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PUSHOVER ANALIZA MOSTOVA SA UTICAJEM INTERAKCIJE TLA I KONSTRUKCIJE

Rezime:

Rad se bavi uticajem interakcije tla i konstrukcije na kapacitet pomeranja AB mosta fundiranog na plitkim temeljima. Kapacitet pomeranja u poprečnom pravcu određen je primenom multi modalne pushover analize. Ciljno pomeranje je određeno primenom N2 metode. Uticaj tla je uzet u obzir zadavanjem opruga odgovarajuće krutosti prema NEHPR. Razmatrana su dva tipa tla prema EN1998-1. Analiza je sprovedena primenom programa SAP2000 za slučaj kruto i fleksibilno oslonjene konstrukcije. Data je analiza dobijenih rezultata.

Ključne reči: AB most; pushover analiza; plitak temelj; interakcija tla i konstrukcije.

PUSHOVER ANALYSIS OF BRIDGES INCLUDING SOIL-STRUCTURE INTERACTION EFFECTS

Summary:

This paper deals with the effects of soil-structure interaction (SSI) on the displacement capacity of multi-span RC bridge founded on shallow foundations. Modal pushover analyses are carried out in transverse direction. Target displacement of the monitoring point is calculated using N2 method. The SSI is considered through boundary springs according to NEHPR. Two different soil properties are considered according to EN1998-1. Analyses are carried out using SAP2000, considering fixed- and flexible-base models. The obtained results are discussed.

Key words: RC bridge; pushover analysis; shallow foundation; soil-structure interaction.

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1. INTRODUCTION

The increasing attention in earthquake analysis of bridges in the last years is associated with the collapse of bridges that have occurred in areas with high seismicity. EU has adopted standards EN 1998-1 [1] and EN 1998-2 [2] that recognized the need for reliable estimation of the nonlinear seismic response of bridges. EN 1998-2 prescribes two nonlinear methods for the analysis and design of earthquake resistant bridges: (a) the nonlinear response-history analysis (NRHA) and (b) the nonlinear static pushover analysis (SPA). NRHA including the SSI effects is complex and time consuming, since it requires the discretization of the bridge superstructure and a large volume of the surrounding soil. Therefore, the SPA has been extended to the SSI problem [3] as less demanding procedure. Standard pushover analysis (SPA), developed and used for the seismic assessment of buildings, is performed subjecting the structure to the monotonically increasing lateral forces with invariant distribution until a target displacement is reached. Both, the force distribution and assessment of target displacement are based on the assumption that the response is controlled by a fundamental mode which remains unchanged. The SPA has been extended to regular bridge analysis [3]. For irregular bridges, multi-modal pushover analysis (MPA) [4] and extended multi-modal pushover (EMPA) have been developed [5]. The detailed investigation of the effectiveness of both approaches in bridge analysis is presented in [5]. The main conclusion of this investigation is that SPA methods are accurate enough for regular bridge configurations, where the effective modal mass of the fundamental mode is at least 80% of the total mass, while the advantages of the MPA become evident when irregular bridges are considered.

In the conventional bridge analysis, SSI is usually neglected due to the prevailing view that the SSI has beneficial effect on the seismic response of structures. Mylonakis and Gazetas [6] have shown that this view is an oversimplification which sometimes may lead to the unsafe design of bridges. However, very limited investigations have focused on the soil-structure interaction effects on pushover analysis of bridges.

The purpose of this study is to investigate the effects of SSI on the displacement capacity of a typical bridge structure founded on shallow foundations. Modal pushover analyses are carried out in the transverse direction, taking into account three different scenarios related to different soil properties below the foundations: (1) fixed-base structure, (2) structure founded on soft rock (soil type B according to EN1998-1) or (3) firm send and limestone (soil type C). SSI is considered through boundary springs using the appropriate dynamic foundation stiffness. The peak ground acceleration is $a_g = 0.30$ g.

Results of the multi-modal pushover analysis (MPA) in transverse direction are presented and the influence of SSI and higher modes on transverse displacements, base shear and plastic hinge formation are discussed.

2. MULTI MODAL PUSHOVER ANALYSIS

The multi-modal pushover analysis (MPA) is proposed by Chopra and Goel (2002) in order to take into account higher modes of vibration in determining the displacement capacity of high raise buildings. Paraskeva et al. [4] extended MPA to seismic assessment of bridges.

According to EN 1998-2, only inelastic behavior is allowed in the piers. The pushover analyses are carried out separately for each significant mode n, and the contributions from individual modes to the calculated response quantities (displacements, drifts, etc.) are combined using an appropriate combination rule (SRSS). The basic steps are summarized below:

1. Compute the natural periods and mode shapes of the bridge. Select n natural periods T_n and mode shapes ϕ_n of the bridge, to account for 80-90% of the effective modal mass in the considered, lateral, direction.

2. Chose the appropriate monitoring point (MP) for tracking the displacement in the considered direction and calculating the inelastic displacement demands. In the paper, the MP is taken as the resultant of modal load pattern of the first transverse mode [5]:

$$x^* = \left(\sum_{j=1}^{N} x_j m_j \phi_{jn} \right) / \left(\sum_{j=1}^{N} m_j \phi_{jn} \right) \tag{1}$$

   where:

   x_j is the distance of the j^{th} mass m_j from a selected point of the MDOF system,
   ϕ_{jn} is the value of ϕ_n at the location of m_j.

3. Carry out pushover analysis for each mode n by applying the modal force distribution s_n*=$\mathbf{m}\phi_n$ (where **m** is the mass matrix of the structure), in order to obtain modal pushover curves V_{bn} - u_{rn} (base shear vs. displacement of the monitoring point).

4. Idealize modal pushover curves for each mode n by bilinear curves using the equal energy absorption rule.

5. Convert idealized pushover curve of the MDOF system to the capacity curve of an equivalent SDOF system, in *ADRS* format (S_{an}-S_{dn}), using modal conversion parameters:

$$S_{an} = \frac{V_{bn}}{M_n^*}, \qquad S_{dn} = \frac{u_{rn}}{\Gamma_n \phi_{rn}} \tag{2}$$

   where, S_{an} is spectral acceleration, S_{dn} is spectral displacement, ϕ_{rn} is the value of the ϕ_n at the selected monitoring point, $M_n^* = L_n \Gamma_n$ is the effective modal mass, $L_n = \phi_n^T \mathbf{m}\mathbf{1}$, $\Gamma_n = L_n / M_n$ is the modal participation factor and $M_n = \phi_n^T \mathbf{m} \phi_n$ is the generalized mass for mode n.

6. From the capacity curve obtained in (5), extract the spectral displacement and spectral acceleration at yielding point ($S_{dy,n}$ and $S_{ay,n}$, respectively), for each mode n.

7. Calculate elastic vibration period of the equivalent SDOF system T_n^* for each mode n:

$$T_n^* = 2\pi \sqrt{\frac{S_{dy,n}}{S_{ay,n}}} = 2\pi \sqrt{\frac{M_n^* S_{dy,n}}{V_{by,n}}} \tag{3}$$

where $V_{by,n}$ is base shear at yielding point.

8. Transform the demand spectrum into the ADRS format $S_a(g)$-S_d, where:

$$S_d = \frac{T^2}{(2\pi)^2} S_a(g) \tag{4}$$

9. Calculate the spectral target displacement, $S_{de,n}$, of the equivalent SDOF system with known period T_n^* and infinite elastic behavior using the following equation:

$$S_{de,n} = S_{ae}(T_n^*)\left(\frac{T_n^*}{2\pi}\right)^2 \tag{5}$$

where $S_{ae}(T_n^*)$ is spectral acceleration obtained for period T_n^*.

10. Calculate the inelastic displacement demand $S_{d,n}$ of the equivalent SDOF system in the n^{th} mode using the procedure based on the N2 method adopted in EN 1998-1. For:

 a) $T_n^* \geq T_C$ (medium and long period range)

$$S_{d,n} = S_{de,n} \tag{6}$$

 b) $T_n^* < T_C$ (short period range)

$$S_{d,n} = \frac{S_{de,n}}{q_u}\left[1 + (q_u - 1)\frac{T_C}{T_n^*}\right] \geq S_{de,n} \tag{7}$$

where q_u is the ratio between the spectral acceleration (base force) in the equivalent SDOF with unlimited elastic behavior $S_{ae}(T_n^*)$ and spectral acceleration (base force) at yielding point $S_{ay,n}$, in the n^{th} mode:

$$q_u = \frac{S_{ae}(T_n^*)}{S_{ay,n}} \tag{8}$$

11. Convert the inelastic displacement demands $S_{d,n}$ at the monitoring point and corresponding base shears of the actual bridge, for each individual mode n, from the inelastic SDOF system to the MDOF system, according to:

$$\begin{aligned} u_{rn} &= S_{d,n}\Gamma_n\phi_n \\ V_{rn} &= S_{a,n}\Gamma_n L_n = S_{a,n}M_n^* \end{aligned} \qquad n = 1,2,\ldots,N \tag{9}$$

where u_{rn} is the modal displacement demand along the actual bridge deck, V_{rn} is the base shear, $S_{d,n}$ is the spectral displacement demand of the SDOF system at the location of monitoring point and $S_{a,n}$ is the spectral acceleration of the SDOF system at the location of the monitoring point.

12. Total value of any response quantity r is determined by combining the peak modal responses r_n from individual modes using SRSS combination rule $r = \left(\sum_i^n r_i\right)^{1/2}$.

3. DESCRIPTION OF STUDIED BRIDGE STRUCTURE

The Nišava Bridge is a seven-span continuous bridge structure of 232.2m total length (21.6+5×37.8+21.6), curved-in-plan, with radius of curvature R=540m and longitudinal slope of 1.82%, (Fig. 1a). The idealized bridge deck cross section is presented in Fig. 1b. The deck is supported by six RC piers P_1-P_6 of unequal clear heights: P_1=17.14m, P_2=17.51m, P_3=16.82m, P_4=16.11m, P_5=16.07m, P_6=9.95m, and equal rectangular hollow cross section. The piers and are founded on prismatic footings. The inner piers (P_2-P_5) are monolithically connected to the deck, while for the outer piers (P_1 and P_6) a bearing type connection is adopted allowing the movement only in the tangential direction. The rotation about longitudinal and transverse axis is unrestrained in all connections. The deck is resting on piers over "Neotopf" bearings.

The bridge was designed according to Serbian regulations as an object of high importance, for VII seismic zone (PGA of 0.1g) and soil type I (equivalent to the soil type B of EN1998-1). The adopted damping in transverse direction is ξ_i=7%. The reinforcement detailing according to Serbian regulations leads to the high amount of confinement of the joints [7], thus the structure is designed as a ductile, allowing plastic hinge formation at the end of the piers. The adopted reinforcement is shown in Fig. 1c.

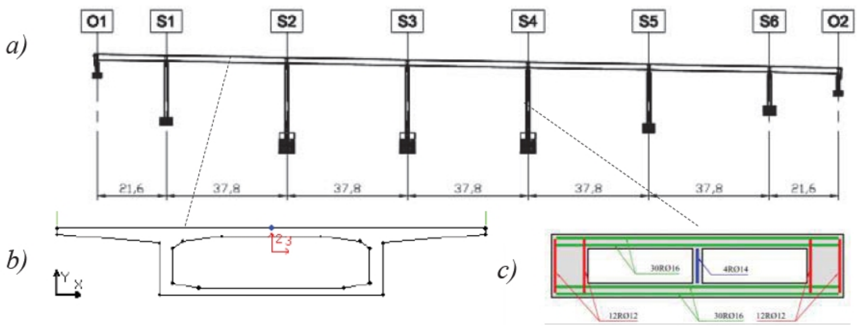

Figure 1 - a) Bridge layout; b) Idealized bridge deck cross section; c) pier cross section with adopted reinforcement

4. NUMERICAL MODEL

The considered structure is modelled in SAP2000. The material properties of RC members and steel reinforcement are given in Table 1. The deck is supposed to remain in the elastic range. The torsional rigidity is reduced to 50%. The reinforcement of the piers is assigned using the Section Designer tool of SAP2000. Bending rigidity of the piers is reduced to 50%.

Mass of the structure is generated from the self-weight (Q_G = 51441.2kN), additional dead loadings ($Q_{\Delta G}$ = 11038.8kN) and traffic loading (Q_T = 2104.6kN), according to EN 1998-2.

To investigate the influence of SSI, three different scenarios are analysed: (1) fixed-base structure, (2) flexible-base structure on a soil type B and (3) flexible-base structure on a soil type C. The soil properties are provided in Table 2. Abutments A1-A2 are modelled as nonlinear springs (K_z = 1800000kN/m, K_{long} = 4644kN/m) and appropriate gap elements (K_{gap} = 1000kN/m, d_{gap} = 5cm).

Table 1 – Material properties of RC members

Element	Material	E [GPa]	ν	γ [kN/m³]	f_c' [MPa]	f_y [MPa]	f_u [MPa]
Beam	C30/37	31.5	0.2	25.0	23.00	-	-
Pier	C35/45	35.0	0.2	25.0	27.75	-	-
Rebar	RA 400/500	210.0	0.3	78.5	-	400	500

Increase of the shear deformation during the earthquake decreases shear modulus and increases damping in the soil. The ratio of effective shear modulus G and shear modulus G_0, associated with small strains is calculated according to [8] for the considered soil types and a_g=0.30g and presented in Table 3. The designed spectral accelerations, S_{DS} = $S_e(T)=2.5a_g(g)S\,\eta$, are taken according to EN1998-1 [1]. The change of damping due to foundation-soil interaction is calculated using FEMA 440 [9]. The result shows that there is neither increase of damping and therefore, nor spectrum reduction.

Table 2 – Soil properties in three considered scenarios

Scenario	Soil	Soil type	G_0 [kN/m²]	ρ [t/m³]	ν [-]
1	Rock	B	-	-	-
2	Soft rock	B	300000	2.20	0.20
3	Firm sand and limestone	C	90000	2.00	0.35

Table 3 – G/G_0 ratios for various site classes [9]

Soil	Site class	S_{DS} / 2.5 [10]			Considered bridge structure				
		≤0.1	0.4	≥0.8	η	S	S_{DS} / 2.5	G/G_0	G [kN/m²]
Soft rock	B	1.00	0.95	0.90	1	1.20	0.360g	0.957	287100
Clay	C	0.95	0.75	0.60	1	1.15	0.345g	0.787	70830

Foundation impedance is the complex function which depends on the frequency of vibrations, geometry of the soil-foundation contact surface and the soil properties. The components of the dynamic stiffness are calculated according to Pais and Kausel [8] as:

$$K_{i,d} = \alpha_i K_i \quad \wedge \quad K_{ii,d} = \alpha_i K_i, \quad i = x, y, z \tag{9}$$

where:

K_i, $i=x,y,z$ are translational static stiffnesses of the foundation,

K_{ii}, i=x,y,z are rotational static stiffnesses of the foundation,
α_i and α_{ii}, i=x,y,z are frequency-dependent dynamic stiffness coefficients.

Having in mind that the earthquake loading is generally of a low frequency, for the practical purpose, without the loss of accuracy, α_{ij}=1 can be adopted. The static stiffnesses of the foundation, which depend on the effective shear modulus G, Poisson's ratio ν and foundation dimensions $B\times L$ [8], are provided in Table 4.

Table 4 – Dimensions and stiffnesses of foundation for soil types B and C

Soil	$L\times B$	G	ν	K_x	K_y	K_z	K_{xx}	K_{yy}	K_{zz}
	[m]	[MPa]	[-]	[MN/m]			[MNm/m]		
B	3.75×2.00	287.1	0.20	4028	4251	4712	19516	49169	54843
C	5.00×3.00	70.8	0.35	1529	1598	2009	18038	38173	36161

5. RESULTS OF THE MULTIMODAL PUSHOVER ANALYSIS

The bridge is assessed using the MPA [4] in transverse direction. The properties of the elastic response spectra are as follows: Type 1, a_g = 0.30 g, soil types B and C according to EN1998 [1] (as shown in Fig. 3). The required dynamic properties (periods T_n and modes ϕ_n) are determined using the standard eigenvalue analyses in SAP2000. Mode shapes are illustrated in Fig. 2. In the MPA, only transverse modes which modal mass participation factors are higher than 2% are considered [5]. The lowest obtained mode is the longitudinal one in all scenarios. Five transverse modes, which account for the 84.5-87.8% of the total mass in the transverse direction, are considered. The slight non-symmetry of mode shapes originates from the different height of columns, indicates that the stiffness of the substructure is not equally distributed along the bridge.

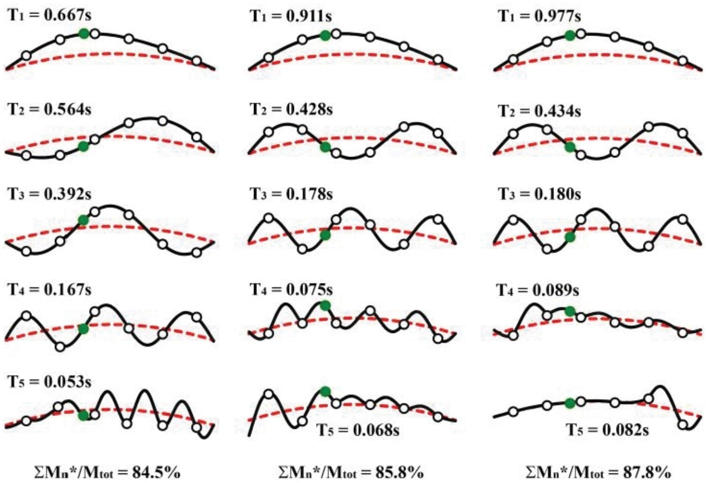

Figure 2 - Mode shapes ϕ_i in transverse direction for the fixed-base model (left), soil B (middle) and soil C (right); white bullets=pier locations P_i, green bullet=monitoring point MP

The pushover analyses are carried out separately for each significant mode. The inelastic behaviour in the piers is modelled using the software built-in P-M2-M3 plastic hinges, according to FEMA 356 [10]. The gravity loads are applied before each pushover analysis, and P-Δ effects are included.

Fig. 3 illustrates the procedure for obtaining the inelastic displacement demands for three considered scenarios. Two considered elastic response spectra are plotted, along with the capacity curves for the first three transverse modes (4th and 5th transverse modes are omitted for the sake of simplicity).

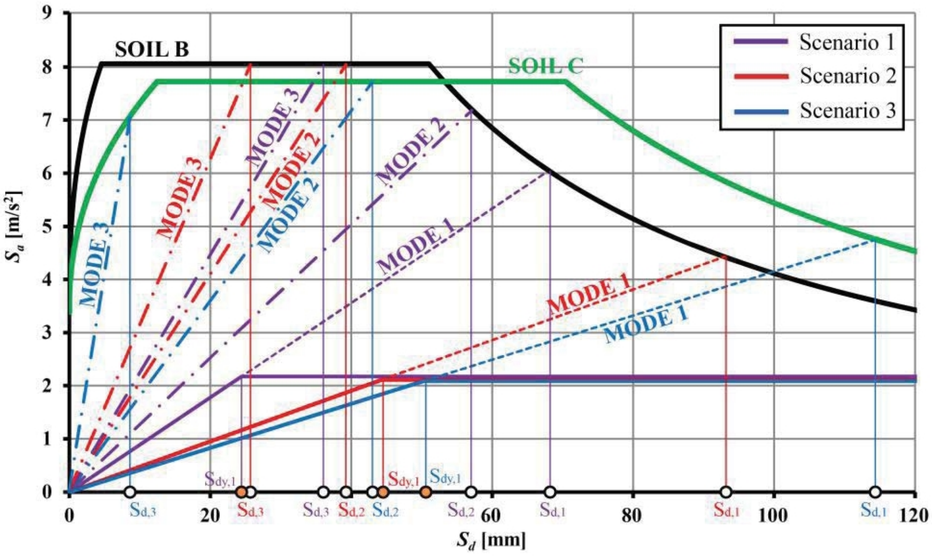

Figure 3 - Black and green lines: elastic response spectra for soils B and C (a_g=0.30g, ξ_i=7%). Capacity curves for three considered scenarios, corresponding to the first 3 transverse modes

The capacity curves corresponding to higher transverse modes are linear, which means that ductility demand $\mu = 1$, i.e. the structure remains elastic in the higher modes. In addition, for the first transverse mode in all three scenarios, $T_1^* > T_C$ and therefore $S_{d,1} = S_{de,1}$, according to [11]. Thus only the elastic spectra have been plotted in Fig. 3.

Fig. 4a illustrates the total transverse inelastic displacements u_r for three considered scenarios (obtained using the SRSS combination), in comparison with the displacement u_1 obtained only by considering the first transverse mode. It is evident that the first transverse mode contributed to the final response significantly in all scenarios. Target displacement in the monitoring point MP for three considered scenarios are: $u_{MP,1}$ = 133.36mm, $u_{MP,2}$=173.48mm and $u_{MP,3}$=212.75mm (obtained using SRSS combination). The corresponding ductility demands are μ_1=2.767, μ_2=2.099 and μ_3=2.266. The total base shear forces are V_{b1} = 25624kN, V_{b2} = 25538kN and V_{b3} = 25439kN. The base shear modal contributions are presented in Fig. 5.

Finally, for the determination of plastic hinge distribution along the structure, the bridge is pushed using the modal force distribution of the first transverse mode (which have the highest contribution to the overall response), to the value of peak displacement $u_{MP,i}$ in each scenario *i*. The formation of plastic hinges is shown in Fig. 4b.

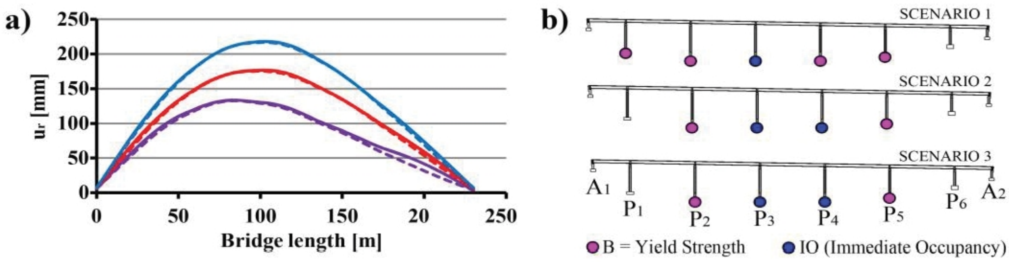

Figure 4 - a) Transverse inelastic displacements u_r for three considered scenarios (solid lines – SRSS combination, dashed lines –first transverse mode), b) plastic hinge distribution along the structure for the scenarios 1-3

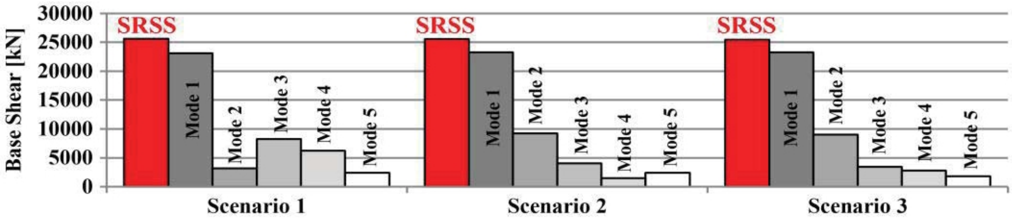

Figure 5 - Base shear modal contributions for the scenarios 1-3 and the corresponding values of base shear for the SRSS combination

6. CONCLUSIONS

In the paper, the multimodal pushover analysis including the effects of SSI of the RC Nišava Bridge has been presented. The seismic demands of the structure have been calculated, considering five dominant transverse modes. The overall bridge performance showed the following:

1. Neither local nor global failure occurred, even under seismic actions that three times exceed the design level,
2. A significant overstrength of the bridge is found due to the partial safety factors used in the design, minimum reinforcement due to the Serbian code requirements and application of conservative code provisions related to the longitudinal bar buckling,
3. The fundamental transverse mode shape contributes to the final response significantly, both for the transverse displacements and base shear,
4. The SSI increased target displacement of the monitoring point from 30% (scenario 2) to 59.9% (scenario 3) in comparison with the fixed-base.
5. The SSI reduced the ductility demand, from 24% in scenario 2 to 18% in scenario 3.
6. The effect of SSI on the total base shear is negligible.
7. In the fixed-base model, plastic hinges have been formed in piers P1-P5, while the SSI leaded to the plastic hinge formation only in piers P2-P5.

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