacements are then used to obtain the shear strain at the mid depth between the location of the accelerometers. The shear stress is plotted against the shear strain for the mid depth points between the locations of accelerometers. The soil stiffness is obtained by fitting ellipses to the shear stress-shear strain curves and by considering the slope of the line joining the extreme points of the ellipse. The results clearly show the increase in soil stiffness with depth, i.e., the deeper the location, the steeper the major axis of the ellipse. However the strain dependency of the soil stiffness is not picked by this analysis i.e. at large the strain amplitudes one would expect to see a deterioration in soil stiffness. This deterioration is not being picked up by this analysis, possibly due to limited range of shear strains that were caused by the 09-10-1995 event.
The shear moduli obtained by the above procedure were used to calculate shear wave velocities which were compared to those obtained by Ps logging carried out at the CAO site. These comparisons were considered to give satisfactory results when the plasticity index of the soil strata is taken into account.

The system identification method described in this paper is a simple and useful technique to obtain the shear stress-shear strain characteristics of soil strata using the measured acceleration traces at site. However as linear interpolation is being used between accelerations measured at locations which are separated by relatively long distances the calculated shear stresses and shear strains are inherently 'averaged out' over the soil strata between the location of the accelerometers.

13 Paper No: 9.37 Kurose, H., Sato, M., Azuma, H., Ozeki, K. and Yoshida, N., *Effective Stress Analysis by Shear Strain Controllable Model and its Application to Centrifuge Shaking Model Test*

This paper describes an effective stress based FEM code which is able to control the growth of shear strains in the soil. The constitutive law used for liquefaction analyses of sandy soils needs to capture the excess pore water pressure generation during undrained events as well as the shear strain growth. To achieve this, the authors propose a change to the traditional shear modulus degradation curve by means of a parameter 'c'. The rate of increase of shear strain due to rise in excess pore water pressure is controlled by selecting a suitable value for 'c'.

The performance of the constitutive law is first compared to experimental results from an undrained torsional shear test. These comparisons show that the parameter 'c' does not affect the excess pore pressure generation but only the growth of shear strains.

The constitutive law is then used to analyse results from a dynamic centrifuge test on a model caisson retaining saturated sand. A simple sinusoidal motion is used as earthquake input motion. The FEM results could pick up the drop in acceleration amplitude after the first few cycles heralding the onset of liquefaction. The computed excess pore pressure histories compare well with those observed in the centrifuge test. However, the horizontal displacement of the caisson is either under-predicted (with c=0.25) or over-predicted (with c=0.45). The authors speculate that this may be due to the densification of soil during the centrifuge test as the centrifugal acceleration is applied to the model. It would have been useful to have back calculated the value of 'c' that gives the correct prediction of horizontal displacement ($0.2 < c < 0.45$).

14 Paper No: 9.17, Choi, J., Park, I. and Kim, S., *Numerical Analysis of Saturated Sand Under Dynamic Loads*

This paper describes the use of a constitutive model based on Disturbed State Concept (DSC) to predict the build up of excess pore water pressures and consequent softening of soil under dynamic loads. The DSC model assumes that the soil moves from a relatively intact (RI) state to fully adjusted (FA) state under the action of the applied loading. Under the RI state the soil behaviour is elasto-plastic with hardening behaviour and HiSS yield surface is used. The FA state is akin to the Critical State where plastic deformation can occur at a constant volume and with no further increase in stress. A disturbance factor is calculated to estimate the fraction of the soil mass that is in FA state and consequently the fraction of soil mass still in RI state.

The authors seek to validate the numerical scheme based on field test data obtained from Inchon Airport in Korea where a 10 ton hydraulic hammer dropping on the ground surface provided the dynamic loading. Field data was obtained in the form of excess pore water pressure time histories at a depth of 5 m below the ground surface. This data indicates clear build up of pore water pressures on the drop of the hammer and also small oscillations indicating possibly the rebound of the hammer on the ground. This pore pressure is dissipated until the next hammer blow occurs. The frequency of hammer drops is approximately 1.25 Hz. The data indicates accumulation of excess pore water pressure with every hammer blow.

The numerical simulation using DSC based constitutive model predicts build up of excess pore water pressure under applied dynamic stresses. However, the dissipation of the excess pore water pressure is not captured well. Also the attenuation of vibrations caused by hammer rebound in the field data is very quick. The numerical prediction has excess oscillations that seem to increase with time (and with every hammer drop), thereby clouding the excess pore pressure generation. In the view of the reporter the DSC model may have very little material damping in it which can cause this spurious result. Also the diffusion of the excess pore water pressures is not captured well by the numerical simulation.

15 Paper No: 9.29 Lee, J., Choi, I., Seo, J and Cho, Y., *Refining Historical Earthquake Data Through Modelling and Scale Model Tests*

The authors wish to evaluate the performance of wooden framed structures under earthquake loading. The wooden frame structure represented structures built for use by commoners in 18th and 19th century Korea. Scaled models of these structures were tested on a shaking table. Two specific site conditions were chosen i.e. the structures were either on rock or on alluvium soil deposits, both of which formed most common types of foundations for these historic structures. Several earthquake motions suitable for the site conditions were used. For example, for the rock foundation site the input motion was from a rock site in Eastern Canada, while for soil site the authors use El-Centro Array No.5 acceleration as the input motion. The intensity of input motion was changed to mimic different epicentral distance of the site under different earthquakes. Vertical accelerations were applied which were 2/3rd of the horizontal accelerations. Spectral responses of the structures under different earthquakes were produced.

The main conclusions of the paper are that wooden houses at soil sites are more vulnerable to either collapse or severe damage. This is reasonable given that alluvium soil deposits lead to amplification of the bedrock motion. Even at rock sites the structures collapse when earthquake intensity reaches VIII on Modified Mercalli (MM) scale. The authors recommend that 0.25g of PGA for soil sites and 0.6g of PGA for rock sites as limiting accelerations for collapse of wooden framed structures of this type.

16 Paper No: 9.36 Horii, N., Toyosawa, Y., Tamate, S and Hashizume, H., *Centrifuge Model Tests on the Stability of a Clayey Ground Improved by Deep Mixing Method With a Low Improvement Ratio*

This paper deals with stability of rapidly constructed embankments on soft clayey soils. The ground improvement takes the form of cemented columns installed by Deep Mixing Method. The improvement ratio (ratio of the cross-sectional area of cemented columns to the plan area of the improved site) is kept to 10 %. The embankment in the centrifuge is constructed rapidly using an in-flight sand hopper.

In each of the centrifuge tests the location of improved site is changed from toe of the embankment to the shoulder of the embankment. Also the improved soil columns were inclined to vertical in some of the centrifuge tests. The displacement vectors were observed in each of the test. The centrifuge test results show that the least amount of horizontal and vertical displacement occurs when the cemented columns are placed at the shoulder of the embankment and are inclined to vertical at an angle of 20°. However, it does appear that even in this case there is a slip surface developing at the toe of the embankment.

The experimental results were also compared to the numerical results obtained using PLAXIS code. The numerical results under-predict both horizontal and vertical movements in the soft clay, compared to the centrifuge test results.

17 Paper No: 9.03 Al-Karni, A. and Buddu, M., *An Experimental Study of Seismic Bearing Capacity of Shallow Footings*

This paper describes a series of experiments conducted to evaluate the seismic bearing capacity of shallow foundations. The experiments were conducted on square and rectangular footings with or without embedment into soil. The seismic bearing capacity was derived in a previous paper by the authors in which the standard Meyerhoff's bearing capacity equation is modified to include the effects of horizontal and vertical accelerations.

The experimental results were used in a variety of ways. The most interesting one is the critical acceleration. The critical acceleration is taken as the acceleration that initiates the vertical movement or rotation of the footing. The experimental results do not compare well the theoretical calculations (in fact the authors point out that they match more closely with those obtained using Richards et al, 1991). The authors also comment on the absence of development of any clear failure surfaces. The experimentally observed deformation profiles did not match any theoretical surfaces obtained from the methods in the literature.

The authors also comment on the shear fluidisation of the soil when subjected to lateral accelerations. Following Richards et al (1991), the slip planes in the soil appear when $\frac{K_h}{1-K_v}=\frac{\sin\phi}{2}\left[(\sin\phi-3)(\sin\phi-1)\right]^{0.5}$ and a state of general shear occurs when $K_h=\tan\phi$ (assuming $K_v=0$) when the soil would behave like viscous fluid. The authors argue that these set the limits on the critical acceleration regardless of the safety factors to which the shallow foundations were designed.

18 Paper No: 9.12 Al-Karni, A., *Shear strength reduction due to excess pore water pressure*

In this paper the author attempts to derive mathematical expressions to calculate the reduction in shear strength of soil due to excess pore water pressure build up (either due to earthquake loading, blasting or any other source such a breaking of buried pipe). He uses the standard Mohr-Coulomb expression to calculate a ratio of reduced shear strength due to excess pore water pressure build up to the original shear strength. This ratio is called the stability factor. In the absence of any cohesion in the soil, this is nothing but one minus the excess pore water pressure ratio $\left(1-\frac{u_{excess}}{\sigma'}\right)$. Using the Mohr's circle he also derives mathematical expressions for the effective stress in the soil. The author suggests that a reduction factor could be included in the seismic bearing capacity equations by calculating a reduced friction angle and a reduced interlocking.

The concept of stability factor is inaccurate as the reduction in shear strength of soil due to rise in excess pore water pressure can be handled by the effective stress principle alone. The rise in excess pore water pressure in the soil is due to the tendency of the soil to undergo volumetric changes under the application of a shear stress. This behaviour cannot be incorporated in the stress space alone using Mohr's circles. Furthermore the behaviour of soil under cyclic loading cannot be captured by the Mohr-Coulomb envelope. The existence of Phase Transformation (PT) lines below the Critical State (or Failure) Line were well established in the literature (Ishihara et al, 1974, Luong and Sidaner, 1981) which show that the soil behaviour changes when the stress path induced by cyclic loading reaches the PT lines. It is important to capture the complete cyclic behaviour of the soil before one can understand the bearing failure mechanisms of shallow footings under earthquake loading.

19 Paper No: 9.15 McCullough, N.J., Schlechter, S.M. and Dickenson, S.E., *Centrifuge modelling of Pile Supported Wharves for Seismic Hazards*

This paper deals with the centrifuge modelling of pile supported wharves subjected to earthquake loading. The authors consider the particular case of wharf construction that involves placement of an embankment on foundation soil and backfilling behind the embankment. On the seaward side of the embankment piles are driven and the pile cap is placed to form the wharf.

The centrifuge modeling is carried out with the aim of understanding the dynamic interaction among the piles, embankment and the foundation soil. This is important for the following reason. The collapse based designs often are very expensive to implement in large scale projects such as seismic retrofit of wharves in ports. Many port authorities therefore wish to allow for performance-based designs where certain amount of deformation will be allowed. Centrifuge modeling therefore offers an excellent tool to validate the proposed seismic retrofit and also understand its performance under different earthquakes of varying intensities.

The centrifuge testing of a typical cross-section of the pile supported wharf was carried out at 40g using the UC Davis large centrifuge facility. Soil improvement was also attempted in the clayey soil in a grid pattern around the piles. This was achieved by mixing the clay (San Francisco bay mud) with 13.7% cement.

The centrifuge test results clearly show a reduction in the magnitude of excess pore water pressures in the models with soil improvement. Also the vertical settlements were much smaller in these models. Bending moments in the piles clearly increase with increasing earthquake intensity and were found to be maximum just below the clay-dense soil interface. The authors do not elaborate on the centrifuge results but conclude that the soil improvement clearly reduces the seismic hazards to the wharf structures.

20 Paper No: 9.09 Sekiguchi, H., Kim, H. and Kita, K., *Shaking table tests on Seismic Deformation of Composite Break Waters*

This paper deals with centrifuge testing of composite breakwaters. Composite breakwaters suffered excessive settlement of up to 2.5 m during the Kobe earthquake of 1995. The composite breakwaters take the shape of a rubble mound placed on the foundation soil. The rubble mound supports a caisson structure. Following the earthquake loading the foundation soil can liquefy causing the rubble mound to settle into the foundation soil. The paper considers two possible mechanisms. The first mechanism deals with accumulated axial strains under shear stresses generated by cyclic loading. The second mechanism deals with the penetration of the rubble into liquefied soil and associated settlement due to mass flow as described by Peires et al (1998). In the current experimental work, the side walls of the container were made of Perspex and hence displacement of rubble can be observed through these walls. High speed CCD camera images were obtained during the earthquakes (at more than 1000 frames per second) to see the development of the vertical settlement of the mound into the foundation soil and lateral spreading of the mound (horizontal extension).

The centrifuge test data clearly shows that following the earthquake loading large suctions develop in the foundation soil right below the rubble mound-foundation soil interface. This is interpreted as strong dilative behaviour of the soil in this region. Deep below the mound as well as at large lateral distance from the rubble mound large positive excess pore pressures were observed confirming the contractile behaviour of the soil in these regions. These results match with those observed earlier with Peires et al (1998). The experimental results also show that when the particle size of the mound material is small relative to the foundation soil particles, the overall settlement of the caisson structure increased substantially. This is due to the rubble mound material dispersing (mass flow mechanism) into the foundation soil. This accounted for nearly 20% of the settlement of the composite breakwaters for the particles sizes used in this study.

This is an interesting paper that forces us to move away from continuum mechanics when considering problems like the composite breakwaters that have two materials with very different particle sizes. Particulate mechanics consideration will be appropriate in these types of problems.

References:

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