DISSIPATION OF SEISMIC ENERGY IN SOIL DURING THE FIRST SHOCK AND AFTERSHOCKS ON RC FRAME FOUNDED ON PILES

B. Folić*, Angelos Liolios, **, Asterios Liolios**, Ž. Žugić*

** University of Belgrade, Innovation Center, Faculty of Mechanical Engineering, Kraljice Marije 16, Belgrade 11000, Serbia, boris.folic@gmail.com*
*** Institute of Information and Communication Technologies, Bulgarian Academy of Sciences, Sofia, Bulgaria*
**** Democritus University of Thrace, Dept. Civil Engineering, GR-67100 Xanthi, Greece, (e-mail: aliolios@civil.duth.gr,*

ABSTRACT

This paper presented the research of seismic energy dissipation in soil on 2D AB façade frame eight floors high founded on bored RC piles. Soil-pile dynamical interaction, i.e. non-linear soil behavior is modeled by horizontal p-y curves for sand, using multiplastic link elements. The soil around the pile is represented with horizontal p-y curves, on which the seismic energy dissipation is determined, during the first shock and aftershocks of an earthquake. For the El Centro accelerogram, using the time analysis procedure (TH), the energy dissipation in soil during the first shock which had PGA of 0,30g and during the aftershock, scaled at 0,21g.

KEY WORDS: Soil Pile Structure Interaction, p-y curve, dissipation of seismic energy, aftershock

DISIPACIJA SEIZMIČKE ENERGIJE U TLU TOKOM GLAVOG I NAKNADNOG UDARA NA AB RAMU FUNDIRANOM NA ŠIPOVIMA

REZIME

U ovom radu prikazano je ispitivanje disipacije seizimičke energije u tlu na 2D AB fasadnom osmoetažnom ramu fundiranom na bušenim AB šipovima. Dinamička interakcija tlo-šip, odnosno nelinearno ponašanje tla je modelovano horizontalnim p-y krivama za pesak, primenom multiplastic link elemenata. Tlo oko šipa je predstavljeno horizontalnim p-y krivama, na kojima je i određena disipacija seizmičke energije, tokom glavnog i naknadnog udara zemljotresa. Za akcelerogram El Centro postupkom vremenske analize (TH) razmatrana je disipacija energije u tlu, tokom glavnog udara zemljotresa PGA od 0,30g i naknadog udara, koji je skaliran na 0,21g.

KLJUČNE REČI: interakcija konstrukcija-šip-tlo, p-y krive, disipacija seizmičke energije, glavni i naknadni udar.

Nomenclature

Maft	Magnitude of aftershock
Mmax	Magnitude of major shock
U_{i+1}	upper joint displacement of floor i
U_i	lower joint displacement of floor i
h_i	floor i height
PGA	Peak Ground Acceleration
FS	First Shock, major shock of earthquake
AfSh	aftershock

INTRODUCTION

This paper presents numerical research of the dissipation of the seismic energy in soil, on a 2D AB frame founded on piles. The soil around the pile is represented by horizontal p-y curves, on which dissipation of seismic energy is tested, during the first shock and aftershocks (Aničić at al.1990). Energy in p-y curves is calculated according to the basic mechanical principle of the force work, and force action along the direction (Folić B. and Folić R., 2018; Folić B., 2017). Therefore, dissipation in the soil along the pile is analyzed based on the force in non-linear “spring” and its displacement, in each step of the TH analysis. Thus, dissipation can be observed by depth of the soil, at each meter of depth, in the way the p-y curves are represented.

„It was long thought that foundation on piles is safe, and that it also protects the superstructure in the event of intensive earthquakes.. In the devastating earthquakes that occurred in the last two decades (of the 20th century) this has been disproven. It is important to realistically evaluate deformations of foundations on piles in the design phase, too, and not only in the study of structure-soil interaction.. The first research of this interaction is usually related to the analysis of standard geotechnical conditions, and lately to liquefiable soil (since the earthquake of Niigata in 1964). For the piles in such soil, intensive damage and sudden failures and/or collapses of structures were recorded. The theory of limit balances is used for the evaluation of the load due to lateral spreading of liquefiable soil“ (Folić, 2017).

Based on the condition of plastic joints of structures and piles and, and conditions of p-y curves the failure probability of the structure, piles or soil can be evaluated. Practically, in this way the structure-foundation-soil system is observed. Maymond, (Maymond, 1998) mentions the assessment systems of pile damage which became visible only several years after earthquakes, based on the geodetic surveying, which was verified by excavation. Tazoh (2000) mentions three standard methods for pile damage investigation, except excavation, which are ultrasound pile inspection (Ćosić at al 2014) and recording using camera through an inspection bore.

There is a relatively small number of numerical soil condition tests, in a way presented in the paper, because the condition of p-y curves is usually ignored, even though the model itself was often implemented, because important experimental studies were conducted several decades ago, as early as in 70's and 80's of the 20th century (Mayer and Rees 1979; Mosher at al., 2000), and the model of p-y curves itself was derived from the correlation of stress-strain diagram with triaxial soil test, as early as by the end of 50's, beginning of 60's of the 20th century, as well as the soil wedge failure model. On the other hand, during the last decade of 20th century and two decades of 21st century, there emerged a number of very important numerical-experimental methods of testing of seismic performance of structures on piles. The research of SSI interaction were performed on complex integral models where the soil was modeled using a finite element network (Kwon and Elnashai, 2006), on the simplified Tajimi method of one-dimensional waves, or on even simpler method of equivalent pile constraint (Pender, 1995). Experiments testing time of passage of waves through different directions of the structure, foundations and the soil were performed, and in correlation, the structural response in characteristic points was studied; (Todoroska and Trifunac, 2006). In the integral model, when the structure-foundation-soil (SFS) system is observed, there occurs the problem of the soil network size (FE Method) in relation to the spectrum of frequencies relevant to the given research, as well as the problem of the total size of the soil model in relation to the structure and foundation, for the purpose of correct inclusion of the SFS interaction and the problems of boundary conditions on the soil contours, due to the effects of refraction, reflection and resonance of waves in the soil.

Majority of studies was originally focused on free-standing piles, then on bridges, and then on buildings. Establishing correlation between the model and the structure often boils down to the reverse calculation method, standard parameters of a common models or boundary equilibrium method, presented by Broms as early as in 1964. Also, the impact in the soil during earthquakes can be observed through the variation diagram of a normalized shear modulus as opposed to the maximum dilatation achieved (Madabhushi at all. 2010; Pitilakis, 1999). Wolf often used the frequency analysis and (Novak, 1974), or integral-decomposition -synthetic method of kinematic (with out mass) and inertial (with mass) interaction of structure-soil-pile (Gazetaš, 1984; Gazetaš, 1997; Mylonakis at al. 1997).

In this work, symmetric (regular) skeletal (frame) constructions founded on piles using more complex models were used. In (Hatzigeorgiou and Liolios, 2010) the effect of an aftershock on 2D RC frames with regular and irregular height is observed, columns reinforced, without piles. Therefore, this paper can be considered one of the possible directions of future research, with piles.

2D FRAME CONSTRUCTION MODEL FOUNDED ON A GROUP PILES WITH P-Y CURVES

The spatial (3D) frame is dimensioned with reference to earthquake action, using SAP 2000 software, including the effects in the perpendicular direction and torsion (with 5% eccentricity), for a behaviour factor of 5.85. The span between frames is 8 m, which is also the distance between the pile axes, in both directions, since the structure in question is symmetrical along two orthogonal axes. The height of the first two stories is 5 m, while for the remaining 6 storeys, it is 3.1 m. (Folić B, and Folić R. 2018; Folić B, 2017).

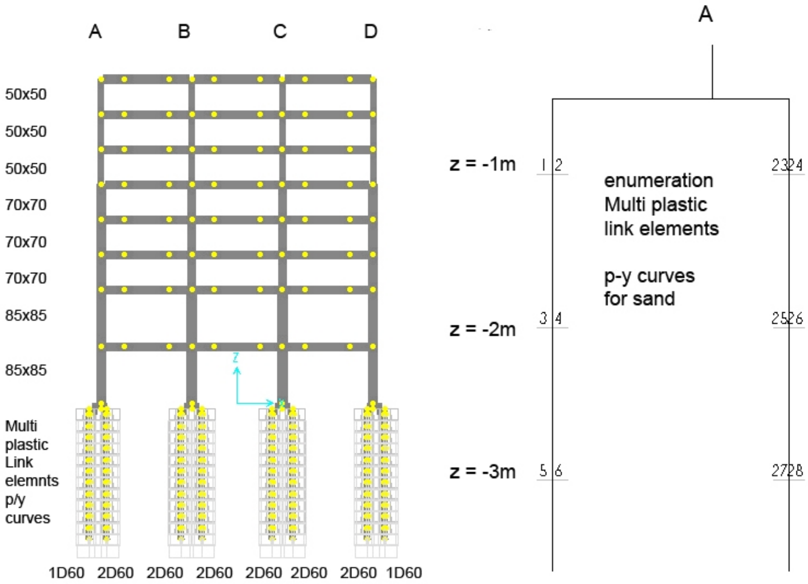


Figure. 1. (b) 2D Frame with dimension of columns and piles with p-y curves as link elements. (c) Enumeration p-y curves of the left group of piles

Slika 1 (b) 2D ram sa dimenzijama stubova i šipova is a p-y krivama kao link elementima; (c) Numeracija p-y krivih leve grupe šipova

The soil was modeled using multiplastic link elements fig. 1, with p-y curves used for sand. For modeling the Takeda type hysteresis envelope, a hysteresis p-y curve was used for submerged dense sand, piles with a 60 cm diameter, and angle of inner friction of 35°.

RESULTS AND ANALYSES. THE STATE OF PLASTIC HINGES

In the images of the state of plastic hinges during an aftershock of 0.20g (Fig. 2b) and 0.21g (Fig. 2a), the emergence of plastic hinges in the top three meters of the pile's depth is noticed. This only refers to the far left independent pile, which is the subject of analysis of the behaviour of soil during the earthquake, while other piles remain undamaged.

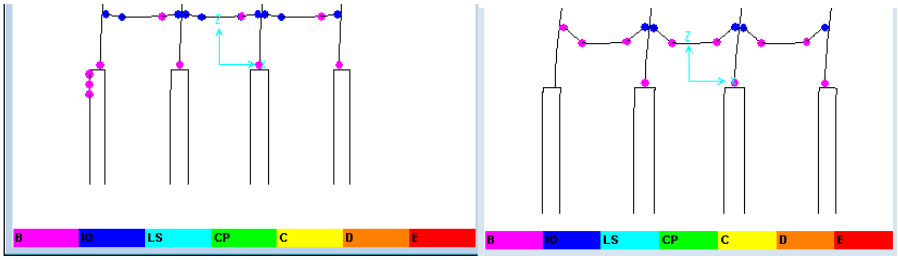
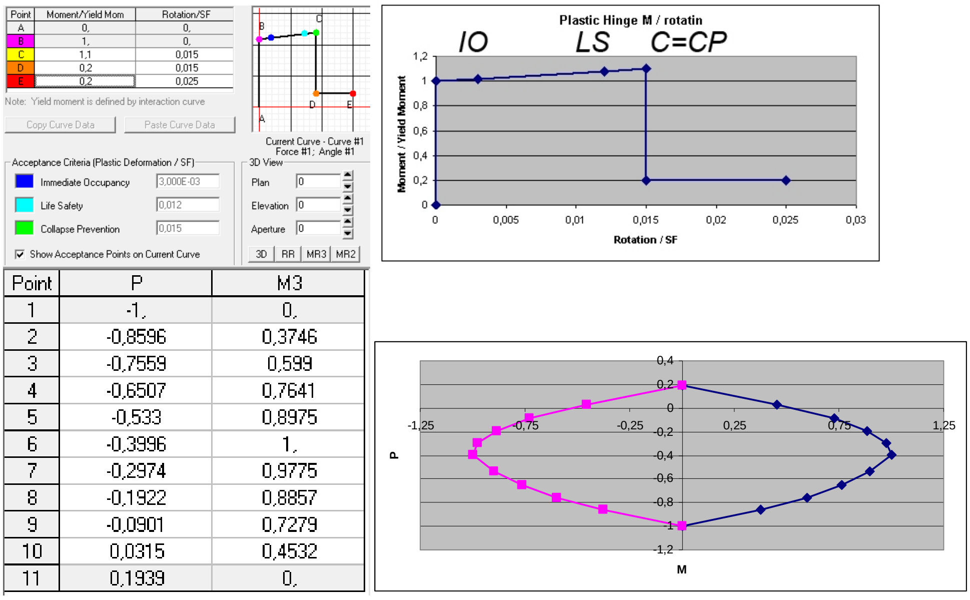


Figure. 2. State of plastic hinges (PHS) at the end ElCentro earthquake FS 0.30g, (a) PGA AfSh 0.21g PHS: 47Y, 32IO, 15LS, 7D, 3C. (b) PGA FS 0.30g PHS: 86Y, 10IO, 3LS.
Slika. 2. Stanje plastičnih zglobova (PHS) na kraju zemljotresa ElCentro FS 0,30g, (a) PGA AfSh 0.21g PHS: 47Y, 32IO, 15LS, 7D, 3C. (b) PGA FS 0.30g PHS: 86Y, 10IO, 3LS.

As the lowest relevant PGA of the aftershock, the value AfSh of 0.15g (50% of PGA FS) is chosen in this research, and the next usual value of 0.20g (67% of FS), and then on the basis of the state of plastic hinges and the assumed relationship between the PGA of the first strike and the aftershock of about 70%, the maximal value of aftershock of 0.21g was chosen.



Point	Moment/Yield Mom	Rotation/SF
A	0,	0,
B	1,	0,
C	1,1	0,015
D	0,2	0,015
E	0,2	0,025

Note: Yield moment is defined by interaction curve

Current Curve - Curve #1
Force #1; Angle #1

Acceptance Criteria (Plastic Deformation / SF)
Immediate Occupancy: 3,000E-03
Life Safety: 0,012
Collapse Prevention: 0,015
☑ Show Acceptance Points on Current Curve

3D View
Plan: 0
Elevation: 0
Aperture: 0

Point	P	M3
1	-1,	0,
2	-0,8596	0,3746
3	-0,7559	0,599
4	-0,6507	0,7641
5	-0,533	0,8975
6	-0,3996	1,
7	-0,2974	0,9775
8	-0,1922	0,8857
9	-0,0901	0,7279
10	0,0315	0,4532
11	0,1939	0,

Figure 3 Plastic Hinge Properties in ground level left column 1H1; P_{max}=-24662.406; M=3288.87 P_{min}=4782.04

Slika 3 Svojstva Plastičnog zgloba u levom stubu prizemlje 1H1; P_{max}=-24662,406; M=3288,87 P_{min}=4782,04

DISSIPATION OF SEISMIC ENERGY IN SOIL

Using the multi linear plastic link elements applied as soil on piles with models of p-y curves, the behaviour of the soil by depth can be estimated in a TH analysis, i.e. the dissipation of seismic energy (Folić B. and Folić R., 2018). During an aftershock, the link elements of the end left pile are analysed here, as groups of three piles, shown in image 1, axis A .

In this model p-y curves for the real diameter of the pile are used for 1 m of depth, as well as for other depths, but if the soil around the pile cap is well compressed, then this "spring" should be adopted for a pile of approximately 2 or 2.5 times wider diameter (for these piles).

The images of reactions of link elements 1 and 2 at the depth of 1 m show whether link 1 is broken or the pile is permanently slanted, so that the Link 1 cannot be activated. According to the diagram of the displacement of link elements 1 and 2, it is estimated that there are permanent displacements in the second part, i.e. appearance of gap link elements of soil.

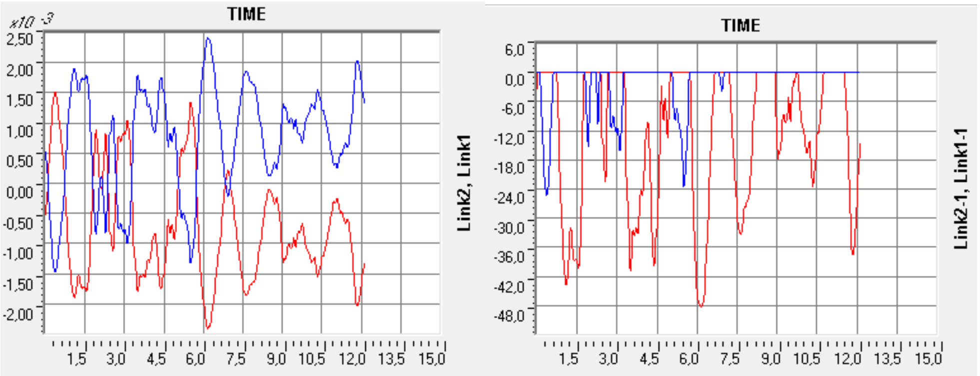


Figure. 4. Links 1 and 2, level -1.0 m. PGA ElCentro 0.30g After Shock 0.21g. NDA
(a) Displacement max 0.242 cm.(b) Force max 48.40 kN.
Slika 4. Link 1 i 2, nivo -1.0 m. PGA ElCentro 0,30g Drugi udar 0,21g. NDA
(a) Pomeranje max 0,242 cm.(b) Sila max 48,40 kN.

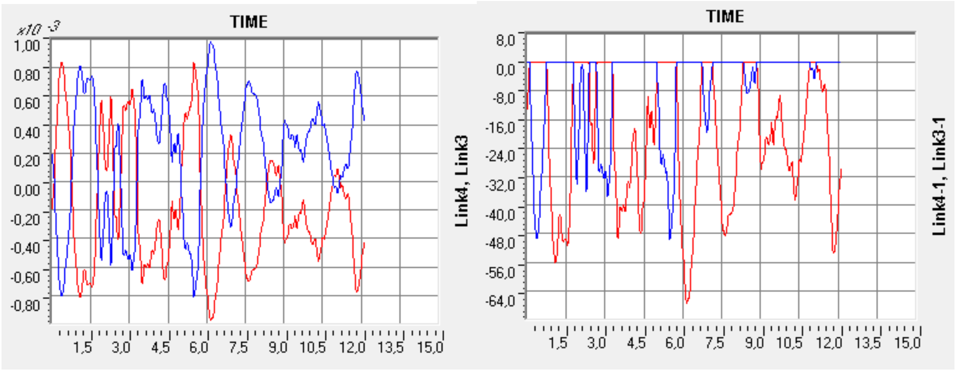


Figure. 5 Links 3 and 4, level -2.0 m. FS 0,30g Aftershock 0.21g ElCentro. NDA
(a) Displacement max 0.980 cm. (b) Force max 67.38 kN.
Slika 5. Link 3 i 4, nivo -2.0 m. FS 0,30g Drugi udar 0,21g. ElCentro NDA
(a) Pomeranje max 0,980 cm.(b) Sila max 67,38 kN.

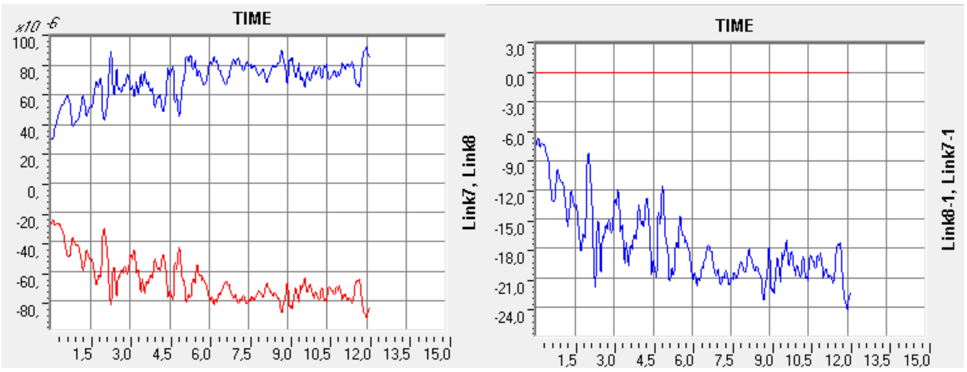


Figure. 6 Link 7 and 8, level -4.0 m. FS 0.30g After Shock 0.21g ElCentro. NDA
(a) Displacement max 0.009 cm. (b) Force max 24.24 kN.
Slika. 6 Link 7 i 8, nivo -4,0 m. FS 0,30g After Shock 0,21g ElCentro. NDA
(a) Pomeranje max 0,009 cm. (b) Sila max 24,24 kN.

"Separation" of the graph of the drift of link elements 7 and 8 is visible, which means that during the action of ElCentro AfSh 0,21g, at the depth of 4 m, only one link element is compressed, as seen in the reaction force diagram. At the depth of 4 m, and deeper, there are no plastic hinges in piles, while at the depth of 1, 2 and 3 meters, there are plastic hinges, but only in this pile, as seen on the figure of the state of plastic hinges at the end of AfSh 0,21g earthquake.

During the first strike, there is a sharp limit of dissipation at 4 m od depth; during the aftershock deeper layers are activated as deep as up down to 6m. The overall dissipated energy down the depth of the pile for AfSh is ~1,0883375 [kNm], while during the first shock it is ~1.513381300 [kNm]. The relationship between energies of the aftershock the first shock is 1,0883/1,5134=0,719, bigger than PGA, which is 0.700 (0,21/0,30=0,70).

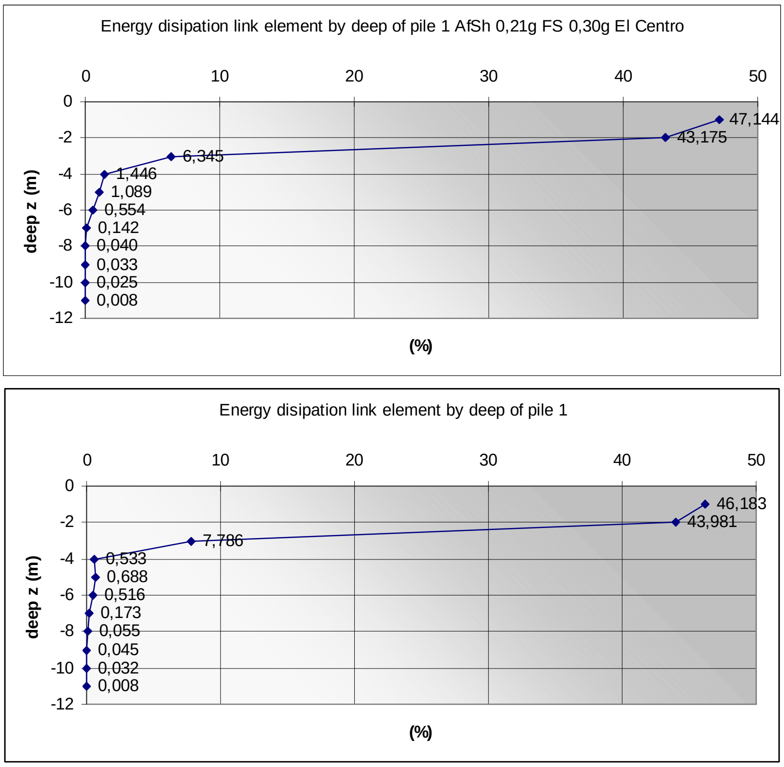


Figure. 7. Percentage of seismic energy dissipation, on links element with depth for the pile 1, (a) Due After Shock 0.21g ElCentro FS 0.30g. NDA. (b) Due First shock ElCentro 0.30g.
Slika 7. Procenat disipacije seismičke energije, link elemenata po dubini šipa 1, (a) Tokom Drugog udara 0,21g ElCentro FS 0,30g. NDA. (b) Tokom Glavnog udara ElCentro 0,30g.

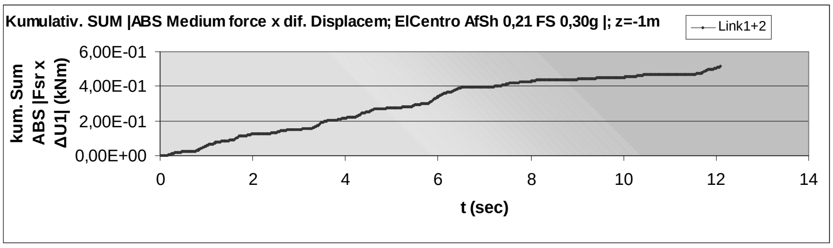


Figure. 8. Cumulative Absolute work of link elements under El Centro impact. Link 1 and 2, level -1.0 m of depth below ground surface. PGA AfSh 0.21g ELCentro NDA.
Slika. 8. Kumulativni Absolutni rad link elemenata pod El Centro udarom. Link 1 i 2, nivo -1,0 m od površine tla. PGA AfSh 0,21g ELCentro NDA.

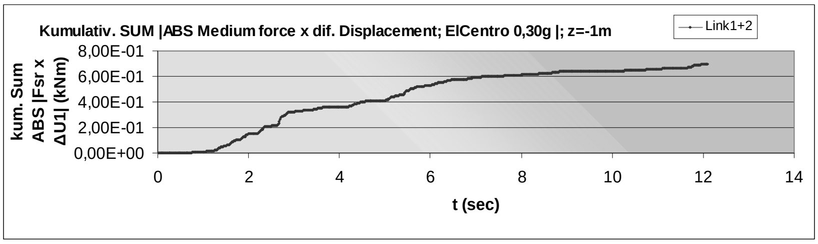


Figure.9. Cumulative Absolute work of link elements under El Centro impact. Link 1 and 2, level 1.0 m of depth below ground surface. PGA FS 0.30g ELCentro NDA.
Slika 9. Kumulativani Absolutni rad link elemenata tokom El Centro udara. Link 1 i 2, nivo 1,0 m od površine tla. PGA FS 0,30g ELCentro NDA.

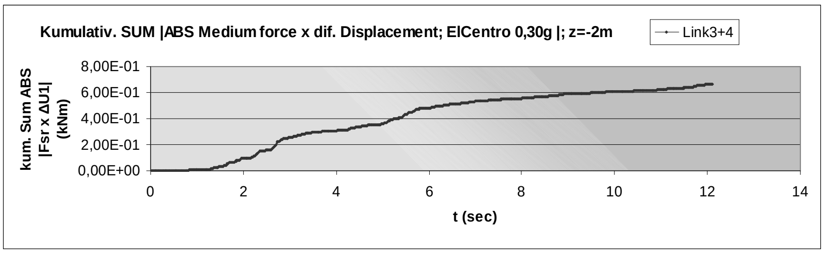


Figure 10. Cumulative Absolut work link elements under action El Centro. Link 3 and 4, lavel 2.0 m depth below ground surface. FS 0.30g ELCentro NDA
Slika 10. Kumulativni absolutni rad link elementa pod dejstvom EL Centro, Link 3 i 4, nivo 2 m ispod površine tla. FS 0,30g El Centro NDA

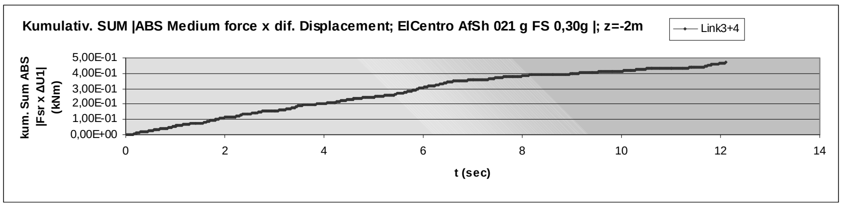


Figure 11. Cumulative Absolute work of link elements under El Centro impact. Link 3 and 4, level 2.0 m of depth below ground surface. PGA AfSh 0,21g FS 0.30g ELCentro NDA.
Slika 11. Kumulativni absolutni rad link elementa pod dejstvom EL Centro udara, Link 3 i 4, nivo 2 m ispod površine tla. PGA AfSh 0,21g FS 0.30g El Centro NDA

The aftershock has a considerably smoother curve of the cumulative seismic work on the link element 3 and 4 than for the same curve after the first strike.

The Change of Characteristic Tone

The change of the characteristic tone of the building-pile-soil system after the First Shock and aftershock of the ElCentro earthquake is given in the table 1.

In table 1, the increase of the characteristic eigen tone for an aftershock of 0.15 (50% FS PGA) is 30% in relationship to the state of the building-pile-soil system after the main strike. After a further increase of PGA aftershock to 0.20g, this change is 100%, so it dramatically increases.

Table 1. Modal periods (eigenvalues) after different PGA of First Shock and After Shocks of El Centro. 2D Frame.
Tabela 1. Modalni periodi (svojstvene vrednosti) nakon različitih PGA za Glavni i Naknadni udar El Centro. 2D okvir.

PGA (g)	T1 (sec)	ΔT1 %
FS 0.30	2,39338	0
AfSh 0.15	3,13098	30,82
AfSh 0.20	4,80789	100,88
AfSh 0.21	4,89092	104,35

CONCLUSIONS AND DIRECTIONS FOR FURTHER RESEARCH

When choosing the model of the soil for frame constructions, the elastic secant stiffness is used (Pando 2013) as a replacement for p-y curves. The change of the secant stiffness can considerably change the shape of the break of the building-foundation-soil system. In massive frame buildings constructed on piles by lowering the secant stiffness, a migration of the plastic joints deeper into soil occurs (Suarez, 2005; Folić B., 2017). P-y curves are applied through multi-plastic link elements, enabling separate modeling of the linear part of damping and stiffness, while the non-linear part is of a hysteresis type. The effect of an aftershock is tested here by assuming the matrix of stiffness, and thereby the state of plastic hinges, at the end the state of TH analysis of the main strike (specifically the ElCentro accelerogram with PGA (0.30g). Regardless of the great number of parameter tests of the static and hysteresis response of the pile (Mayer et all, 1979; Reese, L., Van Impe, W., 2001) it is not always clear whether the cyclic resistance adequately replaces the dynamical impact on the soil . For p-y curves see (Maymond, 1998; Stewart et all 1999).

There is a number of model parameters affecting the seismic performance of the structure, which have an impact on the seismic response, and thus, on its degree of damage. One of the basic problems occurring in the study of 2D frames is the way of separating a 2D frame from the spatial 3D structural model, as well as determination of the corresponding stiffness and load. For a dynamical model, one of the key parameters is the value of the first eigen tone T_1, and the corresponding horizontal load (BS Base shear) at the base.

When a 3D model is used for dimensioning of RC structures, for determination of the first eigen tone T_1 and BS, the introduction of linear stiffness of the soil on piles can significantly affect them. The linear part of stiffness of p-y curves can be determined based on the research by Matlock and Rees (Mosher at all 2000), and for sand, those are three sic stiffness parameters (dense, medium and loose sand) provided in the tables, and each of these stifnesses can occur only as two cases: submerged and dry sand, a total of 6. API

(American Petroleum Institute) norms provide a larger number of continual stiffness parameter. In the future research, it is necessary to determine variations of T_1 and BS for all 6 cases of soil, of one realistic type of structure (which often occurs in different locations and conditions of soil, or only for the theoretical research which can be used on a later occasion). The percentage of increase of T_1, may vary, depending on the type of piles, their length, ways of group formation etc. and this % has a different impact on the response of the structure. Gazetaš proposed, that when an interaction with the soil must be introduced, the change of T_1 is 30%. However, the research of the dissipation of seismic energy in the soil, can, from case to case, change this limit more precisely and provide a good assessment of the reserve in soil and piles. For the effect of damping, see (Park and Hashash, 2004)

Folić B. (2017), presented the impact of several different models of soil: with or without piles, with linear and non-linear soil, to the seismic response of the structure. It was proposed, for the important structures, to observe these parameters as the envelope of the impacts for their dimensioning. Variation of certain soil parameters, depends both the adopted model of the structure-foundation-soil system and on the degree of reliability (parameters) of the geomechanical report and types of piles Pando proposed secant stiffness, (Pando 2013) and they, in the engineering approach of these regular structures, and usual accelerograms, should be used only at first 4 meters of depth. The research ought to be extended to a large number of various spans and heights of buildings.

In this research it has been planned to investigate what happens to a structure when PGA is increased or reduced for 0,01 PGA. The value of 0,20g has been chosen as a rule, and maximum has been 0,21g, which agrees with step of 0,01g, so automatically, the PGA steps have been denser, in order to check the sensitivity of the structure to damage. The condition of the structure at 0,21 g, indicates severe damage of upper floors, so an increase to 0,22 g, would surely lead to a collapse, irrespective of the remaining reserve in the soil and piles. The impact of a denser PGA is provided in Folić B. 2017.

ACKNOWLEDGEMENT

This paper was done with the financial assistance of the Ministry of Science, Education and Technological Development of the Republic of Serbia, within the project for technological development TR 36043.

REFERENCES

Aničić, D., Fajfar, P., Petrović, B., Szavits-Nossan, A., Tomažević, M.; Zemljotresno inženjerstvo –Visokogradnja. Građevinska Knjiga. Beograd. 1990.

Ćosić M., Folić B., Folić R.: Numerical Simulation of the Pile Integrity Test on Defected Piles, Acta Geotechnica Slovenica, Vol. 11, Iss. 2, pp. 5-19. 2014.

Folić B.: Seizmička analiza betonskih konstrukcija fundiranih na šipovima. Doktorska disertacija. FTN. Univerzitet u N. Sadu. Novi Sad. 2017.

Folić B., Folić R.,; Comparative nonlinear analysis of a RC 2D frame soil-pile interaction.. Journal for research in the field of material and structures. Beograd. Pp. 63-89. 2018.

Gazetas, G.: Seismic response of end-bearing single piles. Soil Dynamics and Earthq. Eng. No2. pp. 82-93. 1984.

Gazetas G.: Dynamic soil-structure interaction during earthquakes, Proc. Of the SIRENA Advanced Study Course on Seismic Risk, pp. 301-354. 1997.

Hatzigeorgiou, G.D., Liolios, A.A.; Nonlinear behaviour of RC frames under repeated strong ground motions. Soil dynamics and Earthquake Engineering 30. Pp.1010-1025. 2010.

Kwon O., Elnashai A.: Analytical Seismic Assessment of Highway Bridges with Soil-Structure Interaction. 4th International Conference on Earthquake Engineering, Taipei, Taiwan, October 12-13, Paper No. 142. 2006.

Madabhushi, G., Knappett, J., Haigh, S.; Design of pile foundations in liquefiable soils. Imperial college press. Pp. 217. 2010.

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

Maymond P. J.,; Shaking table scale model test of nonlinear soli-pile –superstructure interaction in soft clay, University of California, Berkley. Ph.D. 1998.

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

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.

Novak, M.: Dynamics stiffness and damping of piles. Canadian Geot. Journal, No 11. pp 574-598. 1974.

Pando, M.,: Analyses of Lateral Loaded Piles with p-y Curves - Observations on the Effect of Pile Flexural Stiffness and Cyclic Loading. NCDOT 7th Geo3T2, Raleigh, NC, Thursday, April 04, 2013. Paper: 3B-1_A49.

Reese, L., Van Impe, W.: Single pile and pile groups under lateral loading, Balkema, Rotterdam, 2001.

Park D., Hashash. Y. M. A.: Soil damping formula tion in nonlinear time domain site response analysis. Journal of Earthquake Engineering, Vol. 8, No. 2 (2004) 249-274. Imperial College Press

Pitilakis, K.D., Anastasiadis, A.J. 1999: Soil and site caharacterization for seismic response analysis. Proceengs of the 11th European conference on earthquake engineering. Balkema Rotterdam,. Pp.65-90.

Pender M.: Earthquake resistant design of foundations. Pacific Conference on EE, Melburn, Novembar 1995.pp. 155-171.

Stewart, J.P., Fenves, G.L., Seed, R.B.: Seismic soil-structure interaction in buildings.1: Analitical methods, Journal Environm. Engineering, ASCE, V. 125:1, pp. 26-37. 1999.

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

Tazoh T.; Ohtsuki A.; Fuchimoto M.; Nanjo A.; , Yasuda F.; Fujii Y.; Nakahira A.; Kuroda C.: 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. (1978). 12WCEE. Auckland. New Zealand. 2000.

Todorovska M.; Trifunac M.: 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