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PERFORMANCE-BASED SEISMIC EVALUATION OF SOIL-PILE-BRIDGE PIER INTERACTION USING INDA

Abstract: The purpose of this paper is to present the methodology for performance-based seismic evaluation of soil-pile-bridge pier interaction using the incremental nonlinear dynamic analysis (INDA). The INDA analysis was post processed separately for the pier and for the pile, so that the constructed *PGA=f(DR)* curves are in the capacitive domain. For these curves the authors identified the IO, CP i GI performance levels, while the regression analyses were conducted based on the specific *DR* and *PGA* parameters. Fragility curves were constructed based on the solutions of regression analysis and the probability theory of log-normal distribution. Based on the results of fragility analysis, reliability curves were also constructed.
Key words: INDA, seismic performance, fragility, reliability, artificial accelerograms

EVALUACIJA SEIZMIČKIH PERFORMANSI INTERAKCIJE TLO-ŠIP-STUB MOSTA INDA ANALIZOM

Abstract: U radu je prikazana procedura evaluacije seizmičkih performansi interakcije tlo-šip-stub mosta inkrementalnom nelinearnom dinamičkom analizom (INDA). Postprocesiranje INDA analiza je sprovedeno posebno za stub, a posebno za šip, tako da su konstruisane krive *PGA=f(DR)* u kapacitativnom domenu. Za ovako konstruisane krive određeni su IO, CP i GI performansni nivoi, a na osnovu određenih *DR* i *PGA* parametara sprovedene su regresione analize. Krive povredljivosti su konstruisane na osnovu rešenja regresione analize i teorije verovatnoće log-normalne raspodele, a za *PGA* meru intenziteta. Takođe, konstruisane su i krive pouzdanosti na osnovu rešenja analize povredljivosti.
Key words: INDA, seizmičke performanse, povredljivost, pouzdanost, veštački akcelerogrami

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1. INTRODUCTION

Due to the complexity of phenomena involved in the wave propagation in soil-structure interaction (SSI), mathematical modelling of this problem is based on a multidisciplinary approach to the engineering seismology and earthquake engineering. The soil-structure interaction can be considered by conducting tests on actual models and/or in the laboratory, using analytical and numerical methods. Seismic performances are considered in several ways: by applying the deterministic concept with a single earthquake scenario, based on parametric analysis and the probabilistic concept. The paper [22] presents a 3D finite element incremental dynamic analysis study of caisson foundations carrying single-degree of freedom (SDOF) structures on clayey soil. The emphasis is given to the interplay between the nonlinearities developed above (superstructure) and, mainly, below ground surface, either of material (soil plasticity) or of geometric (caisson–soil interface gapping and slippage) origin. The pile performance analysis by establishing a correlation between the engineering demand parameters (EDP) and the intensity measure (IM) is presented in [4]. The general approach of modelling the dynamic interaction of piles groups in the soil using the hybrid techniques by connecting the finite element method (FEM) and the boundary element method (BEM) is discussed in [5], while the various aspects of mathematical and numerical modelling of the complex soil-piles interaction are presented in [11]. General approaches to analyzing the seismic performance of piles with the emphasis on various mathematical soil-pile interaction models are presented in [10]. Modelling the piles and soil using 3D finite elements and taking into account the influence of plastic nonlinear soil behaviour in seismic performance assessment is presented in [1].

The number of soil-pile-bridge pier interaction studies based on the incremental nonlinear dynamic analysis (INDA) is considerably fewer. Therefore, the concept of this work is focused on modelling the aspects of soil-pile-bridge pier interaction based on the INDA analysis. In order to understand and complete the methodology of these analyses, in addition to the INDA, the following was also considered: numerical modelling of soil-pile-bridge pier interaction and the generation of artificial accelerograms. Results of numerical simulations were presented and 300 NDA analyses were statistically processed.

2. SOIL-PILE-BRIDGE PIER INTERACTION

There are several approaches to modelling and analyzing the soil-pile-bridge pier interaction based on the finite element method, taking into account the development of geometric and material nonlinearity. Figure 1a shows the actual pile model in the soil, and with the structure above ground level (bridge pier), while figure 1b shows the numerical pile model formed from column finite elements, and with the structure above ground level, also formed using column finite elements. The column finite elements for modelling the pile and the bridge pier are based on the principle of nonlinear deformation along the element, where at the cross section level a specific fibre discretization is implemented. The cross-section is generally considered through three sub domains: unconfined concrete, confined concrete and steel. The stress-strain state at the cross section level is determined by integrating the nonlinear single-axis stress-strain state of each single fibre. According to Mander [15], the constitutive model of behaviour for the unconfined and confined domains of concrete is a nonlinear constant confinement concrete model. The constitutive model of behaviour of steel reinforcement is a bi-linear elastic-plastic model with kinematic strain hardening in the nonlinear deformation zone [20].

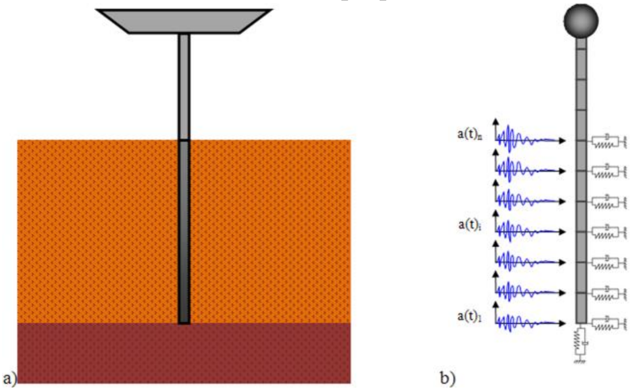

Figure 1. a) the realistic model of pile in the soil, bridge pier and soil, b) the numerical model of pile, bridge pier and implicit modelling of soil action

The nonlinear dynamic soil-pile-bridge pier interaction is modelled using the constitutive model of behaviour for the lateral analysis of piles, where the formation of gaps under cyclic soil deformation is also taken into account [2]. Effects of cyclic degradation/hardening of soil stiffness and strength are also taken into account; in addition, actions in the direction pile axis are also separately modelled, which are orthogonal to the effects that are introduced by applying this model of interaction. The hysteretic constitutive model consists of four major parts: backbone curve, standard reload curve (SRC), general unload curve (GUC) and direct reload curve (DRC) [3]. Defining the mechanical properties of the constitutive model of the soil-pile-bridge pier interaction behaviour requires nineteen parameters.

3. ARTIFICIAL ACCELEROGRAMS

The procedure of generating artificial accelerograms is conducted by determining the spectral density function based on the response spectrum; in this specific case a pseudo response spectra has been used [9]. This function is used to derive the sinusoidal signal amplitude the phase angle of which is generated by a random number function in the range between $0 \div 2\pi$ according to uniform distribution. Sinusoidal signals are compressed in order to generate accelerograms. In order to determine the other properties of the artificial accelerogram, such as duration of recording, it is necessary to obtain additional information about the expected earthquake based on the response spectrum. Upon the generation of artificial accelerograms for representing the record of the free field motion, further analyses are conducted in order to generate accelerograms for soil layers and bedrock motion. In this specific case, the soil is considered as a single-layer system, but given the number of input accelerograms in numerical analyses for simultaneous performance of numerical integration in time, the single-layer system is considered as a multi-layer system with the same geo-technical properties. For each individual layer accelerograms are generated taking that waves are propagating similar to the single-layer system [14].

4. NUMERICAL SIMULATION RESULTS AND DISCUSSION

Numerical simulations of nonlinear pile behaviour in interaction with the soil were carried out using the finite element method in the *SeismoStruct* software [18]. The pile and pier diameter is d_p=1.8m, the pile length is L_p=15m, while the bridge pier height is L_b=10m. The pier and pile are of circular cross-section with radially disposed reinforcement consisting of 25 rods of Ø40mm diameter. The cross-section is discretized to 300 fibres, and a total of 10 integration sectors were considered. The mass applied to the pier top is m=816t. The constitutive concrete model is defined for the C 25/30 strength class, according to EC 2 [7]. The constitutive model of steel reinforcement is also defined according to EC 2 [7]: E_s=200GPa and $f_{s,y}$=435MPa. The following are the parameters of the constitutive model of soil-pile interaction: K_o=15000KN/m³, P_0=0, P_a=0, α=0.5, α_n=1, β=0, β_n=1, Flg=31, e_{p1}=1, p_1=1, p_2=0, p_k=1, e_k=1, p_s=1, e_s=1 and k_s=0.1. Parameters F_c and F_y are determined in the function of changes along the soil depth, so that these values were separately identified for the 16 link elements used for modelling the soil-pile interaction based on the *p-y* curves.

The artificial accelerograms were generated using the *Simqke* software [21] for the horizontal elastic response spectra according to EC 8 [8] for type C soil, the peak ground acceleration PGA_0=0.35g, the soil coefficient S=1.2 and damping ratio ξ=5%. Two groups were considered, each with five artificial accelerograms. The first group consists of accelerograms of shorter total time of acceleration recording t_{acc}=20s and a shorter time of stationary domain, where the times of stationary domain initiation and finalization are $t_{s,i}$=2s and $t_{s,f}$=10s, respectively. The second group consists of accelerograms with longer total time of acceleration recording t_{acc}=40s and a longer time of stationary domain, where the times of stationary domain initiation and finalization are $t_{s,i}$=2s and $t_{s,f}$=15s, respectively. Accelerograms were sampled at a time interval of Δt=0.01s, so that sampling frequency is f_s=100Hz. For all generated artificial accelerograms, *PGA* is obtained to be 0.437g.

After the accelerograms were generated, they were further processed in the *Shake* software [19], in order to generate independent accelerograms along the soil depth $a(t)_i$. The soil domain is discretized to 15 soil layers of 1m thickness, while the bedrock domain is considered separately, so that for each INDA analysis 16 simultaneous accelerograms were used in the processing phase. A total of 160 accelerograms were generated in this manner. In INDA analyses, these accelerograms were simultaneously scaled, so that for a single INDA analysis all 16 accelerograms were scaled with the same scale factor. First, accelerograms were scaled to the initial value of $PGA_{s,1}$=0.1g for h=0 and then incrementally scaled to ΔPGA=0.1g. Given the differences among the accelerograms and the scale factor, the ultimate scale factors among the accelerograms for a single INDA analysis are also different. Due to the large number of generated accelerograms, they are not presented in this paper. For each INDA analysis, accelerograms were scaled to PGA_{max}=3g, so the total of 300 NDA analysis were carried out. By processing the INDA analyses the discrete values $I_i(EDP_i,IM_i)$ were obtained, which were then interpolated and represent the system response in the capacitive domain. For the EDP parameter, a global drift (*DR*), while for the IM parameter a *PGA* was selected. Figures 2 and 3 are depicting the *DR-PGA* ratio curves for the pier top and the pile head, respectively.

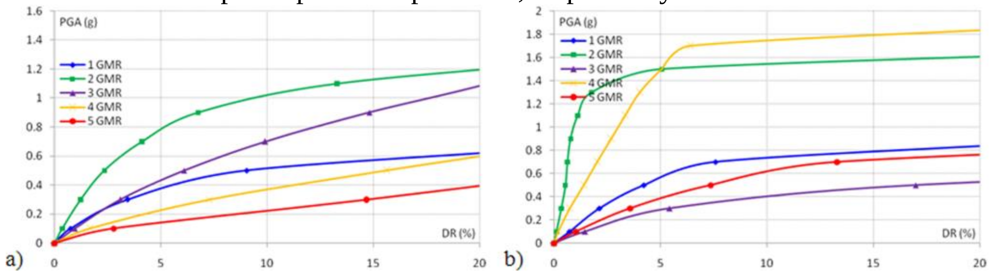

Figure 2. The DR-PGA curve for the pier top: a) the first group of accelerograms, b) the second group of accelerograms

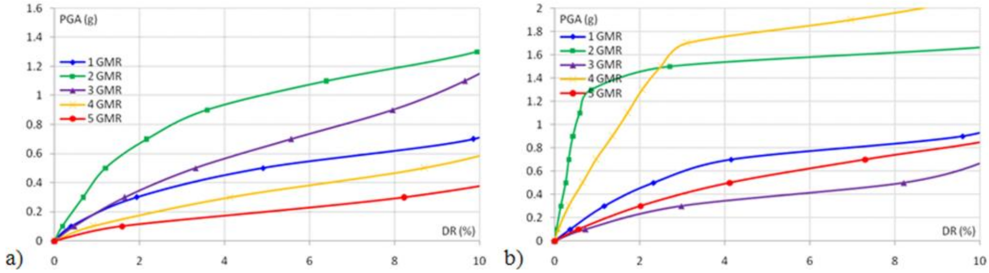

Figure 3. The DR-PGA curve for the pile head: a) the first group of accelerograms, b) the second group of accelerograms

Generally, it can be concluded that there is a discrepancy in the soil-pile-bridge pier system response for two different groups of accelerograms. A difference also exists when considering the pier and pile response, where slightly higher *PGA* values were registered for the pile, as compared to the pier. The drift interval value for the pier is considered in the range of *DR*=[0÷20]%, while for the pile this range was *DR*=[0÷10]%. The limit states of the soil-pile-bridge pier system were determined by considering the structural performance level (SPL): immediate occupancy (IO), collapse prevention (CP) and the global dynamic instability (GI). For the purpose of the present study, the appropriate limit state for the IM parameter has been established based on the EDP parameter according to codes.

The IO performance level is determined by considering the *PGA* value for the global drift DR_{IO} of reinforced concrete systems according to SEAOC [17], where D_{IO} is the displacement for the IO performance level and *H* is the height. The CP performance level is determined when the tangent slope to the *PGA=f(DR)* curve is equal to 20% of the initial elastic slope DR_e of this curve or when *DR*=10%, where D_{CP} is the displacement for CP performance level. The GI performance level is determined for the condition that the *PGA=f(DR)* curve asymptotically approaches the horizontal line, where D_{GI} is the displacement for the GI performance level. Based on the above set criteria for determining the performance level, statistical analyzes were conducted for each *PGA=f(DR)* curve. Results of these analyzes are shown in table 1, sorted separately for the pier and pile. Tags in the table are as follows: PGA_m mean value of the maximum acceleration values, PGA_{med} median value of the peak acceleration values, PGA_{min} minimum value of peak accelerations, σ standard deviation, v variance.

Table 1. Discrete DR_i and PGA_i values of specific performance levels for the soil-pile-bridge pier interaction

pier									
performance level		IO_{min}		IO_{max}		CP		GI	
DR_m (%)	*PGA* (g)	0.5	0.11	1	0.23	8.32	0.75	-	0.96
DR_{med} (%)	*PGA* (g)	0.5	0.06	1	0.11	10	0.65	-	0.87
DR_{min} (%)	*PGA* (g)	0.5	0.02	1	0.04	1.4	0.24	18.6	0.39
σ		0.132		0.305		0.446		0.473	
v		0.017		0.093		0.199		0.223	
pile									
performance level		IO_{min}		IO_{max}		CP		GI	
DR_m (%)	*PGA* (g)	0.5	0.22	1	0.36	6.71	0.88	-	1.03
DR_{med} (%)	*PGA* (g)	0.5	0.11	1	0.20	7.35	0.80	-	0.90
DR_{min} (%)	*PGA* (g)	0.5	0.03	1	0.07	0.9	0.36	-	0.36
σ		0.290		0.387		0.420		0.535	
v		0.084		0.150		0.176		0.287	

In this specific case, a lower drift value has been realized of DR_{min}=0.9% for the CP performance level, as compared to the drift value of DR_{min}=1% for the IO_{max} performance level at the pile head. The consequence of this situation is that the pile can much faster develop the state of pre-collapse in the second group of accelerograms. The determination of the GI performance level is much more complicated as compared to the previous IO and CP performance levels. More precisely, it is obligatory that the *PGA=f(DR)* curve is horizontal; in many cases, however, this condition is optional, unless the sign of inclination of the *PGA=f(DR)* curve changes from positive to negative value. This condition is achieved only in one case, in DR_{min}=18.6% and *PGA*=0.39g for the pier, while in other cases the GI performance level is determined based on the maximum drift value.

Unlike the previously presented deterministic methods of evaluation of performance levels and the conditions of the soil-pile-bridge pier interaction system, based on the theory of probability it is possible to consider the system's fragility. The probabilistic concept in the analysis of the soil-pile-bridge pier interaction system is based on a qualitative consideration of the damage level according to HAZUS [12]: slight, moderate, extensive and complete. These damage levels are defined as a function of the system ductility μ, so that the level of slight damage is equivalent to $1<\mu<2$, the level of moderate damage is equivalent to $2<\mu<4$, the level of extensive damage is equivalent to $4<\mu<7$, while the level of complete damage is equivalent to $\mu>7$ [6]. The intensity parameter IM is commonly considered by identifying the appropriate response spectra with the variation of standard deviation $\pm\sigma$, which is a function of uncertainty of the seismic demand that is imposed to the structure. However, in this study, a variation of seismic demand is applied which is a function of scaling the IM parameter, i.e. the *PGA*, according the INDA analysis. In this sense, it is possible to consider a much wider range of seismic demand variations *PGA*=[0÷1]g without any further extrapolation. The relation between μ and *PGA* was determined based on regression analysis for the linear function of lnμ=k·ln*PGA*+*n*.

The fragility curve was constructed in relation to the *PGA* intensity measure by using the log-normal distribution, the probability density function. The cumulative distribution function on the occurrence of damage is determined by [13], where *erfc* is the complementary error function and Φ is the cumulative distribution function. The discrete probability functions for the pier and pile are shown in figures 4a and 4b, respectively. A lower level of damage is typical up to *PGA*=0.2g for the pier model, while for the pile, this value is up to *PGA*=0.3g.

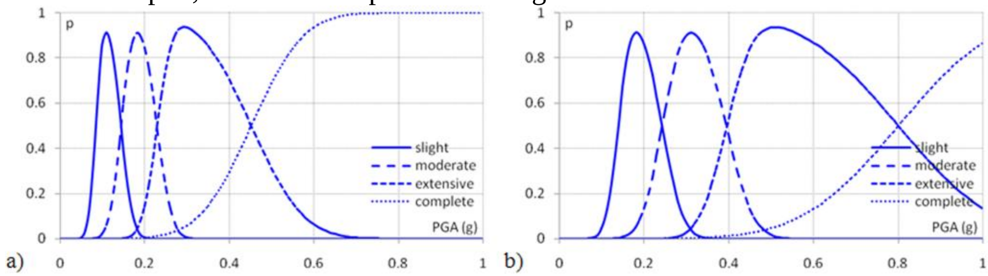

Figure 4. Discrete probability functions: a) pier, b) pile

The cumulative probability distribution function of damage for the seismic soil-pile-bridge pier interaction is shown in figures 5a and 5b for the pier and pile, respectively. The upper limit of the complete damage level is considered for μ_{sup}=20, whereby the changes of this limit significantly affect the cumulative probability distribution function of complete damage. By comparing the obtained solutions for the pier and pile, it can be concluded that the pier is more sensitive to the changing levels of intensity measures *PGA*. The consequence of this is that the same *PGA* level results in larger damage to the pier, where the development higher intensity damage is also more likely.

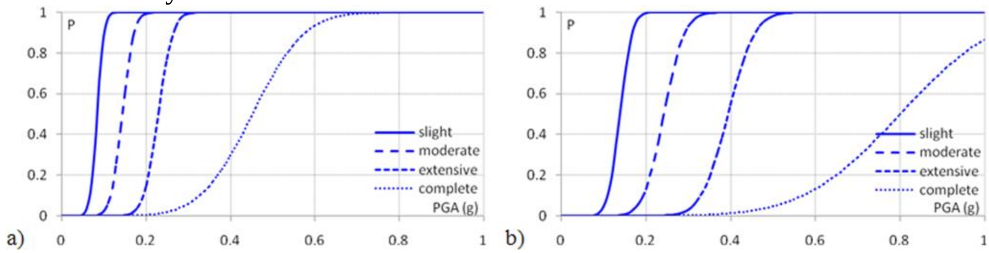

Figure 5. Fragility curves for seismic soil-pile-bridge pier interaction: a) pier, b) pile

Typical values for seismic intensity measures *PGA*=[0.1÷0.5]g and the corresponding probabilities of fragility P_i for seismic soil-pile-bridge pier interaction are shown in table 2. Values of fragility probability beneath the diagonal in table 2 are typically equivalent to 1 or very close to this value, while those above the diagonal are typically equivalent to 0 or very close to this value. The values on the diagonal itself and near to it in table 2 are declining. If, for example, the value of *PGA*=0.1g, then it can be concluded that at all fragility levels of the pier are higher than that of the pile. Thus, for the level of slight damage, the probability of pier and pile fragility are equal to P=0.88 and P=0.04, respectively, while for the level of extensive damage this value is P=0 for both the pier and the pile. On the other hand, for PGA=0.3g, the probability of pier and pile fragility for the level of slight damage are P=1, while for the level of extensive damage is P=0.99 and P=0.03, respectively.

Table 2. Probability of fragility P_i for the typical seismic intensity measure PGA_i of the soil-pile-bridge pier interaction

		pier				pile			
damage		slight	moderate	extensive	complete	slight	moderate	extensive	complete
PGA=0.1g	P_i	0.88	0.02	0	0	0.04	0	0	0
PGA=0.2g	P_i	1	0.99	0.15	0	0.99	0.13	0	0
PGA=0.3g	P_i	1	1	0.99	0.06	1	0.93	0.03	0
PGA=0.4g	P_i	1	1	1	0.29	1	1	0.55	0.01
PGA=0.5g	P_i	1	1	1	0.69	1	1	0.98	0.05

Evaluation of the system performance is also performed by analyzing the system reliability state. When applying this analysis a more complete answer is obtained regarding the system state, and it is based on the previously considered fragility analysis. System reliability R is defined by [16]. A negative R coefficient value indicates a possible failure and system unreliability, while a positive R coefficient value indicates that the failure probability is approximately equal to 0, i.e. that the system is reliable to a significant degree. When the R coefficient value is ≈6, then the system reliability is ≈100%, while in the case when R≈0, the system failure probability is P=50%. Reliability curves for the seismic soil-pile-bridge pier interaction are shown in figures 6a and 6b for the pier and pile, respectively.

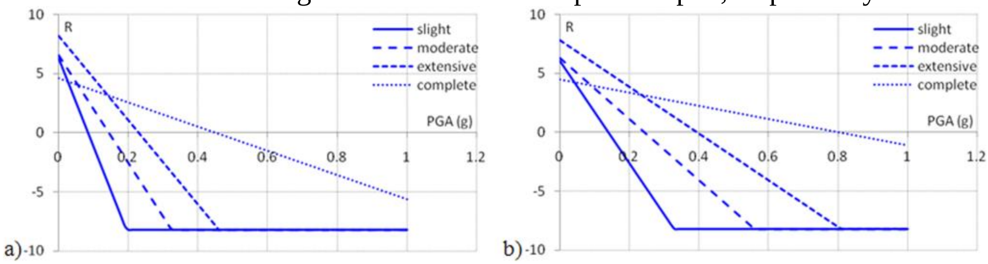

Figure 6. Reliability curves for the seismic soil-pile-bridge pier interaction: a) pier, b) pile

Comparing the solutions obtained for the pier and pile, it can be concluded that the pier is more sensitive to the changing levels of intensity measure PGA, so that higher levels of uncertainty can be expected at lower PGA values, as compared to the pile. For P>50%, pier reliability at slight level of damage is PGA≤0.08g, at moderate level of damage is PGA≤0.14g, at extensive level of damage is PGA≤0.22g and at complete level of damage is PGA≤0.45g. For P>50%, pile reliability at slight level of damage is PGA≤0.13g, at moderate level of damage is PGA≤0.24g, at extensive level of damage is PGA≤0.39g and at complete level of damage is PGA≤0.8g.

5. SUMMARY AND CONCLUSIONS

In this study, a numerical model has been developed for the soil-pile-bridge pier interaction in order to evaluate the system's seismic performance. Effects representing the influence of soil were introduced by applying the principle of implicit modelling the nonlinear dynamic soil-pile-bridge pier interaction. The input signal to the system is treated through the generated artificial accelerograms, which were further processed by layers of soil and bedrock. The system response is analyzed in the capacitive domain using the incremental nonlinear dynamic analysis (INDA). The INDA analysis was processed in a successive manner by scaling the nonlinear dynamic analysis (NDA) according the defined scaling criteria.

The NDA and INDA analyses were post processed according to the global drift *DR* and the corresponding *PGA* values separately for the pier and separately for the pile, so that curves *PGA=f(DR)* were constructed in the capacitive domain. The IO, CP and GI performance levels were determined for these curves, and based on specific *DR* and *PGA* parameters regression analyses were carried for the linear function $\ln\mu = k \cdot \ln PGA + n$. The fragility curves were constructed based on the solutions of regression analysis and the probability theory of log-normal distribution for the *PGA* intensity measures. The intensity measure *IM* is typically considered by identifying the corresponding response spectra with the variation of standard deviation $\pm\sigma$, which is a function of uncertainty of seismic demand that is imposed to the structure. However, in this study the authors applied a variation of seismic demand in a function of scaling the *IM* parameter, or *PGA* according to the INDA analysis. In this sense, it is possible to consider a much wider range of variation in seismic demand $PGA=[0\div1]$g without any further extrapolation. By comparing the obtained solutions of the fragility curve for the pier and pile, it can be concluded that the pier is more sensitive to the changing levels of intensity measure *PGA*, than the pile. Thus, the same *PGA* level results in larger damage to the pier, where the development of higher intensity damage is also more likely. Based on the solutions obtained in fragility analysis, reliability curves were also constructed. By comparing the obtained solutions for the pier and pile, it can be concluded that the pier is more sensitive to the changing levels of intensity measure *PGA*, so that it can develop higher levels of uncertainty at lower *PGA* values, as compared to the pile. The methodological procedure for seismic performance analysis presented in this study provides an integrated quantitative and qualitative consideration and evaluation of the complex soil-foundation-structure interaction (SFSI).

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