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A REFINED COMPUTATIONAL FRAMEWORK FOR THE ASSESSMENT OF THE INELASTIC RESPONSE OF AN IRREGULAR BUILDING THAT WAS DAMAGED DURING THE LEFKADA EARTHQUAKE

A. SEXTOS
Structural Division, Civil Engineering Department
Aristotle University Thessaloniki, GREECE

K. PITILAKIS
Geotechnical Engineering Division, Civil Engineering Department
Aristotle University Thessaloniki, GREECE

E. KIRTAS
Geotechnical Engineering Division, Civil Engineering Department
Aristotle University Thessaloniki, GREECE

V. FOTAKI
Geotechnical Engineering Division, Civil Engineering Department
Aristotle University Thessaloniki, GREECE

ABSTRACT

This paper presents a comprehensive numerical study of a 4-storey RC building which was heavily damaged during the Lefkada, Greece earthquake in 2003. This is one of the few cases where all important earthquake input and structural configuration and response data where both available and reliable (i.e. recorded ground motion, in-situ measured soil properties, structural design drawings and observed damage). The structure is supported on pile foundation due to the very soft and potentially liquefiable soil profile and it has been designed according to the 1959 Greek Seismic code. It is also of particular interest that its irregularities in plan and height lead to a complex dynamic behavior that is primarily characterized by torsion. Towards the evaluation of the earthquake performance of the structure, the analysis approach employs advanced modeling techniques for a) the assessment of the actual, varying with depth, seismic input based on the available records and accounting for the site amplification as well as the presence of liquefiable layers b) the simulation of the complete foundation below the building and the relative stiffness interplay between the piles and the soil layers c) the inelastic dynamic response of the structure accounting for plastic hinge development and short column failure. The results indicate that a number of parameters affect the seismic response of the building. At the same time, they highlight the importance of structural configuration and regularity that, independently of the modeling refinement adopted, are proved to be of paramount importance to the overall structural behavior.

INTRODUCTION

A particularly challenging problem concerning the seismic behavior of a building is the way in which the ground, the foundation and the superstructure interact with each other during a strong seismic event. It is apparent that buildings are not fixed at their base as their foundation is flexible, dissipates energy and interacts with the surrounding soil and the superstructure in such a way, that it filters seismic motion (kinematic interaction) while it is subjected to inertial forces generated by the vibration of the superstructure (inertial interaction). This phenomenon is very complex and its beneficial or detrimental effect on the dynamic response of the bridge is dependent on a series of parameters such as [1, 2 ,3, 4]: the intensity of ground motion, the dominant wavelengths, the angle of incidence of the seismic waves, the stromatography, the stiffness and damping of soil as well as the size, geometry, stiffness, slenderness and dynamic characteristics of the structure. Although considerable research was carried out over the last twenty years in all the aforementioned directions, they are only partially reflected in modern seismic codes. Further more, soil-structure-interaction (SSI) is often treated as a beneficial phenomenon on the basis of the anticipated period elongation of the structure as well as on the energy dissipation at the foundation level caused by wave radiation and hysteretic damping.

The Lefkada earthquake that occurred on the 14th August 2003, was a good opportunity to obtain an insight to the particular and complex issues of structural interaction with the soil. Exceptionally, many important components of the problem were available and quite reliable; the earthquake motion during the main shock was recorded few hundred meters away, as close were the soil properties measured in-situ. Of interest was also a particular 4-storey residential building that suffered moderate to high level of damage during the earthquake. For this building, most of the superstructure and foundation configuration design drawings were available while the observed damage highlighted the locations of structural weaknes. Another important factor of additional interest was the very soft, potentially liquefiable, soil and the massive pile group foundation. Moreover, the irregularities in plan and height (i.e. due to the presence of pilotis) posed a very crucial question with respect to whether the observed damage should be primarily attributed to soil-structure interaction related phenomena, site amplification and liquefaction or, to the overall structural irregularity of the building. This paper therefore, attempts, in the light of the particular well known earthquake-soil-structure environment to combine state-of-the-art computational tools in order to shed some light both on the feasibility of performing a comprehensive seismic analysis that would account for all the aforementioned issues as well as on the relative contribution of the involved parameters to the inelastic dynamic behavior of the particular building, while making an effort to extrapolate the building-specific observations to identify potential conclusions of wider significance.

OVERVIEW OF THE LEFKADA EARTHQUAKE GROUND MOTION

The Lefkada earthquake took place on August 14, 2003, at 8.15am local time measuring 6.4 on the Richter scale. The epicenter according to Athens Geodynamic Institute was located 8.5 miles under the sea, approximately 20 miles north-west of Lefkada island and it is the most powerful to have hit the island since 1995. Four strong aftershocks of a magnitude 5.3 to 5.5 followed the main shock in a time period of 24 hours. The shock caused severe damages to buildings, roads, quay walls, water and wastewater systems. Moreover quite dangerous rock slides occurred all over the island, causing interruption of the road system function and resulting to access disruption at several locations on the island.

The acceleration time histories recorded by the permanent array of the Institute of Engineering Seismology and Earthquake Resistant Structures of Thessaloniki (ITSAK) [5] during the main event of 14/08/2003 (shown in Figure 1), clearly indicate that the motion was indeed very strong (i.e. a maximum 0.40g horizontal peak ground acceleration was recorded). The damage observed though, was not proportionally significant as a result of the traditionally earthquake-resistant construction practice mentioned as well as due to the lack of resonance between the fundament period of the structures (primarily one or two storey stone-wood structures with period T=0.1-0.25sec) and the frequencies of higher spectral amplification. The latter is illustrated in Figure 2, where the elastic spectra of the recorded motions in the two horizontal directions, the current Seismic Code elastic spectrum and the corresponding inelastic spectra (calculated for ductility factors μ=2-4 with widely used signal processing software [6]) are compared. In terms of R/C buildings on the other hand, it is notable that most structural damages to R/C buildings took place in the northern part of the island and in particular within Lefkada city. Again, despite the undoubted earthquake intensity, the number of the totally or partially collapsed buildings was surprisingly low and the performance of most R/C buildings was deemed very satisfactory.

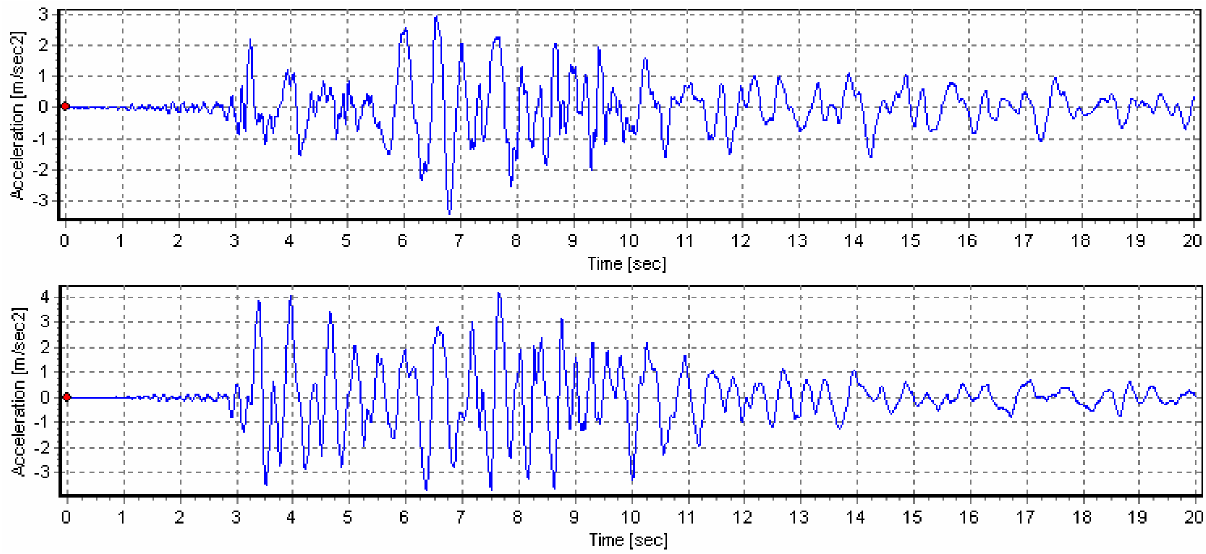

Figure 1: Acceleration Time Histories as recorded by the permanent array of the Institute of Engineering Seismology and Earthquake Resistant Structures of Thessaloniki [5].

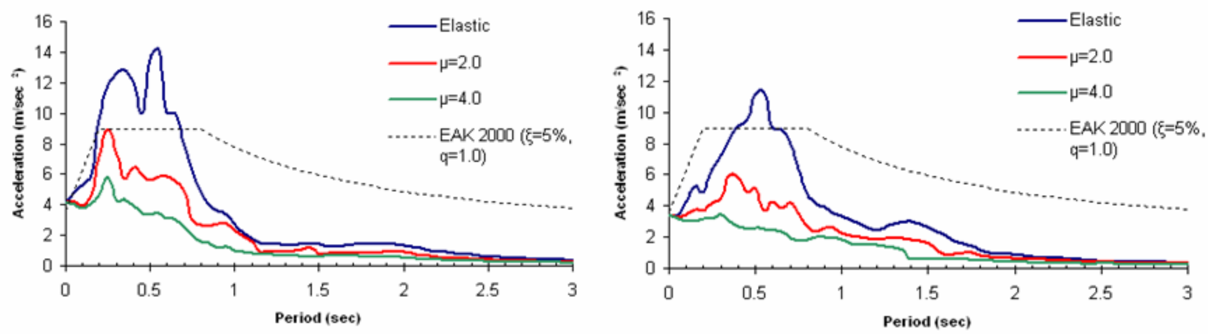

Figure 2: Inelastic spectra derived for ductility factors μ=2-4 based on the main shock response spectrum. The Greek code elastic spectra (i.e. for Soil Category C) are also plotted for comparison.

Another reason for the relatively minor damage that was observed was the fact that the vast majority of R/C buildings in Lefkada island, had already been designed towards at least a minimum level of earthquake forces. In particular, according to the Greek Seismic Code which has been revised in 2000 and 2003 in terms of seismic zonation, the structures in Lefkada should be designed for a peak ground acceleration equal to 0.36g, since most Ionian islands are considered to be located in the highest seismic zone (i.e zone III). This value of design acceleration was already prescribed as well by the previous versions of the Seismic Code that was initially issued and enforced in 1992 (and revised later in 1995). Buildings constructed between 1959-1992 on the other hand, where designed according to the allowable stress-based code of 1959, which prescribed a level of design earthquake forces (i.e. seismic factor ε=0.08-0.16g) for stiff, medium and soft soil conditions respectively but independently of the structural period. Based on these observations, the building under study was one of the few RC buildings that suffered such level of damage, a fact that contributes to the interest of the specific case study.

OVERVIEW OF THE STRUCTURAL CONFIGURATION AND DESCRIPTION OF THE DAMAGE OBSERVED

The structure examined in the present study is an R/C 4-storey building with pilotis, located in P. Filippa - Panagou street, in the town of Lefkada (Figure 3a). The basement floor with a height of 5.65m, has been used as a super market store. In order to serve the needs of the store, a loft has been constructed at the back of the shop at the height of 3m, as seen in Figure 3b. The structural design drawings of the building available were the plan views of the basement, the (typical) first floor, illustrated in Figure 4 and the building foundation. Based on these drawings, existing photos and on-site inspection, the overall structure configuration was determined.

The building was constructed in 1980, hence it was designed according to earthquake forces defined in the Greek Seismic Code of 1959, whereas member design was performed on the basis of the 1954 Reinforced Concrete Code. The latter was essentially expressing the structural knowledge at the time and hence, the earthquake action was considered as lateral forces uniform with height. The horizontal loading values were calculated taking into consideration a seismic coefficient as described previously. In general, it has to be noted that design according to the 1959 seismic code is very common as the particular code was enforced for approximately 25 years. Nevertheless, the vast majority of the building stock designed according to this code throughout Greece had satisfactory seismic performance when subjected and tested by real earthquakes. What has to be noted though is that the above design scheme was primarily used for relatively low-rise buildings with small openings and well constructed infills. Nevertheless, due to the lack of specific terms of code use and especially during the 70s, many buildings were also designed according to this code even if they had longer bays, higher number of storeys as well as storeys with no infills at all (i.e. pilotis) [7]. Consequently, the relative effect of seismic input, soil-structure interaction and structural configuration for the specific building is of particular interest.

In terms of foundation, the structure was supported on a set of small pile groups with piles of a diameter equal to d=0.52m. Depending on the geometry of the supporting vertical member, the piles were either single or in groups of two to four, the latter connected with pile caps. A number of groups was also connected to each other with tie beams of dimensions 30x80cm.

Figure 3a: The building under study (left)
Figure 3b: The inner loft at the back side of the basement store (right)

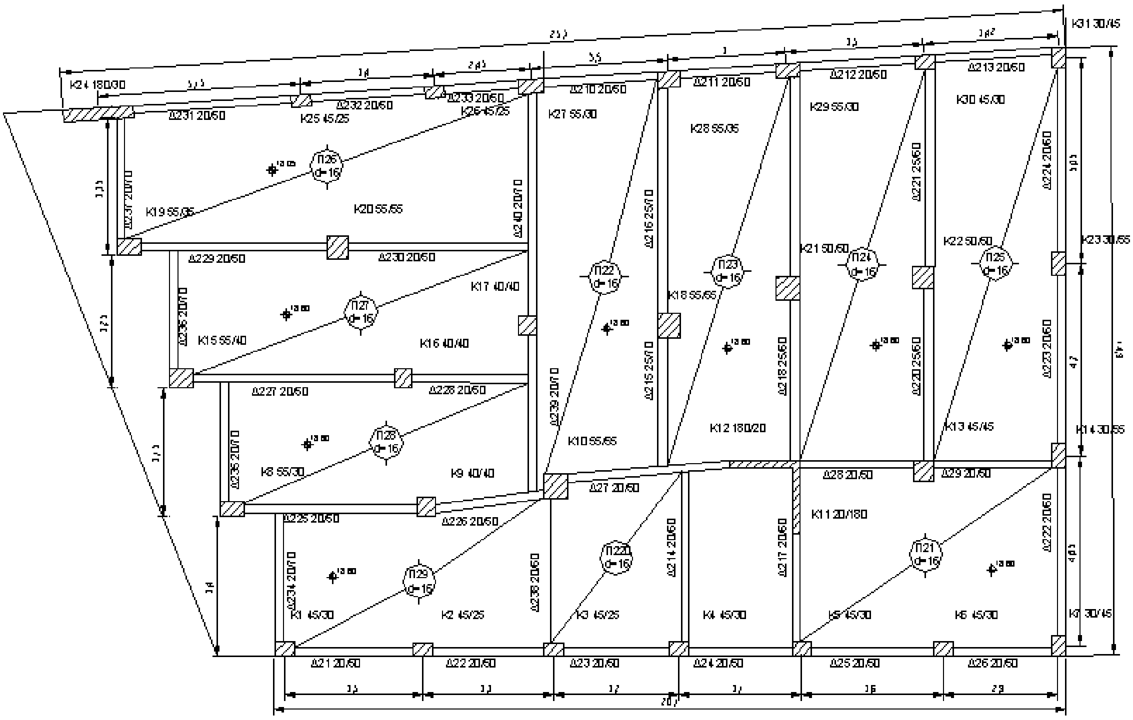

Figure 4: The plan view of the typical building storey

Figures 5a (top): Flexural failure of the columns of the entrance of the Super Market.
Figures 5b (bottom): Short columns failure on the lateral side of the building

From the existing pile and pile caps design calculations it was determined that the piles had been considered as end-bearing at a depth of 17-18m and the lateral friction with the soil was ignored during initial design. Based on design calculations and in-situ observations, the soil layering under the building was assumed similar to the soil conditions of the area in the vicinity of the structure, where soil drillings existed. As a result, the superficial layer was taken to be consisting of debris to a depth of 3.5m. A layer of clay of high or medium density until the depth of 4.6m was also considered. From 4.6m to 10.3m a loose, susceptible to liquefaction soil layer was assumed, consisting of silty sand. The soil type of the next 1m was silt with varying percentage of loose sand. Finally, the subsoil was agricallaceous marga of medium plasticity.

The observed damages of the building during the 14th August 2003 earthquake, are concentrated at the front and one side of the building all at the basement level. In particular, flexural failure of the columns in the store entrance was observed (i.e. spalling, buckling of the longitudinal bars and fracture of some hoops due to the expansion of the core -Figure 5a). Additionally, short column failure was also observed on the lateral side of the structure due to the masonry infill of approximately 2m height. Diagonal cracking extending to the body of the masonry infills was also observed. The intense spalling and the expansion of the short column core is apparent in Figure 5b.

OVERVIEW OF THE NUMERICAL SIMULATION FRAMEWORK

Along with the rapid progress in the computer science, numerical simulation methods are widely used to the study of complex SSI phenomena. It is true, that advanced software exists for the refined simulation of site response (inclusive of liquefaction and material non-linearity) whereas specialised programs are also available for pile design, soil-to-pile and pile-to-pile interaction studies. Moreover, powerful tools are widely used for the inelastic dynamic analysis of buildings. Nevertheless, although different aspects of the very complex and multiparametric nature of the interaction of the structure with its own foundation and the surrounding soil, in the presence of an incident wavefield can be successfully dealt using stand-alone specific tools, it indeed very difficult either to perform a refined comprehensive analysis or even to combine the various specific tools. In other words, notwithstanding the significant progress in analysis and design, all the available specialized tools are inherently restricted in one way or another to deal with a particular problem, inevitably ignoring the strong coupling between earthquake motion, and the foundation-soil-structure components.

The aforementioned limitation is also posed to the study of the particular building. Indeed, nowadays, it is widely accepted that the pile-soil stiffness interplay modify seismic motion depending on the frequency of the incoming waves, while the vibration of the superstructure due to the (modified) ground excitation increases the lateral pile displacements and causes greater strains in the soil, leading to smaller moduli, increased damping and further modification of the motion [2, 3, 8]. This fact is also recognised by Eurocode 8 – Part 5 [9] which prescribes that kinematically induced bending moments can be computed for structures in regions of moderate to high seismicity that are founded on soils susceptible to liquefaction. Moreover, soil and pile nonlinearities affect both the seismic input and the natural frequency of the building which for cases of strong ground motion experiences further period elongation due to the development of plastic hinges.

As a result, for the assessment of the particular building in Lefkada, it was of major significance to study the soil-structure system as a whole, as it was subjected to the actual recorded earthquake excitation, the latter appropriately modified to account for site-dependent amplification, soil nonlinearities, potential liquefaction and simultaneous excitation along the two horizontal axes. For this purpose, the analysis framework was set by incorporating state-of-the-art and practice structural and geotechnical analysis software (both commercially available and research oriented), together with analytical solutions from the literature in order to: a) to define an earthquake motion representative for the specific building site b) highlight the most salient features of the inelastic dynamic behavior of the soil-foundation-structure system by coupling its components at the highest possibly degree.

Definition of a 'realistic' input motion

Input motion identification was performed adopting a detailed procedure (illustrated in Figure 6) that was based on the motions recorded at the location of the city hospital. At first, the recorded seismic motion was deconvoluted to the bedrock level at the position of the hospital area were the accelerograms were recorded, using the FE code Cyberquake [10] and equivalently accounting for soil non-linearity. The anticipated 1-D site response at the location of the building under study was performed with program Cyclic1D [11] that is able to accurately represent the seismic motion modification due to liquefaction at particular soil layers. The distinct accelerations that were calculated along the piles length every 1m were used as the free field input motion for the, essentially, asynchronous excitation of the foundation.

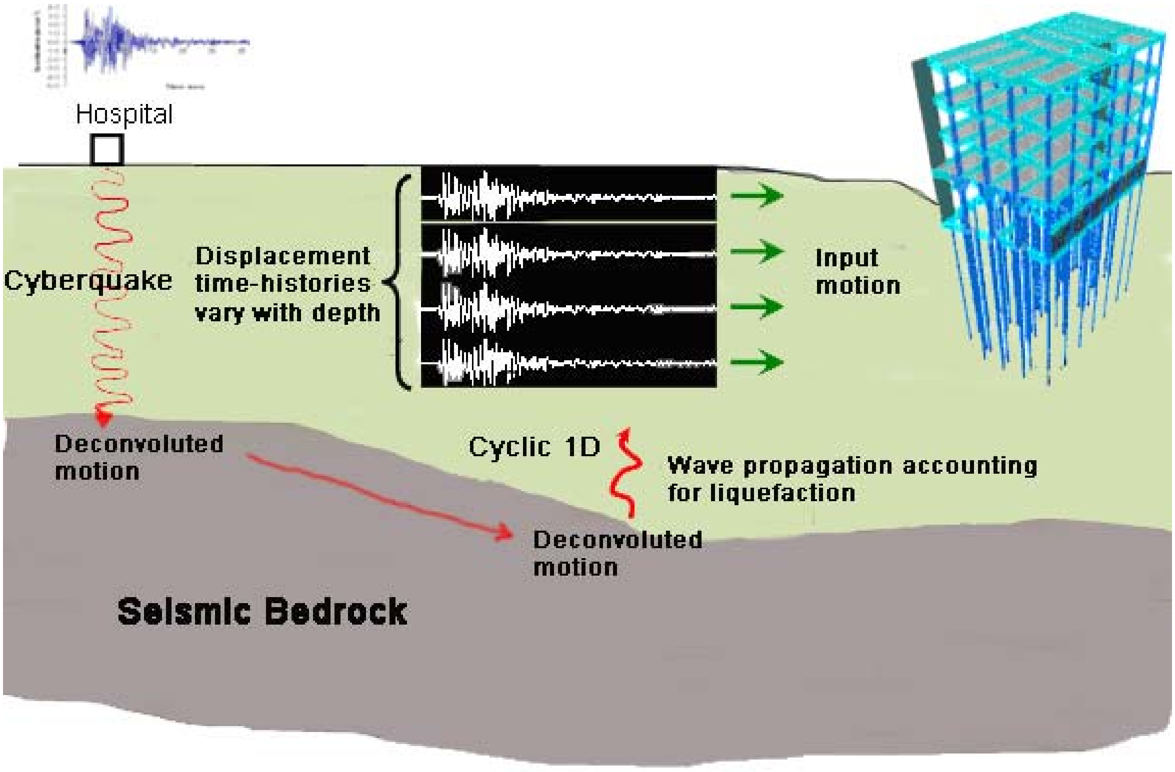

Figure 6: Definition of a 'realistic' ground motion for the excitation of the SSI system

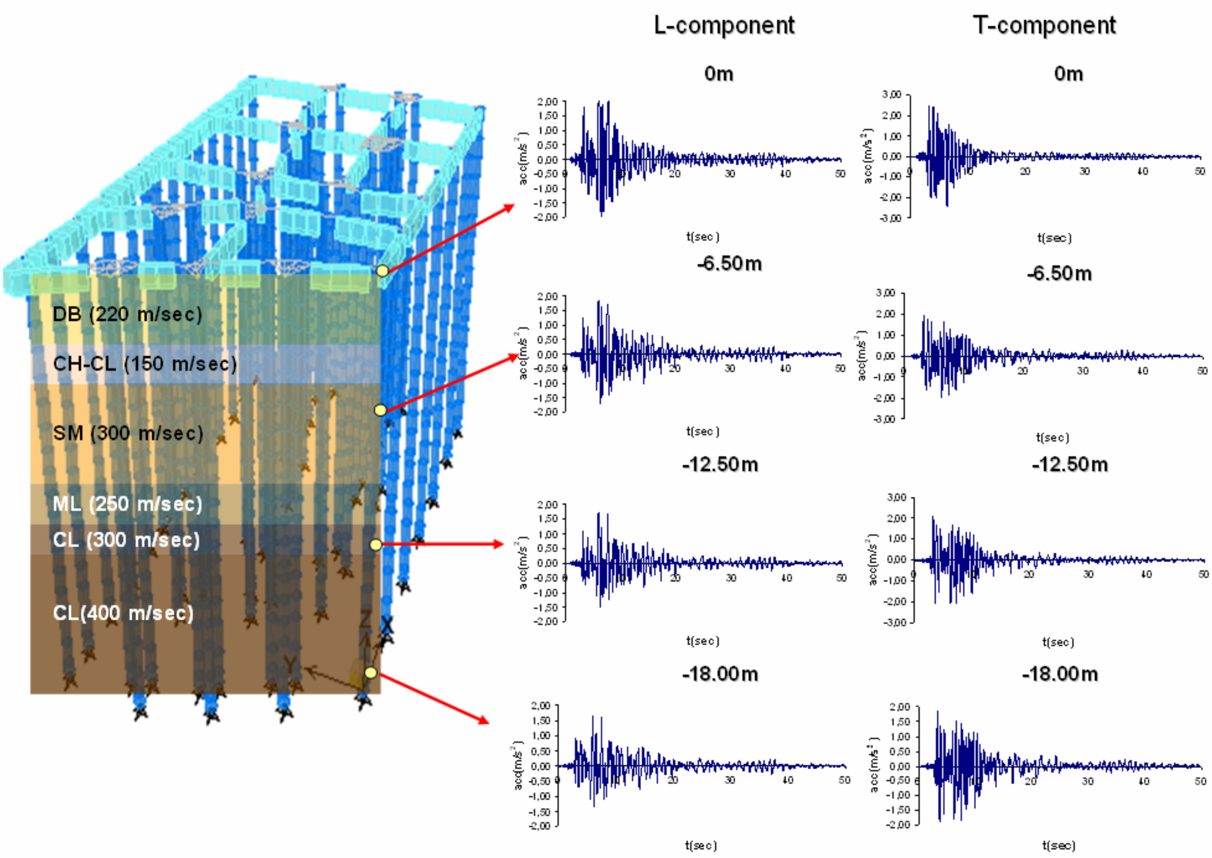

Figure 7: Estimation of the imposed earthquake excitation at various pile depths

Modeling of the soil-pile-superstructure system

For the evaluation of the inelastic dynamic response of the foundation – structure system, the FE model illustrated in Figure 8 was developed. The piles are modeled as beam elements of 18m length, with appropriately modified section modulus to account for cracking at the particular levels of seismic excitation. The piles are assumed to be laterally supported on horizontal Winkler-type springs in two directions. For the determination of the appropriate spring constants, a typical solution would have been the use of the American Petroleum Institute nonlinear load-deflection (p-y) curves [12]. Even though they have been proved very effective for static and pushover analysis [13], their applicability to the more realistic case of dynamic excitation is limited [8] since they essentially neglect kinematic effects and complex issues related to soil non-linearity and liquefaction. On the other hand, the refined approach proposed by Wu and Finn [8] was not feasible to be applied due to the complexity of the superstructure and the foreseen inelastic analysis. Within the context of the present study therefore, and since the field motions were computed at each depth using comprehensive site response analysis tools, the dynamic Winkler spring formulation proposed by Makris & Gazetas [14] was used, appropriately modified to account for the liquefaction induced soil stiffness reduction and damping increase along with pile depth:

$$k_x = 1.2E_s \text{ and } Cx = 1.6 \cdot \rho s \cdot Vs \cdot d \cdot \sqrt[4]{\left(\frac{\omega \cdot d}{Vs}\right)} + 2 \cdot \beta s \cdot \frac{kx}{\omega} \quad (1)$$

According to the available drawings of the initial design the piles are connected through rigid caps of 1.0x1.0m and 0.8x0.8m, which are implemented with rigid beam elements and the corresponding constraint. The vertical and horizontal stiffness of the pile cap itself was calculated according to the classical relationship of Gorbunov-Possadov [15] that matches well with more recent solutions [16]:

$$K_v = \frac{G}{1-v} \cdot \beta_z \cdot \sqrt{4cd} \quad (2)$$

where, βz and βx coefficients are damping coefficients, 2c and 2d the dimensions of the rectangular part of the pile cap, v is the Poisson ratio taken equal to 0.2 and G is the soil shear modulus for the very soft silty sand observed at the surficial layer.

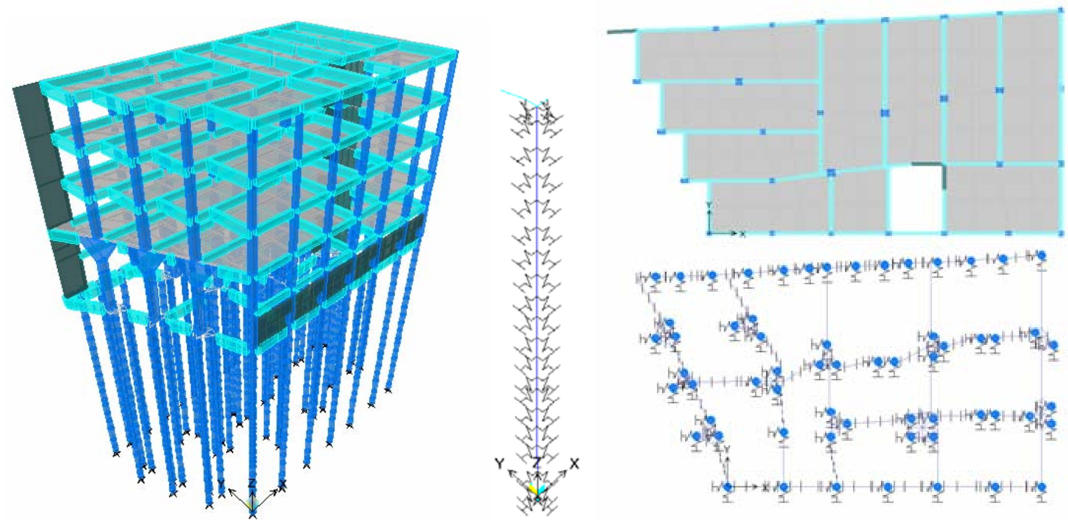

Figure 8: 3D model of the soil-foundation-superstructure system

The tie-beams are modeled as elastic frame elements of section dimensions 30x80cm and 80x80cm. Using a mesh density of 1m, tie-beams are supported on vertical Winkler type springs resulting from the K_s=15 MN/m^3 subgrade reaction of the soil superficial layer. The complete soil-foundation-superstructure approach is illustrated in Figure 8. The limitations of the aforementioned simulation procedure are related to the fact that the dynamic pile-to-pile interaction as well as the interaction among group of piles is essentially neglected although it is widely recognised that the complex dynamic stiffness matrix of such a system may be considerably different and strongly frequency dependent. Yet such an approximation is necessary within the desired framework of the inelastic behaviour of the building and the varying with depth liquefaction-dependent seismic excitation.

Modeling the superstructure

The three dimensional model of the superstructure is an exact representation of the structural configuration, inclusive of the short columns and the disrupted masonry infills at the side of the structure. The total height of pilotis (i.e. 5.65m) was discretised in three separate sub-levels in order to account for both the loft and the varying with height cross section of the front columns. The beams that connect the main columns at the front of the building were modelled as non-prismatic elements, while the relative 22° rotation of their main axes of the front columns with respect to the grid of the building was also accounted for. The material properties correspond to C16/20, equivalent to the B225 concrete used at the time of construction. The few shear walls were modelled using (2D) shell elements. The upper floors are essentially identical to the first floor except from the column cross section dimensions. A 3-Dimensional visualization of the superstructure is illustrated in Figure 8. The widely used FE code ETABS [17] was used for all analyses except from the inelastic dynamic analysis that was performed with SAP2000 [18] due to the numerical instability that was observed when the ETABS built-in plastic hinges were implemented in analysis in the time domain. Inelastic behaviour at the locations of potential plastic hinge development is performed through well controlled plastic link elements (i.e. bi-linear, Wen-type, two node springs) assigned at the end of each frame element. The moment-curvature relationship required was derived using the computer program RCCOLA [19]. Dynamic inelastic analysis is then performed using direct integration in the time domain. In order to evaluate the relative contribution of the various phenomena discussed previously on the overall dynamic response of the foundation-soil-structure system, as well as to define the effect of the analysis assumptions, additional reference analyses were performed in simpler systems; modal analysis of the fixed base model, response spectrum analysis using the (elastic and design) Greek Seismic Code spectrum [20], time-history analysis directly using the two horizontal acceleration time histories recorded at the city hospital during the main shock and time-history analysis using an equivalent uniform excitation along the pile length. The results of the above analysis scheme can be found elsewhere [21].

EVALUATION OF THE INELASTIC DYNAMIC RESPONSE OF THE BUILDING

Elastic dynamic behavior of the complete Soil-Foundation-Structure system

The fundamental period of the fixed-base building was calculated equal to T=0.527sec, hence, it can be considered as relatively more flexible than a typical 4-storey structure. This is illustrated in Figure 9 from which it is observed that the dominant vibration mode is almost purely torsional. This fact is primarily attributed to the presence of the inner loft at the back of the building and the soft-storey mechanism that is created by the height of the front columns that essentially form a pilotis as well as to absence of shear walls. In fact, as it is seen in Figures 4 and 8, only two L-shape walls exist at the center of the structure together with a 180x30 wall at the perimeter to resist seismic forces along the x-x direction whereas there is no clear frame system and no shear walls at all along the y-y direction. As a result, the asymmetry and irregularity in plan and height of the building are expected to play an important role on its overall dynamic behavior. When the stiffness of the pile foundation is introduced, the fundamental period of the building is shifted by 10% to T=0.584sec (Figure 10). The most notable modification of the structural response though, is the fact that the participation of higher modes of vibration has been significantly enhanced. In particular, 25 modes are required in order to activate the 75% of the total mass whereas only 6 modes were enough to activate more than 90%. The resulting overall distress due to the modified by SSI effects modal participation though, does not exceed 30% in the extreme case. As a result, the damage observed can only partially attributed to the foundation compliance.

Inelastic dynamic response of the complete Soil-Foundation-Structure system

Having studied the response of the building in the frequency domain, it was deemed necessary to extend the analysis to the time domain, while accounting for the non-linear behavior of the crucial structural elements as it was described previously. Along these lines, the ductility demand of all basement beams and columns was sought and was compared to the corresponding supply in an effort to match the observed damage distribution and pattern. The input motion (varying along the pile length and accounting for soil non-linearity and liquefaction) was imposed at the foundation, simultaneously at the (perpendicular to each other) horizontal directions and at an angle of 30^{o} with respect to the building axes. This was due to the actual orientation of the building and the recorded motion components. By comparing the ductility demand and supply as well as the corresponding Moment-Rotation (M-θ) diagrams of the entrance columns C18 and C19 (Figure 11) it is seen that the analysis results match very well with the flexural damage of the front columns. Due to the torsional sensitivity of the building, it is apparent that the front and back columns of the building suffer primarily from bending around their X-axis whereas the side columns are subjected to bending around their Y-axis, a fact that is also confirmed by the inelastic analysis performed. Additionally, the remaining basement columns remain essentially elastic as it was also verified by their condition after the earthquake.

Effect of the angle of ground motion incidence on the imposed ductility demand

Having assessed the inelastic dynamic response of the building, it was of particular interest to investigate whether the direction of excitation played an important role on the extent of structural damage. This is feasible to be studied with the adopted analysis and simulation scheme since, as is described above, the excitation is simultaneously applied along the two horizontal axes. Such excitation pattern is considered to be a more realistic approach, especially for the particular case where the input motion is derived on the basis of actual recordings, compared to the standard practice according to which each component of ground motion is applied separately and the peak responses of the structure are combined according to a directional combination rule. The importance of the angle of ground motion incidence is a very interesting issue; it has been highlighted by many researchers, working primarily on the Penzien–Watabe model (i.e. assuming the translational components of ground motion uncorrelated along a well-defined orthogonal axes of the building) [22].

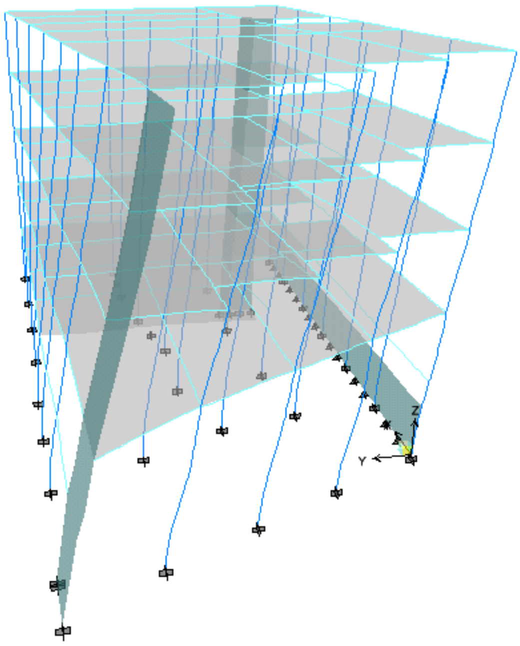

Figure 9: First mode of the fixed-based model- T=0.527sec

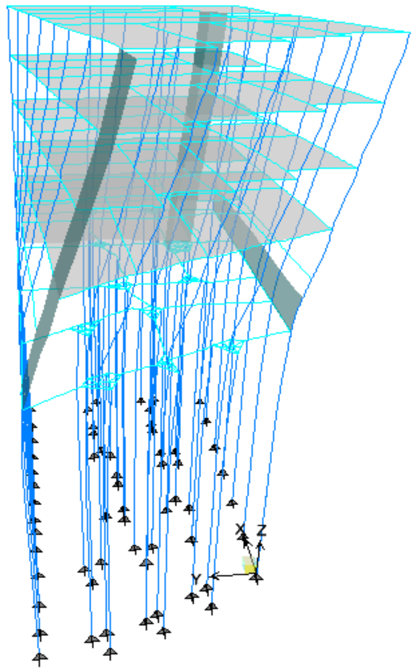

Figure 10: First mode of the model with the pile-elements- T=0.584sec

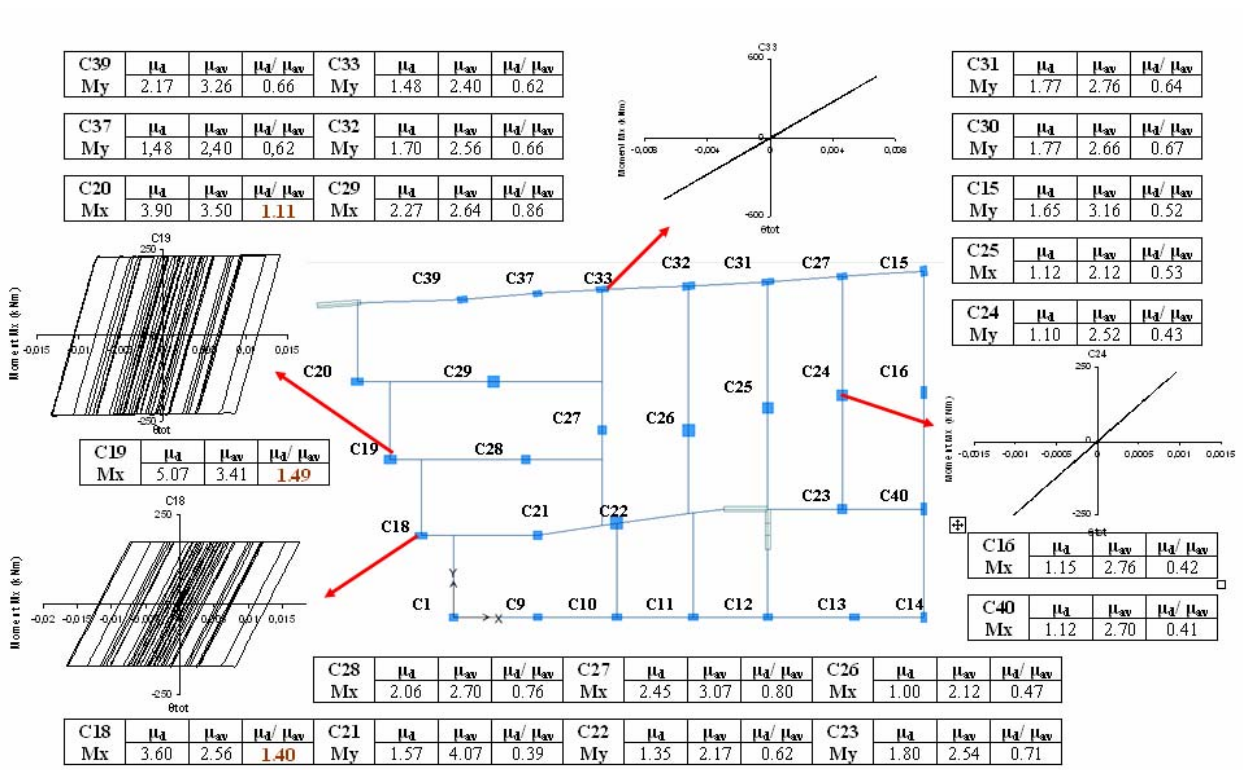

Figure 11: Ductility demand and supply of the basement columns

Recent research [23, 24] has also shown that the maximum value of a response quantity can be up to 176% larger than the value produced when the seismic components are applied along the structural axes. For the investigation of this important aspect of the response, a scheme of five different analyses was adopted in order to evaluate the effect of the angle of incidence of ground motion to the inelastic response of the building. Both the computed L and T components of the seismic excitation are therefore successively rotated and applied at angles of 0°, 30°, 45°, 60° and 90°. As it was presented earlier, the relative orientation between the ground motion components recorded at the location of the city hospital and the building axes, was 30°. It is very interesting to notice in Figure 12 that the ratio of ductility demand over ductility for columns C18, C19 and C20 can vary up to 50% for different angles of excitation; unfortunately for the building, the actual relative angle of 30° is found to be the most critical.

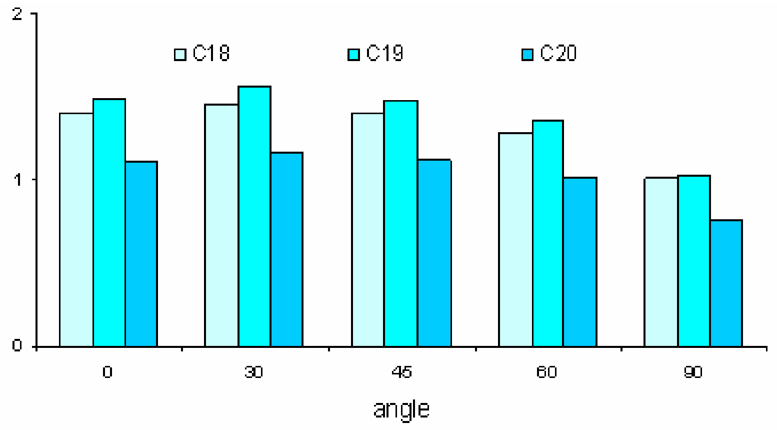

Figure 12: Effect of the excitation angle of incidence on ductility demand of the columns

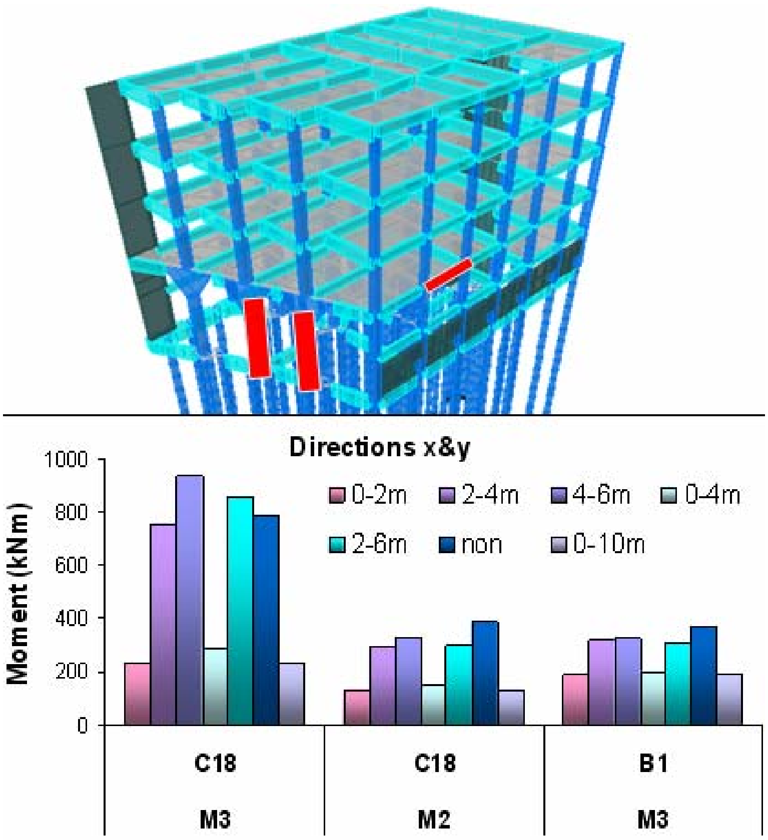

Figure 13: Bending moments of characteristic front columns and side beams for various liquefiable soil layers depth

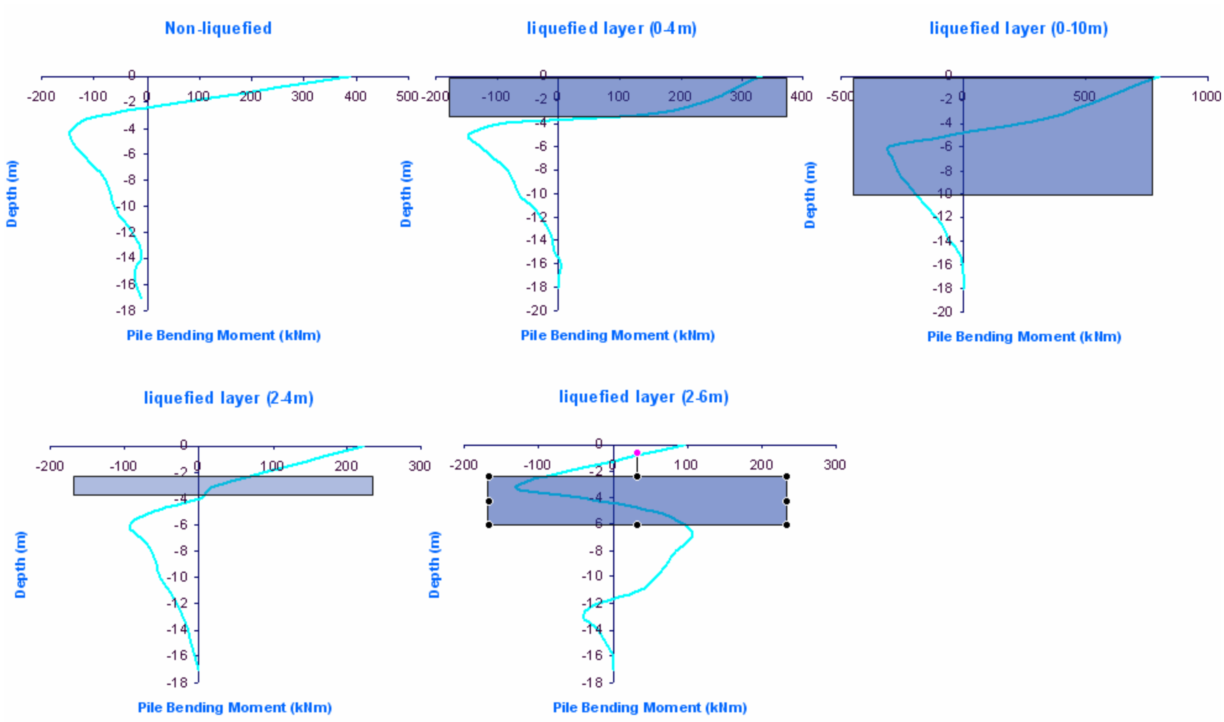

Figure 14: Kinematically induced pile bending moments for various liquefiable soil layers depth

<u>Effect of liquefaction depth to the pile bending moments</u>

Apart from the influence of the structural configuration, the foundation compliance and the direction of the incident seismic waves, it was deemed necessary to attempt o evaluate the role of liquefaction since there was physical evidence that took place at the overall harbour area of the Lefkada city. For this purpose, a liquefiable soil layer was assumed at various depths and structural response was obtained and compared to the reference analysis presented above. In Figure 13 it is depicted that the depth of the liquefiable soil strongly affects the seismic response of the pile supported structure, an observation that is in agreement with other studies as well [25]. It is particularly noticed that although in principle liquefaction may filter certain particles of seismic motion and hence the non-liquefaction case (highlighted in blue) is often the most critical one, certain coupling of (liquefaction-induced) spectral modification, structural dynamic characteristics and foundation-structure interaction may lead to increased seismic demand of the front columns (i.e. column C18 when the liquefiable layer is located at depths 2-6 and 4-6m). In terms of foundation behavior, as it is seen in Figure 14, depending on where liquefaction occurs, the pile foundation may also undergo substantial distress. In particular, extensive liquefaction at depths of 0-10m may lead to up to 4 times higher bending moments, whereas the the moment pattern along the pile length can be also altered considerably even when liquefaction is not extensive. It has to be noted herein that the most probable location of the liquefiable soil material is the depth of 4-10m, hence at least up to a certain degree, the structural damage observed can also be attributed to soil liquefaction.

CONCLUSIONS

The present study focused on the evaluation of the seismic response of an irregular 4-storey building that was damaged during the Lefkada earthquake. For this purpose, a three step assessment was performed involving a) the definition of a ‘realistic’, varying with depth, seismic input based on the available records and accounting for site amplification, liquefaction and simultaneous excitation along the two horizontal axes b) the simulation of the complete foundation below the building and the relative stiffness interplay between the piles and the soil layers c) the study of the inelastic dynamic response of the soil-foundation-structure system accounting for plastic hinge development and short column failure. The main conclusions that were drawn from this study can be summarized as follows:

- The structural irregularity and the initial design of the building have a significant effect on its overall seismic behaviour. In particular, the lack of shear walls and the presence of a soft basement lead to the torsional sensitivity of the structure which in turn results into large displacements of the perimeter columns and their subsequent distress.
- When the interaction of the structure with its foundation and surrounding soil is accounted for, it is shown that dynamic response of the structure is more complex and that higher modes are triggered. Consequently the complete modeling of the soil-structure system highlights some aspects of the dynamic behavior that could not be assessed otherwise.
- With the use of inelastic dynamic analysis, the observed damage is analytically confirmed and the agreement between numerical results and actual structural behavior is very good, since both the flexural damage of the basement front columns damage and the short column failure was verified.
- It is shown that the direction of excitation has to be accurately accounted for in order to match the ductility demand that was actually imposed to the structural member during the particular earthquake event.
- The comprehensive computational framework for the detailed representation of both the earthquake input and the dynamic response of the overall soil-foundation-structure system has revealed that in the particular case, both liquefaction and dynamic SSI effects play an important role in the behavior of the building.
- The observed damage pattern can be primarily attributed to the characteristics of input motion, the direction of excitation, the liquefiable soil depth and apparently the building's significant irregularity in plan and height. The latter observation though has to be considered within the framework of the building code that was enforced at the time of construction and the lack of detailed regularity criteria.

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