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A COMPARATIVE STUDY ON VERTICAL SETTLEMENT OF A MULTI STORIED BUILDINGS SUPPORTED BY PILE GROUPS EMBEDDED IN VARIOUS SOILS BY ETABS SOFTWARE

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ABSTRACT:

The shaking table test is purposely designed to confirm the ability of the numerical substructure technique to simulate the SSI phenomenon. A model foundation-structure system with strong SSI potential is embedded in a dry bed of sand deposited within a purpose designed shaking-table soil container. The experimental system is subjected to a strong ground motion. The numerical simulation of the complete soil-foundation- structure system is conducted in the linear viscoelastic domain using the substructure approach. Model simulations of a series of large scale seismic soil pile-superstructure tests conducted on the 6.1m x 6.1m the multidirectional shaking table are used to calibrate and verify the proposed numerical technique. Detailed comparison with measured structural and 'free field' soil response in the shaking table tests show that the proposed formulation gives very good descriptions of maximum spectral accelerations and frequency response. The model accurately captures the transition from kinematic and inertial response of these 'simple' systems.

I.INTRODUCTION

In the recent decades, the city blocks that contain clusters of closely spaced buildings have come forth in the world. During earthquake events, the radiation energy would be emitted from a vibrating structure to other structures through the soil. There will have influence on the dynamic characteristics and the earthquake response among the closed spaced buildings, which calls Structure-Soil-Structure Interaction (SSSI). Notably, for structures located in dense urban environments composed of the city blocks, the assumption of buildings being isolated from each other is invalid, and can lead to erroneous results. Therefore, consideration of the interaction of soil, foundations, and structures requires a more holistic approach. Study on the SSSI is of great importance to predicting the seismic responses of structure exactly and have become a hotspot in the fields of earthquake engineering research. Some interrelated research has received much attention in the theoretical studies and numerical analysis Developed a boundary element method (BEM) model for considering interaction between adjacent, massless foundations. developed a lumped mass model to consider ground-motion induced interactions between adjacent, massless foundations. developed a numerical, hybrid model to investigate the dynamic interaction systems submitted to time-harmonic loads. used ANSYS program to simulate two steel moment frames with concrete shear walls on three types of soil. Utilized FEM-BEM method in frequency domain to analyze including pile foundations Besides, some experiment and field observation also have been conducted. Clarified actual phenomena of the SSSI by a series of experiments such as forced vibration tests and

earthquake observations for a full-scale building and a model structure The Nuclear Power Engineering Corporation (NUPEC) carried out forced vibration tests and shaking table tests using models of reactor buildings and adjacent structures from As part of a collaborative program, make calculation study on the tests using the SASSI program and FEM-BEM method. analyzed the data from two adjacent 16.3m seven layers steel structure and the soil which is the certain depth from the basement of a structure nearby in the Whittier Narrows earthquake in the United States. More recently, some work have been done on analyzing the influence of large groups of buildings, as well as that of site effects due to subsoil configuration, on the seismic response of the overall system by means of several experimental and numerical models However, limited by facilities and cost, current studies have been mostly focused on theoretical study and calculation analysis, and little experimental work has been conducted. Actually, many theoretical outcomes were difficult to guide practical engineering due to the lack of test validation.

Soil-structure interaction:

A summary of the SSI principles and guidance is reported in NIST the complete soil-structure interaction can be modelled using two different approaches: (i) a continuum-based model or (ii) a substructure-based approach. a schematic illustration of the main features that should be taken into account for soil-structure interaction analysis using a continuum approach (also called direct method). In this method, the non-linear behaviour of each part of the model can be simulated, by using appropriate constitutive models.

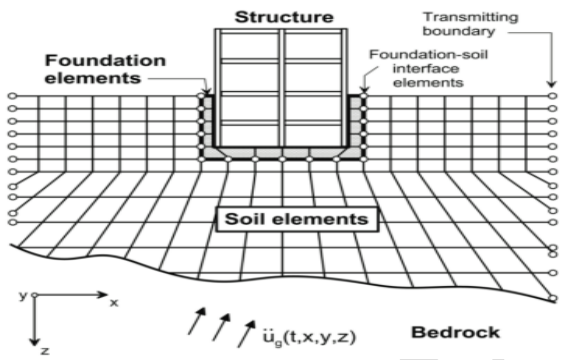


Figure: Schematic illustration of a direct analysis of soil-structure interaction using continuum modeling by finite elements

The substructure approach (also called indirect method). The idea of this method is to summarize the soil response with springs and dashpots that are calibrated to represent the continuum response. According to this approach, the input imposed to the structure In general, building-soil interaction consists of two parts; kinematic and dynamic (or inertial) interaction. The former is a result of wave nature of excitation and is manifested through the scattering of incident waves from building foundation and through filtering effect of the foundation that may be stiffer than the soil and therefore may not follow the higher frequency deformations of soil. This interaction depends on frequency, angle of incidence and type of incident waves, as well as shape of foundation and on the depth of embedment. It develops due to presence of stiff foundation elements on or in soil cause foundation motion to deviate from free-field motions. The later is due to inertia forces of building and of the foundation which act on soil due to contact area. And it depends on the mass and height of the building and the mass and depth of foundation, on the relative stiffness of soil 2 compared with the building and on the shape of foundation. It develops in structure due to its own vibrations which gives rise to

base shear and base moment, which in turn cause displacements of the foundation relative to free field. Dynamic analysis of soil-structure interaction can be done using

- Direct Method
- Substructure Method

Direct Method:

Direct Method is one in which the soil and structure are modeled together in a single step accounting for both inertial and kinematic interaction. Inertial interaction develops in structure due to own vibrations give rise to base shear and base moment, which in turn cause displacements of the foundation relative to free field. Kinematic interaction develops due to presence of stiff foundation elements on or in soil cause foundation motion to deviate from free field motions.

Sub-Structure method or Multistep Method:

Sub-Structure Method is one in which the analysis is broken down into several steps that is the principal of superposition is used to isolate the two primary causes of soil-structure interaction, inability of foundation to match the free field deformation and the effect of dynamic response of structure foundation system on the movement of supporting soil.

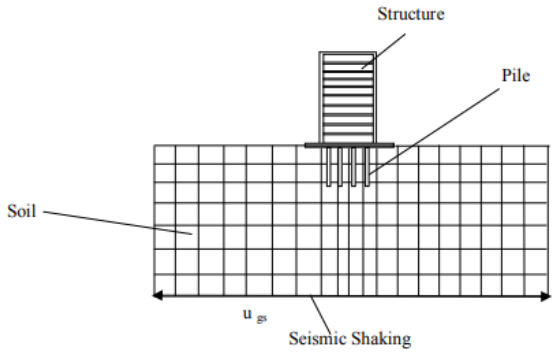


Figure: Direct Method of Soil-Structure Interaction

Kinematic interaction:

n The deformation due to kinematic interaction alone can be computed by assuming that foundation has stiffness, but no mass as shown in Figure

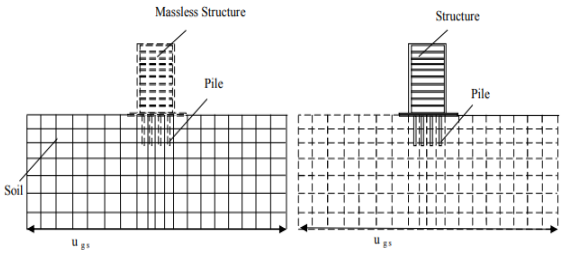


Figure: Substructure method of Soil Structure Interaction

Inertial interaction:

The structure and foundation do have mass and this mass cause them to respond dynamically. The deformation due to inertial interaction can be computed from the following equation of motion

The right side of the above equation represents the inertial loading on the structure-foundation system. This inertial loading depends on the base motion and the foundation input motion, which reflects the effects of kinematic interaction. In the inertial interaction analysis, the inertial loading is applied only to the structure; the base of the soil deposit is stationary. The solution to the entire soil-structure interaction problem is equal to the sum of the solutions of kinematic and inertial interaction analysis Generally, in modeling the infinite media problems, two complementary regions can be distinguished; namely the interior (i.e., a neighborhood of the structure encompassing heterogeneities, irregularities and nonlinearities) and the exterior (typically a horizontally layered medium extending to great extent, usually assumed infinite, distance from the structure).

Pile Foundations:

Pile foundation is a popular method of construction for overcoming the difficulties of foundation on soft soils. But, until nineteenth century the design was entirely based on experience It is only too convenient for an engineer to divide the design of major buildings into two components: the design of the structure and the design of foundations. But in reality, the loads on foundation determine their movement, but this movement affects the loads imposed by the structure; inevitably interaction between structure, foundation and soil or rock forming the founding material together comprise one interacting structural system Significant damage to pile supported structures during major earthquakes (such as 1906 San Francisco earthquake, 1964 Niigata and Alaska earthquakes) led to an increase in demand to reliably predict the response of piles. Since then, extensive research have been carried out and 7 several analytical and numerical procedures have been developed to determine the static and dynamic response of piles subjected to horizontal or vertical loads. Also, full scale experimental observations on the pile's behavior and numerous model testing have been carried out.

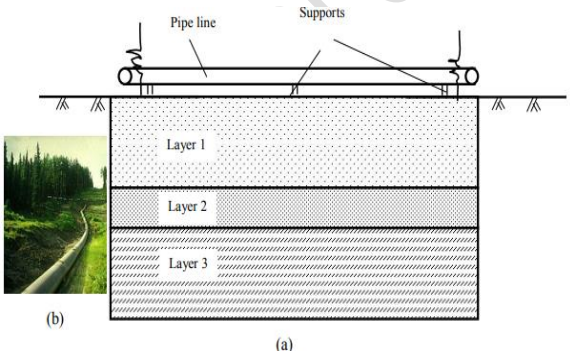


Figure: (a) Schematic diagram showing on ground pipe line (b) Surface Pipe Line

Generally, a pile can be regarded as a stiff, slender body embedded in a much softer medium which is soil. When a load is applied to pile, it deforms and interacts with the soil, resulting in the development of interface pressure along the two media, the distribution of which depends on applied load and soil-pile properties. So, the study of SSI is important in understanding the complex behavior of soil and pile. In this thesis, an attempt is made to understand this behavior. Obtaining appropriate radiation conditions for large-scale engineering problems is the most challenging part of the dynamic soil-structure interaction. The disturbance travels as a wave in the ground affecting a very large area, contrary to the static case, where the influence of load is confined to a limited area around the application point of load. In keeping with this point of view, while performing dynamic analysis, care should be taken in modeling the boundaries

NONLINEAR SOIL COLUMN MODELING:

During an earthquake event, there is a significant difference between the excitation recorded at the ground surface and the excitation recorded in the bedrock or outcrop rock, and this difference will vary depending on the depth of the data recording set and the soil type of the site. Based on this concept, the nonlinear cyclic behavior of soil in high strains can be simulated by an equivalent linear system. The equivalent linear model is in fact a simplification of the viscoelastic model known as the Kelvin-Voigt model Although the equivalent linear approach has an acceptable performance in many cases and shows quick performance and adequate accuracy in intense vibrations, the method generates responses far from reality, especially when an incremental dynamic analysis (IDA) is in progress Thus, in the present study, accurate simulation of site response considering the nonlinear behavior of the soil was placed in the agenda.

Scope of present study:

In an effort to identify the stiffness degradation of the pile-soil system due to the nonlinearity of the soil, the full scale study of a free head single pile lateral response was initiated. This study was also performed due to an absence of field data for Auckland residual clay, which is widespread in Auckland region. Based on the field tests results, the nonlinear equations of were used to analyses in accordance with the stiffness degradation and gap deepening of the pile-soil system. This leads to a better representation of the pile-soil response when in comparison to the original nonlinear Davies and Budhu equations, which do not incorporate these two parameters. Thus, the use of nonlinear equations by the iterative procedure of the curve-fitting method to predict the load-displacement curves can be recommended To determine and analyses the dynamic response of the pile-soil system in the linear and progressively to the nonlinear soil deformation range. Dynamic pile head loading was applied at increasing forcing amplitude in an attempt to produce the nonlinear response and to assess if possible permanent degradation had occurred in the soil.

II.LITERATURE REVIEW

Chore, H. S., Ingle, R. K. and Sawant [1] Civil engineering structures involve structural elements with direct contact with the ground. When the external actions, such as earthquakes, act on these systems, neither the structural displacements nor the ground displacements, are independent of each other. The process in which the response of the soil influences the motion of the structure and the motion of the structure influences the response of the soil is termed as soil-structure interaction (SSI) The response of structure relies upon on the properties of soil, structure and the way of the excitation. Implementing soil-structure interaction impacts empowers the designer to assess real displacements of the soil-foundation structure framework precisely under the influence of seismic action. Present design practice for dynamic loading assumes the building to be fixed at their bases. But in reality, supporting soil medium permits movement to some degree because of their common capacity to deform which diminish the overall lateral stiffness of the structural system resulting in a lengthening of lateral natural time periods. This, in turn, effects the response of the building considerably.

III.METHODOLOGY

Soil boundary condition is a simulation condition, which represents the accuracy of the experimental test output. In reality, the soil is unbounded. However, the effect of the soil boundary is required to be studied due to the influence boundary on the behaviour of soil during experimental tests. In the shaking table experiments, the container influences soil behaviour of SFSI system due to wave reflection on the container boundary and variation of system vibration mode. A suitable simulation of soil boundary is necessary to enable the soil in the container to get the same deformation as the soil prototype and to minimize the impact of boundary condition. Many researchers proposed different kinds of soil container to simulate the boundary conditions of dynamic soil tests. The main available studied container is laminar box, winged wall box, generally rigid wall box with inner lining, and the flexible container. The region near the boundary is more affected by the boundary condition in comparison with the area far from the boundary. It is found that the ratio D/d should be taken as 5 by controlling the size of the structure plan, where (D) and (d) is the

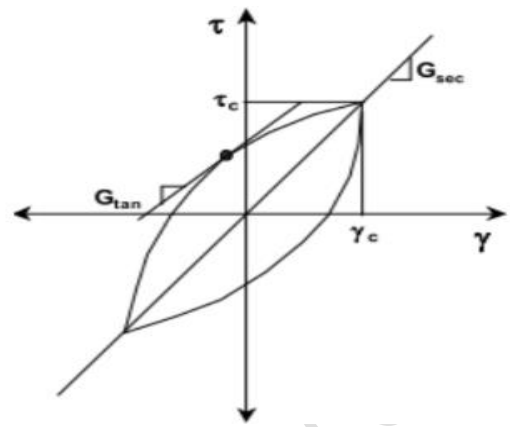
diameter of the soil container and the structure base diameter, respectively.

Dynamic behaviours of soil

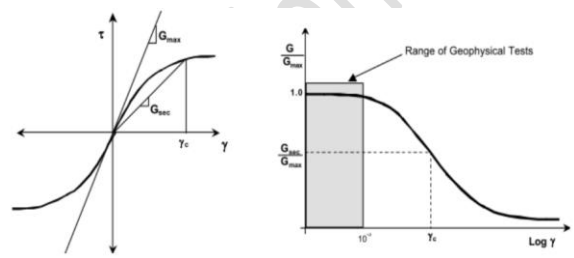
The response of soils to dynamic loads is controlled mostly by the soil mechanical properties of soil. The soil mechanical properties associated with dynamic loading are shear wave velocity (ـ0), shear modulus (G), the damping ratio (D), and Poisson's ratio (ν). Wave propagation effects control the engineering problems. These effects induce low levels of strain in the soil mass. However, when soils are subjected to dynamic loading that may cause a stability problem, then considerable strains are induced. References show the hysteresis soil behavior when the soil under dynamic load. This hysteresis response of soils can be estimated by considering two important parameters of hysteresis loop shape the loop inclination represents the stiffness, the tangent shear modulus varies with the dynamic loading.

$$G_{sec} = \frac{\tau_c}{\gamma_c} \quad (1)$$

Therefore, ـ0 describes the general inclination of the hysteresis loop. If the result of the damping ratio (D) is less than 1, the damping ratio is defined as under damping, while if those values are equal to 1 and more, the damping ratio is defined as critical damping and over damped, respectively. Most problems in earthquake engineering are within under damped limits. The damping ratio represents the ability of material to dissipate dynamic load or dampen the system. It should be noted that many parameters affect the stiffness of soils during dynamic loadings, such as relative density, plasticity index, main principal effective stress, over consolidation ratio, the number of load cycles and void ratio.



(a) Hysteresis loop



(b) Secant and tangent shear modulus

the loss of soil element stiffness with an amplitude of strain. The damping force increases and causes the energy dissipated in the ground by friction, heat, or plastic yielding. Damping or the damping ratio (D) is defined as the damping coefficient divided by the critical damping coefficient. The damping (D) can be estimated from the hysteresis loop, the area of the loop divided by the triangle area created by the secant modulus and the maximum strain (energy dissipated in one cycle of the peak energy during a load cycle),

$$\zeta = \frac{w_D}{4\pi w_s} = \frac{1}{2\pi}\frac{A_{loop}}{G_{sec}\gamma c^2}\frac{\tau_c}{\gamma_c} \quad (2)$$

The indication of the low-strain soil models is based on the approach of the equivalent linear model. This method is simple and used in a dynamic model commonly, but they have a limited ability to represent many aspects of soil behavior under dynamic loading conditions.

Model Soil and Model Piles:

A model soil with appropriately scaled stiffness and strength properties was developed for the project, and consisted of 72% kaolinite, 24% bentonite, and 4% type C fly ash (by weight). The model soil has a unit weight of 14.8 kN/m3, a plasticity index of 75, an untrained shear strength of 4.8 kPa and a shear wave velocity of approximately 32 m/second (the last two parameters measured at a water content of 130% and cure time of 5 days). Bender element and cyclic triaxial laboratory tests were performed to characterize the modulus degradation and damping curves of the model soil. provide a more detailed discussion of the model soil development. From standard Caltrans design, a 410 mm diameter x 12.7 mm wall concrete-filled steel pipe pile was selected as the target prototype. Scaling constraints dictated a maximum prototype pile length of 12.8 m, which provided a L/d ratio of 32, acceptable for a slender pile. The fixity conditions of the pile, known to be significant in lateral response, were established as fixed against rotation at the head, and fixed against (relative) translation at the tip. This corresponds to a pile driven into a firm strata at the base, and cast into the pile cap. The flexural rigidity of the prototype pile was computed as 79,120 kN-m2 Accordingly, the model piles were fabricated using 50.8 mm diameter 6061 T-6 aluminum tubing with a wall thickness of 0.71 mm, which provided the correctly scaled flexural rigidity (EI).

EXPERIMENTAL 1-G MODELLING:

Ad-hoc laboratory investigations are essential for studying complex soil-structure interaction, due the paucity of the field (or full scale model) data, which tend to be expensive to conduct and hard to interpret. Data from instrumented piles under buildings of different vibrational characteristics subjected to actual earthquake motions would be ideal. These data are rare due to high cost and the unpredictable nature of earthquake occurrence. Therefore, well-controlled laboratory investigation on model piles along with analytical and/or numerical simulations is pivotal for understanding the seismic response of both single piles and pile groups (see, for example, The approaches based on the use of the 1-g shaking table possess certain advantages over centrifuge procedures, such as the valuable benefit of working on a larger and more reliable physical models, which allow detailed measurements of pile response and numerous combinations regarding soil profile, pile-head boundary conditions, superstructure features. Additionally, it is possible to plan and carry out a wide set of tests with reasonable operating expenses.

MODELLING:

The theoretical model employed in piles modelling is an Euler-Bernoulli beam in pure bending It is assumed that the pile is not being so heavily loaded that it is stressed by axial load or in bending beyond its elastic range. The maximum lateral load capacity of the pile moving relative to soil is not exceeded.

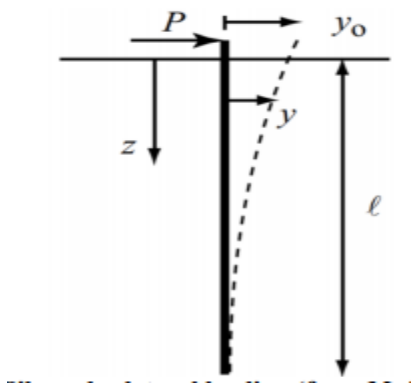


Figure: Pile under lateral loading

The pile can be hence considered as a beam subjected to loading at the ground surface or at the head of the pile and by the resistance of the ground to relative movement of pile and soil. If the soil responds

elastically, then the resisting force is proportional to relative displacements according to some coefficient of subgrade reaction k and the equation governing the deformation of the pile will be of the form:

Experimental set-up:

The 6-degree-of-freedom earthquake simulator of BLADE and the equivalent shear beam (ESB) container was utilized to perform the aforementioned series of tests. More details on the devices used are given in the following paragraphs.

Experimental procedure:

The testing procedure includes the following dynamic input motions: - White noise excitation: during white noise exploratory testing, a random noise signal of bandwidth 0-100Hz and peak ground acceleration varying between 0.01g and 0.10g was employed. In total 45 horizontal (first phase), 28 horizontal and two vertical (second phase) white noise tests were performed; - Harmonic excitation: sined well acceleration time-histories were imposed. In the first stage of testing 291 different sined wells were used. Each one is formed by 12 steady-state cycles; 15 different frequencies of excitation (varying from 5Hz to 30Hz with a step of 2.5Hz and from 30Hz to 50Hz with a step of 5Hz) and acceleration amplitudes varying between 0.01g÷0.18g were applied. In the second stage 142 horizontal and 26 vertical sinusoidal excitations were employed, characterized by 16 steady cycles; 7 frequencies (varying from 5Hz to 45Hz with a step of 5Hz) were employed with acceleration amplitudes varying between 0.01g÷0.13g. - Earthquake excitation: three earthquake records from the SISMA database (The earthquake motions were modified by a frequency-scaling factor of 5 or 12 to account for different size between model and prototype. The frequency-scaled signals were applied at 16 acceleration amplitude varying from 0.043g to 0.577g. 19 earthquake input motions were used in total. More details about earthquake excitation are provided in the next paragraph. The experimental program included also two pseudo-static tests: - Pullover test: small increments of lateral load were applied, while monitoring the pile head response by means of displacement transducers; - Snapback test: after the application of traction on the pile head through a pulley system (several weight on the pulley were applied) the wire was cut and the horizontal motion of piles was measured for the estimation of the natural frequency and damping ratio of the embedded pile. One of the five snapback test was performed at the end of the pullover test

Earthquake Simulator (shaking table):

The 6 degree-of-freedom Earthquake Simulator (ES) consists of a 3m x 3m cast aluminium platform weighing 3.8 tonnes and capable of carrying a maximum payload of 15 tonnes. The platform has the shape of an inverted pyramid made by four sections and has a honeycomb-like network of stiffening diaphragms giving it high strength and bending stiffness. The platform surface is an arrangement of five aluminium plates with a regular grid of M12 bolt holes for attaching to the platform body and for mounting the specimens. The platform sits inside a reinforced concrete seismic block that has a mass of 300 tonnes. The block is located in a pit in the Earthquake Engineering Laboratory and is isolated from the rest of the laboratory by a 20 mm cork filled gap running between the block and the rest of the laboratory. Hydraulic power for the ES is provided by a set of 6 shared variable volume hydraulic pumps providing up to 900 litres/min at a working pressure of 205 bar. The maximum flow capacity can be increased to around 1200 litres/min for up to 16

seconds at times of peak demand with the addition of extra hydraulic accumulators.

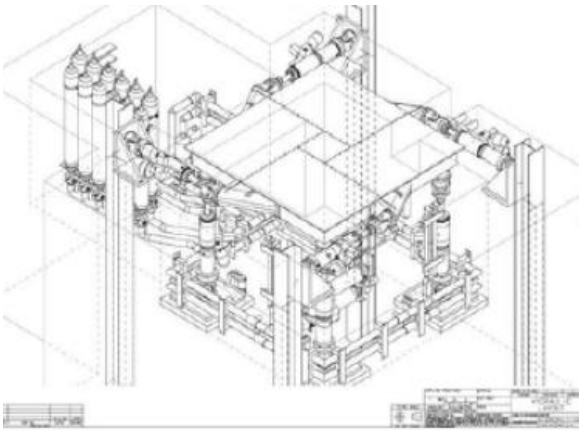


Figure: Earthquake Simulator

Equivalent Soil Container The equivalent soil container ensures that it follows the soil movements without influencing its response. Several different soil containers were proposed to minimize boundary effects; the most common are: (1) the Laminar box 2006; the Equivalent Shear Beam box

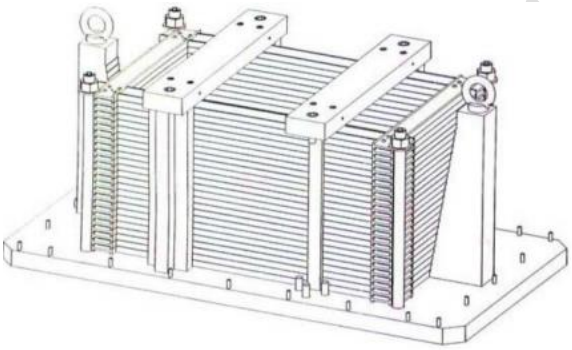


Figure: Laminar box - 3D view

Laminar box soil containers are constructed from a stack of stiff rings, which may experience independent and unrestrained lateral displacements because of the box negligible shear stiffness. Internal walls of the rings are made smooth to restrict boundary shear stresses. Laminar box is often used to model liquefaction.

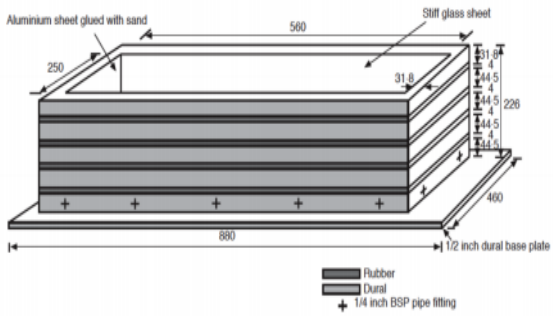


Figure: Equivalent Shear Beam - 3D view

Equivalent Shear Beam (ESB) centrifuge container, has roughened internal walls to enable shear stress transmission. It was constructed from an alternating stack of aluminium alloy and rubber rings for flexibility. The composite shear stiffness of the ESB was tuned to the dynamic stiffness of a test soil by careful detailing of the rubber layer thickness. This type of containers should be ideally designed to match the shear stiffness of the inner soil. However, the shear stiffness of the soil varies during shaking depending on strain level. Therefore, the matching between the end wall and the soil stiffness would be possible only at a particular strain level. The ESB of BLADE Laboratory is designed considering a value of strain in the soil close to failure conditions. It is thus more flexible than the soil deposit at lower 19 strain amplitudes and, consequently, the soil will always dictate the overall behaviour of the container The resonant frequency and damping of the empty container in the first shear mode in the longitudinal direction were measured prior to testing as 5.7 Hz and 27%, respectively; such values are sufficiently different from the soil material properties. The Bristol University ESB was designed by and it is a direct descendant of the 'large flexible shear stack'. original: 1.2m long as opposed to 5m, 0.55m wide as opposed to 1m, and 0.814m deep as opposed to 1.2m,

significantly reducing the costs and time scales associated with testing.

Figure: Shear stack

Structural elements configurations and properties:

The structure components of the model consist of piles, a cantilever system (a single degree-of-freedom, SDOF) provided with its foundation and two kinds of connection, namely one between a small group of piles (so called short-cap connection) and the other connecting all the five piles (so called large-cap connection). In both experimental phases, the five piles embedded in the bi-layer soil consist of an alloy aluminium tube (commercial model 6063-T6) with thickness t = 0.71mm, outer diameter D = 22.23mm and length l = 750mm. Pile 3, 4 and 5 are closer to each other with a relative spacing s 70mm (s/D ≈ 3); pile 1 and 2 are placed at a distance of 140mm.

Table: Pile properties

Geometrical details [mm]	Unit weight [kN/m3]	Length [mm]	Young's modulus [GPa]	Poisson's ratio
De = 22.23 t = 0.71	27	**750**	**70**	**0.3**

In the first stage of the tests the oscillator was formed by an aluminium column with extra masses added to its top to achieve different dynamic response. In the second phase, three different columns were used to vary the pier stiffness.

SIMULATION OF SHAKING TABLE TEST EXPERIMENTS:

A series of scaled physical model experiments have been performed at U.C. Berkeley on the shaking table to examine the seismic response of soil-pile-superstructure interaction, as described a the companion paper by A model soil with appropriately scaled stiffness and strength properties was developed for the project, and consisted of 72% kaolinite, 24% bentonite, and 4% type C fly ash (by weight). The model soil has a unit weight of 14.8 kN/m3, a plasticity index of 75, an undrained shear strength of 4.8 kPa and a shear wave velocity of approximately 32 m/second (the last two parameters measured at a water content of 130% and cure time of 5 days). Bender element and cyclic triaxial laboratory tests were performed to characterize the modulus degradation and damping curves of the model soil. From standard Caltrans design, a 410 mm diameter x 12.7 mm wall concrete-filled steel pipe pile was selected as the target prototype. Scaling constraints dictated a maximum prototype pile length of 12.8 m, which provided a L/d ratio of 32, acceptable for a slender pile.

Table: Soil properties of layers

Soil Material Type	Thickness of Layers (m)	Maximum Shear Modulus (Mpa)	Total Unit Weight (kN/m^3)	Shear Wave Velocity (m/sec.)	Location of Input Motion	Vertical Effective Stress (kpa)
1 (soft to medium)	0.2	0.49	14.80	18.09		0.51
1	0.3	0.82	14.80	23.25		1.62
1	0.3	1.22	14.80	28.43		3.04
1	0.5	1.55	14.80	32.04		4.94
1	0.5	1.97	14.80	36.18		7.33
1	0.5	3.64	14.80	49.10		9.72
1	0.3	5.06	14.80	57.89		11.61
1	0.2	9.48	14.80	79.25		12.74
2 (bearing layer)	0.2	46.09	18.00	158.49		13.74
		89.70	22.00	200.00	OUTCROP	14.37

The fixity conditions of the pile, known to be significant in lateral response, were established as fixed against rotation at the head, and fixed against (relative) translation at the tip. provide a more detailed discussion of model soil development and pile selection. The work presented here focuses on the analytical simulation of one shaking table experiment, referred to as Test 1.18. The layout of this experiment is shown in Figure 2 and consists of four single piles with head masses ranging from 10 to 160 lbs. in approximately 6 ft of cohesive model soil. The modeling criteria are almost exclusively based on the un drained shear strength and soil stiffness, which was found to be controlled by water content. As shown by the results of the T-bar pullout tests a non-uniform soil strength profile was obtained The model itself was subjected to a series of seismic events including sine sweeps and earthquake records the Loma Prieta earthquake was used as the input to the shaking table for the test analyzed herein. the location of the vertical arrays of accelerometers and T-bar tests. The figure also shows the approximate head masses and location for each of the piles.

Figure: Test Arrangement in the Large Shaking Table

Free-Field Response:

The free field response of the container was evaluated by comparing the motions recorded at two of the vertical arrays placed inside the container with those numerically simulated. The equivalent linear method of analysis in the time domain was used for describing the site response analysis. The model uses an equivalent Rayleigh damping formulation and the implementation is essentially similar to that of QUAD4M shows results suggesting that for this level of shaking the equivalent linear method as well as the use of other nonlinear constitutive laws give very similar results. In general, the frequency content of the spectral accelerations is well predicted except at frequencies higher than 5 Hz. In particular, the amplification of spectral acceleration at the site period (i.e., T ≈ 0.5 sec.) was well captured by both linear and nonlinear analyses. The equivalent linear method gives better overall predictions for periods shorter than 0.2 sec. However, the spectral accelerations at frequencies higher than 5 Hz were under predicted by up to 50% by both methods of analyses. Most significantly, the amplification of the spectral acceleration at about T=0.2 sec, corresponding to the predominant frequency of the earthquake motion, was under predicted in the results of both simulations.

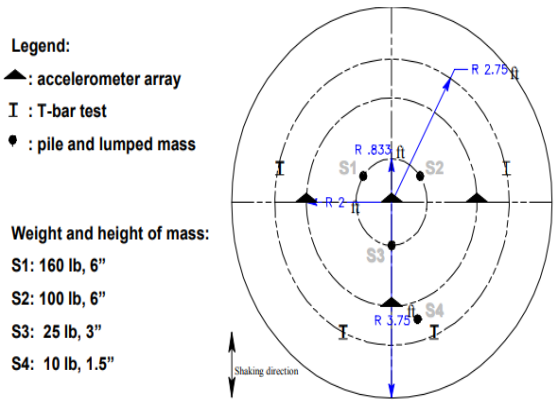


Figure: Plan View of Shaking Table Test 1.1

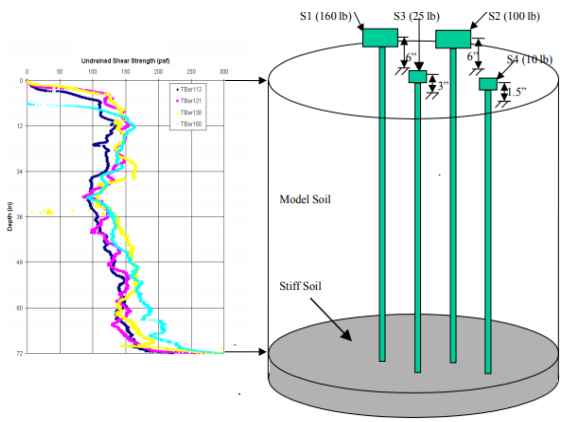


Figure: b) Shear Strength Profile and Set-up of Single Piles

The predicted responses for the two heavy structures, S1 and S2, match the observed spectral response very well. The predicted responses for the two small structures, S3 and S4, match less favorably with the observed response, which is partially attributed to the prediction of the free field response. compares the computed and recorded maximum accelerations. The predicted maximum accelerations of the structures are within 5-10% to the observed one except for the pile S3. In particular, the error for the two small structures, S3 and S4, are much higher than the two large structures, S1 and S2, which indicates that the accuracy of the structural prediction is dominated by the accuracy of the free field prediction. Therefore, greater error was observed for the smaller structures, which were essentially dominated by kinematic interaction. As the mass of the structure becomes larger, inertial interaction has greater influence to the overall response of the structure, and the influence of the free field accuracy is less significant.

Free-Field Soil Analysis

At a large distance from pile foundation (the so called free-field), the motions of these piles have a smaller effect on soils and the one-dimensional wave propagations are adequately assumed for the behavior of layered soil deposits. Because of using the results of shaking table tests, the free- field response of the container was evaluated by comparing the motions recorded at two of the vertical arrays placed inside the container with those of the numerically simulated. As a result, the equivalent linear method of analysis in the frequency domain was used for describing the site response.

A computer program was written for the free field procedure formulations in frequency domain dependent approach in DYFRA program which can be used to solve the ground response problem. In simple terms, the input motion is represented as the sum of a series of sine waves. A relatively simple solution for the response of the soil profile to sine waves of different frequencies (in term of transfer function) is used to obtain the response of the soil deposit to each of the input sine waves. The overall response is obtained by summing the individual response to each of the input sine waves.

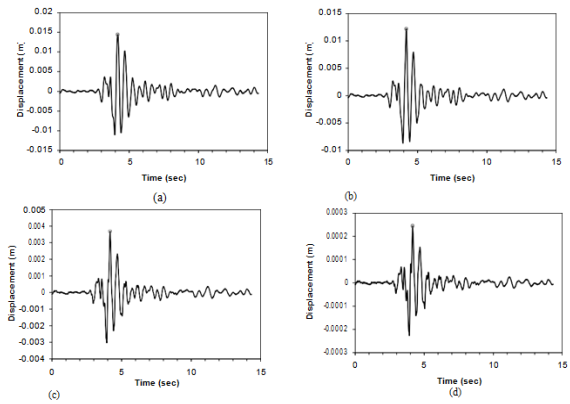


Figure: Recorded displacement from DYFRA program (a) second layer; (b) fourth layer; (c) sixth layer; and (d) eighth layer.

the recorded displacement for second, fourth, sixth and eighth layers obtained of DYFRA program. The results state that the magnitude of displacement (and maximum displacement) reduces from second layer to eighth layer, significantly. Also the maximum displacement for all the layers happened in time of 4.2 sec.

IV.RESULTS AND DISCUSSIONS

The analysis of seismic soil-pile-structure interaction problems is often oversimplified and the effects of two-directional shaking are ignored. Nevertheless, two-directional shaking may significantly increase the amount of non-linear behavior in the near, intermediate and free field domains. In addition, larger excess pore pressures and strain-induced softening may be generated due to two-directional loading, which can reduce the soil stiffness and increase permanent deformations of the structure and/or soil. Cyclic movements/displacements in one direction can soften the near field soil, and can significantly

PILE-SOIL INTERACTION ANALYSIS WITH BNWF MODEL:

The dynamic BNWF model by is a degrading polygonal hysteretic model encompassing multi linear backbone curve with defined rules for loading, reloading and unloading. This model is able to capture the dynamic nonlinear behaviour of soil through the following features. It accounts for cyclic soil degradation through simulating unloading-reloading behavior considering a set of rules such as those proposed by It can simulate gap formation and closing along the soil-pile interface for cohesive soils and reloading in the slack zone (by means of a strain-hardening curve) for cohesionless soils. In addition, the model can handle cyclic soil degradation/hardening as well as reduced radiation damping due to increased soil non-linearity. The initial confining pressure at zero pile displacement is modeled as a pre straining effect applied to the compression-only elements attached to both sides of the pile. Several parameters must be calibrated and provided as input in the model to assess the phenomenological model and the soil mechanical behaviour. In this analysis, different types of soil that feature typical cyclic hardening/degrading behaviour are considered. For saturated soils (sand or soft clay), the cyclic response of the soil along the upper portion of pile is generally considered unconfined and is characterized by an inverted S-shaped hysteresis curve due to slack zone development On the other hand, the cyclic response of soil along the lower segment of pile is considered confined and is characterized by an oval-shape hysteresis curve In the case of dry soils (loose sand in particular), soil cave-in is expected to occur, hence the soil cyclic response is characterized by an oval-shape hysteresis curve along the upper portion of the pile as well. Undergoing cyclic loading, soils may exhibit both stiffness and strength degradation depending on the maximum strain amplitude and number of loading cycles experienced. For saturated soft clay, stiffness

degradation is usually more significant than strength degradation, while for dry sands a typical hardening response is expected.

Analysis Cases:

Six different homogeneous soil deposits are considered, all with constant thickness of 20 m and resting upon a uniform linear visco-elastic bedrock (characterized by shear velocity Vs= 800 m/s and soil damping ratio ξ = 5%), presents the soil type and properties of the different soil deposits considered. Two shear wave velocity values, Vs= 100 m/s and Vs= 200 m/s, and three different soil types are considered: dry sand (DS), saturated sand (SS) and saturated clay (SC). The foundation consists of a single vertical fixed-head pile with a circular cross-section with diameter, d = 1 m and a total length, Lp = 20 m. The concrete pile has a Young modulus, Ep =3×107 kPa and density, ρp = 2.5Mg/m 3. The pile is modeled as a beam element and is discretized into 0.5 m long finite elements to achieve a suitable level of accuracy. Non-linear springs (spring–dashpot combinations) are attached to each pile node in both sides and are excited at their end with the freefield motion. The initial confining pressure is modeled by imposing a pre-straining displacement to the springs considering a coefficient of lateral earth pressure KH equal to 1.0 since the pile is assumed to be driven

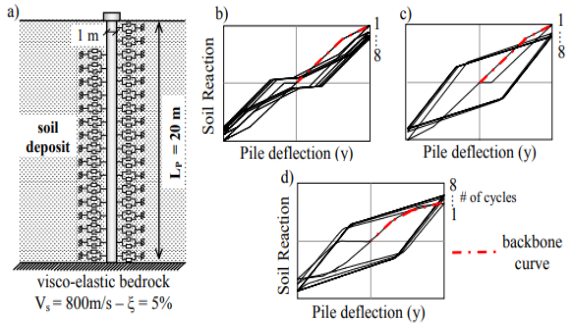


Figure: Soil profile and BNWF model; Hysteretic curves: b) S-shaped hysteresis curve; c) oval-shape hysteresis curve and d) hardening response

Nonlinear soil-pile-structure interaction:

Using the method applied in the previous section together The studied problem is depicted in Figure 6: a pile-column embedded in different homogeneous soil profiles, rotationally restrained at the pile head to simulate the presence of a pile cap. It is assumed that the transverse response of the bridge may be described by the response of a single pier, as would be the case for a multi-spam bridge with coherent ground shaking applied to all piers. The pier height, H = 6 m, its diameter, d =1 m. The deck mass at the top of the pier is 115 t (calculated by assuming that the 3 columns carry equal loads) and the fundamental period of the fixed-base pier is T=0.55 sec. The profiles of the bending moments along the pile obtained from the fully coupled SSI analyses are compared with those previously obtained form the kinematic interaction analyses in absence of the superstructure.

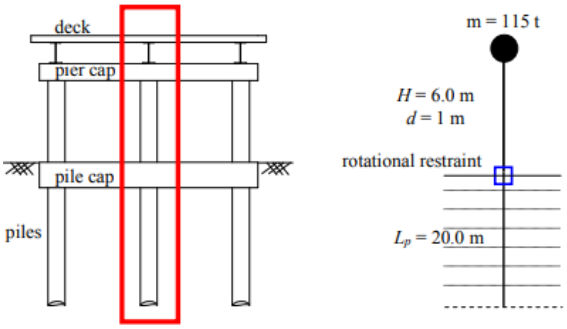


Figure: a) Multi-column bent; b) Schematic illustration of the analyzed problem.

With-in the pile obtained from the IDAs for all soil profiles for the Imperial Valley earthquake. The responses are generally characterized by a maximum moment at the pile head since the inertial effects arising from the superstructure have a significant influence at the pile head and attenuate rapidly with depth. For soil profiles with shear wave velocity equal to 100 m/s, it is observed that the kinematic interaction has a strong effect on the pile response

both at the head and at greater depth. In the particular case of 100DS soil profile, kinematic bending moments along the shaft are greater than those obtained at the pile head from the non-linear SSI analyses. Furthermore, for the soil profiles with shear wave velocity equal to 200 m/s, kinematic bending moments are less important but are still predominant in the lower portion of the pile.

Numerical Modeling of Soil-Pile interaction:

The numerical analysis is performed by using Open Sees PL. Developed by the Regents of the University of California (2000), is a graphical user interface (GUI) for three dimensional (3D) soil-pile interaction responses. The base shaking simulation is performed with a control boundary conditions and zero inclination mode. The pile is modeled with linear beam element and soil with nonlinear beam element. The interface between pile and soil is simulated with zero-length elements. These elements connect the fixed node of pile with slave spring nodes of soil.

Soil Properties:

The different types of soils considered for the analysis are homogeneous, namely, purely cohesive (C-soil), purely non-cohesive (Ø-soil) and combination of both (C-Ø soil). Table explains the different soil parameters of the soils considered for the analysis.

Table: Soil types and their parameters

Soil Parameters	C-Soil	Ø-Soil C-	Ø Soil
Shear Modulus, G (kPa)	$6x10^4$	$13x10^4$	$10x10^4$
Bulk Modulus, K (kN/m2)	$3x10^5$	$39x10^4$	$2.3x10^4$
Cohesion, C (kN/m2)	37	0.3	**25**
Coeff. of Permeability, (m/s)	$1x10^{-9}$	$1.0x10^{-7}$	1x10-^{4}
Mass density (Mg/m3)	1.5	2.1	**1.9**
Friction Angle (Degrees)	0^0	40^0	**33.5^0**

Pile Properties Numerical analysis is carried out for circular rigid short and flexible long concrete piles. The lengths of 4.5 m and 18 m respectively are modeled with a diameter 0.5 m. The pile is fixed at the bottom and pinned at the top to find its response against the given dynamic load. The short and long piles are modeled with a semi-infinite soil medium of size 25m x 25m x 9m, and 40m x 40m x 36m, to simulate the soil structure interaction phenomenon. Fig.5. shows half-mesh Soil-Pile interactive model considered for the dynamic analysis.

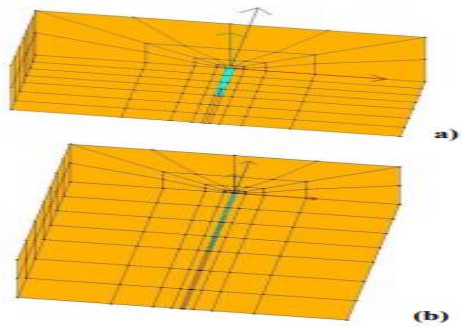


Figure: Soil-Pile finite element models generated for (a) Short Pile (b) Long Pile

DYNAMIC BEHAVIOR OF SOIL-PILE SYSTEM:

A number of approaches are available to account for dynamic soil-pile interaction but they are usually based on the assumption that the soil behavior is governed by the law of linear elasticity or visco-elasticity and the soil is perfectly bonded to a pile. In practice, however, the bonding between the soil and the pile is rarely perfect and slippage or even

separation often occurs in the contact area. Furthermore, the soil region immediately adjacent to the pile can undergo a large degree of straining, which would cause the soil-pile system to behave in a nonlinear manner. Many efforts have been spent on the numerical analysis with a 3D finite element method (FEM) to model the soil-pile interaction. However, it is too complex, especially for group piles in nonlinear soil. A rigorous approach to the nonlinearity of a soil-pile system is extremely difficult and time consuming. As an approximate analysis, a procedure is developed using a combination of the analytical solution and the numerical solution rather than using a general FEM. This procedure is considered as an efficient technique for solving the nonlinear soil-pile system The relationship between the foundation vibration and the resistance of the side soil layers was derived using elastic theory by Both theoretical and experimental studies have shown that the dynamic proposed including a cylindrical annulus of softer soil (an inner weakened zone or so called boundary zone) around the pile in a plane strain analysis. One of the simplifications involved in the original boundary zone concept was that the mass of the inner zone was neglected to avoid wave reflections from the interface between the inner boundary zone and the outer zone. To overcome this problem, proposed a scheme that can account for the mass of the boundary zone. Some of the effects of the boundary zone mass were investigated and found that a homogeneous boundary zone with a non-zero mass yields undulation impedance due to wave reflections from the fictitious interface between the two media.

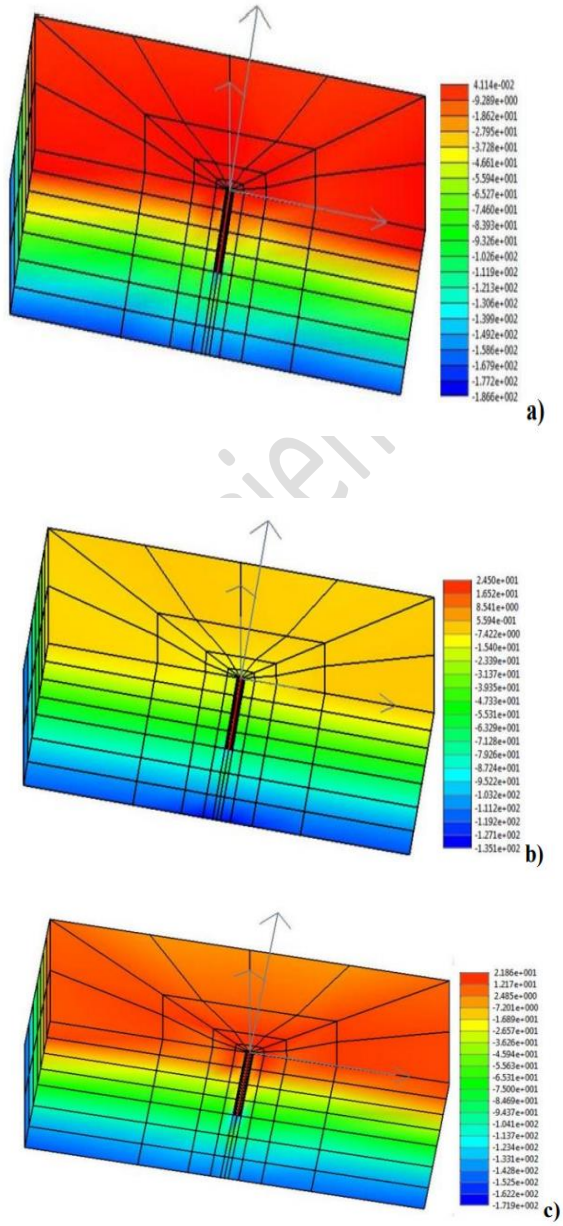


Figure: Longitudinal stress profiles observed in the soil and short pile subjected to El Centro earthquake ground motion for a) Ø-Cohesionless soil b) C-Cohesive soil c) C-Ø soil

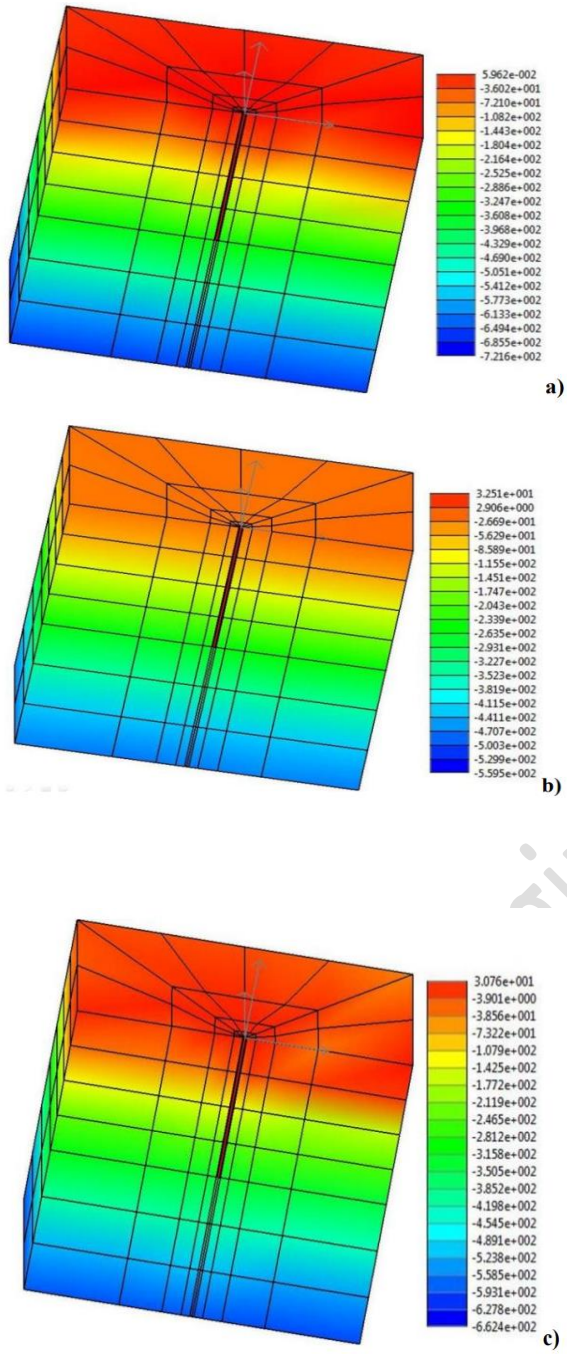


Figure: Longitudinal stress [kPa] profiles observed in the soil and long pile subjected to El Centro earthquake ground motion for a) Ø-Cohesionless soil b) C-Cohesive soil c) C-Ø soil

A dynamic load in the form of El Centro ground motion (Mw 6.9) with peak ground acceleration of 0.348g is applied at the base of the soil model, to perform the analysis. The acceleration time history of the ground motion used for the study is shown in Fig. shows the Fourier Amplitude Spectra of El Centro ground motion. Fig. shows the IS 1893 design response spectra and El Centro ground motion response spectra.

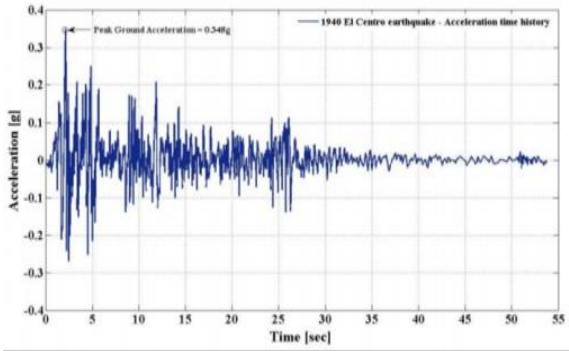


Figure: Acceleration time history of 1940 El Centro earthquake ground motion (Mw 6.9)

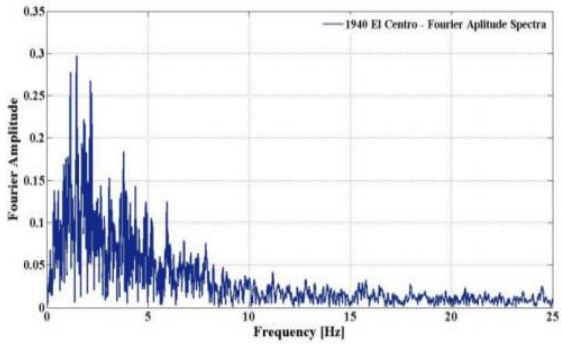


Figure: Fourier Amplitude Spectra of El Centro ground motion

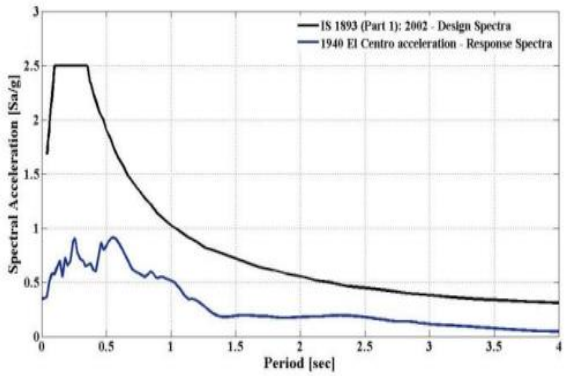


Figure: design response spectra and El Centro ground motion response spectra

long piles for different soil types are presented in Fig. Amplification of displacement response is observed at the top of the pile. Fig. shows the stress contours for short and long piles modeled in different soil strata. It is observed that the displacements and stresses high for C-Ø soil.

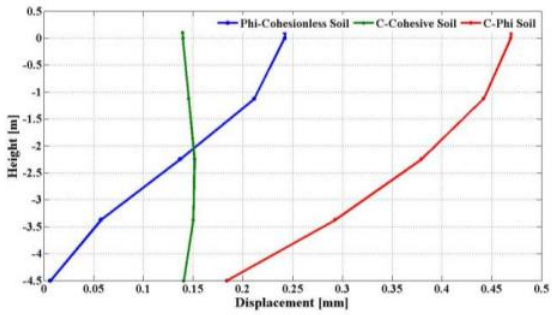


Figure: Short pile response profile for different soil types

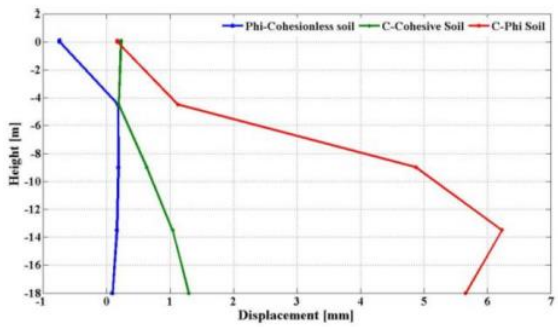
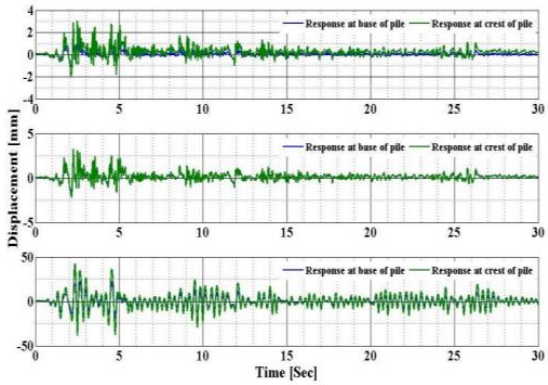


Figure: Long pile response profile for different soil types



Displacement response time history of short pile at the base and at the crest for a) Ø-Cohesionless soil b) C Cohesive soil c) C-Ø soil

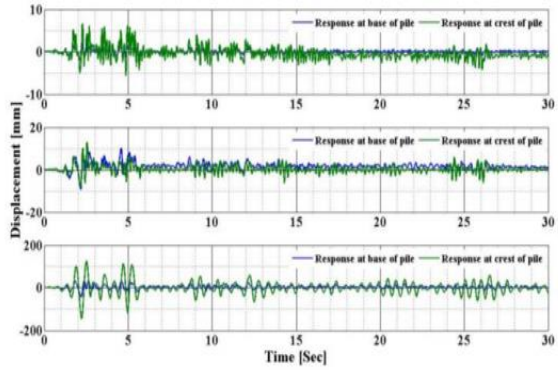


Figure: Displacement response time history of long pile at the base and at the crest for a) Ø-Cohesionless soil b) C Cohesive soil c) C-Ø soil

An important consideration in evaluating the seismic performance of pile-group supported structures, particularly in soft clay or liquefiable soils. Additionally, dynamic deformations can also get induced within the structure due to the underneath soft soils. An understanding from historical catastrophic earthquakes evidently demonstrated that the ground motions were responsible for the failure of foundations, which in turn have caused damage to the structures, leading to the majority of property loss and casualties. In view of the necessity to precisely predict the response of a pile in a soil-structure interaction problem during an earthquake, in this paper, numerical analysis of a single pile subjected to the El Centro earthquake (Mw 6.9) ground motion was carried out to understand the soil-pile interaction for different soil conditions, namely, C-soil, Ø-soil, and C-Ø soil. The axisymmetric numerical model was developed by using Finite Element Program "Open Sees PL", to understand the soil pile interaction for the dynamic earthquake loading condition. Response profiles, displacements response time history at the base and at the crest, and stress contours are studied to understand the behavior of short and long piles in various soil conditions.

V.CONCLUSION:

Present study is a limited effort to assess the effect of dynamic interaction of soil-pile foundation-structure on seismic response of structure. It may be concluded from this limited study that rigid pile group exhibiting higher foundation stiffness or lower ratio of lateral stiffness of superstructure and pile foundation may significantly reduce period lengthening of the stiffer structure embedded in very soft clayey layer which may be equivalent to fixed base condition. The coupled dynamic soil-pile-superstructure and site response analysis using the finite element computer code Geo FEAP was presented. The computer code incorporates a 1-D element to model the nonlinear near field response simulated through p-y springs and a 2-D solid element using the equivalent linear approach to represent the free field site response. The responses of single piles were calculated and compared to the observed behavior in the large shaking table behavior of soil-pile-superstructure interaction and describes the transition from kinematic to inertial interaction. In particular, the predicted frequency content of the response spectra for the superstructures matches most favorably with the experimental results.

- C-Ø soil, time history analysis has been carried out on the finite element models. Subsequent are some interpretations drawn from the existent study
- For short rigid short pile and flexible long pile, the peak displacements at the top of the pile are found to be less for C-Ø soil as compared to C-soil and Ø-soil.
- The values of peak displacements are more for long pile than the short pile; hence length of the pile is one of the important parameter for displacement controls.
- The grade of the concrete is not influencing the displacement characteristics of the pile. Change in displacements is negligible for the parametric grades considered for the present study.
- The type of surrounding soil of the pile plays an important role in the displacements at the top of the pile.

References:

1. Chore, H. S., Ingle, R. K. and Sawant, V. A. "Building frame pile foundation-soil interaction analysis: A Parametric study." Interaction and Multi scale Mechanics, Vol. 3, No. 1, 55-79, 2010.
2. Deepa B. S., Nandakumar C.G., "Seismic Soil Structure Interaction Studies on Multi storey Frames." International Journal of Applied Engineering Research and Development, Vol.2, Issue 1, 45-58, Mar 2012.
3. Abghari A. and Chai, J. (1995) "Modeling of soil-pile-superstructure interaction in the design of bridge foundations," Geotechnical Special Pub No. 51, Performance of Deep Foundations under Seismic Loading, ASCE, John Turner (ed.), 45-59.
4. Bowen, H. J. and Cubrinovski M, “Pseudo-static analysis of piles in liquefiable soils: Parametric evaluation of liquefied layer properties”. Bulletin of the New Zealand Society for Earthquake Engineering, Issue 41, pp.234-246, 2008.
5. Budhu M. and Davies T. G, “Nonlinear analysis of laterally loaded piles in cohesionless soils”, Canadian Geotechnical Journal, vol.24 pp.289-296, 1987

6. Chopra AK, "Dynamics of Structures: Theory and Applications to Earthquake Engineering", Englewood Cliffs, NJ: Prentice-Hall, 1995.
7. Cubrinovski M., Ishihara K. and Poulos H, "Pseudostatic analysis of piles subjected to lateral spreading", Bulletin of the New Zealand Society for Earthquake Engineering, vol.42 pp. 28-38, 2009.
8. Hushi S. P. G., Knappett J. A., and Haigh, S., Design of Pile Foundations in Liquefiable Soils London, Imperial College Press, 2010.
9. Pender M. J., "A seismic pile foundation design analysis," Bulletin of the New Zealand National Society for Earthquake Engineering, vol. 26 pp. 49160, 1993
10. Pradeep, Kumar R., "Need for Pre-earthquake Risk Assessment of Buildings in Moderate-to-Severe Earthquake Prone Areas in India". The International Conference on Forensic Civil Engineering, Nagpur, India, 21-23 January 2016.