American Scientific Research Journal for Engineering, Technology, and Sciences (ASRJETS)

ISSN (Print) 2313-4410, ISSN (Online) 2313-4402



http://asrjetsjournal.org/

# Towards a Consistent Simplified Approach on the Strength of a Pier Based on Piles Founded on a Ground of Insufficient Stability

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**Abstract**

When a construction presents a huge vertical concentrated load (e.g. a bridge) and has to be founded on a ground of insufficient stability, the procedure to be followed is the use of pillars. However, in the case of a seismic event, the pillars are in danger due to: a) their fraction owed to high bending moments, due to the presence of significant horizontal soil movements along their length, b) shear failure of the foundation soil, in the case where the shear stress on a sliding surface, due to the static loading along with the seismic excitation, exceeds the local soil strength, c) a usually upwards pillar shift with respect to its surrounded soil, owed to the liquification of the underground. A description on the methods of analysis corresponding to the dynamic behavior of pillars along with the seismic event, concerning the collapse of the Higashi – Nada bridge of Kobe in Japan – after the earthquake of 17th January 1995 – is then to be presented.

***Keywords:*** Ground; Kobe; Pier; Pile; Soil.

## 1. Introduction

The problem is focused on the strength of a simple block system of a bridge in the form of a single column, presenting a concentrated mass, $M_{deck}$ and a second moment of area on the head of block, $I_{deck}$, which is founded through a base fixed on a group of friction pillars which present a full or partial fixity at their heads and are surrounded by a multi-layered soil formulation (Figure 1).

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The **pier** presents a modulus of elasticity $E_{pier}$, a diameter $D_{pier}$, a cross sectional area $A_{pier}$, a second moment of area with respect to the centroidal axis $I_{pier}$, a mass per meter of length $m_{pier}$ and a height H. The **pile** is a flecturally elastic straight bar having a circular cross section with a modulus of elasticity, $E_{pile}$, a diameter, d or a radius, R, a cross sectional area, $A_{pile}$, a second moment of area with respect to a horizontal centroidal axis, $I_{pile}$, a mass per meter of length, $m_{pile}$ and a height, L.

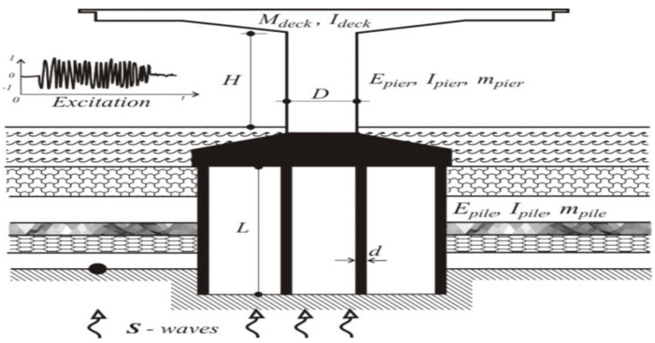


**Figure 1:** The whole block system

The whole system is set under a seismic vibration which is considered to be applied on the basis of the soil formulation (usually a rocky base) or on the ground surface. The analysis of the dynamic behavior of the system will be realized taking into account both the non-linear dynamic behavior for each single pile of the pile-group and the dynamic interaction between soil – foundation – over construction.

## 2. Methodology of analysis

The solution of problem is faced in stages. Initially the non-linear dynamic response of the single piles in axial and flexural vibration is calculated [1,2,3]. This calculation actually constitutes the core of the problem. Then the influence of soil non-linearity on the factors of dynamic interaction [4,5,5a,5b] between the piles is estimated and finally the seismic response [6,7,8,9,10,11,12,13,14,15,16,17,18,19] of the whole system of bridge is solved. More precisely:

### *2.1. Stage I: Determination of flexural stiffness [20,21,22,23,24,25,26]on the head of a single pile, under axial or shear loading*

The head of a pile undergoes a dynamic axial or shear load, the width of which is varying harmonically with respect to time:

$$P = P_c cos(\omega t) \qquad (1)$$

Where:

$P_c$: is the width of the applied load and

ω: is the cyclic frequency in $rad/s$

The environment pile – ground is characterized from the:

- shear modulus for small deformations, $(\gamma < 10^{-5}) G_{max}$,
- Poisson's ratio, ν,
- percentage of ground dumping for small deformations, $\xi_o$,
- index of plasticity, PI and
- density, ρ.

The ground can be respectively described with the modulus of elasticity for small deformations, $E_{max}$, which is connected with the shear modulus, $G_{max}$, through the relation:

$$E_{max} = 2\, G_{max}\, (1 + v) \qquad (2)$$

### *2.1.1. Methodology for solving the problem of axial and shear vibration*

#### *2.1.1.1. The fundamental approach*

The problem of axial or shear vibration of a pile is based on the acceptance that the environment of the pile – soil can be substituted from a series of springs and dumpers which operate independently, taking into account that the soil consists of a large number of thin layers independent to each other. The soil reaction at any depth is therefore independent to any other movement at different depths. This means that the seismic waves created on the surface between the pile and soil are propagated only horizontally (Figure 2), or equally a biaxial stress – strain analysis is dominant on the plane of propagation. The above acceptance is known as a *Winkler* spring – like – soil simulation and has a wide application on many similar problems of soil mechanics.

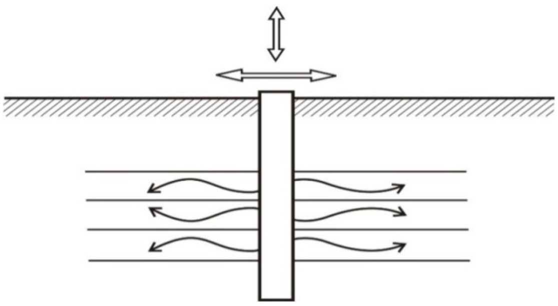

**Figure 2:** The fundamental approach: The waves are propagated only horizontally, creating a plain of radically symmetric or antimetric deformation

#### *2.1.1.2. Calculation steps*

Initially, before the application of any static or dynamic (axial or shear) load on the head of the pile, every soil layer that surrounds the pile is considered to be radically uniform. This uniformity means that there are not any of the unavoidable effects coming from the pile driving. The unloaded soil can be described through the shear modulus, $G_{max}$, which is a parameter that characterizes every layer, as depicted in Figure 3: a.

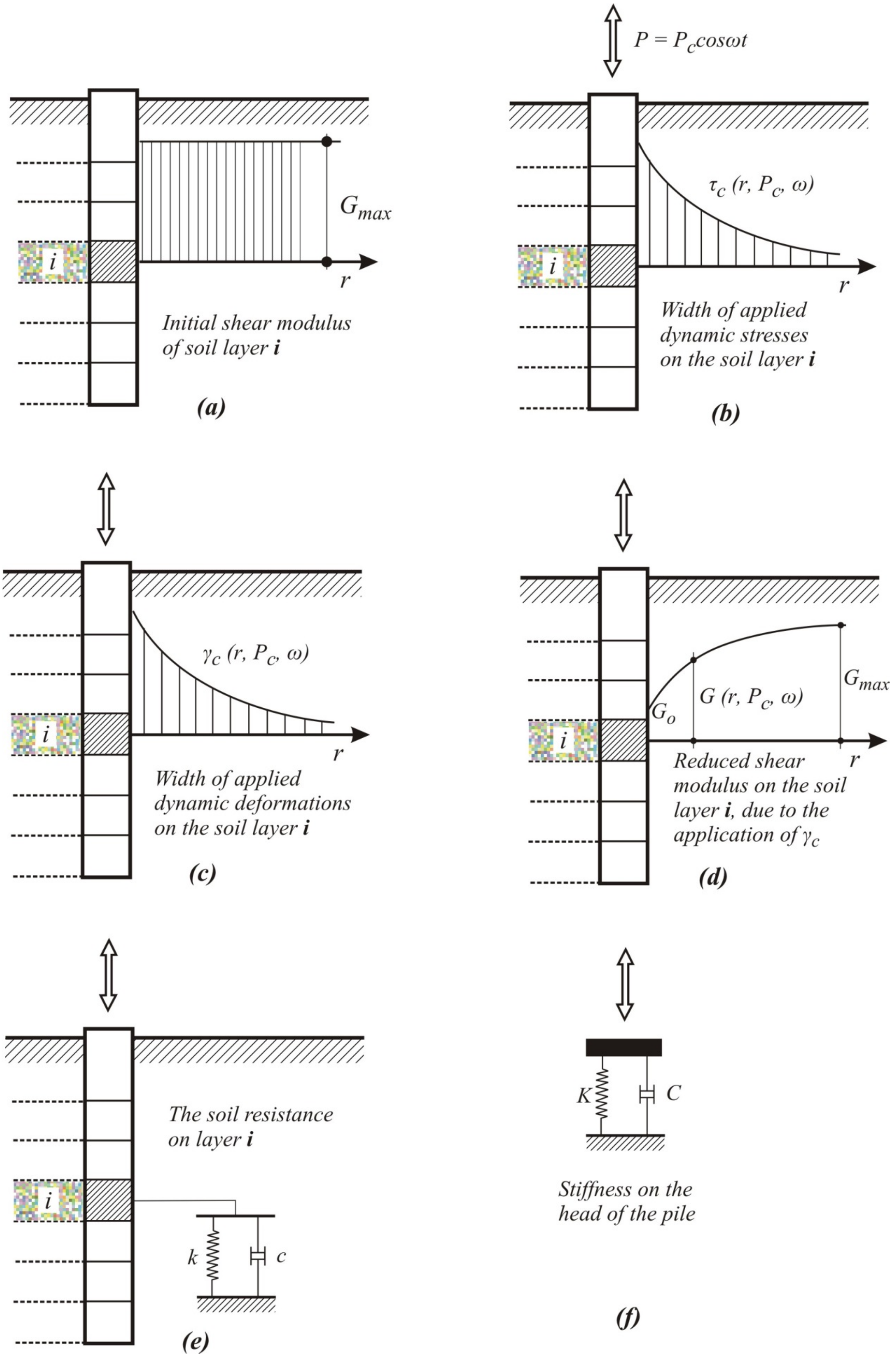


**Figure 3:** The calculation steps for the determination of the dynamic stiffness on the head of a single pile

For reasons of simplicity, without any constrain of generality, the case of axial loading of pile is here described in the presence of axial symmetry. Therefore, the application of axial load, $P_o$, on the head of a pile, results in the development of shear stresses, $\tau_o$, on the soil elements of each layer.

Through a good approximation, these shear stresses, $\tau_o = \tau_o(r)$, can be considered to be reduced inversely proportional to the radial distance, r, from the pile (Randolph & Wroth) [27,28]:

$$\tau_o(r) = \tau_o(R)\frac{R}{T} \qquad (3)$$

The application of dynamic load, $P_c\cos(\omega t)$, exerts on the soil elements further shear stresses, $\tau_c\cos(\omega t+\theta)$, on the top of the previous ones (Figure 4), where:

$\tau_c$: is the width of shear stresses and

$\theta$: is the phase difference between the shear stresses and the externally applied load.

In Figures 3: b and 3: c, a qualitative radial alteration of the dynamic shear stresses along with the corresponding shear deformations, $\gamma_c$, is depicted.

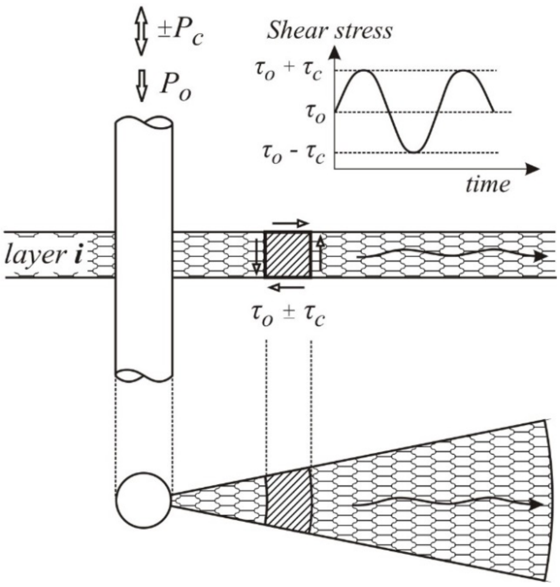


**Figure 4:** The stresses that are applied from pile on the soil elements

The application of shear deformations on the surrounding soil leads to a reduction of the soil shear modulus G. This reduction is due to the non-linearity of the stress – strain diagram for a soil element and is a function of the width of the applied deformation. It is hence expected that the reduction of G will be greater in the vicinity of the pile where the shear deformation is maximum, while it weakens with the increase of radial distance from the pile.

Similarly, the shear modulus, G, will be minimum on the surface between pile and soil and will be radially increased, tending to reach the value of $G_{max}$, for great radial distances (Figure 3: d)

In a similar way, the percentage damping coefficient, $\xi$, which is increased for greater shear deformations on the soil element, will present its maximum value on the surface between pile and soil, tending to reach the value of $\xi_o$, for greater radial distances.

The above comments lead to the conclusion that after the loading of pile, the environment pile – soil, in substance, ceases to be radially uniform.

The degree of the above so called ‘equivalent’ radially non-uniformity, is a function of both the level of the applied load and, of course, the material characteristics of soil. A fully elastic soil material would continue to remain radially uniform even after the pile loading. In reality, the soil materials, almost always, present a high level of inelasticity. In this analysis, the inelastic behavior of piles is realistically simulated through the use of experimental curves.

The determination of the modified – due to loading – soil parameters is followed by a calculation of the so called ‘dynamic coefficients of resistance’ corresponding to a layer of unit width. This resistance, forming the reaction coming from every soil layer under an applied deformation, is presented by a system consisting a simple spring and a simple damper connected in parallel (Figure 3: e).

The two constants of this system (k and c) can be calculated through the solution of the elastodynamic problem, dealing with the equilibrium of a unit width soil layer which is extended sideways to the infinity and surrounds the vibrated elementary part of the pile.

This calculation is realized for a certain each time frequency, which is separate for the axial and shear form of vibration. Finally, for each form of vibration, the ‘Composite Dynamic Stiffness’ on the head of the pile (Figure 3: f) can be determined through the solution of the differential equation that expresses the movement of the pile which is surrounded by the previously mentioned springs and dumpers and have substituted the surrounding soil.

For any type of vibration, the Composite Dynamic Stiffness is expressed through a ‘concentrated’ spring presenting a constant K and a ‘concentrated’ dumper presenting a constant C, connected in parallel.

### *2.2. Stage II: The effect of non-linear behavior of soil on the dynamic interaction between the piles which form a group*

Using the solutions of elastodynamic problem which has been described in the stage I, the radial change of deformation of the surrounding soil is determined, for each specific value of the applied load.

For each value of radial distance, this change is then compared with the elastic (linear) solution of the same problem, thus calculating the increase of tempo in the radial reduction of soil movements, due to the non-linearity of the soil material. Simple analytical expressions are suggested for the calculation on the effect that has the non-linearity of soil on the coefficients of interaction between the piles.

### *2.3. Stage III: Calculation of the seismic response for the foundation system – pier – deck*

The method is applied in the analysis of seismic response for the Higashi-Nada bridge, which failed during the earthquake on the $17^{th}$ of January 1995 in Kobe, Japan.

From stages I and II the frequential dynamic stiffness for the bridge's piles of group is calculated. For the solution of problem, concerning the seismic response of the whole system soil – foundation – pier – deck, the modified software code SPIAB (Mylonakis & Gazetas) [14] is used to analyze the system in substructures, where the seismic excitation is input as an accelerograph on the basis of the soil formation. The analysis in 'time steps' is realized by a consecutive use of the direct and inverse Fast Fourier Transform. Through the help of the wave theory and after taking into account the kinematic interaction of soil – foundation by the use of simulation of a springy soil, the seismic movement which excites the (flexibly founded) system, i.e. head node – pier – deck, is calculated.

Finally, the response of the over-construction is estimated through the solution of the conjugated equations of oscillation for a two-degree oscillator which is resting on the exciting soil through a system of springs and dumpers (dynamic stiffness of the pile group).

The non-linear behavior of pier is simulated through the equivalent – linear method while the outcoming results are compared with those resulting from the solution of a finite element elastoplastic code.

## 3. Comments - Remarks

In our previous discussion, the calculation of non-linear behavior for each pile of the group (Stages I and II) can be realized through the use of the equivalent linear method in a frequential analysis. In Stage III, where the seismic response of the whole system foundation – pier – deck is calculated, the analysis is materialized 'by time', using the Fourier Transform, which is known to be based on the principle of successiveness.

Given the 'incompatibility' for the principle of successiveness with the non-linear analysis, we have to highlight the following:

- Indeed, the 'by frequency' calculation of the non-linear stiffness of the pile group along with the 'by time' determination of the seismic response for the whole system can be adopted only as a simplified approach.
- In the most software programs widely used in practice by P/Cs, the foundation is simulated by springs (one for each direction), that are rarely accompanied by dampers, the constants of which are not dependent on the frequency of loading, but they correspond to a properly chosen frequency, which is usually the fundamental frequency of the system. In this case the non-linear behavior of the soil is taken quite 'grossly' into account, i.e. independently of the frequency, using the experience of reduction on the springs' constants (percentage of their elastic value, e.g. 50%). Therefore, a more accurate calculation for the reduced soil stiffness (for each of the 'properly chosen' frequency) constitutes a significant improvement for a more realistic simulation of the whole problem.

## 4. Application: Dynamic failure analysis of the piers of Higashi – Nada bridge during the earthquake of 17.01.1995 in Kobe [29,30,31,32,33,34,35], Japan

### *4.1. Introduction*

A method of antiseismic design for a pier foundation of bridge will be here presented.

It came after a real incident, concerning the failure of a bridge, in Japan.

The method comprises equivalent elastic analyses, where the non-linear behavior of piers is taken into account. A special emphasis is given in the simulation of the dynamic non-linear interaction of soil – piles – pier. Here the harmful role of the foundation subsidence has been proved, the ignorance of which could lead to incorrect estimations for the seismic response of the bridge.

The lacks of shear reinforcement, combined with the extraordinary high levels of the developed accelerations, along with the existence of flexible foundation, are proved to be crucial for the failure.

### *4.2. The earthquake at Kobe 1995*

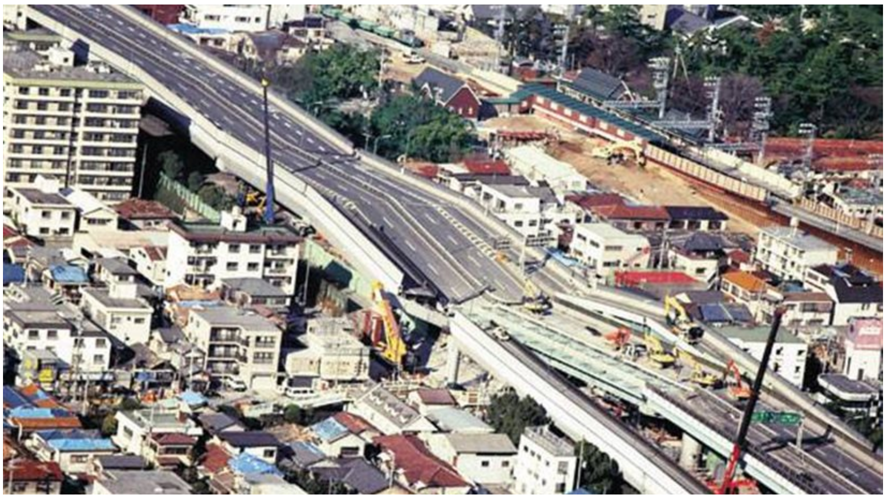

**Figure 5:** The Bridge Higashi – Nada, the whole length of which failed on a direction perpendicular to its axis, during the earthquake of Kobe, Japan

On the 17th of January, 1995 in the region of Nikawa city in Japan, a huge landslip was the result of the Hyogokken – Nambu earthquake. The magnitude of earthquake was 7.2 on the Richter's scale and the distance from the tectonic strike fault was less than 10 km. According to estimations, the maximum acceleration magnitude occurred in the region came to circa 0.5g. The declivity consisted of sand and clay, while the two thirds of the sliding surface were under the level of aquifer. The sliding was surface and the infected material volume circa 110,000 to 120,000 $m^3$. The moving of mass was greater than 80 m. In the following Figure 6, the region where the landslip took place is depicted.

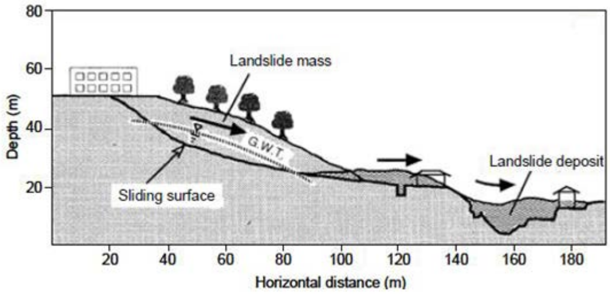


**Figure 6:** The region where the landslip took place

The earthquake, having shaken the city of Kobe in Japan, will certainly constitute a crucial point in the worldwide 'seismic' history. The extraordinary high levels of the developed ground-acceleration were the main reason for the constructions to suffer an unpredictable tension of trial. There was no kind of construction that had not suffered a serious damage. Port quay walls were moved up to 5m, multistoried buildings along with small houses were collapsed, large ground areas were liquified, hill slopes were glided, underground constructions and networks were failed.

Significant damages suffered also decades of bridges, among the hundreds that existed in the populous and technologically developed city of Kobe. Piers made of concrete and steel were failed and bridge decks were fallen from piers. However, more spectacular was the transversal turnover of the Higashi – Nada Bridge (Figure 7), 630m of length, which constituted a part of the super elevated motorway Hanshin Expressway Route No 3. This motorway crosses along the city of Kobe and consists of a series of consecutive bridges made of single-column piers, which present a length of more than 30 kilometers.

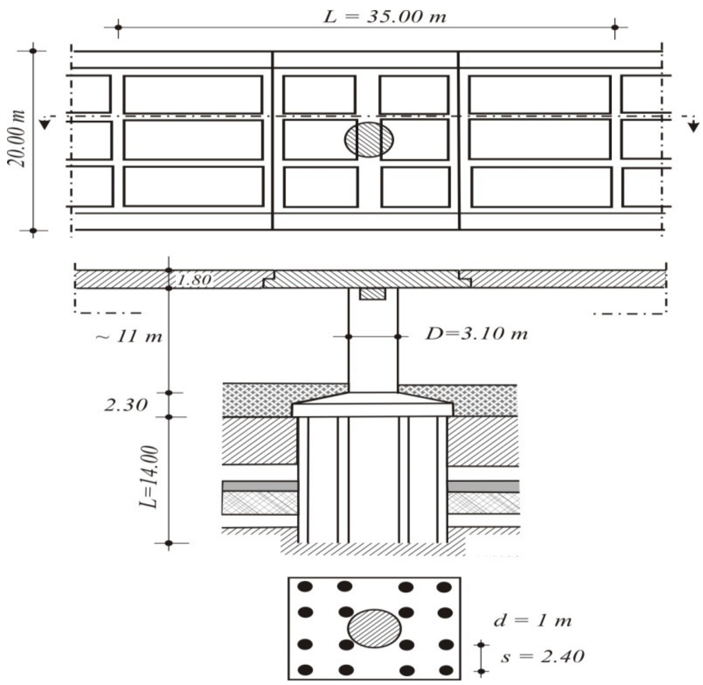


**Figure 7:** The geometrical characteristics of bridge

### *4.3. The local land effect on the form and intensity of the exciting seismic vibration*

Kobe is built along an oblong seaside strip (more than 30km in length and less than 3km in width) as shown in the photo of Figure 8.

The limits of the city are defined from the mountainous area which is raised almost next to the seaside. The granite rocky underground of the area which appears on the surface, sinks in a steep inclination under softer formations, so that along the seaside it reaches the depth of 1 to 1.5 km, as approximately depicted in Figure 9.

**Figure 8:** Aerial photo of the Kobe – Osaka district

In fact, some publications (Kawase) [31] and (Tokimatsu and his colleagues) [34] present the granite underground to sink almost quite perpendicular from the isobathymetric point 0.0km of Figure 9: a, in a depth of 1km. The ground layers consist of a soft rock (newly developed ground layers) and alluvial fans, i.e. sand gravel -mainly ground of 10 to 100m thickness- with alternating layers of clay, as shown in Figure 9: b.

The width of the ground layer affects and modifies the intensity of the passing seismic waves. Therefore, in deep and soft ground formations it is expected to record long period vibrations, while in positions where the rock appears on the surface, or the ground is hard, or it has a very small thickness, it is expected a vibration of high frequency. This general prediction has been verified also during the earthquake of Kobe, as descriptively is shown in Figure 9, where also acceleration spectra of different positions, i.e. ground layers of different thicknesses, are presented. Obviously, the softer becomes the ground the greater is the period to which the maximum spectral acceleration is corresponding.

On the other hand, it has to be noted that the spot positions of Figure 9 present a difference not only to the width of layer but also to the distance from the fault, something which, more or less, affected the intensity of the seismic vibration. The break stone zone, as defined from geological metaseismic observations, practically coincides with the zone of maximum damage, shown in Figure 11.

The width difference between the alluvium layers at different positions (geotechnical factor) but also the distance from the fault (geological factor) resulted in a different vibration at various positions, thus defining a different distribution of damages in both the city of Kobe and the surrounding area. Of course, it is obvious that the characteristics of seismic vibrations are also affected from other factors like the brake mechanism, the fault characteristics, the rock inclination and the geology of the interpolated seismic wave route.

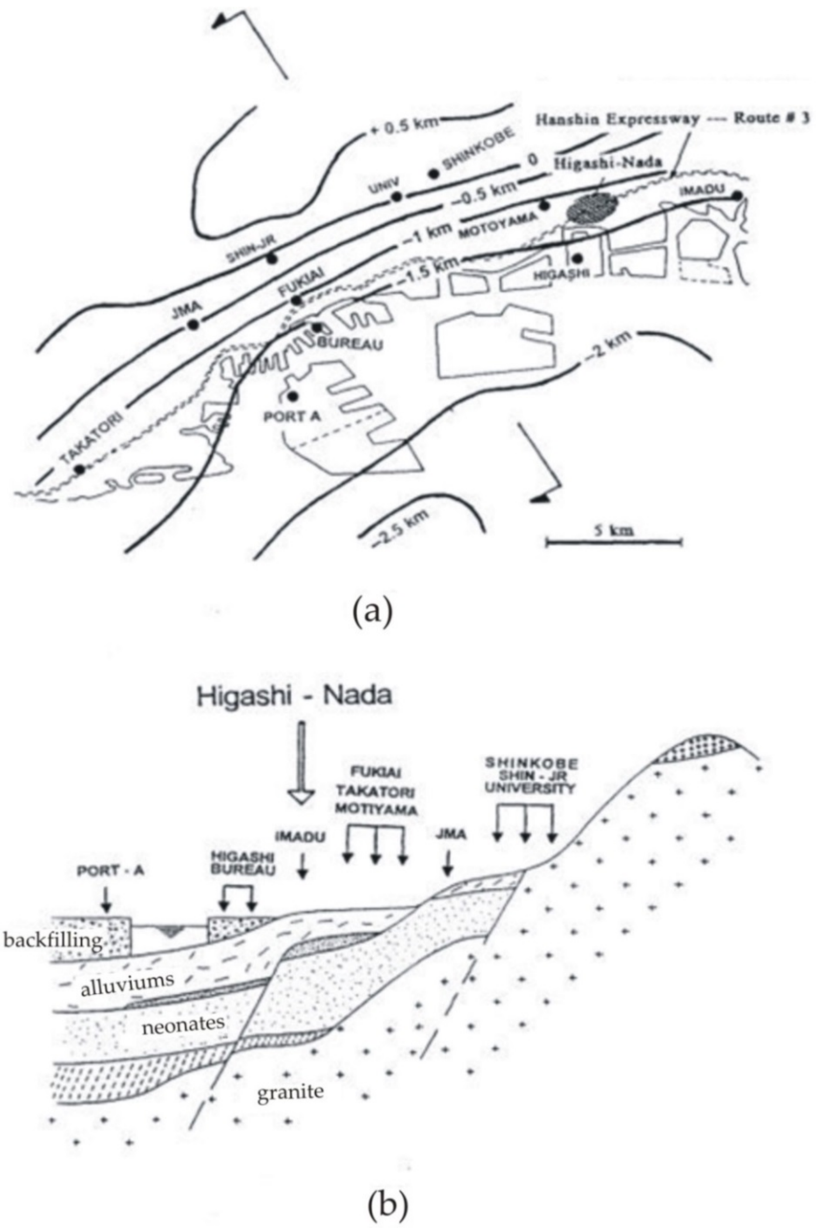


**Figure 9:** a. The area of Kobe with the contour lines showing the depth up to the maternal rock

b. An approximate geological cut across the Kobe area, showing the spots of accelerographs along with the position of bridge

It is therefore concluded that the seismic characteristics of a vibration at any position were also a function of the local width of ground layer.

The *Japan Meteorological Agency (JMA) seismic intensity scale* (Wikipedia) [36] is a seismic scale used in Japan and Taiwan to measure the intensity of earthquakes. Unlike Richter magnitude scale, which attempt to describe the energy released by the earthquake, the JMA scale describes the degree of shaking at various points on the Earth's surface, and is analogous to the Mercalli intensity scale. The intensity of an earthquake is not totally determined by its magnitude, but varies with event's depth, and distance from the event.

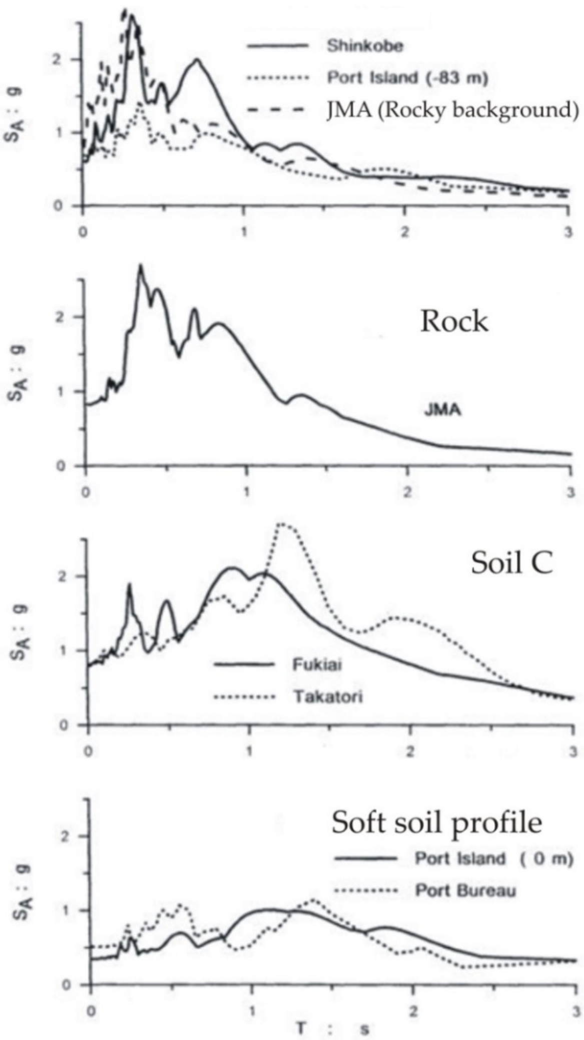


**Figure 10:** The acceleration spectra of the principal records during the Kobe earthquake, discriminating the nature of ground and the location of recording

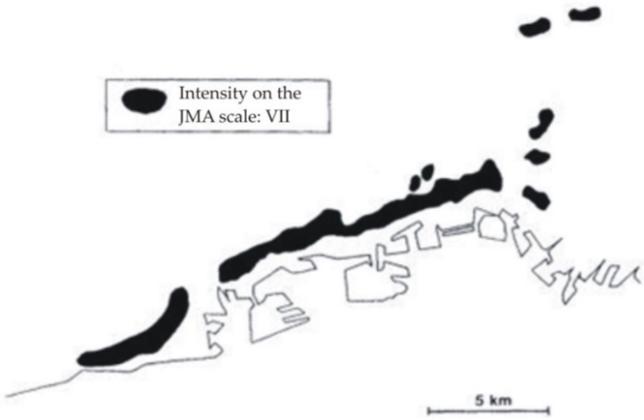


**Figure 11:** The damaged zone during the Kobe earthquake. It practically coincides with the brake stone zone

In the analysis that follows the recordings of JMA and FUKIAI are used (modified where necessary), two recordings with maximum accelerations PGA = 0.85g, but far different spectral characteristics, the time histories and the response spectra of which are given in Figure 12. In the same figure the TAKATORI recorded spectrum is depicted, the case of which, for the particular bridge, consists a specially unfavorable but completely probable possibility.

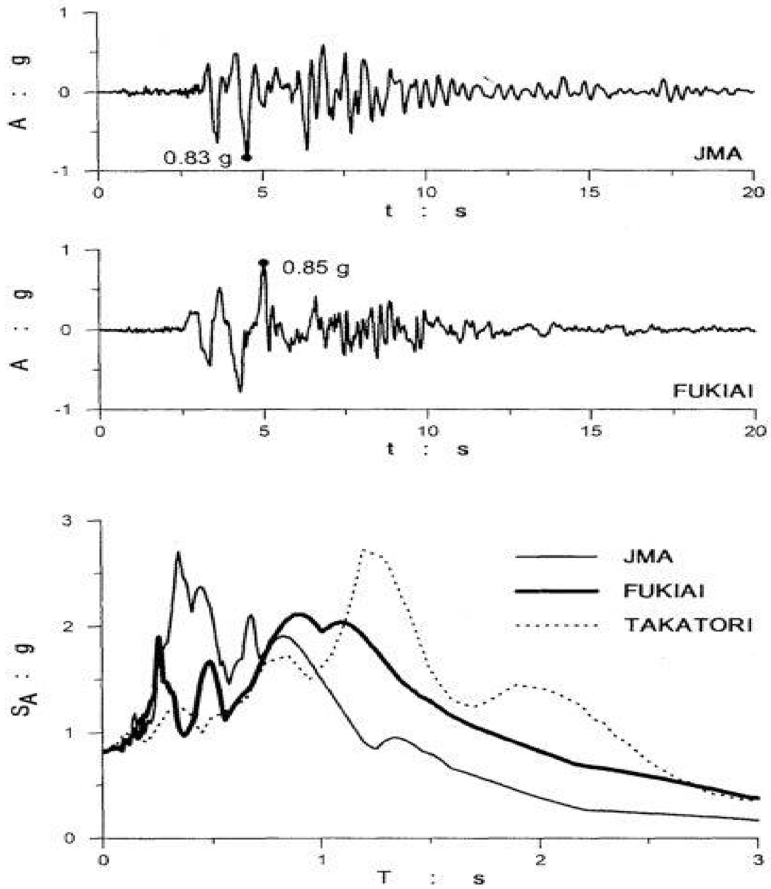


**Figure 12:** Time histories and acceleration spectra of three recordings at Kobe

### 4.4. Elastoplastic analysis

For a more precise documentation on what has been so far discussed, we take resource to existing elastoplastic dynamic analyses, e.g. through the use of ADINA, 1995 software. In Figure 13 the 'route to failure' has been designed, according to the existed technical construction characteristics but mainly from the observations that followed.

## 5. Conclusions

In this paper a new method of antiseismic design for bridge foundations and piers has been presented, where, the full system soil – foundation – pier – deck has been analyzed, taking into account both the non-linear behavior of the foundation ground and the non-linear behavior of the pier of bridge. The presentation of method was due to a real seismic event, through which the following conclusions are being drawn:

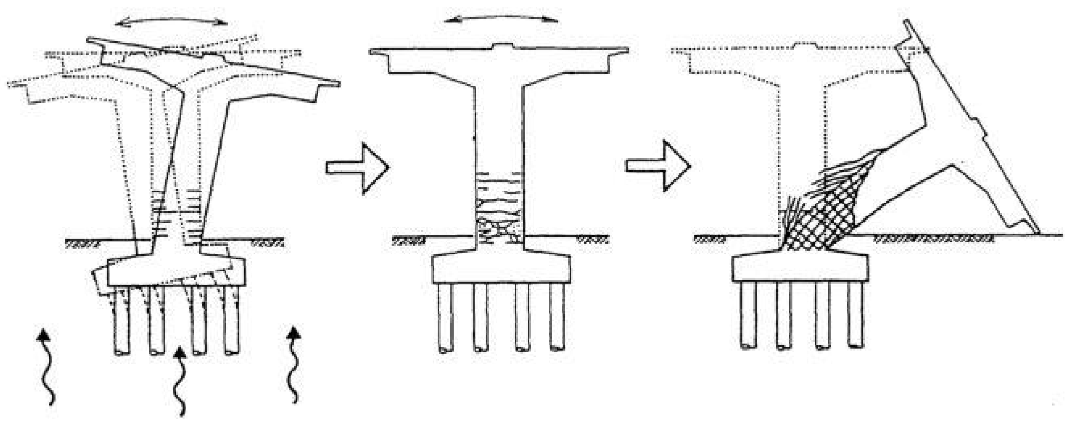

**Figure 13:** The route to the failure: The simultaneous flexural and shear failure

- The affection of soil flexibility is not (as unfortunately there is a widespread impression) always in favor for the structure. Specifically in cases where the strength of construction is marginal, the soil flexibility may play a decisively negative role. This is dependent on the flexural flexibility of the construction along with the precise spectral content of the seismic vibration. The continuous decrease of $S_\alpha$ with an increase of the natural period is a solution for the needs of regulations and not for the natural reality, a realistic approach of which can be realized through the analysis of the full system of bridge.
- The influence of soil non-linearity in both the behavior of a single pile and the interaction among the piles is possible to be significant and affect the dynamic characteristics of the whole system.
- The non-linear behavior of pier is usually proven to be a significant factor for the seismic response of bridges. However, the equivalent linear method seems to be capable for a realistic description of the whole phenomenon [37].
- A serious seismic event puts all the structures through a hard test. As a result all the weaknesses generated in the structure, due to either code imperfections or analysis and design errors, or even bad construction are readily apparent. This is why strong earthquakes usually lead to improvements or even

drastic changes to the design codes along with modifications on the design and execution of the construction works. It is difficult to classify the damage caused by an earthquake. This is due to dynamic character of the seismic action and the inelastic response of the structure [38]. It is therefore obvious that an earthquake design must be realized by people carrying a deep knowledge of the seismic phenomenon along with its parameters that affect the response of structures.

**References**

[1] O. Michaelides, G. Gazetas & G. Bouckovalas (1996), "Axial and transverse nonlinear dynamic analysis of a pile", Proceedings of 3rd PanHellenic Conference of Geotecnique, Patras I, 435-442.

[2] O. Michaelides, G. Gazetas, G. Bouckovalas & E. Chrysikou (1996), "Approximate nonlinear dynamic axial response of piles", Geotechnique.

[3] O. Michaelides, G. Bouckovalas & G. Gazetas (1996), "Non – linear soil properties and impedances for axially vibrating pile elements", Soils & Foundations, The Japanese Society of Soil Mechanics and Foundation Engineering.

[4] G. Gazetas, K. Fan, A. Kaynia & E. Kausel (1991), "Dynamic Interaction factors for floating pile groups", Journal of Geotechnical Engineering, ASCE vol. 117, pp. 1531-1548.

[5] N. Makris & G. Gazetas (1992), "Dynamic pile – soil – pile interaction, Part II: Lateral and seismic response", Earthquake Engineering and Structural Dynamics vol. 21, pp. 145-162.

[5a] A. Comodromos, (2008), "Interaction between soil and constructions" University of Thessaly, Volos.

[5b] A. Comodromos, (2008), "Calculating Geotechnical Mechanics. Interaction between soil and constructions", Kledarithmos Publications, Athens.

[6] G. Gazetas, (1996) "Soil dynamics & Seismic Mechanics", Athens: Symeon Editions.

[7] K. Fan, (1992), "Seismic response of pile foundations evaluated through case histories", PhD thesis, New York: State University of New York at Buffalo.

[8] G. Gazetas, (1983), "Analysis of machine foundation vibrations: State of the art", Soil Dynamics and Earthquake Engineering 2, No. 1, pp. 2-43.

[9] G. Gazetas & N. Makris, (1991), "Dynamic pile – soil – pile interaction. Part I: Analysis of axial vibration", Earthquake Engineering and Structural Dynamics 20, No. 2, pp. 115-132.

[10] K. Ishihara, (1986), "Evaluation of soil properties for use in earthquake response analysis", Geomechanical Modelling in Engineering Practice, R. Dingar and J.A. Studer, eds., Rotterdam, the Netherlands: A. Balkema, pp. 241-275.

[11] A.M. Kaynia & E. Kausel, (1982), “Dynamic stiffness and seismic response of pile groups”, Research Report R82-03, Massachusetts Institute of Technology.

[12] K. Kawashima, (1995), “Impact of Hanshin – Awaji Earthquake on Seismic Strengthening of Highway Bridges”, US National Seismic Conference on Bridges and Highways, San Diego, California, USA.

[13] N. Makris & G. Gazetas, (1992), “Dynamic pile – soil – pile interaction, Part II: Lateral and seismic response”, Earthquake Engineering and Structural Dynamics, vol. 21, pp. 145-162.

[14] G. Mylonakis & G. Gazetas, (1995b), “SPIAB: A computer program for seismic response of pile – supported bridges including soil – pile – structure interaction”, New York: State University of New York at Buffalo.

[15] G. Mylonakis, (1996), “Contributions to static and seismic analysis of piles and pile – supported bridge piers”, Ph.D. thesis, New York: State University of new York at Buffalo.

[16] M.J. Pender & S. Pranjoto, (1996), “Gapping effects during cyclic lateral loading of piles in clay”, Proceedings 11th World Conference on Earthquake Engineering, Mexico, Paper No. 1007.

[17] PILEGROUP (1994), “A computer program for the dynamic impedance and seismic response of pile groups”, New York: Department of Civil Engineering, State University of New York at Buffalo.

[18] P.B. Schnabel, J. Lysmer & H.B. Seed, (1992), “SHAKE91: A computer program for conducting equivalent linear seismic response analysis of horizontally layered soil deposits”. Centre for Geotechnical Modelling, Davis, California: Department of Civil Engineering & Environmental Engineering, University of California.

[19] J.P. Wolf & G.A. Von Arx, (1978), “Impedance functions of a group of vertical piles”, Proceeding of the Speciality Conference on Earthquake Engineering and Soil Dynamics, ASCE, Pasadena California: ASCE, Vol. 2, pp. 1024-1041.

[20] G. W. Blaney, E. Kausel & J.M. Roesset, (1976), “Dynamic stiffness of piles”, Proceedings of 2nd International Conference on Numerical Methods in Geomechanics, Virginia Polytechnic Institute and State University, Virginia 2, pp. 1001-1012.

[21] R. Dobry & G. Gazetas, (1988), “Simple method for dynamic stiffness and damping of floating pile groups”, Geotecnhique 38, pp. 557-574.

[22] R.J. Jardine, M.J. Symes & J.B. Burland, (1984), “The measurement of soil stiffness in the triaxial apparatus”, Geotechnique 34, No 3, pp. 323-340.

[23] J.M. Roesset & D. Angelides, (1980), “Dynamic stiffness of piles”, Numerical Methods in Offshore Piling, pp. 75-81, London: Institution of Civil Engineers.

[24] J.M. Roesset, (1980), “Stiffness and damping coefficients of foundations”, Special Technical Publication on Dynamic Response of pile Foundations: Analytical Aspects, eds. O’ Neil & Dobry, New York: ASCE.

[25] J.M. Roesset, (1984), “Dynamic stiffness of pile groups”, Pile Foundations, New York: ASCE.

[26] F. Tatsuoka & Y. Kohata, (1995), “Stiffness of hard soils and soft rocks in engineering applications”, Report of the Institute of Industrial Science, No. 242, Tokyo: The University of Tokyo.

[27] M.F. Randolph & P.C. Wroth, (1978), “Analysis of deformation of vertically loaded piles”, Journal of the Geotechnical Engineering Division, ASCE 104, No GT12, pp. 1465-1488.

[28] M.F. Randolph & P.C. Wroth, (1979), “Analysis of vertical deformation of pile groups”, Geotechnique 29, No. 4, pp. 423-439.

[29] G. Gazetas, O. Michaelides, K. Lukakis & G. Mylonakis, (1995), “The Kobe earthquake (Japan) of 17.01.1995. Review, comparisons, preliminaries, conclusions”, Bulletin of Technical Chamber of Greece, No 1851.

[30] E. Goufa, (1995), “Hanshin Expressway, Kobe 1995: Analysis of a pier that has not failed, in the view of soil – pile – over construction interaction”, Thesis, Athens: Geotecnique Sector, National Technical University of Athens.

[31] H. Kawase, (1996), “The cause of the damage belt in Kobe: The basin – edge effect. Constructive interference of the direct S – Wave with the basin – induced diffracted/ Rayleigh waves”, Seismological Research Letters 67, No 5, pp. 25-34.

[32] O. Michaelides, G. Gazetas & K. Loukakis, (1995), “The Kobe Earthquake. The role of ground, the damages in bridges and harbors: overview, analyses, conclusions”, Bulletin of the Society of Civil Engineers in Greece, Issue 227, pp. 33-53.

[33] O. Michaelides, G. Mylonakis, G. Gazetas, E. Louka & E. Goufa, (1996), “Analysis of the Higashi – Nada (Hanshin Expressway) bridge failure in Kobe: The double role of ground”, Proceedings of 3rd PanHellenic Conference of Geotecnique, Patras I, pp. 519-526.

[34] K. Tokimatsu, H. Arai & Y. Asaka, (1996), “Three – dimensional soil profiling in Kobe area using microtremors”, Proceedings of the 11th World Conference on Earthquake Engineering, Mexico, Paper No. 1486.

[35] O. Michaelides, G. Gazetas, (1996), “A dynamic analysis of failure for the piers of Higashi – Nada bridge during the Kobe earthquake”, Proceedings of the 12th Greek Conference of Concrete, Lemesos III, Cyprus, pp. 236-247.

[36] Wikipedia, (2017), "The headword JMA was retrieved from the official site of Wikipedia" https://en.wikipedia.org/wiki/Japan_Meteorological_Agency_seismic_intensity_scale.

[37] V. Margaritopoulou, (2013), "Non-linear dynamic seismic analysis of friction piles", Thesis, Technological Educational Institute of Thessaly, Larissa, Greece.

[38] P.G. Lokkas, (2011), "Design of Concrete Structures Subjected to Seismic Actions", Textbook for MSc Postgraduate Courses in the Structural Design and Construction Management.