SOME ASPECTS OF THE ANALYSIS OF PILE FONDATIONS BEHAVIOUR UNDER SEISMIC ACTION

ABSTRACT

The paper presents a seismic analysis of the structure-pile-soil system, of a 2D RC frame. The analysis of individual system elements and some potential damage on two Vrancea accelerograms, VR77NS and VRfoc86NS are presented. The impact of the response spectra is provided for VR77NS, because the structure enters the resonant area and the damage increase considerably. Local drift diagrams during the earthquake, and the model damage featured as plastic hinges condition at the end of accelerograms are provided. It is indicated that it is necessary to introduce a dynamic interaction of the structural system, which includes not only the piles, but soil as well, because it became possible at the present level of scientific and technological progress of the human kind.

KEY WORDS: seismic analysis, piles, dynamics SPSI, plastic hinges, response spectra

NEKI ASPEKTI ANALIZE PONAŠANJA TEMELJA NA ŠIPOVIMA POD SEIZMIČKIM DEJSTVIMA

Boris Folić*, Radomir Folić**, Asterios. Liolios***, Mladen Ćosić

** Univerzitet u Beogradu, Inovacioni centar Mašinskog fakulteta, Kraljice Marije 16, e-mail: boris.folic@gmail.com*

*** Univerzitet u Novom Sadu, FTN, Dep. za građevinarstvo i geodeziju, Trg Dositeja Obradovića 4, r.folic@gmail.com*

**** Democritus University of Thrace, Dept. Civil Engineering, Xanthi, Greece, e-mail:* aliolios@civil.duth.gr

***** IMS institute, Bul. Vojvode Mišića 43, Beograd. e-mail: mladen.cosic@ymal.com*

REZIME

U radu su prikazani neki elementi analize šipova. Detaljnije je prikazana seizmička analiza sistema konstrukcija-šipovi-tlo, jednog 2D AB rama. Prikazan je analiza pojedinih elemenata sistema, neka moguća oštećenja na dva akcelerograma Vrančee, VR77NS i VRfoc86NS. Uticaj spektra odgovora dat je za VR77NS, jer konstrukcija tada ulazi u rezonantno područje i značajno se povećavaju oštećenja. Dati su dijagrami pomeranja, krajeva stubova (local drift) tokom zemljotresa i oštećenja modela kao stanja plastičnih zglobova na kraju akcelerograma. Ukazano je na neophodnost uvođenja dinamičke interakcija sistema konstrukcija u koje je potrebno uključiti na samo šipove, već i tlo, jer to sada omogućava postignuti naučno tehnološki razvoj čovečanstva.

KLJUČNE REČI: seizmička analiza, šipovi, dinamička interakcija konstrukcija-šip-tlo, plastični zglobovi, spektar odgovora

INTRODUCTION

Piles are ostensibly simple structures, they resemble piers, but since they interact with the soil, they require special attention. Static pile-soil interaction is a relatively simple problem, when the system is observed separately and linearly, but it often becomes a complex structure-pile-soil system, especially when the seismic action is introduced. Behavior of piles in a dynamic interaction with the soil, and its special case of seismic action, was studied by many authors such as: Penzien 1970, Novak 1980, Mayer and Rees 1977, Nogami 1987, Dowrick 1978, Pender 1983, Gazetas 1984, Tazoh 2000, Poulos and Davis 1970, Mylonakis at all 1997, Prakash 1981, Meymand 1996, Makris and Badoni 1998, JSCE 2000, Finn 2002, Bhattacharya at all 2004, Suarez 2005, Todorovska and Trifunac 2006, Milović and Đogo 2009, Madabhushi at all 2010 etc.

A considerable number of other references can be attributed to most of the authors/researchers mentioned above, often preceding those stated above, but this selection can be considered sufficient for this scope of the paper. For that reason, the valuable contribution of individual authors is further briefly described.

In (Poulos, 2017) a simplified approach was set out whereby a practicing foundation designer can undertake the relevant calculations to satisfy the requirements for deep foundation design in seismic areas. It includes pile design for axial loading, including the possible effects of liquefaction, and pile design for lateral loading where liquefaction does and does not occur. Measures to mitigate the liquefaction effects are recommended.

Todorovska and Trifunac researched the VN7S hotel in Los Angeles, which is founded on piles. Ambient vibrations (small dilatations), as well seismic tests were studied under a number of earthquakes. Changes of values of structural oscillation eigenperiods due to the earthquake damage were analyzed, but also propagation, refraction and reflection of the waves through the specific paths of the superstructure and in interaction with the soil around the structure. Trifunac, in the research lasting several decades from the end of the 70's of the 20th century, observed that on this building, after the San Fernando earthquake, a torsion (ambient) oscillation tone emerged. It was also observed that in a number of years, the soil may "consolidate" and partially recover its bearing properties, but not so the superstructure.

Novak, as early as by the beginning of the 70's provided a considerable contribution to the study of the dynamic interaction of piles and soil, using FEM. He also presented solutions in the analytical form, continuing the research of Bereduga at all, but through the Fourier transform, Henkel and Bessel functions. He studied the dynamical effects of a group of piles (as well as Nogami, Gazetas, Mylonakis godine?), and determined that there often was a considerable difference in the dynamical behaviour of an individual pile and a group of piles. He analyzed composite vertical-horizontal-rotating vibrations, in homogenous and stratified soil, as well as the effect of the intensity of normal force.

Wolf introduces frequency analysis, dividing the pile and the soil into conical disks. Makris, Badoni, Gazetas, (Rovithis, Pitilakis, and Mylonakis, 2009) also consider frequency analysis, and combine it often with the dynamical impedance, and inertial and kinematic interaction.

Mayer and Rees, Matlock, introduced p-y and p-z curves for experimental static and hysteretic load, and cone and block as a sand failure mode (after Mosh er and Dawkins, 2000).
Dowrick explained radiation damping, as well as the pile model in a stratified soil.

Finn, Meymand, Madabhushi, Gazetas, Bhattacharya, Tazoh, Dobry at all, studied liquefaction in piles. Bowen, Čubrinovski and Jacka, 2007 considered seismic strengthening by adding piles in liquefiable soil, because of the potential lateral spreading.

RESEARCH METODS OF PILES IN EARTHQUAKES

Table 1. Linear and non linear behaviour of soil - pile –structure system elements
Table 1. Linearno i nelinearno ponašanje elemenata sistema konstrukcija temelj tlo

System Element	Linear (or nonlinear) analysis	Nonlinear analysis	Exists or not	Analysis
Structure	Linear	Nonlinear		PO/TH/FA
Foundation - raft	Linear	Nonlinear	Yes/No**	PO/TH/FA
Foundation – pile	Linear	Nonlinear		PO/TH/FA
Weak or slip and inner zone	-	Nonlinear	Yes/No***	PO/TH/FA
Link elements	Linear: Elastic (or secant*)	Nonlinear p-y or p-z curve	Yes/No	
Soil	Linear	Nonlinear		

* Secant method is practically a linearized nonlinear soil model
** For some types of bridge piers, no top beams or decks are constructed.
*** If necessary, for instance because of a more precise analysis, negative friction etc.:
PO PushOver, TH Time history, FA Frequency Analysis can be introduced

In essence, the piles can be considered using the decomposition and integral methods. When the model of the structure-pile-soil system is divided into substructures, it is then the decomposition method. The decomposition method is usually used to analyze cinematic and inertial interactions. Frequency analysis is used in the determination of dynamic impedance as well as in the integral method. A special method can be introduced, by analogy with structural statics methods, but adapted for these models: i.e. element

substitution method. The element substitution method can be used with both the decomposition method and the integral method. To facilitate the determination of these methods (and combinations thereof), Table 1 is formed.
In table 1 it is possible that the weak zone of soil around the pile and the soil is completely replaced with nonlinear curves, or that the pile-weakened soil contact zone is modelled with an added linking element. *P-y* curves are used for the horizontal direction, and *p-z* for vertical reaction etc. The Nogami model is presented in figure 2 with a number of nonlinear springs and damping. All these methods can be quasi-static PO, dynamic TH or frequency FA.

Stratified soil additionally complicates this problem, but it is not considered in this paper, except for explanation of standing piles, or through use of substituting soil models (figure 1).

When applying the *p-y* and *p-z* curves, it is important to use hysteresis curves, since they determine the dynamic behaviour of the soil in contact with the pile in more detail. Correction of hysteresis curves with respect to behaviour under dynamic action is only possible if dynamic testing exists, and it is usually performed in tanks on platforms, or on centrifuges on scaled-down models.

Figure 1 Typical stiffness profiles for foundation strata a) Constant Stiffness (Typical of overconsolidation clay) b) Parabolic stiffness (Typical of sand), c) Linearly increasing stiffness (Typical of soft clay)
Slika 1 Tipični modeli promene krutosti tla po dubini: a) konstantna krutost (tipična za prekonsolidovane gline) b) parabolična krutost (tipična za pesak) c) linearna krutost (tipična za meke gline) SLIKA DA BUDE NA ENGLESKOM …

Each of the soil models in figure 1, have a corresponding stiffness and damping. Pile damping is provided in table 2. When using p-y curves as hysteresis multiplastic (MP) link elements it is necessary to determine a linear and nonlinear part of the link (element). The linear part of an MP link in sand is linearly variable (increasing) by depth, figure 1c. The variable of the initial stiffness (formula 1) k_o by depth is provided in API recommendations, (Folić B. at all 2018), for saturated and dry sand, and three states of compactness.

$$k = k_o \cdot y \tag{1}$$

Static stiffness can be seen as a boundary problem of stiffness, when frequency tends to zero, then dynamic stiffness tends to be static value. Dynamic stiffness is generally calculated as dynamic impedance, which actually consists of two parts, dynamic stiffness and twisting. Both dynamic stiffness and damping are generally frequency dependent.

$$S(\omega) = \frac{R(t)}{U(t)} \text{ dynamic impedance}$$

(2)

$$S(\omega) = K(\omega) + i\omega C \tag{3}$$

$S(\omega)$ - Impedance for a specific characteristic form (tone) of response: translation, rotation, etc. (referring to the ratio of dynamic force vs. corresponding displacement, or dynamic moment to rotation)

$R(t)$ - Dynamic force or moment

$U(t)$ - Dynamic displacement or rotation

$K(\omega)$ - Dynamic stiffness of the pile (kN/m)

ω - Frequency (rad/s)

C - Damping coefficient (kNs/m)

i - Imaginary number

$$\zeta(\omega) = \frac{\pi f C}{K} = \frac{\omega C}{2K} \tag{4}$$

$$S(\omega) = K[k(\omega) + 2\zeta(\omega) i] \tag{5}$$

Table 2. Dimensionless pile head damping coefficients for $f > f_n$ (after Gazetas 1984)

Tabela 2. Bezdimenzionalani koeficijenti prigušenja glave šipa za $f > f_n$ (prema Gazetasu 1984)

Soil Model	ζ_{HH}	ζ_{HM}	ζ_{MM}
$E = E_s$	$0.80\beta + \frac{1.10 f D}{v_S}($	$0.80\beta + \frac{0.85 f D}{v_S}($	$0.35'\beta + \frac{0.35 f D}{v_S}$
$E = E_s\sqrt{z}$	$0.70\beta + \frac{1.20 f D}{v_S}($	$0.60\beta + \frac{0.70 f D}{v_S}($	$0.22\beta + \frac{0.35 f D}{v_S}($
$E = E_s \cdot z /$	$0.60\beta + \frac{1.80 f D}{v_S}$	$0.30\beta + \frac{1.00 f D}{v_S}$	$0.20\beta + \frac{0.40 f D}{v_S}$

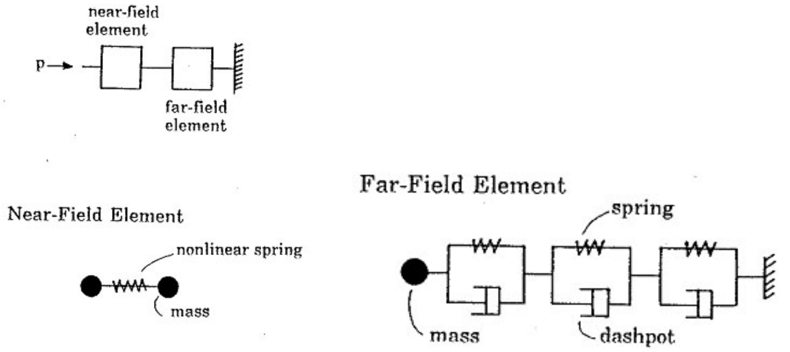

Figure 2 Nogami's Far Field Soil-Pile Models for Horizontal Excitation (after Nogami et al., 1987)
Slika 2. Nogamijev model sa bliskim i daljim poljem tla za horizontalnu pobudu (prema Nogami 1987).

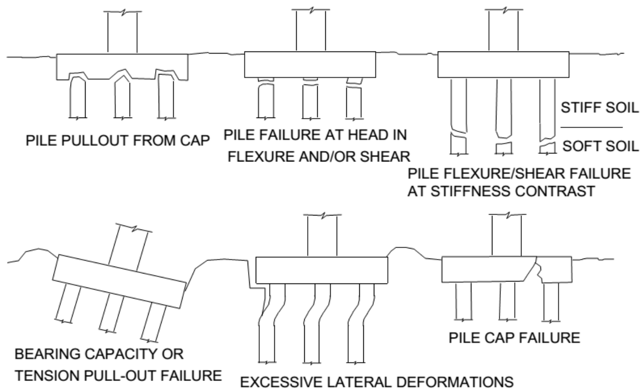

Figure 3 - Potential Failure Modes for Pile Group Foundations Subjected to Seismic Shaking
Slika 3 Potencijalni oblici loma grupe šipova kada je temelj izložen seizmičkim potresima

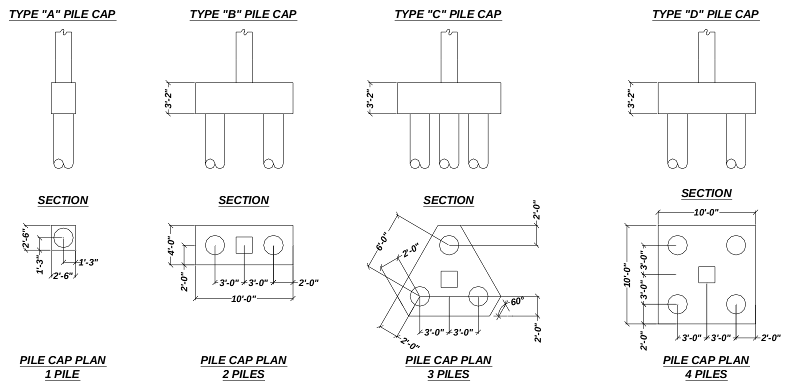

Fig. 4 Foundation plan and primary types of (raft) pile-pile-cap combinations.
Slika 4 Osnova temelja i osnovni tipovi kombinacije (naglavice) šip-glava-šip.

SEISMIC ANALYSIS OF THE RC FRAME OF THE OVERBRIDGE

In the paper (Folić B., 2017; Suarez 2005) for the analysis of the seismic response of the middle frame of an overbridge, different soil models were studied. The frame consists of 4 piles, which extend as piers above the soil. The soil models such as linear elastic springs and nonlinear models using p-y curves for sand are researched. P-y curves for saturated and dry sand, according to Matlock and Rees, are used, but also the modified curves. The soil is observed as single layer and two layer soil, and the standing piles, restrained at the pile toe. Earthquake action during the time history (TH) for four types of accelerograms is researched: first ElCentro, second Vrancea 77 and 2 accelerograms Vrancea 86. Basic models, without tie beams are examined. A brief research report is provided here.

SEISMIC RESPONSE TO VR77NS SINGLE-LAYER SOIL

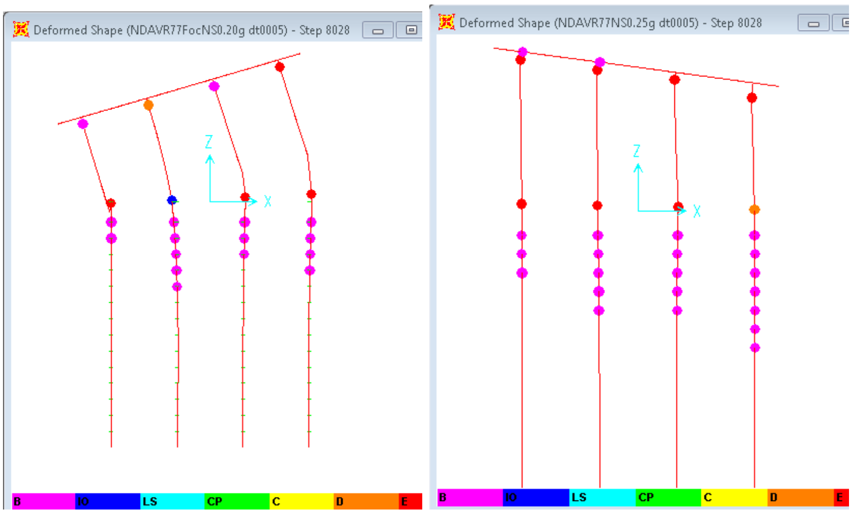

Figure 5 NDA State at the end of earthquake VR77NS acc., PGA 0.20 g. Soil as single layer p-y: φ=34; b=1.2 m; γ=17.6 kN/m3; k=16307 kN/m2, left PGA 0.20 g fracture of construction, right PGA 0.25g,.

Slika. 5 NDA Stanje na kraju zapisa ubrzanja VR77NS. Tlo jednoslojno p-y: φ=34; b=1,2 m; γ=17,6 kN/m3; k=16307 kN/m2, gore levo PGA 0,20 g slom konstrukcije, gore desno PGA 0,25g,

Table 3 Plastic hinge state, at the end of earthquake VR77 acc
Tabela 3 Stanje plastičnih zglobova na kraju akcelerograma zemljotresa VR77

	PGA 0.20 *g*		PGA 0.25*g*
	-	Road deck-RK:	2Y
Pier tops:	2Y+1D+1E	Pier tops:	4 E (1Y/pier)
Pier bases:	1IO+3E	Pier bases:	1D+3E
Tie beams:	-	Tie beams:	-
Piles:	(2+5+3+4)=14Y	Piles:	(3+5+5+7)=20Y
	Σ 22 PH		Σ 30 PH

Table 3 deals with the change of the state of plastic hinges at the end of the VR77NS earthquake, for the change of peak acceleration from PGA 0.20 to PGA 0.25g. In the case of PGA 0.25g two new plastic hinges occur, in the road deck. These hinges are the start of the yield, so as much as the road deck is concerned, the emergency vehicles can pass, however, the final conclusion requires also the analysis of the status of damage in the piers, local drift and residual displacement. In figure 6 are presented the corresponding displacements during the TH analysis of VR77NS, for the model and the accelerograms and plastic hinges status from figure 5. The assessment of status in the soil after the earthquake, in p-y curves is provided in the paper (Folić B. and R, 2018).

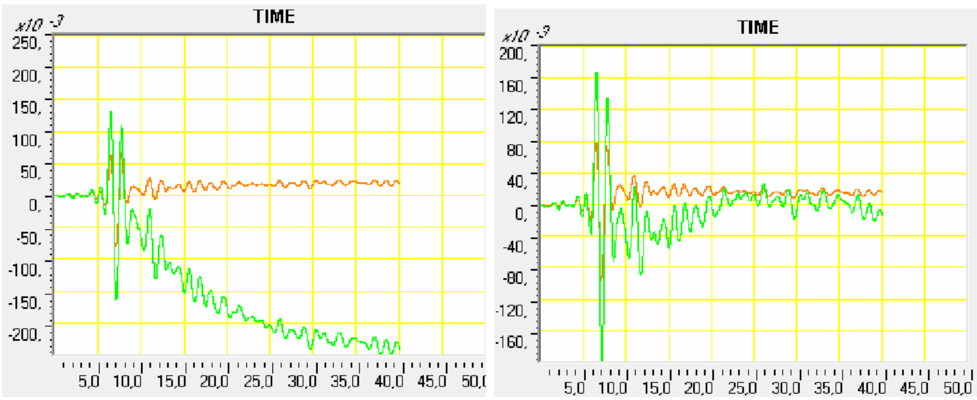

Figure 6 NDA Displacement of column joints, Left PGA 0.20 g. Umax=13.16 cm, Umin=24.99 cm diverg., Right PGA 0.25g. Umax=16.80 cm, Umin=19.93 cm
Slika 6. NDA Pomeranje čvorova stuba, Levo PGA 0,20 g. Umax=13,16 cm, Umin=24,99 cm divergira, Desno PGA 0,25g. Umax=16,80 cm, Umin=19,93 cm.

The damage cause is evident in the response spectrum of this accelerograms, figure 7, because the eigenperiod of the structure is around 0.9 sec.

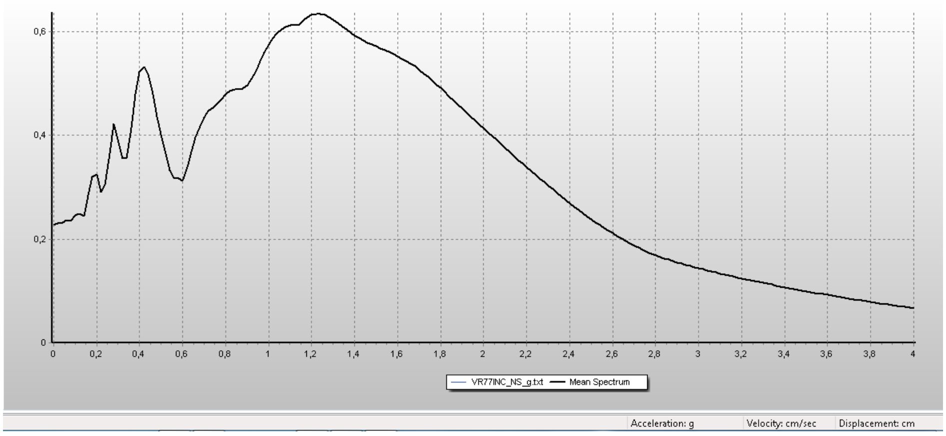

Figure 7 Response spectra elastic 5% damping Vrancea 77: VR77NS
Slika 7 Elastični spektar odgovora 5% prigušenje Vrančea 77: VR77NS.

What is used is the initial corrected value of the response spectra of 0.228 g. For the VR77NS earthquake, and the structure with the period of 0.90 sec the value of the spectrum increase is slightly above number 2 (more accurately 0.497/0.228=2.18), and with the period of 1.1 sec it increases to 2.7 (0.62/0.228=2.72). Practically, for this direction of earthquake action, smaller structural damage with the initial period of 0.90 to 1 sec, cause the structure to enter resonance and cause more severe damage.

In figure 6, for PGA 0.20g there is a divergent displacement of the pier top, (but it stops at the end of acc.) so the extreme displacement for PGA 0,20g, is 20% higher than the displacement for PGA 0,25g. This is an anomaly, which occurs rarely, but it is possible as a result if nonlinear TH analysis is used in dynamic interaction with the soil. For the purpose of the anomaly verification, the accelerograms of PGA 0.19g and 0.21g, can be run, and this would provide a better assessment of the seismic response.

The mean value of normal force per pier is around 2500 kN, so the additional moment from the residual drift is: 2500*0.20m=500 kNm (this moment can be compared to the second order moment according to EC 8, with behaviour factor assessment). The residual displacement is over 20/590=3.4% of the pier height. Although the road deck damage after the Vrancea 77NS earthquake is satisfactory, the damage status of bridge piers after this earthquake does not permit using the bridge, not even temporarily, without considerable additional supporting. In figure 5 and table 4, it can be seen that the status of PH at the bases and tops of the piers are such that they have no bearing capacity, i.e. that they are very close to the mechanism and do not have sufficient kinematic stability.

Table 4 Displacement, during earthquake VR77 ac.c
Tabela 4 Pomeranje tokom akcelerograma zemljotresa VR77

Displacement	PGA 0.20 *g*	PGA 0.25*g*	%
U_{min}	-24.99	-19.93	-20.25
U_{max}	13.16	16.80	27.66

U_{extr}:	24.99	19.93	-20.25

SEISMIC RESPONSE TO VR86FocNS SINGLE-LAYER SOIL

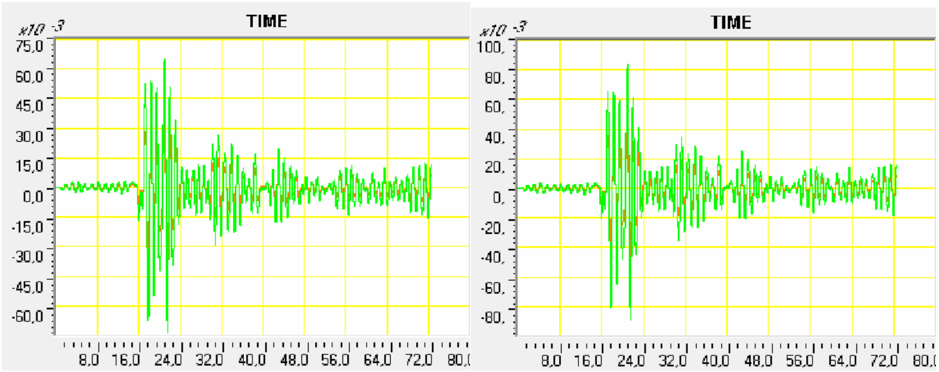

Figure 8 NDA Displacement of column joints, Left PGA 0.20 g. VR86FocNS. Umax=6.529 cm, Umax=8,331 cm, Right PGA 0.25g Umin=8.864 cm, Umin=7.343 cm.
Slika 8. NDA Pomeranje čvorova stuba, Levo PGA 0,20 g. VR86FocNS. Umax=6,529 cm, Umax=8,331 cm, Desno PGA 0,25g Umin=8,864 cm, Umin=7,343 cm

The mean value of the normal force per pier is around 2500 kN, so the additional moment of the presumed drift of 1 cm (realistic is around 2-3mm) is: 2500*0.01m=25 kNm (this moment can also be compared to the second order moment according to EC 8, with the assessment of the realized behaviour factor).

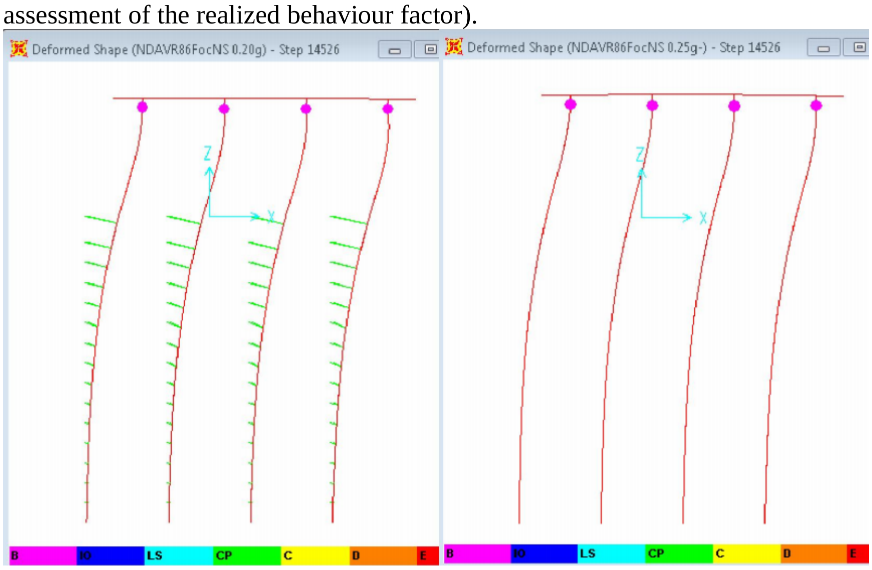

Figure 9 NDA State at the end of earthquake acc VR86FocNS PGA 0.20 g. Soil as single-layer p-y: φ=34; b=1.2 m; γ=17.6 kN/m3; k=16307 kN/m2, left PGA 0.20 g fracture of construction, right PGA 0.25g.
Slika. 9 NDA Stanje na kraju zapisa ubrzanja VR86FocNS. Tlo jednoslojno p-y: φ=34; b=1,2 m; γ=17,6 kN/m3; k=16307 kN/m2, gore levo PGA 0,20 g slom konstrukcije, gore desno PGA 0,25g.

The model parameters are the same as in the previous section, the only changed thing is the accelerograms (earthquake) used for the seismic analysis. The peak values of this new accelerograms are also the same: PGA 0.20g and 0.25g.

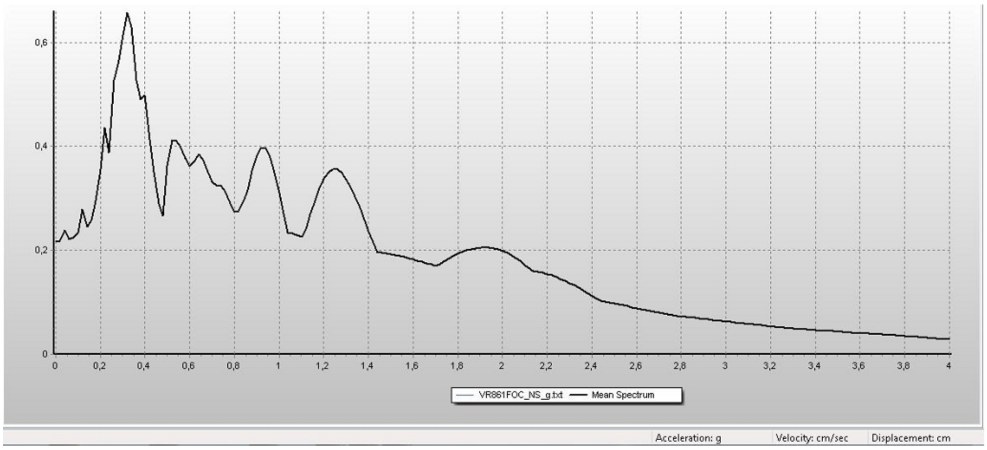

Figure 10 Response spectra elastic 5% damping VR86FocNS
Slika 10. Elastični spektar odgovora 5% prigušenje VR86FocNS.

The peak value is obtained for the period of 0.32 sec (0.657/0.217=3.03). The initial value 0.203g (corrected 0.217 g). This spectrum is considerably inconvenient because of the local peaks, one is at the period of 0.95 sec, and it represents an increase of almost 2 numbers in comparison to the initial value (in this case for around 60%). The next peak is at the period of around 1.25 sec.

Here, considerably smaller displacement and damage of structure are evident, due to the VR86FocNS accelerograms. The road deck is intact, and vehicle passage can be permitted.

It is necessary to obligatorily inspect the pier tops (status of cracks, concrete cover layer and reinforcement, if visible) and also the other parts of the structure, and if it is proven that the damage is in accordance with the anticipated status, the PH needs to cleaned, and tops of the piers should be grouted with fast-setting mixture. If a quality fast-setting grouting mixture is used, that is produced by a manufacturer with a known standard quality of the product, and stored in the prescribed storage conditions, and if there is an experienced team for such works, a bridge could be in a matter of days be repaired for temporary operation. This does not hold for the VR77NS earthquake.

CONCLUSION

Development of design software, computers and models for dynamic interaction of the structure-piles-soil system, increasingly demonstrates that introduction of this analysis is necessary. It has been demonstrated, on only two relatively simple examples, that unless an analysis of a structure as a structure-foundation-soil system is performed, there cannot be sufficiently precise predictions of the seismic response of the structure. Therefore, introduction of the structure-pile-soil system is necessary for any precise damage assessment, both of the structure and of the piles, and it is also necessary for the soil status assessment during and after earthquakes.

The presented methods, of p-y curves, provide a good seismic assessment, but they must be combined with the approximate calculation of eigenperiods of the soil layers and with the verification of mutual relation of stiffness of the layers (figure 3) also in the pape r Folić R. et al. 2018.

Acknowledgement:

The research described in this paper was financially supported by the Ministry of Education and Sciences Republic of Serbia within the Project TR 36043. This support is gratefully acknowledged.

REFERENCES:

Bowen H. J., Cubrinovski M., Jacka M. E.: Effective stress analysis of pile foundations in liquefiable soil. 2007 NZSEE Conference.

Bhattacharya S., Madabhusi S., Bolton M.: An alternative mechanism of pile failure in liquefiable deposits during earthquakes, Geotechnique54. No. 3, 2004, pp. 203-213.

Dowrick, D.: Erthquake resistant design, Wiley, 1978.

Finn W.D.L., Fujita N.: Piles in liquefiable soils: seismic analysis and design issues. Soil Dynamics and Earthquake Engineering 22 (2002) 731–742. www.elsevier.com/locate/soildyn

Folić B, Folić R. Comparative nonlinear analysis of a RC 2D frame soil-pile interaction. Building materials and structures 61 (2018). Beograd. Serbia.

Folić B.: Seizmička analiza betonskih objekata fundiranih na šipovima, Univerzitet u N. Sadu, Fakultet tehničkih nauka, Doktorska disertacija, 2017. Serbia.

Folić R., Folić B., Miličić I.: Strut and tie model for analysis of pile cap. 6th international conference Subotica. 2018. Serbia.

Gazetas, G. (1984): Seismic response of end-bearing single piles. Soil Dynamics and Earthquake Engineering. No.2. pp. 82-93.

JSCE Japan Society of Civil Engineers. Earthquake Resistant Design Codes in Japan. 2000.

Mayer B.J., Reese L.C. (1979): Analysis of single piles under lateral loading, Res. St. 3-5-78-244, Texas Sdof Highways PT

Madabhushi G., Knappett J., Haigh S.: Design of pile foundations in liquefiable soils. Imperial College Press. London, 2010.

Meymand P. J.: Shaking Table Scale Model Tests of Nonlinear Soil-Pile-Superstructure Interaction In Soft Clay. Dissertation, D. of Ph. in Civil Engineering. Univ. of California, Berkeley. 1998.

Mylonakis G., Nikolaou A., Gazetas G.: Soil-pile-bridge seismic interaction: kinematic and inertial effects. Part 1: soft soil. Earthquake Engineering And Structural Dynamics, VOL. 26, 337-359 (1997)

Миловић, Д., Ђого. М.: Проблеми интеракције тло-темељ-конструкција. Српска академија наука и уметности огранак у Новом Саду. Нови Сад 2009. Serbia.

Mosher R., Dawkins W.: Theoretical Manual for Pile Foundations, U.S. Army Corps of Engineers, Report ERDC/ITL TR-00-5, Washington, USA, 2000.

Novak, M.: Soil-pile interaction under dynamic loads. Institution of Civil Engineerings. Numerical methods in offshore piling, London, 1980. pp. 59-68.

Pender, M.J. (1993): Aseismic pile foundation design analysis, Bulletin of the New Zealand NS of EE, Vol. 26, No.1, March, 49-160.

Penzien J. (1970): Soil-pile foundation interaction. Earthquake engineering. Prentice-hall, inc., Englewood Cliffs. N.J. London.

Poulos H., Davis E.: Pile Foundation Analysis and Design, Rainbow-Bridge Book Co, 397p, 1980.

Prakash S.: Soil Dynamics. McGraw-Hill Book Company. New York 1981.

Rovithis E.N, Pitilakis K.D., Mylonakis G.E.: Seismic analysis of coupled soil-pile-structure systems leading to the definition of a pseudo-natural SSI frequency. Soil Dynamics and Earthquake Engineering 29 (2009) 1005–1015. www.elsevier.com/locate/soildyn

Suarez, V.: Implementation of Direct Displacement Based Design for Pile and Drilled Shaft Bents. NCSU. North Caroliona State Univesity. October. 2005.

Tazoh T.; Ohtsuki A.; Fuchimoto M.; Nanjo A.: Analysis of the damage to the pile foundation of a highway bridge caused by soil liquefaction and its lateral spread due to the 1995 Great Hanshin Earthquake. Paper 1978. 12WCEE. 2000. Auckland. New Zealand.

Todorovska M.; Trifunac M. (2006): Impulse response analysis of the Van Nuys 7-story hotel during 11 Earthquake (1971-1994): One -dimensional wave propagation and inferences on global and local reduction of stiffness due to earthquake damage .Report CE 06-01. July, 2006. University of Southern California.

Pecker, A.: Earthquake Foundation Design; in Advanced Earthquake Engineering Analysis, Ed. A. Pecker, Springer, Win New York, 2007, pp. 43-62

Pecker, A.: Soil Structure Interaction; in Advanced Earthquake Engineering Analysis, , Ed. A. Pecker, Springer, Win New York, 2007, pp. 33-42