EFEKAT NAKNADNIH UDARA NA 2D AB RAMU FUNDIRANOM NA ŠIPOVIMA PREKO P-Y KRIVIH

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REZIME

U ovom radu prikazan je način modelovanja drugog udara (zemljotresa), pojave čiji je značaj zapažen tokom druge polovine 20 veka. Navedeni su konstruktivni sistemi kod kojih je bitno razmatranje drugog udara. Detaljnije je prikazan model skeletne AB konstrukcije, odnosno 2D AB fasadnog okvira fundiranog na bušenim AB šipovima, prečnika 60cm. Dinamička interakcija tlo-šip, odnosno nelinearno ponašanje tla je modelovano horizontalnim p-y krivama za pesak, primenom multiplastic link elemenata. Za akcelerogram El Centro tokom vremenske analize (TH) razmatrani su: globalno pomeranje, (lokalni) drift i stanje plastičnih zglobova.

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

THE EFFECT OF AFTERSHOCK ON 2D RC FRAME FOUNDED ON PILES WITH P-Y CURVES

ABSTRACT

This paper presents the manner of modeling the aftershock (in an earthquake), the phenomenon whose significance has been noted in the second half of the 20th century. It lists the constructional systems in which the consideration of the aftershock is important. It shows in more detail the model of the skeletal RC construction, i.e. the 2D RC facade frame constructed on the 60cm-diameter drilled RC piles. The dynamic interaction soil-pile, i.e. the non-linear behaviour of the soil is modelled with the horizontal p-y curves for sand, applying multiplastic link elements. For the accelerogram El Centro during time history (TH) the global and (local) drift, the condition of plastic hinges were considered.

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

Nomenclature

Maft	Magnitud of after shock
Mmax	Magnitud of mayor shock
U_{i+1}	displacement upper joint of flour i
U_i	displacement down joint of flour i
h_i	high of floor i
PGA	Peak Ground Acceration
FS	First Shock, mayor shock of earthquake
AfSh	aftershock

INTRODUCTION

During the 20th century it has been noted that in some specific cases of earthquake effect, the aftershock, which is weaker than the first, has caused the destruction of buildings (Aničić at al.). People have often returned to their seemingly preserved buildings with relatively little damage after the first (main) strike, which led to unexpected loss of human lives. Even though there are no precise data about the type of these constructions, one could assume that those were one of the three types of systems in which the reserve of seismic load capacity was spent, and the state could have been additionally worsened due to the age or inadequate maintenance of the building. After the experience of the Skopje earthquake in 1963. this has been occuring in buildings with masonry construction, and mixed systems (skeletal construction with masonry infill), and also in frames buildings. For this reason, it is necessary to test certain systems and find adequate models in order to examine when certain constructions can be destroyed after the aftershock. This paper suggests the model of examining possibilities when destruction can occur with the aftershock, and under what circustances, for the assumed types of constructions.

In masonry buildings, the dissipation of seismic energy during lower levels of earthquake intensity can occur between the joints of brick elements, usually in mortar joints, micro cracks of brick blocks or in the joints of mortar and brick. In the case of stronger and longer earthquakes, oblique and diagonal cracks could form, especially near the wall openings. The direction of cracks and fissures follows the trajectory of tension, and it depends on the relationship between modules and resistance of the brick and the mortar. In these constructional systems, a significant role in the seismic response can belong to non-constructional elements (bricks narrower than 19cm). The degree of damage of non-constructional, but also constructional elements can change the eccentricity of the centre of stiffness and the centre of mass. This influences the change of the regularity of the construction. If the change is small, then it is covered by the projected eccentricity of 5 percent in the foundation (recommendation EN 1998), and in the case of higher eccentricity a special estimation procedure is needed in order to include this phenomenon.

In frames constructions the following factors also influece the seismic resistance of the building: age, maintenance, local soil conditions, the range of earthquake responses etc. During and after the effect of earthquake, a change in the condition of the soil can occur, and also a change of the static system of the building. Different layouts of plastic hinges change the characteristic eigen tone of the construction due to damage, and thus coordinates of the range of responses (Ćosić et al, 2006). The change of the characteristic tone in the range of responses changes the degree of amplification, and the change of the entry accelerogram changes the shape of the range of responses, as shown on (image 1a) according to (Datta, 2010). The change of the module of slippage and absorption of the soil due to dynamic effect are shown depending on the level of maximal dilatations during the earthquake (image 1b). These diagrams are arranged according to types of soil and the corresponding effective vertical stress at the given depth of the soil (Madhabushi et all, 2010).

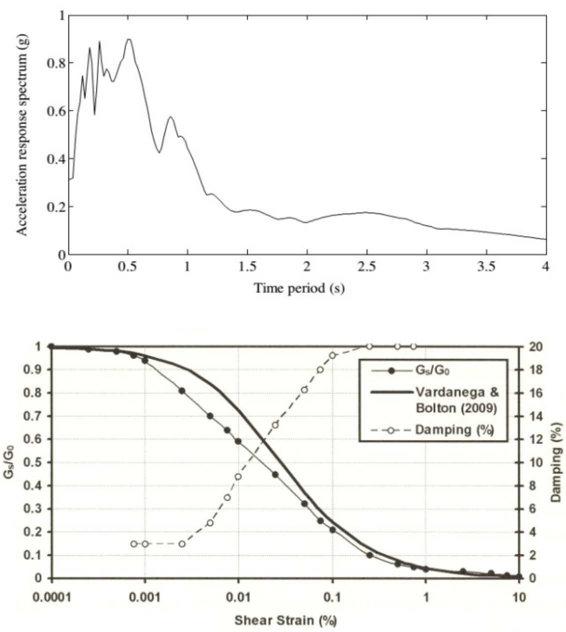


Figure 1. (a) Pseudo acceleration spectrum of El Centro earthquake (Datta, 2010); (b) Variation of normalised shear modulus and damping with shear strain, after (Madabhushi et all 2010).

Slika 1. (a) Spektar pseudo ubrzanja zemljotres ElCentro (Datta, 2010); (b) Promena normalizovanog mosula smicanja i prigušenja sa smičućom dilatacijom, prema (Madabhushi i dr. 2010).

In mixed constructional systems, the skeletal system with masonry walls, the degree of damage of the bricks and the layout of plastic hinges can also change the characteristic tone and shape of the building, but also the eccentricity in the foundation of the bulding (torsion in the base).

All these contstructions can have shallow foundation, or deep with piles. The pile construction usually has a greater seismic reserve than shallow foundations, which depends on local soil characteristics, and the range of responses of the used accelerograms.

For precise research the effects of more different earthquakes to the structure need to be examined. The selection of earthquake must satisfy the criteria of the EC 8 (mean value and deviation). However the methodology for presenting the effects of dynamics interaction would excessively increase the scope of the paper, therefore it is one of the topics for further research.

In this work, symetric (regular) skeletal (frame) constructions founded on piles using more complex models were used. In (Hatzigeorgiou and Liolios, 2010) the effect of aftershock on 2D RC frames with regular and irregular height is noted, with two manners of reinforcing columns.

CONSTRUCTION AND MODELS

AFTER-SHOCK AND FOR-SHOCK

"The process of accumulating tension in the zone of the future epicenter of earthquake is often followed by the creation of smaller earthquackes in the epicenter. So, in the zone of the preparation for an earthquake, a series of forschocks almost always occur, with considerably lower intensity compared to the main shock. Also, right after the occurence of the main shock, during the phase of consoldation and return to the stable state, an entire process of aftershocks start occurring" (Glavatović 2005).

The empirical dependence of the power of magnitude of the main aftershock (Maft) and the magnitude of the main shock (Mmax) in the entire series of earthquakes is called Båth's law, which can be expressed by the following linear ratio:

$$Maft = 1.1 + 0.71 \cdot M \max \quad (1)$$

The application of the formula (1) on Montenegro, where Mmax=7; one arrives at Maft=1.1+4.97=6.07.For ElCentro, also according to the formula (1) where Mmax=6.6; one arrives at Maft=1.1+4.62=5.72.

THE CHOICE OF ACCELEROGRAM

In some specific cases, when after the main shock the seismic reserve of the construction, i.e. the possibility of the dissipation of energy is spent, the destruction of such constructions occurs during a powerful aftershock. This phenonomenon, which is fortunately rare, is sometimes marked by a significant number of human casualties. The calculation attempts to include a realistic effect of the aftershock on the construction and the soil. There is a series of formulas for including the relationship between the mangitude and the intensity and PGA, but this procedure will be greatly simplified here, and only the PGA ratio will be used instead of magnitude. After the main shock ElCentro 0.30g, "aftershocks" of 0.15g, 0.20g and 0.21g are naw calculated. After the main shock, new characteristic forms of the now damaged constructions of 0,20; 0.25 and 0.30g are calculated (Folić B. and Folić R. 2018), with which a new matrix of damping for the aftershock. For the main shock of 0.30 g the dissipation of seismic energy according to the depth of the soil for one pile is calculated, and since for that pile and the same conditions of the soil, damage (plastic hinges) of the aftershock is analysed, this work is a logical continuation of the previous one.

Even though during the aftershock a migration of the earthquakes's epicentre occurs, and thereby a change of physical/mechanical characteristics of the epicentre, as well as the trajectory of seismic waves, as the first iteration in the analysis the same accelerogram, only with changed PGA, is taken for the analysis. So, although there is no doubt that the existing records of aftershock should be used, with or without the use of certain signal filters, the authors thought that, since a non-linear analysis of the dynamic interaction (SPSI), the effect of estimates of the change of the state of construction is involved, the best calculations can be done with the same accelerogram, but with changed PGA.

MODEL CONSTRUCTION 2D FRAME FOUNDED ON PILES GROUP 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 previously described façade 2D frame (fig.2) with its corresponding loads is then taken out of a 3D model dimensioned in this way. The span between frames is 8m, 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 5m, while for the remaining 6 storeys, it is 3.1 m. (Folić B, and Folić R. 2018; Folić B, 2017).

The soil was modeled using multiplastic link elements fig. 2a, 2b, and 2c, with the help of p-y curves for sand. For modeling hysteresis enveleope Takeda tip, hysteresis p-y curve was used for submerged dense sand, piles with 60 cm diamter, and angle of inner friction 35°.

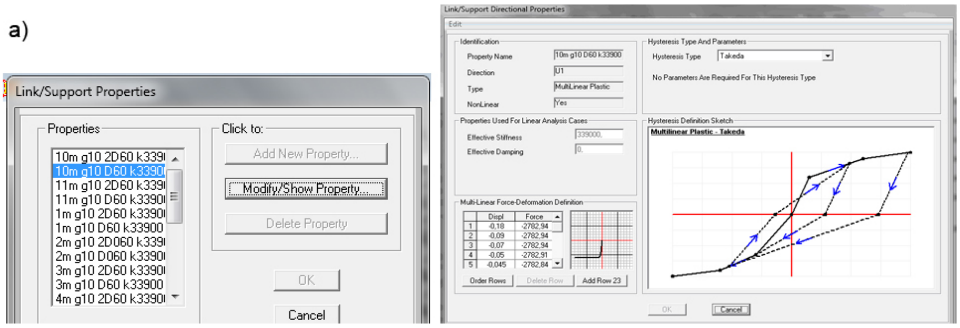


Figure. 2. (a) Input p-y curve in SAP2000;
Slika 2 (a) Ulazne p-y krive u SAP2000;

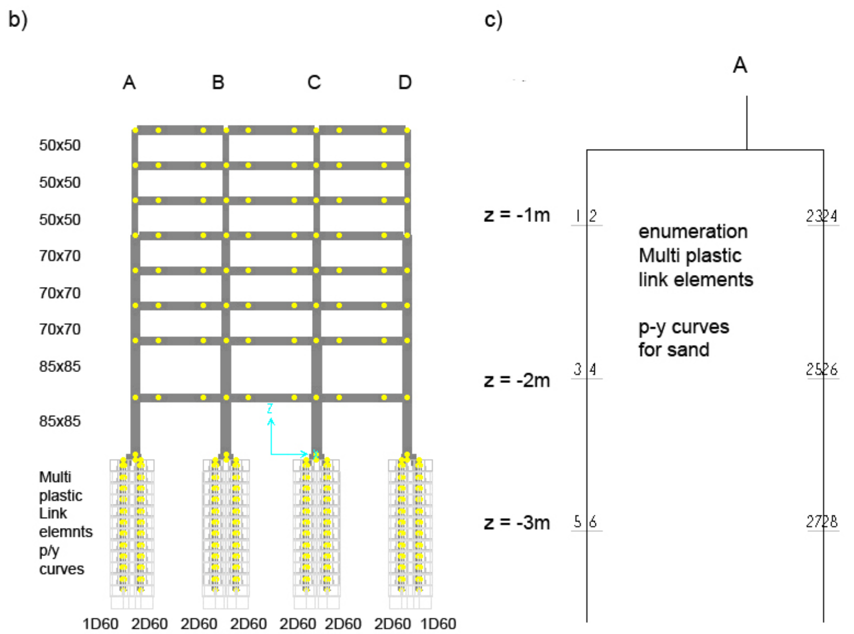


Figure. 2. (b) 2D Frame with dimension of columns and piles with p-y curves as link elements. (c) Enumeration p-y curves left group of piles
Slika 2 (b) 2D ram sa dimenzijama stubova i šipova is a p-y krivama kao link elementima; (c) Numeracija p-y krivih leve grupe šipova

RESULTS AND ANALYSES

LOCAL DRIFT AND MOVEMENT OF JOINTS AT THE TOP OF THE BUILDING

In the following images the movement of joints at the top of the building is shown, for aftershcok 0.15; 0.20 (Fig. 3) 0.21g, and main strike 0.30 g (Fig. 4).

In Figure 3 the displacement of the joint at the top of the building during aftershock of earthquake is shown ElCentro for PGA 0.15; 0.20g. In Figure 4a for PGA 0.21g AfSh the displacement of column joints is shown (in the axis above the tested pile) in height, used for calculating local drift (interstory drif is overall, Fig. 5). In figure 4b following the effect of the main strike 0.30g displacement of the joint at the top of the building is shown. It can be seen that the movement of the joint at the top of the building during the aftershock is divergent. This divergence becomes clearer when the state of the plastic hinges after the first earthquake is observed, in images 6 and 7, which are shown below.

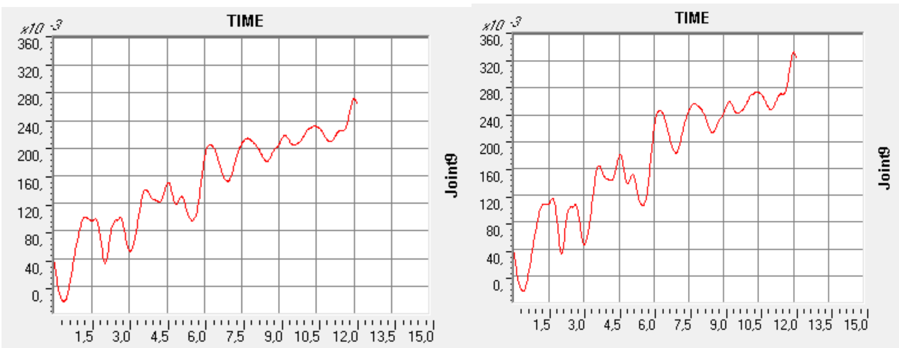


Figure. 3. Displacement plot of the joint at the top of the building, due earthquake acc. ElCentro: (a)After Shock PGA AfSh 0,15g. FS Mayor Shock 0,30g. Ux max=27,18 cm. Ux min=-2,204 cm; (b) PGA AfSh 0,20g. FS 0,30g.Ux max=33,33 cm. Ux min= -2,277cm.

Slika 3. Grafik pomeranja čvora u vrhu zgrade, tokom zemljotresa ElCentro. (a) Naknadni udar PGA AfSh 1,15g. Glavni udar FS 0,30g Ux max=27,18 cm. Ux min=-2,204 cm; (b) PGA AfSh 0,20g. FS 0,30g.Ux max=33,33 cm. Ux min= -2,277cm.

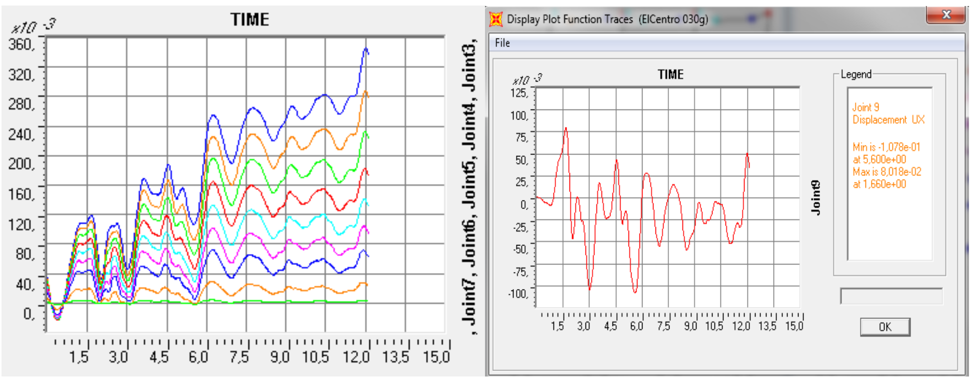


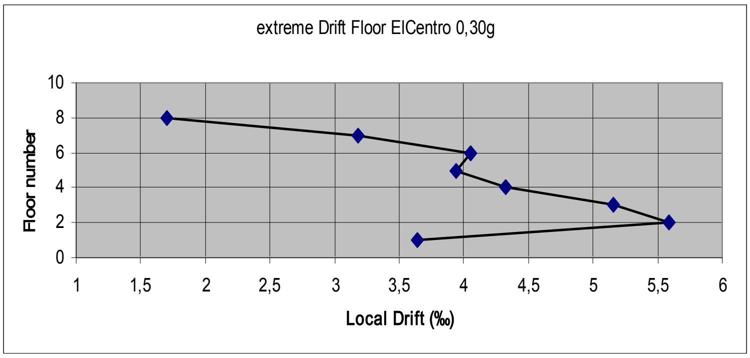
Figure. 4. Displacement plot of the joint at the top of the building, due earthquake acc. ElCentro: (a) All Joints per height building PGA After Shock AfSh 0,21g. FS 0,30g.:Ux max=34,61 cm. Ux min=-2,292 cm.; (b) Top joint FS mayor shock PGA 0,30g.

Slika. 4. Grafik pomeranja čvora u vrhu zgrade, tokom zemljotresa acc. ElCentro: (a) Svi čvorovi po visini zgrade PGA Nakanadni udar AfSh 0,21g. FS 0,30g.:Ux max=34,61 cm. Ux min=-2,292 cm.; (b) Čvor u vrhu zgrade FS glavni udar PGA 0,30g.

Local drift in milipercents [‰] was calculated as an extreme value of all temporal intervals of accelogram's activity, according to the formula:

$$extrDrift = \underset{i=1}{\overset{n}{extrem}} [ABS(U_{i+1} - U_i) / h_i \cdot 1000]_t$$

(2)



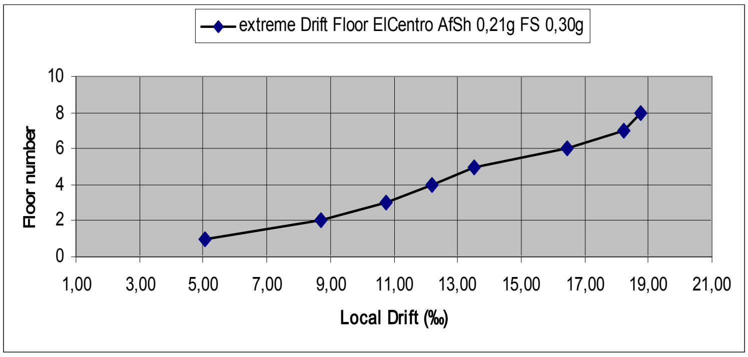


Figure. 5 Extreme Local (storey) drifts IDR. ElCentro. NDA
(a) FS PGA 0,30g (b) Aftershock PGA 0,21g.
Slika 5 Ekstremni lokalni (spratni) drift IDR ElCentro. NDA
(a) FS PGA 0,30g (b) Aftershock PGA 0,21g.

THE STATE OF PLASTIC HINGES

The adoption of the dimensions for cross sections of 2D (and 3D) frame columns fig. 1 is conducted according to the old seismic rules, so that the columns in the lower two storeys have a greater width than EC8, and smaller in the top three storeys. So, according to the existing project (Čaušević, 2010), dimensions of the top columns are 60x60cm, which gives better seismic performances on those storeys, mostly in joints at the top of the building, than the ones here adopted 50x50cm. All the joints are counted, only the two joints at the top of the building for the two inner columns are not considered. There are actually three plastic joints here (Elnashai, and Di Sarno, 2008), but without magnification only two are visible, the ones at the ends of the beams.

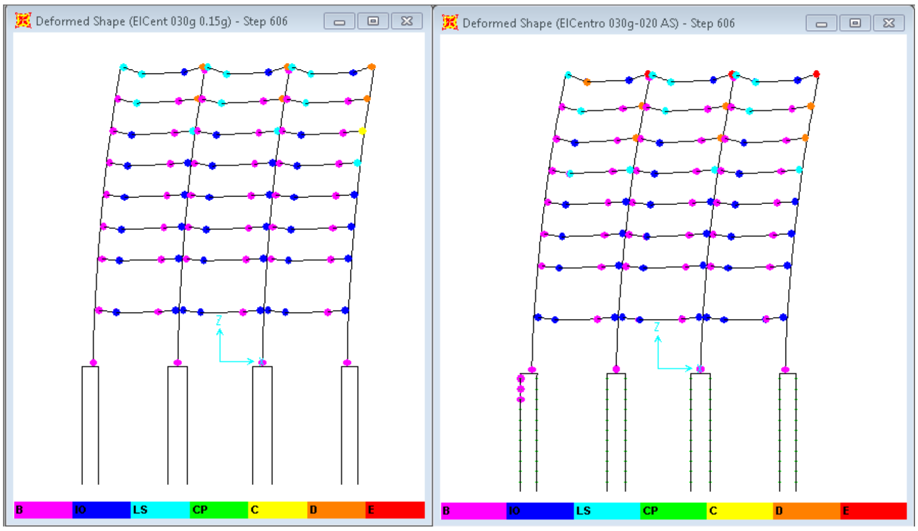


Figure. 6. State of plastic hinges (PHS) at the end earthquake ElCentro FS 0,30g, (a) PGA AfSh 0.15g PHS: 44Y, 37IO, 12LS, 1C, 6D. (b) PGA AfSh 0.20g PHS: 47Y, 36IO, 12LS, 7D, 3E.
Slika. 6. Stanje plastičnih zglobova (PHS) na kraju zemljotresa ElCentro FS 0,30g, (a) PGA AfSh 0.15g PHS: 44Y, 37IO, 12LS, 1C, 6D. (b) PGA AfSh 0.20g PHS: 47Y, 36IO, 12LS, 7D, 3E.

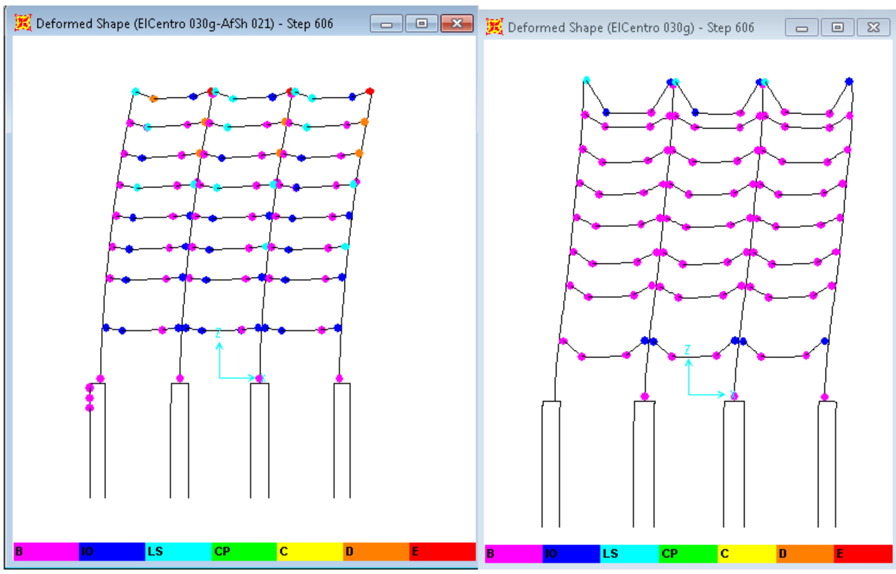


Figure. 7. State of plastic hinges (PHS) at the end earthquake ElCentro FS 0,30g, (a) PGA AfSh 0.21g PHS: 47Y, 32IO, 15LS, 7D, 3C. (b) PGA FS 0.30g PHS: 86Y, 10IO, 3LS.

Slika. 7. 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.

In the images of the state of plastic hinges during an aftershock of 0.20g (Fig. 6b) and 0.21g (Fig. 7a), the opening 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.

In table 1 the number of plastic hinges states is given for FS, and differences between plastic hinges at the end of the earthquake ElCentro, after the first strike and the aftershock (for PGA 0.15g, 0.20g and 0.21g) are given.

The lowest relevant PGA of the aftershock, in this research the value AfSh of 0.15g (50% of PGA FS) is chosen, 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. The number of plastic hinges in PGA AfSh 0.20 in relationship to AfSh 0.21g (70% of FS) is practically negligible, and it is given in table 1. However, after a more detailed observation of the state and the arrangement of plastic hinges, a significant difference is noticed: at the bottom of the 6th storey column a plastic joint appears. This tells us that a further increase of the PGA value of afteshock would lead to the destruction of the building, but this sort of increase is highly unlikely in most earthquakes.

Table 1. Number of Plastic Hinge State for First Shock and Diferent Number of Plastic Hinge State for First Shock and After Shock.

PGA (g)		Number of Plastic Hinge State							
FS	AfSh	Y=B	IO	LS	CP	C	D	E	Σ PLH
0.30	-	86	10	3	0	0	0	0	99
State1→State 2		Diferent Number of Plastic Hinge State							
FS or AfSh		Y=B	IO	LS	CP	C	D	E	Δ Σ PLH
FS → 0,15 AfSh		-42	27	9	0	1	6	0	1
FS → 0,20 AfSh		-39	26	9	0	0	7	3	5
FS → 0,21 AfSh		-39	22	12	0	0	7	3	5
0,20 AfSh → 0,21 AfSh		0	-4	3	0	0	0	0	-1

CONCLUSIONS AND DIRECTIONS FOR FURTHER RESEARCH

The interpretation of the results of the non-linear SPSI analysis showed that not everything has to be logical, at least not at first sight. Because of this, for more detail analyses it is necessary to take several different values PGA, and separately, but also integrally, take into consideration the results of the dynamic interaction of the system construction-pile-soil.

When choosing the model of the soil for frame constructions, elastic secant stiffness is used (Pando 2013) or p-y curves. The change of secant stiffness can considerably change the shape of the break of the system building-foundation-soil. 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 modelling of the linear part of damping and stiffness, while the non-linear part is of hysteresis type. The effect of afteshock is tested here by overtaking the matrix of stiffness, and thereby the state of plastic hinges, at the end the state of TH analysis of the main strike (concretely accelerogram ElCentro 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) in which the soil is modeled by p-y curves, it is not always clear whether the hysteresis curves adopted in such a way are safe in earthquakes, so it is recommended to use variation of parameters of curves depending on the reliability of entry data of geo-mechanical testing. Such a variation of parameters could be used as an envelope of the effect for dimensioning RC buldings, and then a dynamic analysis should be done, i.e. seismic performaces tested. For p-y curves see (Maymond, 1998; Milović and Đogo 2009; Mosher and Dawkins, 2000 ; Stewart et all 1999; Ćosić et all, 2014).

In dissertation (Folić B., 2017.) showed a possibility of widened accelerogram ElCentro, on the extended response of 20 and 40 sec, where movements were convergent. In this kind of divergent movement, the use of extended temporal response of TH analysis is recommended after the effect of earthquake because only then a truly divergent or convergent movement can be established.

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REFERENCES

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

Datta, T.K., 2010. Sismic analysis of structure. Indian Institute of Technology Delhi, India. John Wiley & Sons (Asia) Pte Ltd. Pp 454.

Elnashai, A.S., Di Sarno, L., 2008. Fundamentals of earthquake engineering, Wiley, West Sussex, UK

Čaušević M., 2010. Dinamika konstrukcija. Golden Marketing. Tehnička knjiga. Zagreb

Ćosić, M., Lađinović, Đ., 2006. Connventional and modified pushover methods. INDIS, Zbornik radova . Novi Sad. Pp. 267-274.

Ćosić, M., Folić B., Folić R., Sedmak, S., 2014. Performace-based seismic analysis of highway E75 overpass at kovilj. Structural integrity and life. Vol.14. Beograd. pp.17-28.

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

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

Glavatović, B., 2005. Inženjerska seizmologija. Skripta. Građevinski fakultet u Podgorici.

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

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

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

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

Milović, D., Đogo. M. 2009. Problemi interakcije tlo-temelj-konstrukcija. Srpska akademija nauka i umetnosti ogranak u Novom Sadu. Novi Sad.

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

Pando, M., 2013. Analyses of Lateral Loaded Piles with p-y Curves - Observations on the Effect of Pile Flexural Stiffness and Cyclic Loading. NCDOT 7thGeo3T2, Raleigh, NC, Thursday, April 04, 2013. Paper: 3B-1_A49.

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

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

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