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Multidisciplinary approach to the assessment of seismic performances and rehabilitation of bridges: nonlinear analyses, probability theory and optimization theory

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Abstract

The paper presents a multidisciplinary approach to the assessment of seismic performances based on the *Performance-Based Earthquake Engineering* (PBEE), taking into account the multi-criteria optimization theory in analyzing the priority methods for bridge rehabilitation/strengthening. One bridge model was subjected to nonlinear static pushover analyses (NSPA), target displacement analyses using the spectrum capacity method (CSM), vulnerability analyses, and reliability analyses, while for a damaged bridge, in addition to be considered using the above methods, was also analyzed using the VIKOR method of multi-criteria optimization. Seismic performances were determined based on monitoring the system's plastification and analyzing the relevant parameters for the level of target displacement, such as target displacement, total shear force, spectral displacement, spectral acceleration, vibration period, damping and ductility. The phases of damage were considered using the probabilistic analysis of vulnerability and reliability: slight, moderate, extensive and complete, as a function of system ductility.

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Keywords: earthquake; nonlinear analyses; performances; damage; vulnerability and reliability analysis; multi-criteria optimization

1. Introduction

Designing bridges in everyday engineering practice is based on the application of national or international technical codes. More often than not, existing regulations do not define specific problems that may occur during designing of

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bridges with enough detail. Existing regulations, in most cases, are not in compliance with modern regulative, such as EN 1998-2:2005 [1], AASHTO [2, 3] or CALTRANS [4]. On the other hand, even the application of modern regulations has certain limitations in analyzing structure systems on a higher level, for example in modern earthquake engineering, which requires employing of experts for these very sensitive issues. Assessing a bridge damaged by earthquake presents a complex engineering, technological and economic problem to be solved by analyzing a number of parameters in the process of making an optimum decision. Some of the key aspects of solving this problem are related to the creation of adequate numerical models of the damaged bridge structure and application of nonlinear methods, taking into account modern design rules and regulations. Within the PBEE methodology, consideration of non-linear behavior of bridges is based on, among other things, the application of non-linear static pushover analysis (NSPA). The basis of bridge calculations using non-linear seismic analysis and the formulating of Finite Element Method (FEM) are given in [5]. Application of NSPA in solving of problems related to bridges founded on piles is shown in [6, 7, 8], whereas a general approach to bridge analysis using NSPA is shown in [9, 10, 11].

However, in addition to the engineering aspects of this complex issue, the economic aspects have also particular importance; they are reflected through the effects of intervention costs, the choice of the method and analysis of the period of time required for interventions on the bridge structure. The solution to these problems can be found in the application of the MCDM approach (*Multi-Criteria Decision Making*). Today, there are a considerable number of multi-criteria optimization methods in which the solution for a multi-criteria problem is obtained by choosing the best alternative from a set of predefined alternatives (multi-attribute decision making) or programming the best alternative (multi-objective decision making).

In this paper, results of classic design method and of the *Performance-Based Seismic Design* method were compared for a given structure - an overpass. The overpass was designed according to Serbian regulations (SRP) and built at the section of E-75 Highway Novi Sad - Belgrade, as part of the Kovilj loop. Given that the problem of strengthening the structure of RC bridges can be taken into consideration and presented as a discrete mathematical problem, this research is based on the VIKOR method.

2. Design of overpass by SRP regulations

The overpass was designed according to Serbian regulations [12], which are older than the modern regulations for design and analysis of bridge structures. Longitudinal overpass girders, which were prefabricated and monolithized, were modelled as surface girders. Along the transverse direction, the structural system of the overpass was modelled as a frame static system, which included the piles as well, and the soil interaction was introduced by using elastic springs. For these springs, equivalent stiffness was defined for both pressure and tension. The constitutive model of material behaviour is linear-elastic, and linear static analysis was used for determining cross-section forces. For determining of seismic effects, the equivalent static method was used, whereas sizing was performed using stress-strain work diagrams according to PBAB 87 [13].

3. Numerical modelling of a non-linear soil-pile interaction

Numerical model of the overpass used for non-linear analysis was taken into consideration by applying the decomposition of overpass structure into sub-groups, which were then considered in detail from the point of possibility of optimal modelling as a plane model. From the aspect of seismic effects and system deformation, it is of particular interest to analyze the overpass structure along the transverse direction, hence only this direction was analyzed during this research. Figure 1 shows a 3D model of the overpass created as a solid using an especially emphasized transverse frame of the overpass. The overpass consists of two spans of 24m each, whereas the transverse cross-section is made of 9 hollowed-out prefabricated elements with dimensions of 95/100 cm, with a span of 16.3m. Central columns are of a circular cross-section with Ø90 cm and length of 5.9m, connected to piles with Ø130cm and length of 14.2m via RC cubes with side length of 130cm. Total width of the overpass is 16.33m, whereas the plate in the central part has a height of 115cm and width of 800cm. The beam was made of MB40 concrete, while MB30 was used for columns and piles. Columns, piles and beams were modeled using linear finite elements. Plasticization of the system takes places at the locations of local plasticization - plastic hinges, therefore 74 plastic hinges were applied [14]. For the beam, plastic hinges were defined at the places where plasticization is enabled via bending moments, whereas in case of columns, plastic hinges were defined on locations were plasticization occurred through the interaction of normal forces and bending moments. Plastic hinges of the beam and columns were applied on ends, while in case of piles, they were applied uniformly along the length. Soil-pile interaction is covered by applying link elements, hence a sophisticated hysteresis interaction model with incorporated contact elements (gap element) for simulation of reactions to pressure was applied. Backbone curves of a hysteresis model were defined as multi-linear *p-y* curves according to [15], in such a way that the tangent stiffness is determined from discrete values of force and displacements, which change along soil depth. In addition, when these curves were generated, the effects of groundwater level, defined according to the geotechnical profile of soil, were taken into account.

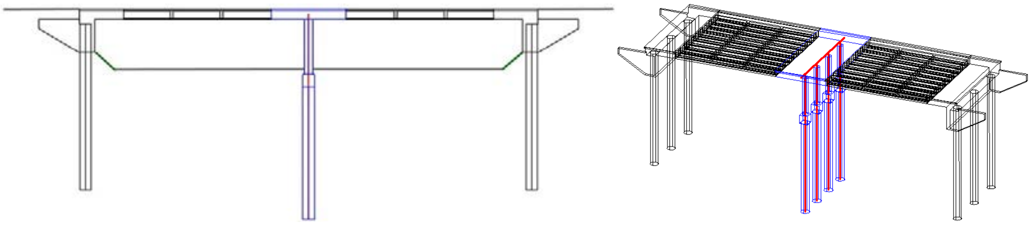

Fig. 1. A 3D model of the overpass created as a solid model with especially emphasized transverse frame of interest for this consideration

4. NSPA analysis of an overpass in the transverse direction

NSPA is performed for a non-linear model of structure behaviour, whereas seismic effects are generated and applied to the structure in form of seismic forces. In this paper, three procedures for generating of seismic forces were applied: as seismic load; based on inertial seismic forces (accel) and according to the first mode. Material non-linearity is introduced via development of elastic-plastic deformation in plastic hinges, whereas as geometric non-linearity includes *P-Δ* effects and large displacements. Non-linear static analysis of effects of vertical load was performed first, where the stiffness matrix of the system obtained at the end of this analysis is used as the initial matrix for NSPA. Load from the non-linear static analysis of effects of vertical loads is transferred and used in NSPA, so that the overpass analysis in realistic seismic conditions can be simulated. Maximum available structure displacement is determined using an iterative procedure, by gradually increasing displacement and controlling the number of increments realized. The shapes of all NSPA pushover curves are nearly identical, the only difference appearing in relative values of total shear force. Highest relative values of the total shear force were obtained for seismic force generation using inertial forces. This was caused by the influence of both the first mode and higher modes of a total system response, especially the third, fourth and eleventh. Development of plastic hinges along the incremental stages via performance states: A, B, IO, LS, CP, C, D and E for applied seismic forces, inertial forces, according to first mode, are shown in figure 2.

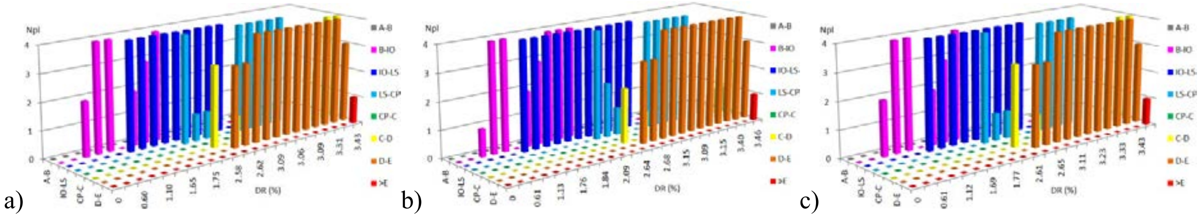

Fig. 2. Development of plastic hinges along the incremental stages via performance states: A, B, IO, LS, CP, C, D and E for applied: a) seismic forces (load), b) inerital forces (accel), c) according to the I mode

Values of global drifts and achieved performance states are given on the abscissa, whereas the number of plastic hinges N_{pl} which was achieved in specific inter-performance states A-B, B-IO, IO-LS, LS-CP, CP-C, D-E and >E are given on the ordinate. The number of plastic hinges and distribution of achieved performance states in case of applied seismic forces and according to the first mode is nearly identical, whereas in case of seismic forces generated as inertial, they are somewhat different. This difference is reflected in the number of formed plastic hinges for specific values of global drifts.

The second part of NSPA includes the determining of the target displacement analysis of the overpass, hence following methods were used for the purpose of this research: *Capacity Spectrum Method* - CSM, according to ATC 40 [16] and *Equivalent Linearization Method* - ELM, according to FEMA 440 [17]. Parameter analysis of target displacement was performed for the applied seismic forces and for inertial seismic forces as pushover load, since modeling of seismic forces according to the first mode resulted in solutions extremely similar to those obtained by directly applied seismic forces. Parameters C_a and C_v were considered in the interval of 0,1 to 1, and levels of target displacements were determined for these values using CSM and ELM methods. A total of 400 target displacement analyses were carried out for target displacement levels calculated in this way by using parameter analysis. The following have been taken into consideration and shown: displacement D_t (figure 3), total shear force V_t (figure 4), spectral displacement $S_{d,t}$ (figure 5), spectral acceleration $S_{a,t}$ (figure 6), vibration period T_t (figure 7), damping ξ_t (figure 8) and ductility μ_t (figure 9) for applied seismic forces as pushover load.

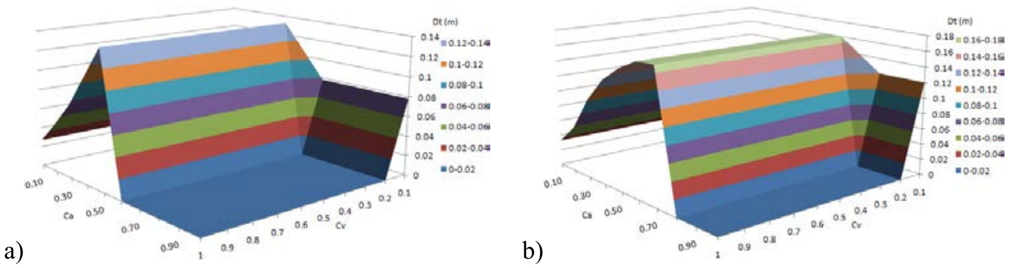

Fig. 3. Values of displacement D_t obtained from C_a and C_v parameter analysis of target displacement for applied seismic forces as pushover load: a) CSM method, b) ELM method

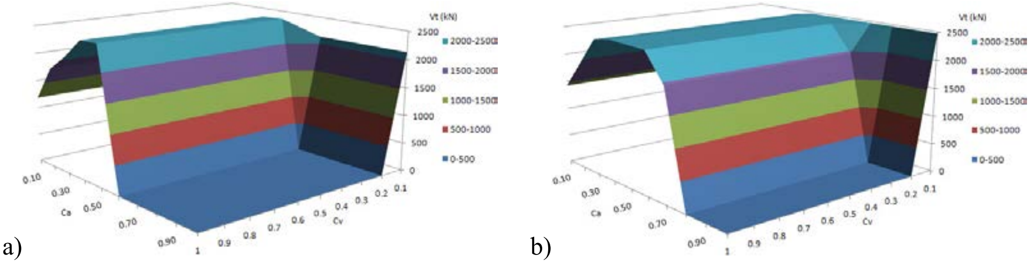

Fig. 4. Values of shear force V_t obtained from C_a and C_v parameter analysis of target displacement for applied seismic forces as pushover load: a) CSM method, b) ELM method

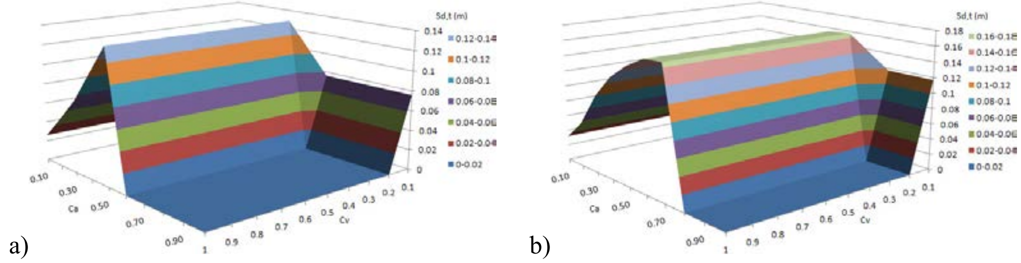

Fig. 5. Values of spectral displacement $S_{d,t}$ obtained from C_a and C_v parameter analysis of target displacement for applied seismic forces as pushover load: a) CSM method, b) ELM method

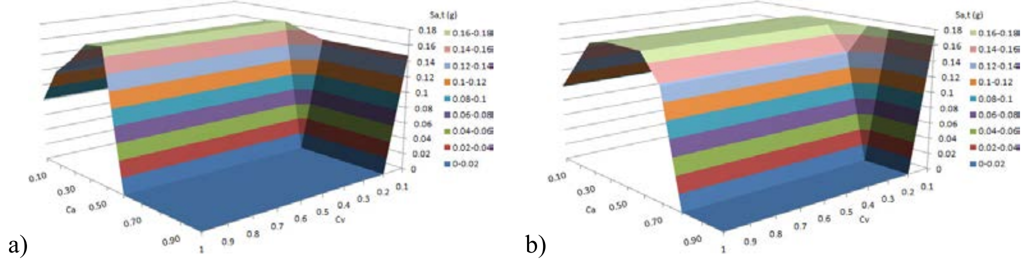

Fig. 6. Values of spectral acceleration $S_{a,t}$ obtained from C_a and C_v parameter analysis of target displacement for applied seismic forces as pushover load: a) CSM method, b) ELM method

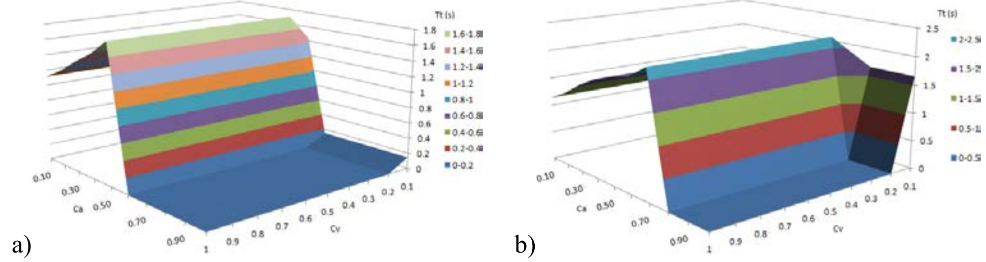

Fig. 7. Values of vibration period T_t obtained from C_a and C_v parameter analysis of target displacement for applied seismic forces as pushover load: a) CSM method, b) ELM method

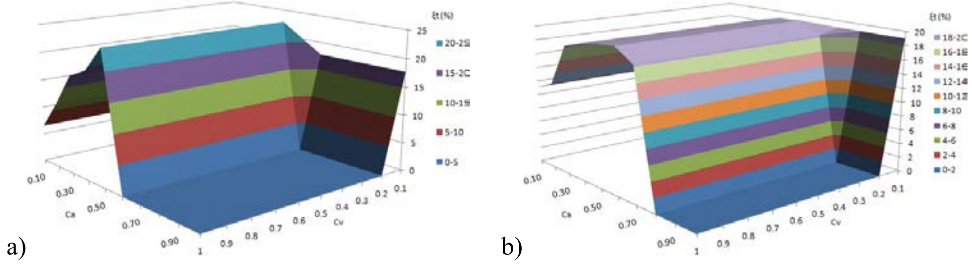

Fig. 8. Values of damping ξ_t obtained from C_a and C_v parameter analysis of target displacement for applied seismic forces as pushover load: a) CSM method, b) ELM method.

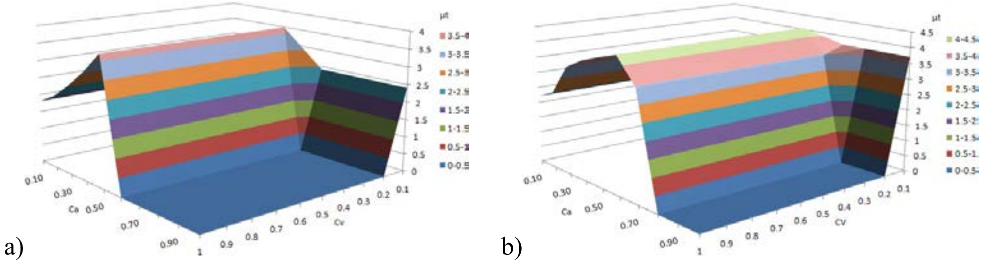

Fig. 9. Values of ductility μ_t obtained from C_a and C_v parameter analysis of target displacement for applied seismic forces as pushover load: a) CSM method, b) ELM method

Applying ELM method, in comparison to CSM method, resulted in higher values for all considered parameters, except damping. Parameter C_a and C_v analysis of target displacement realized the following highest values:

displacement D_t≈0.16m, total shear force V_t≈2500kN, spectral displacement $S_{d,t,max}$≈0.158m, spectral acceleration $S_{a,t,max}$≈0.17g, vibration period $T_{t,max}$≈2.1s, damping $\xi_{t,max}$≈22% and ductility $\mu_{t,max}$≈4. Level of damping, determined using the target displacement analysis, is significantly higher than the initial value. The reason for this is that both viscous and hysteresis damping takes part in this damping. Ductility, determined by using target displacement analysis, is relatively high, since initiation of system plasticization is realized already during the second step of NSPA.

5. Vulnerability and reliability of the overpass in transverse direction

Probabilistic concept of performance analysis of the overpass takes into account the level of overpass structure damage as a function of probability of occurrence, i.e. a function of vulnerability probability (fragility). Damage level were considered according to HAZUS [18]: slight, moderate, extensive and complete. These damage levels were defined as a function of system ductility μ, in such a way that $1<\mu<2$ corresponds to slight damage, $2<\mu<4$ corresponds to moderate damage, $4<\mu<7$ corresponds to extensive damage and $\mu>7$ corresponds to complete damage. A relation between discrete values of spectral acceleration S_a and ductility μ was previously established by using a regression analysis for a linear function. Creating of vulnerability curves was performed in relation to S_a measure of intensity by applying log-normal distribution, taking into account probability density function and cumulative distribution function. Figure 10a shows vulnerability curves for the overpass structure along the transverse direction. Probabilities of initiating P>0 of corresponding damage levels of the overpass (along the transverse direction) are: S_a=0.03g for slight, S_a=0.06g for moderate, S_a=0.13g for extensive and S_a=0.16g for complete damage. Probabilities of reaching P=1 of the corresponding levels of overpass damage (for transverse direction) are: S_a=0.09g for slight, S_a=0.19g for moderate, S_a=0.34g for extensive and S_a>1g for complete damage. System performance assessment, in addition, was performed using reliability state analysis of the overpass. This analysis is based on the previous vulnerability analysis. Figure 10b shows the reliability curves for the overpass structure along the transverse direction. Negative value of coefficient R indicates the possibility of failure and system unreliability, whereas positive value of this coefficient indicates failure probability close to zero, i.e. significant system reliability. When the value of the coefficient is R≈6, system reliability is ≈100%, whereas in case of R≈0, system failure probability is P=50%. Overpass reliability (along the transverse direction) for P>50% is at S_a≤0.05g for slight damage level, S_a≤0.12g for moderate damage level, S_a≤0.23g for extensive and S_a≤0.62g for complete damage level.

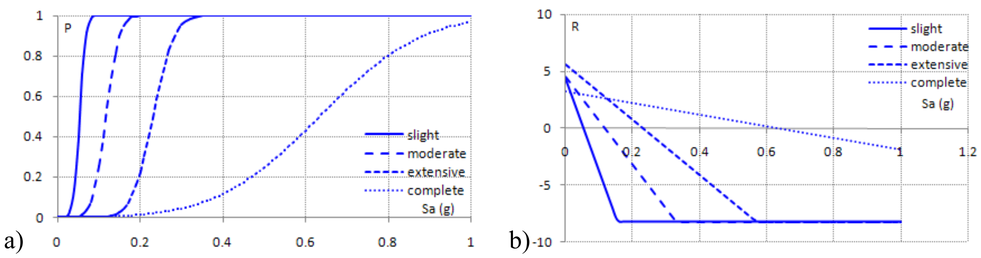

Fig. 10. a) vulnerability curves, b) reliability curves

6. VIKOR method of decision making in the analysis of bridge rehabilitation/strengthening

The VIKOR method was developed for the purpose of multi-criteria decision making in strategic projects [19], by first identifying the best (compromise) solution in a multi-criteria sense from a set of J permissible alternatives evaluated based on a set of n criteria functions. This method requires knowing the values of all criterion functions for all alternatives in the form of a matrix $\|f_{ij}\| n\text{x}J$, where f_{ij} denotes the value of i-th criterion function for the j-th alternative. The compromise solution $F^c=(f_1^c,\ldots f_n^c)$ is the permissible solution that is the closest to the ideal solution F^* (best values of criteria functions). Here, compromise implies an agreement reached by mutual trade-offs, represented by $\Delta f_i=f_i^*-f_i^c$, $i=1,\ldots n$. The VIKOR method is carried out in several steps:

- determining the ideal point based on the values of criteria functions,

- transformation of various criteria functions (different measures of value),
- assigning criteria weights w_i, i=1,...n, representing the relative importance of the criterion that is based on the preference of decision-maker,
- determining S_j (weighted and normalized *Manhattan* distance), R_j (weighted and normalized *Chebyshev* distance), Q_j, j,...J.

The priority list of methods of bridge rehabilitation/strengthening was defined and decided based on the following activities: forming a list of methods of rehabilitation/strengthening, forming criteria for multi-criteria optimization, numerical procedure of multi-criteria optimization based on the VIKOR method, evaluating the solutions formed in accordance with the adopted criteria, ranking the alternative solutions and analyzing and selecting the priority methods of bridge rehabilitation/strengthening. In the preliminary stage, a number of rehabilitation/strengthening methods were taken into consideration, which in the final stage were reduced to a total of 8 methods (Met1,...Met8): refurbishing rehabilitation, cross-section strengthening, adding stiffeners, rehabilitation using FRP strips, adding dampers, installing base insulation, changing the static system, and strengthening foundation with piles, while 6 criteria being selected for multi-criteria optimization: capacity c_1, ductility c_2, deformation c_3, global stability c_4, rehabilitation costs c_5 and time needed for rehabilitation c_6. Values of coefficients c_1,...c_6 are shown as normalized. For the purpose of this study eight possible scenarios were defined for which different values of weight coefficients w_i were considered across the criteria. Table 1 shows the decision matrix and the predefined scenarios along with the weight coefficients.

Table 1. Matrix decision and predefined scenarios along with the weight coefficients in the analysis of priority methods of bridge rehabilitation/strengthening

	c_1	c_2	c_3	c_4	c_5	c_6		w_1	w_2	w_3	w_4	w_5	w_6
Met1	1	1	1	1	1	1	S1	6	5	3	4	1	2
Met2	4	2	3	4	3	5	S2	5	6	3	4	1	2
Met3	5	3	4	5	4	5	S3	5	3	6	4	1	2
Met4	2	4	3	3	4	4	S4	5	4	3	6	1	2
Met5	2	2	3	4	5	4	S5	4	3	1	2	6	5
Met6	2	3	3	4	6	5	S6	2	1	3	4	6	5
Met7	7	5	5	6	5	5	S7	4	3	1	2	5	6
Met8	4	3	4	4	7	8	S8	2	1	3	4	5	6

The first four scenarios (S1, S2, S3, S4) are related to situations when higher weight coefficients values were taken into account in order to increase the quality of behaviour of bridge structural system. In the case of the remaining four scenarios (S5, S6, S7, S8) higher importance is attributed to the economic effects and the time needed for rehabilitation/strengthening of the bridge. The ultimate solution obtained using the VIKOR method is shown in the form of priority ranks of bridge rehabilitation/strengthening methods (Figure 11).

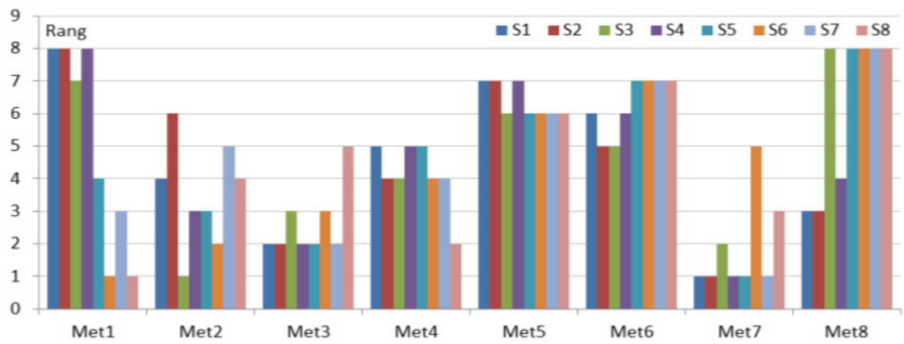

Fig. 11. Ranking/priority chart of bridge rehabilitation/strengthening methods

In the case of the first four scenarios (S1, S2, S3, S4) with higher values of weight coefficients, in order to improve the quality of behaviour of the bridge structural system, the method of changing the static system Met7 is the priority approach to rehabilitation/strengthening. In the case of the last four scenarios (S5, S6, S7, S8), with the emphasis on economic effects and the time needed for rehabilitation/strengthening, Met1 (refurbishing rehabilitation) is the priority approach. However, in specific cases, the method of refurbishing rehabilitation is insufficiently adequate for bridge rehabilitation, so when considering the rank of trade-off solutions the Met3 approach (adding stiffeners) is obtained as optimal solution.

7. Conclusion

Research determined the maximum value of global drift of 3.5% for columns, whereas for drift values of 2.6%, significant reduction of load bearing capacity. On the other hand, ductility for levels of maximum realized displacement and level of system target displacement are almost larger than 6 and 3, respectively, which indicates favourable ductile behaviour. Fracture mechanism development occurs via plasticization of column ends, which also indicates a favorable fracture mechanism for seismic activity conditions. System vulnerability analysis has shown that up to S_a=0.2g nearly all levels of damage are initiated, but level of complete damage (P=1) can be expected only for S_a>1g. However, already at values of S_a≈0.3g, extensive damage to the overpass is expected to occur along the transverse directions with probability P≈1. Reliability level analysis of the overpass along the transverse direction showed that reliability R>0 is ensured up to S_a<0.23g for levels of extensive damage and lower. Based on that, it can be concluded that the overpass (along the transverse direction) is very vulnerable up to the level of S_a≈0.3g, and that in this case repairs are justified.

Acknowledgements

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