DEVELOPING A METHODOLOGY FOR THE INTEGRATED NUMERICAL EVALUATION AND PERFORMANCE ASSESSMENT OF SOIL-PILE-PIER

Mladen Ćosić[1], Boris Folić[2], Radomir Folić[3]

***Abstract**: In the paper is developed a discrete numerical solid pile model with a discontinuity and defects. Model included performance-based seismic evaluation of the soil-pile-bridge pier interaction. The pile discontinuity and defects are modelled by reducing the specific finite elements and elastic modulus of concrete. The wave-propagation response of the pile was analyzed based on a step-by-step numerical integration using the Hilber-Hughes-Taylor (HHT) method in time domain (THA). The response analysis is performed with an integration of individual reflectograms into a reflectogram surface, which is generated in a 3D cylindrical coordinate system. Non-linear response of the system is considered using the incremental-iterative Newton-Raphson`s method, while the stability analysis is performed according to the modified geometrical nonlinearity analysis of stability. Determination of critical load and effective length of the pile are performed based on numerical solution and using regression analysis of the power function. The procedure of the soil-pile-bridge pier performance evaluation is based on the incremental nonlinear dynamic analysis (INDA). The system's input signal is treated through the generated artificial accelerograms, which were subsequently processed by soil layers and for the bedrock. Fragility curves were constructed based on solutions of the regression analysis and the probability theory of log-normal distribution, while the generation of reliability curves is based on a solution of vulnerability.*

***Key words**: soil-pile-pier of bridge, reflectogram surface, nonlinear analyses, performance*

1. Introduction

Pile-funded structures are calculated in accordance with the criteria of bearing capacity, stability, serviceability and durability according to the limit state theory. The standard approach to the development of a structural model with piles is based on the structure's discrete numerical modelling, while the piles are subsequently calculated. Another solution is to model both the structure and the piles in a discrete numerical manner. In this case, the constitutive model of material behaviour is linear-elastic, while the system is dimensioned according to the interaction diagrams. In most cases, the combination participated by the seismic influence is the applicable load combination. Seismic analysis of the structure and piles in practical engineering problems is based on spectral-modal analysis or the equivalent static method. On the other hand, the behaviour

[1] MrSc, doctorand, Faculty of Civil Engineering, University of Belgrade, Serbia, mladen.cosic@ymail.com
2 MrSc, Innovative Centre, Faculty of Mechanical Engineering, University of Belgrade, Serbia, boris.folic@gmail.com
3 Professor emeritus PhD, Faculty of Technical Sciences, University of Novi Sad, Serbia, folic@uns.ac.rs

of pile-funded structures in earthquake conditions cannot be adequately modelled and analyzed based on the previously described numerical models and methods of analysis. Also, discontinuities, defects, irregularities and damage in piles cannot be modelled in details without applying advanced approaches to numerical modelling. In order to enable the adequate numerical modelling and analysis of the soil-pile-pier of bridge system (SPP) a methodology has been developed for the integrated numerical evaluation of state and seismic performance. This methodology is based on the evaluation of the system behaviour through three different procedures: at the level of a detailed discrete numerical model for the evaluation of defects, irregularities and damage, at the level of nonlinear system response and at the level of the system's seismic performance evaluation.

2. Presentation of the developed methodology

The methodology for integrated numerical evaluation of state and seismic performances of the soil-pile-pier (SPP) system is based on the interaction of a number of different methods, which are conducted using the *finite element method* (FEM). Figure 1 shows the flow chart of the proposed methodology. This methodology begins with considerations regarding the domains participating in the system analysis. At this stage, key physical domains are identified, such as the domain of structure, the domain piles and that of the soil. Domains identified in this manner are the basis for the stage of numerical modelling. This is the stage of discretization and abstraction of the physical model, whereby abstraction implies the aspect of translation the physical model into a numerical model, while discretization implies the aspect of system formation, i.e. the finite element mesh. The domain of structure is modelled using beam or shell finite elements, while the domain of piles is modelled using beam or solid finite elements. Depending on how the domain of soil is modelled, there are generally two approaches: direct (explicit) numerical modelling using finite elements and indirect (implicit) modelling using replacement elements (replacement springs and damping elements). Given that modelling the soil domain using solid finite elements requires large hardware resources, when the soil domain is of considerable dimensions and when there is a high number of solid finite elements, it is acceptable to shift from 2D to 3D modelling or implicit modelling. Another option is to apply parallel processing techniques. After geometrical and numerical modelling, the model is further analyzed and corrections are introduced in order to improve the SPP model, the finite element mesh, boundary and intermediate conditions, and the like. As noted above, this methodology is based on the evaluation of the system behaviour through three different procedures.

In the first part, the pile (piles) is numerically modelled using 3D solid finite elements, with the further introduction of corrections by modelling discontinuities, defects, irregularities and damage to the pile. The general approach to modelling defects and damage to the pile is applying the principle of elimination of certain solid finite elements and reduction of the pile cross section, specifically in the pile base (toe) zone. Damage at the level of materials, instead of the level of geometry, is modelled by reducing the pile's level of elastic modulus in a particular zone. Figure 2 presents the discrete numerical models of the pile with defects and damage. The pile-soil interaction is realized using the implicit principle of modelling based on replacement elements. The next stage consists of *pile integrity test* (PIT) by simulating wave propagation through the pile and analyzing numerical integration using the *Hilber-Hughes-Taylor* (HHT) method of time history analysis (THA). The PIT test is based on the one dimensional wave propagation theory. The overall period of wave propagation through the pile is measured from the time of initiation by external excitation, through the propagation through the pile, reflection at the base and return to the pile head. The recorded changes in wave propagation through the pile over time are presented in reflectograms.

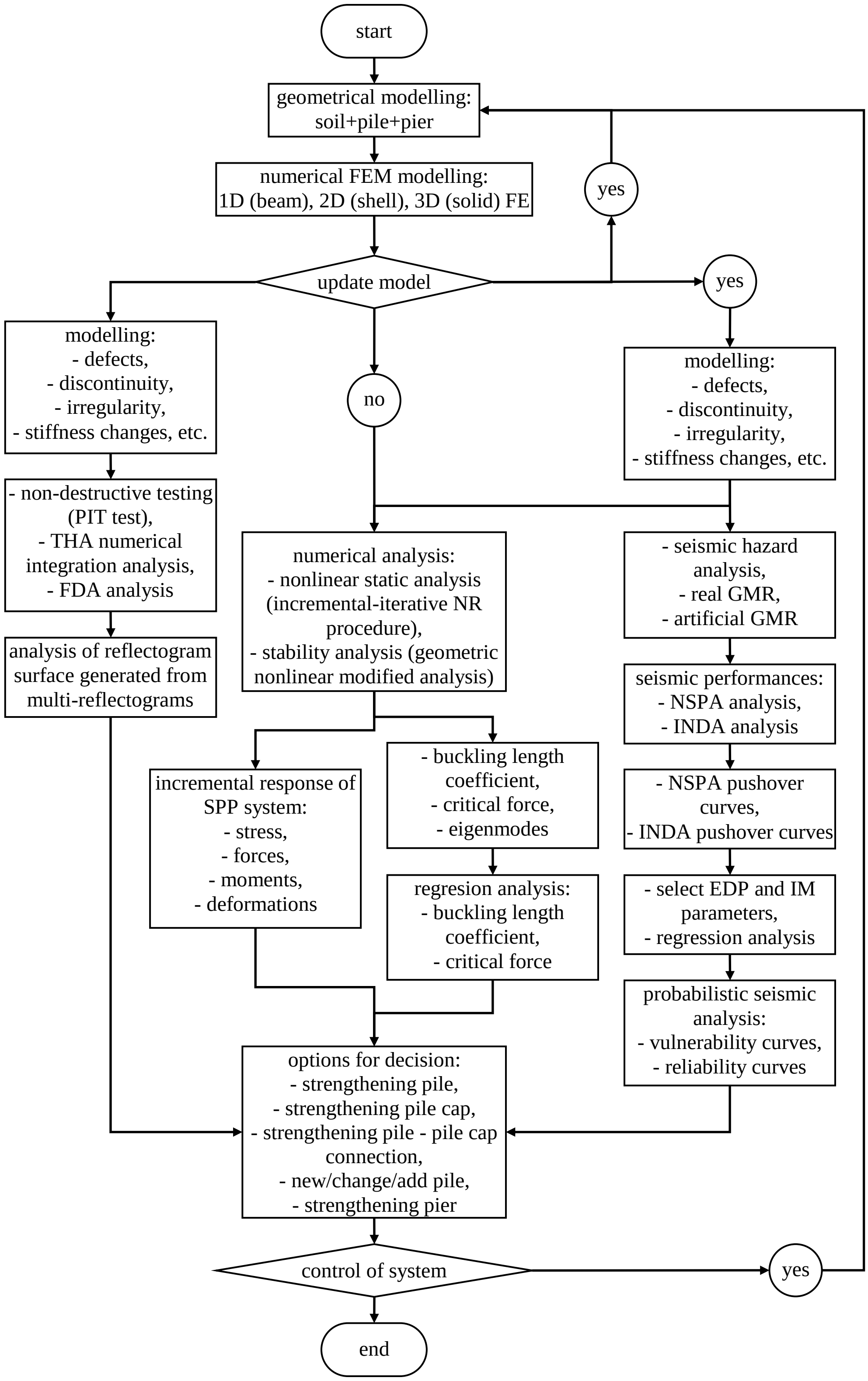


Figure 1. Flow chart of the methodology for numerical evaluation of irregularity and assessment of seismic performances for the SPP system

The reflected waves generated by impedance change (discontinuity) are propagating to the pile head, where changes are recorded in the reflectogram. Changes in the reflectogram occur as a result of changes in the pile base, changes in pile diameter along the pile skin, partial inclusion of soil into the pile domain, due to cracks, variations in the quality of pile material, variations in the soil layers and the influence of reinforcing steel in the pile (heavily reinforced pile). Also, this is the stage where the frequency domain analysis (FDA) is carried out.

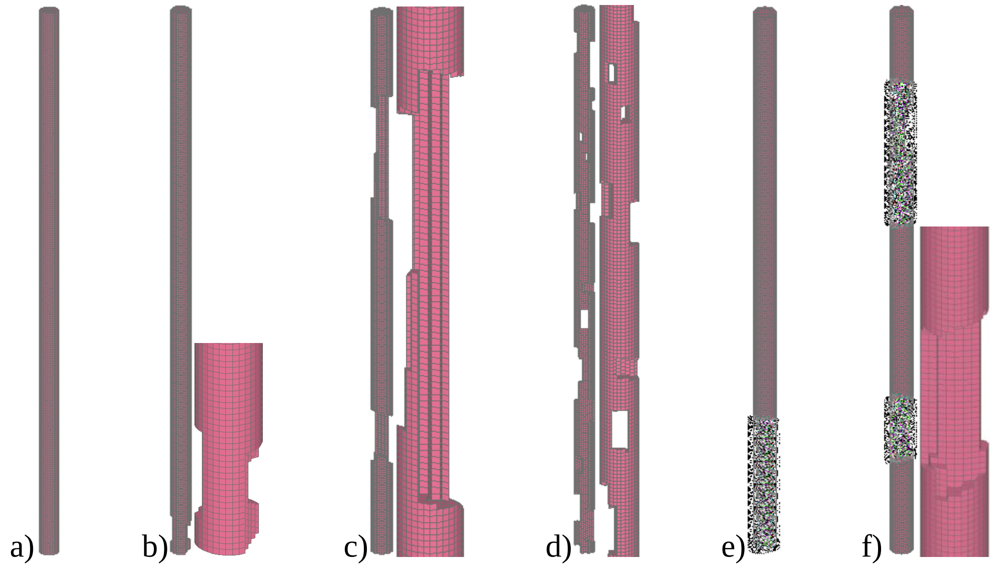


Figure 2. Numerical pile models: a) model 1 - without discontinuities and defects, b) model 2 - asymmetric pile toe defect, c) model 3 - asymmetric reduction of cross section in two locations along the pile length, d) model 4 - significantly degraded pile along its full length, e) model 5 - different elastic modulus in pile toe zone, f) model 6 - different elastic modulus and asymmetric reduction of cross section in two locations along the pile [1]

The system response using non-destructive methods is analyzed by integrating the individual reflectograms into a reflectogram surface, which is generated in a 3D cylindrical coordinate system [1]. Figure 3 presents reflectograms for the 12 measuring points, the generated 3D reflectogram surface and the generated 3D surface of the *Fourier amplitude spectrum* (FAS) for a significantly degraded pile along its whole length (model 4). The assessment of the pile is performed based on the analysis of reflectogram surface and identifying the level of discontinuities, defects, irregularities and damage.

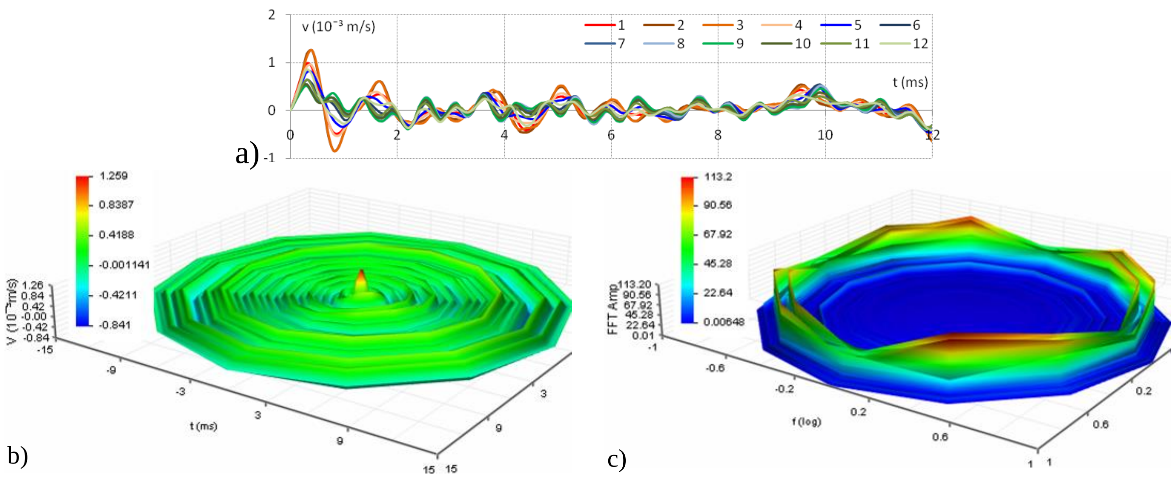
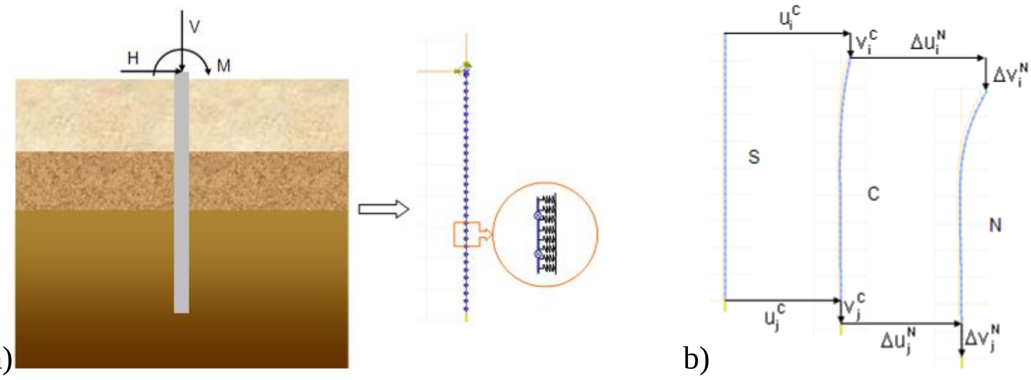


Figure 3. Numerical model of pile 4 - significantly degraded pile along its full length: a) reflectograms, b) 3D reflectogram surface, c) 3D FAS spectrum surface [1]

In the second part, the nonlinear system response is considered using the incremental-iterative *Newton-Raphson* method by controlling the incremental load increase. Consideration of the pile's bearing capacity and its deformation in the soil using linear models and in case development of geometric and material nonlinearities with added material nonlinear soil behaviour is provided in Figure 4a, as presented indirectly through springs [2]. In this case, the geotechnical soil model is presented as a tri-component elastoplastic model. Figure 4b shows the pile in its incremental equilibrium configurations: strart, current and next. The final equilibrium state is calculated by applying the updated or total formulation.



a) b)

Figure 4. a) pile model in realistic conditions and a numerical model made of linear finite elements b) start, currently and next incremental configuration of pile [2]

The incremental-iterative method allows load to be increased in an incremental manner and the analysis of static influences in the pile and of soil response. The load is divided into a number of increments, whereby parameter 0 corresponds to the unloaded state of the system, while parameter 1 corresponds to a 100% loaded system where the maximum number of increments is reached. The loading parameter is divided into ten parts through which the distribution of static influences in the pile is monitored. Figure 5 shows the horizontal pile displacement component U_h and the bending moment M for the different loading parameters at specific depths z.

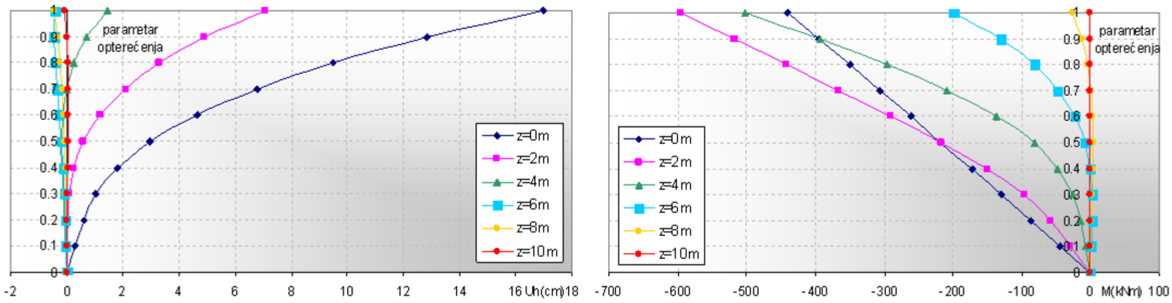


Figure 5. Horizontal displacements of pile U_h and bending moments M of pile on specific depths z, depending on load parameters [2]

The pile under the influence of torque, horizontal and vertical forces in conditions of nonlinear soil behaviour is analyzed using the incremental-iterative procedure. First, the linear static analysis was conducted, without taking into account the nonlinear parameters for the pile and soil. This was followed by the nonlinear analysis, taking into account only the development of material nonlinearity; finally, the analysis was conducted by taking into account the development of geometric nonlinearities as well. Diagrams of horizontal pile deformation (Figure 6a), diagrams of transverse forces along the pile (Figure 6b), and diagrams of bending moment along the pile (Figure 6c) are presented to compare the three different analyzes. When comparing the nonlinear models, results for the horizontal displacement obviously agree to a satisfying degree. Here, at the location where the load was applied, the difference is 10%, while in the lower soil levels this difference decreases. The difference between the horizontal displacements obtained by nonlinear and linear analyses at the point where the load was applied is as much as 450%. This huge difference in displacements is a result of the development of geometric and material nonlinearities that were taken into account in the pile model in conditions of nonlinear soil behaviour. In the linear model, the transverse force reaches its maximum value at the depth of z=2m, while in the non-linear model its maximum value occurs at the depth of z=5m. The absolute maximum values of the transverse force for the model with nonlinear behaviour are higher than those in the linear model. The maximum value of the bending moment obtained by linear analysis occurred at the depth of z=0.5m, while going deeper a reduction of bending moment appears, so that at the depth in excess of half the pile length the moment changes its sign. In nonlinear analysis, for the loading parameter of 1, the sign of the bending moment is constant with the depth.

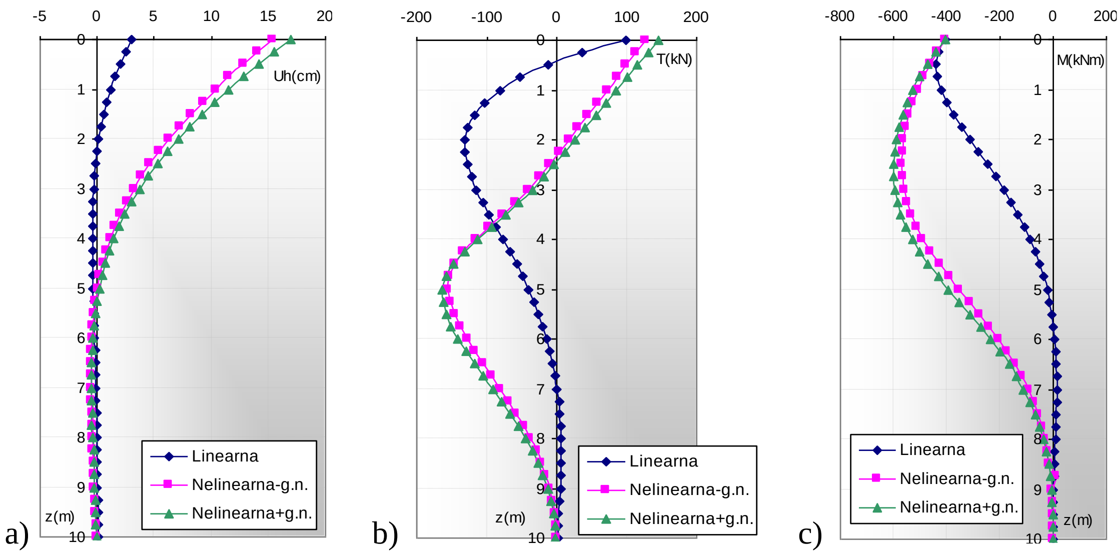


Figure 6. a) horizontal deformation, b) transversal force, c) bending moment [2]

Stability analysis of the SPP system is performed according to the modified geometrically nonlinear analysis of stability [3]. The soil and pile domains are discretized by using 3D solid finite elements, while the geometric modelling of a single pile is based on a rotationally-symmetrical procedure. The medium in which the pile is situated can be of a highly varying nature, from a single through a multi-layered environment with different geotechnical properties. The interface zone is treated numerically by applying special nodal link finite elements, or applying gap elements, where the pressure stiffness is defined while tensile stresses are eliminated. The pile-soil connection is established through the nodes of the solid elements of soil and the solid elements of pile, whereby compatibility of nodes is a key requirement for generating the finite element mesh. The gap element is used for modelling the contact between the model's two points, which is characterized by two states: active (the contact is established - a very high stiffness) and inactive (the contact is not established - a very low stiffness). Applying the gap elements in modelling the transition pile-soil zone, it is necessary to apply the geometrically nonlinear incremental-iterative analysis. The second stage of calculation consists of stability analysis of the soil-pile-structure with the stiffness matrix taken from the previous geometrically nonlinear incremental-iterative analysis. Based on the previously described procedure, as a result of the development of geometric system nonlinearities and nonlinearities in the system gap elements from the first analysis, the corrected elastic stiffness matrix of the system and its corrected geometric stiffness matrix are obtained. Figure 7a shows the vertical cross section of the 3D soil-pile cap model formed of solid finite elements, while Figure 7b shows a detail from the soil-pile connection and the zone of transient finite elements.

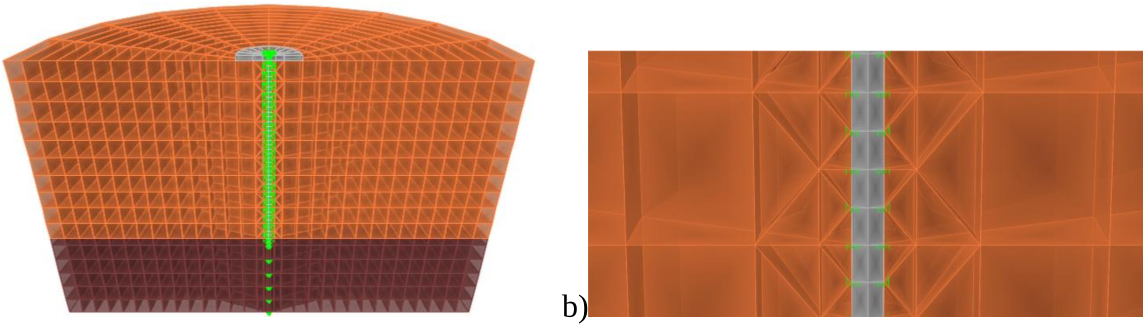


Figure 7. Pile-pile cap-soil 3D model made of solid finite elements: a) vertical cross-section, b) a soil-pile connection detail including the finite element transition zone [3]

Figure 8a shows the characteristic first shape of buckling for L/d=50 and $K_R=10^{-6}$ of a two layer system, where $K_R=E_pI_p/E_sL^4$. The shape of buckled piles is formed from a number of half-waves, instead of a single normal sinusoidal half-wave. This is due to the fact that, in this case, the value of stiffness K_R is very low, so that only 2/3 of the pile is subjected to buckling. Due to the very low value of stiffness K_R, the pile transfers the load through its both base and skin. Figure 8b shows the characteristic first shape of buckling for the L/d=50 and $K_R=10^{-4}$ two layer system. Compared to the previous situation, a pile buckling form has been developed in the shape of a sinusoidal half-wave, so that due to the lower value of stiffness K_R the pile transfers the load mainly through its base instead of its skin. Level of the critical buckling force P_{cr}/P_E and the pile's buckling length coefficient β are identified based on the numerical solutions, as well as using regression analyses for the power function. The derived pile buckling length coefficients for two and single layer as the function of pile length, pile stiffness and soil stiffness are presented in [3].

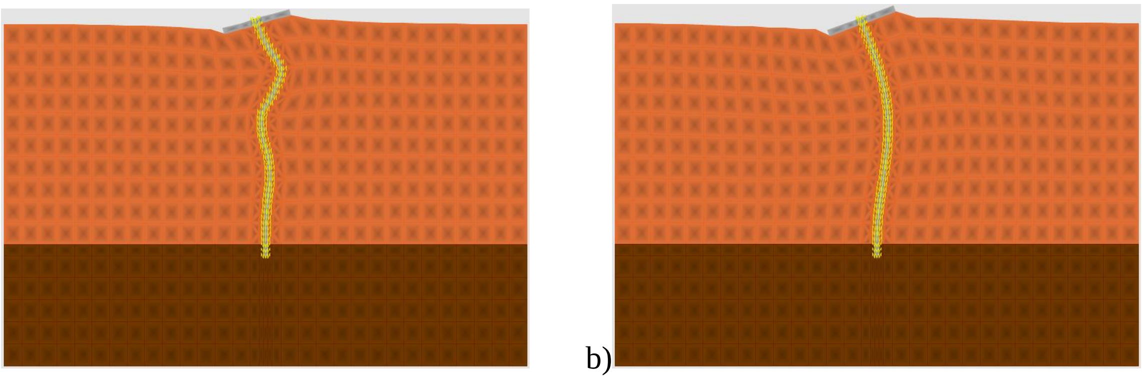


Figure 8. First form of buckling for two-layered system L/d=50: a) $K_R=10^{-6}$, b) $K_R=10^{-4}$ [3]

In the third part contains the procedure of evaluation of seismic performance when the system responds in a nonlinear manner. This evaluation is based on the use of the *nonlinear static pushover analysis* (NSPA) and the *incremental nonlinear dynamic analysis* (INDA) [4]. Beforehand, the input signal to the system was treated both in terms of actual and generated artificial accelerograms, which were further processed by layers of soil down to the bedrock. The pile and pier were modelled using beam finite elements, while effects representing the impact of soil are introduced by applying the principle of implicit modelling of the nonlinear dynamic soil-pile interaction (Figure 9). The beam finite elements are based on the principle of propagation of nonlinear deformation along the element; here, at the cross-section level, specific discretizations to fibres occur. Generally, the cross-section is considered through three sub domains: unconfined concrete, confined concrete and reinforcement steel. Modelling the nonlinear dynamic soil-pile interaction is carried out by applying the constitutive model in lateral pile analysis, which takes into account also the formation of gaps during the cyclic soil deformation.

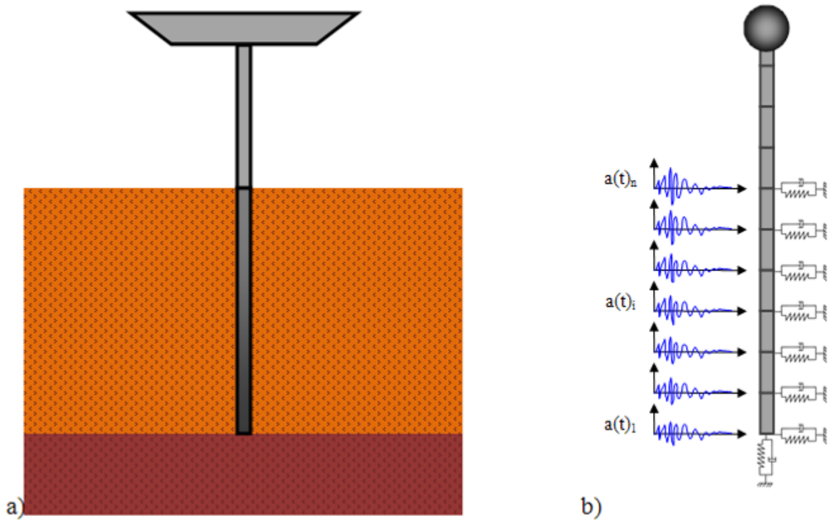


Figure 9. a) the realistic model of pile in the soil, bridge pier and soil, b) the numerical model of pile, bridge pier and implicit modelling of soil action [4]

INDA analyses were post-processed according the global drift *DR* and the corresponding *PGA* values, enabling them to construct the curves *PGA=f(DR)* in the capacitive domain (Figure 10). The limit states of pile-soil system were determined by considering the structural performance levels (SPL): immediate occupancy (IO), collapse prevention (CP) and the global dynamic instability (GI). Fragility curves are constructed based on resolving the regression analyses and the probability theory of log-normal distribution, while reliability curves are constructed based on resolving fragility analysis.

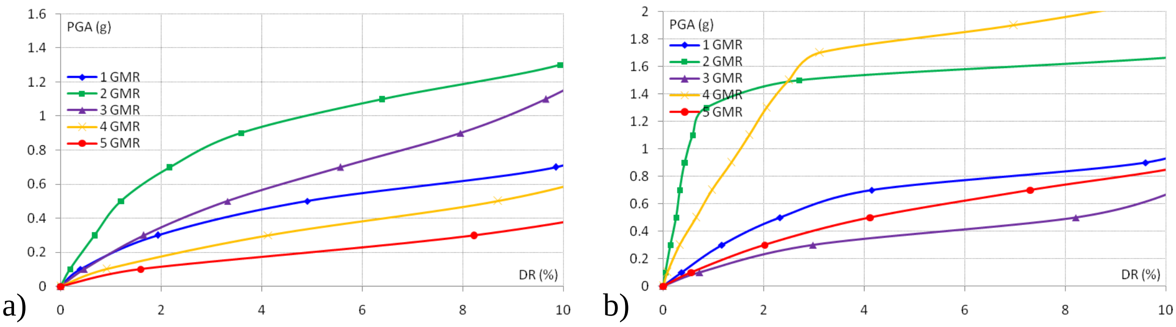


Figure 10. Curve *PGA=f(DR)* for the pier head: a) the first group of accelerograms $t_{s,i}$=2s, $t_{s,f}$=10s, t_{acc}=20s, b) the second group of accelerograms $t_{s,i}$=2s, $t_{s,f}$=15s, t_{acc}=40s [4]

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

REFERENCES

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

[2] Ćosić M. Analiza šipa metodom konačnih elemenata u uslovima nelinearnog ponašanja tla, Geotehnički aspekti građevinarstva, II naučno-stručno savetovanje, Zbornik radova, Ed. R. Folić, Soko Banja, Srbija, 2007, str. 303-310.

[3] Cosic M., Folic B., Sedmak S. Buckling Analysis of 3D Model of Slender Pile in Interaction with Soil Using Finite Element Method, Structural Integrity and Life, Vol. 12, No. 3, 2012, 221-232.

[4] Ćosić M., Folić B., Folić R. Performance-Based Seismic Evaluation of Soil-Pile-Bridge Pier Interaction Using Incremental Nonlinear Dynamic Analysis, 2013, (in a process of publ.)