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Seismic assessment of existing reinforced-concrete arch bridges

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Original scientific paper

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Seismic assessment of existing reinforced-concrete arch bridges

A new seismic assessment procedure for arch bridges is presented in the paper. The linear response spectrum analysis and the nonlinear static pushover methods are combined in this procedure through various assessment levels and appropriate checks. Guidelines for collecting arch-bridge data needed to reach the required level of knowledge on structural properties are proposed. Criteria for seismic assessment, such as the required participation of effective modal masses, adequate stiffness distribution of spandrel columns, and determination of reference point for forming capacity curves, are improved and adjusted for arch bridges.

Key words:
arch bridges, seismic assessment, linear response spectrum analysis, nonlinear static pushover methods

Izvorni znanstveni rad

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Seizmičko ocjenjivanje postojećih armiranobetonskih lučnih mostova

U ovome je radu prikazana nova metoda seizmičkog ocjenjivanja lučnih mostova. Linearni proračun metodom spektralne analize i nelinearne statičke metode postupnog guranja kombiniraju se u ovoj proceduri kroz razine ocjenjivanja i pripadne provjere. Predlažu se smjernice za prikupljanje podataka o lučnim mostovima s ciljem ostvarivanja tražene razine poznavanja konstrukcije. Pritom su kriteriji za seizmičku ocjenu, primjerice zahtijevano sudjelovanje djelotvorne modalne mase, prikladna raspodjela krutosti nadlučnih stupova i utvrđivanje poredbene točke za formiranje krivulja kapaciteta, ovom metodom poboljšani i prilagođeni lučnim mostovima.

Ključne riječi:
lučni mostovi, seizmičko ocjenjivanje, linearni proračun spektra odgovora, nelinearne statičke metode postupnog guranja

Wissenschaftlicher Originalbeitrag

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Seismische Begutachtung bestehender Bogenbrücken aus Stahlbeton

In dieser Arbeit wird eine neue Methode der seismischen Begutachtung von Bogenbrücken dargestellt. Dabei werden lineare Antwortspektrum-Analysen und nichtlineare statische Pushover-Methoden durch Bewertungsstufen und entsprechende Nachweise kombiniert. Ebenso werden Richtlinien für die Zusammenstellung von Datenbanken gegeben, die das Erzielen der angestrebten Kenntnisstufe ermöglichen. Kriterien für die seismische Begutachtung, wie beispielsweise die erforderliche Teilnahme der effektiven modalen Masse, die angemessene Steifigkeitsverteilung der Brückenstützen und die Berechnung des Leistungspunktes bei der Ermittlung der Kapazitätskurve sind durch diesen Vorgang verbessert und der Bewertung von Bogenbrücken angepasst worden.

Schlüsselwörter:
Bogenbrücken, Seismische Begutachtung, lineare Antwortspektrum-Analyse, nichtlineare statische Pushover-Methode

1. Introduction

A considerable number of the existing Croatian bridges have been designed according to former design codes, i.e. with no regard to seismic actions, and so changes in requirements of new standards, and deficiencies and degradation during years of service, have resulted in different levels of reliability of these bridges. The current European seismic code does not offer a procedure for seismic assessment of bridges, arch bridges in particular. The European standard EN 1998-3 [1] covers the assessment and retrofitting of buildings, and its Part 2 [2] focuses primarily on the seismic design of new bridges. Nevertheless, a combination of these normative prescriptions may be used in the assessment of the existing bridges, but with additional improvement of certain aspects, as discussed in this paper. Non-linear static pushover methods have been the focus of extensive research in recent years [3], particularly in order to enable their use for structures with significant higher mode effects, as is the case for many bridge types.

Due to their robustness, reinforced concrete arch bridges are a rather specific type of structure, and not much can be found in the existing literature about seismic assessment of this type of bridges. Some authors [4-6] question the applicability of pushover analysis to arch bridges using reasonable counterarguments that this kind of analysis does not take into account a highly important vertical response of the arch. However, the authors of this paper believe the pushover method is quite applicable to the entire arch bridge structure, especially when evaluating the spandrel columns response and the bridge deck displacements, which generally respond in horizontal directions. In this paper, the suitability of combining the linear spectral analysis and the non-linear pushover analysis for the arch bridge assessment is proven by means of a new procedure running through appropriately developed levels of assessment. The first level will be decisive for the arch, and the second one for spandrel columns, particularly for the short ones near the crown.

2. Bridge inspection and project oversight

To define a correct structural model of the existing structure, and to perform an appropriate structural analysis, the existing and desired levels of knowledge about the existing structure must be specified based on the bridge importance. These knowledge levels may be obtained through an appropriate collection of data on geometrical properties of structural and non-structural elements which may affect structural response, including the data about structural details, such as the amount and detailing of reinforcement, concrete cover, connection between members and mechanical properties of constituent materials in conjunction with appropriate confidence factors. We need to be aware that the extent of inspections and testing would greatly depend on the funding provided by the investor and so the engineer is very often required to assess the bridge condition based on a limited amount of data. That is why it is highly significant to properly specify the most significant locations on arch bridges, i.e. the locations that must be inspected and tested.

Recommendations for the collection of data on arch bridges are developed based on the analogy with EN 1998-3 [1] for buildings, guidelines provided for bridges [7], and research on Croatian arch bridges [8-11]. Required knowledge levels, adequate data collection methods such as in-situ inspections (location and extension) for arch bridges, and confidence factors *CF* for determining properties of existing materials to be used in the analysis, are presented in Table 1. The knowledge level KL2 is required for bridges of average importance that are not critical for communication. However, the knowledge level KL3 would be more appropriate for bridges that are of critical importance for ensuring an appropriate flow of traffic, especially in the immediate post-earthquake period, and for major bridges where longer design life is required.

3. Seismic assessment procedure for reinforced concrete arch bridges

The assessment procedure presented below (Fig. 1) is conducted through successive levels. They provide answers about the expected bridge performance under a seismic event with acceptable accuracy, and indicate which are the most critical bridge details and elements. Based on the research conducted on Croatian arch bridges [8-11], it may be concluded that arches are not the most critical elements because they are characterized by great robustness as a result of their importance in crossing the obstacle. If bridge performance is deemed inadequate, appropriate counter-measures may be recommended based on these two assessment levels.

3.1. Levels of seismic procedure with required participation of effective modal masses

The first level of assessment is based on the linear multimodal spectral analysis using the effective stiffness of columns (see 3.2), which is performed on the bridge model formed based on the bridge inspection and project oversight results (cf. Section 2).

According to papers [8-10], the linear multimodal spectral analysis, performed in both horizontal and vertical directions, covers the assessment of arches quite adequately because their response under a seismic event is generally linear as a consequence of their robustness. So, with regard to arches, an acceptable performance under a seismic design situation may be proven already at the first level.

Multimodal analysis of numerous existing arch bridges [8-11] shows that capturing all modes whose effective masses

Table 1. Data collection for assessing performance of arch bridges

Knowledge level	Data collection and minimum requirements of in-situ inspection and testing			Confidence factor
	Geometry	Details	Materials	
	Arch and pier axes, cross-section dimensions.	Amount and detailing of longitudinal reinforcement, amount and detailing of confining reinforcement in critical regions, depth of concrete cover, connection between members (arch-pier, pier-superstructure)	Concrete strength,steel yield strength, ultimate strength, and ultimate strain	
KL2	Sample geometry measures at selected element's locations to be compared with available outline construction drawings from the original bridge documentation. If bridge documentation does not exist, a full survey must be conducted to reconstruct the bridge geometry and dimensions.	Checking correspondence between actual details of 20 % of the most critical structural cross-sections and available incomplete detailed construction drawings. If bridge documentation does not exist 40 % of the most critical structural cross sections are to be inspected.	Complementing the information on material properties derived from the original design or test reports or standards at the time of construction with the in-situ tests at 20 % of the most critical structural cross-sections. If bridge documentation or test reports do not exist 40 % of the most critical structural cross-sections are to be tested in situ for material properties.	1,2
KL3		Checking correspondence between the actual details at 20 % of the most critical structural cross-sections and complete detailed construction drawings. If bridge documentation does not exist, 60 % of the most critical structural cross-sections are to be inspected.	Complementing the information on material properties derived from previous test reports with the in-situ tests at 20 % of the most critical structural cross-sections. If bridge documentation or test reports do not exist, 60 % of the most critical structural cross-sections are to be tested in-situ for material properties.	1,0
Notes	Based on seismic assessment of existing arch bridges performed in the research, the most critical structural cross-sections are next to the arch abutment, at the arch crown, in the quarter point of the arch span and cross section at both extremities of the columns extending from the one twentieth to the one tenth of the length. Additionally, the most critical structural cross-sections are the ones evaluated as damaged during visual inspection. Inspection methods will depend on available funding (ultrasonic devices, concrete cover removal). Non-destructive test method (Schmidt hammer test) will be considered in conjunction with destructive tests (extraction of concrete cores and rebar samples).			

add up to 90 % of the total mass according to EN 1998-2 [2] is overly conservative, and that it might require a very high number (hundreds) of modes most of which are negligible [3] as their participation factor is very low (less than 1 %). The reason for this lies in the fact that it is extremely difficult to activate parts of the arches near their abutments (5-10 % of an arch length) that have significant weight, especially in horizontal directions. Therefore, the authors suggest that a less rigid rule, requiring participation of effective mass in the amount of 80 % of a total mass, should be adopted for the accuracy of a linear multimodal spectral analysis for arch bridges.

However, the second level of assessment based on the non-linear pushover analysis is needed for spandrel columns (particularly short ones near the arch crown) because their response to seismic actions is inelastic [10]. This kind of analysis has been proven appropriate for assessment of girder bridges supported with piers of variable height and short central column [3, 12], which could be analogized with spandrel columns on arch bridges.

The second level of assessment is based either on the non-linear single-mode pushover analysis N2 [13, 3], or the Modal Pushover Analysis MPA [3, 14], if necessary. If the assessed arch bridge has a dominant mode shape in longitudinal or transverse direction, with the effective mass of more than 75 % of the total mass [15], then the single mode pushover N2 based on this mode shape may be applied, which will neglect the higher mode shapes. Otherwise the Modal Pushover Analysis is necessary using, in this case, all modes whose participation factor is higher than 1 % and which will altogether capture more than 80 % of the total mass.

The dynamic specificity of arch bridges is the flexibility of an arch as support for spandrel columns, and the fact that a great amount of the bridge mass is generally located in the middle of the bridge, which is due to the position and mass of the arch. So, due to this bridge flexibility and position of the centre of the mass, it has been recognized for concrete arch bridges [8, 9] that

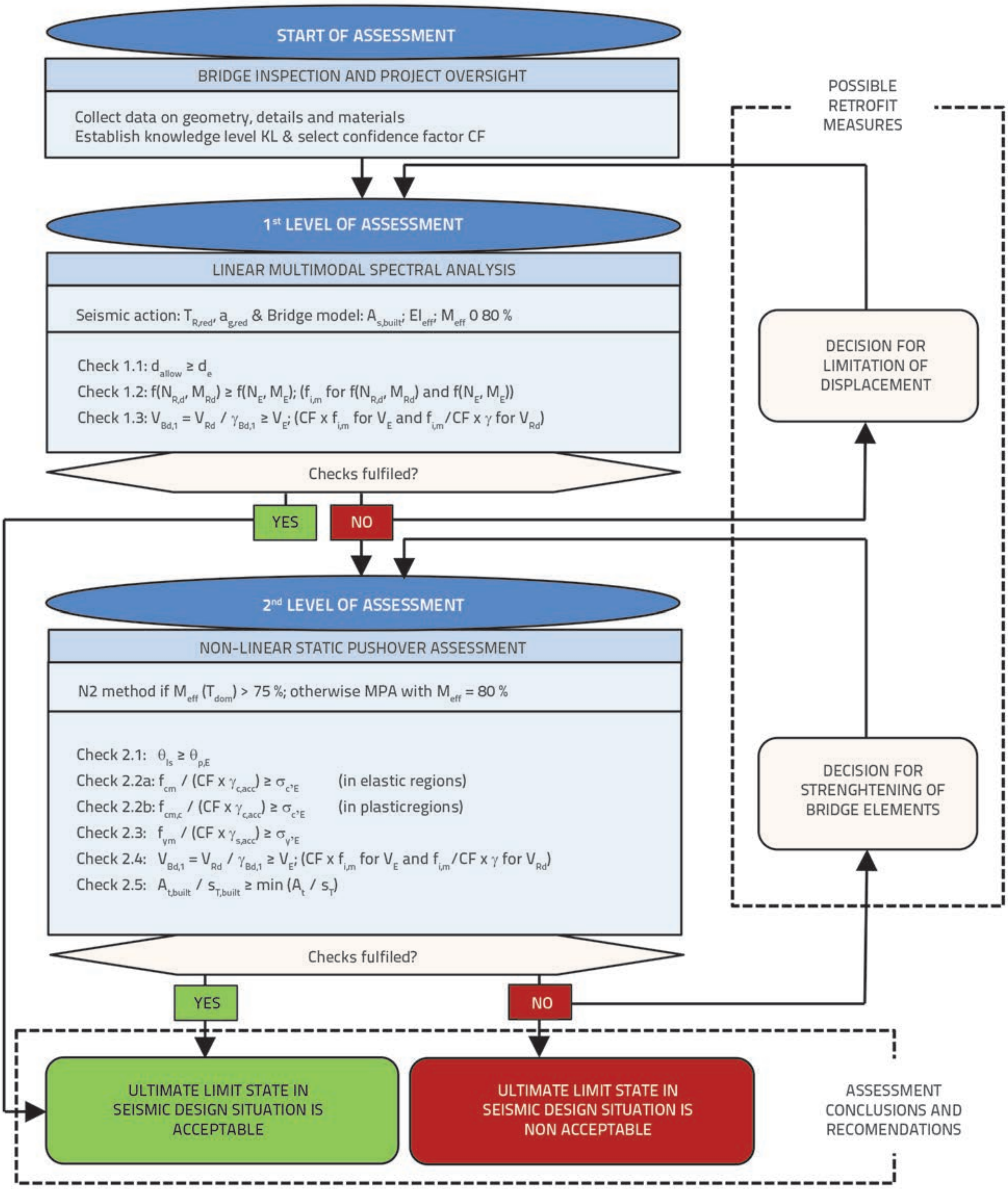

Figure 1. Seismic assessment procedure flowchart

first few horizontal mode shapes (transverse parabolic shape and longitudinal translation of the structure), with medium to long periods, are extremely dominant (m_{eff} > 60 %). If the analogy between spandrel columns on arch bridges and bridges with short central column is withdrawn (greatest demands are posed on the shortest columns) it may be noticed that, together with the dominant parabolic mode shape in transverse direction, a significant mode for MPA will be a diagonally symmetric (S-shaped) mode with the largest transverse displacement at the tops of the highest abutment piers.

3.2. Seismic load on existing bridges

For existing bridges, with the remaining life (t_L) of less than 50 years, it is appropriate to take into the account a reduced peak ground acceleration value $a_{g,R}$, marked as $a_{g,red}$, which has the probability of exceedance of p = 0.1 over a shorter reference return period $T_{R,red}$ compared to the reference return period of seismic action for the no-collapse requirement T_{NCR}. The Eq. (1) [16] offers an acceptable approximation for reduction of the peak ground acceleration, where the value of the exponent k depends on the seismicity of the region and normally ranges from 0.30 to 0.40.

$$\frac{a_{g,red}}{a_{g,R}} = \left(\frac{T_{R,red}}{T_{NCR}}\right)^k; \quad T_{R,red} = 1/(1-(1-p)^{1/t_L}) \tag{1}$$

In the linear multimodal spectral analysis of the first assessment level, the seismic load exerted on the bridge is presented with the response spectrum reduced with respect to the elastic one (design spectrum) by introducing the behaviour factor q, which reflects the ductility capacity of the structure. If there is no seismic detailing of bridge elements (which is the case for old bridges designed according to former codes), we can not rely on their ductility capacities (see Section 3.6, Check 2.1 or Equation (10)) and, for that reason, the authors propose verification of existing bridges with the behaviour factor of q = 1.0.

The applicability of the pushover method, which is to be used for the second assessment level, greatly depends on the selection of an appropriate load pattern that will produce an adequate dynamic response of the bridge. If the N2 method is applied according to the Annex H of the EN 1998-2 [2], two possible distributions are proposed: constant horizontal load along the deck and the second one, horizontal load proportional to the dominant mode shape with the largest participation factor in the considered direction (cf. Figure 2).

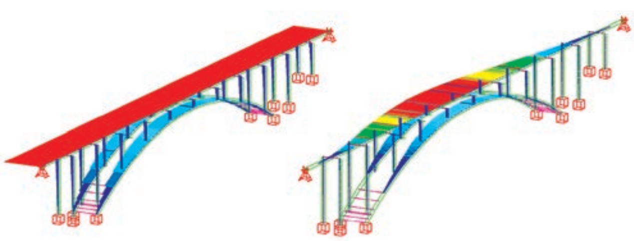

Figure 2. Constant horizontal distribution of load along the deck, and horizontal load proportional to the dominant mode shape in transverse direction

In case these two load types are applied to arch bridges, the first type (constant load along the deck) produces a greater overall seismic force on the bridge structure, and covers possible higher mode effects on coastal columns closer to the abutments. The second type of loading (load proportional to mode shape) imposes greater demands on spandrel columns closer to the arch crown. The bridges are subjected to constant vertical gravity loads composed of self-weight and additional dead loads from bridge equipment. If MPA needs to be applied, load distributions proportional to each significant mode of the bridge are to be used, with both horizontal and vertical seismic load components, the latter being of particular significance for the arch bridge analysis in longitudinal direction.

3.3. Guidelines for numerical modelling

In general, arch bridges (such as the three assessed Croatian bridges, cf. Section 4) are founded on sound rock, and so the support points of numerical models (made of beam type final elements) may be defined as fixed. The extremities of the column constitute the locations for potential plastic hinges, which may be assumed to extend from one twentieth to one tenth of the member length, depending on the boundary conditions [17]. For this reason, each pier is subdivided into six elements, with lengths equalling to 5 %, 10 %, 30 %, 40 %, 10 %, and 5 % of the pier length [18] (Fig. 3). $E_c I_{eff,in,i}$ is assigned for the cracked extremities of the column (see 3.3.1), while $E_c I_{gross,i}$ is assigned for the inner part of the column. Each span of the deck is discretized into four elements of length equalling to 10 %, 40 %, 40 %, and 10 % of the span. The linear elastic behaviour of the element does not strictly call for this subdivision into elements of different lengths, but it has been nonetheless preferred, for sake of accuracy, to refine the mesh near the connections to columns, where the change of stiffness and properties of the mesh are important [18]. Also, arch segments between spandrel columns are subdivided into four elements of length equalling to 10 %, 40 %, 40 %, and 10 % of the arch segment length.

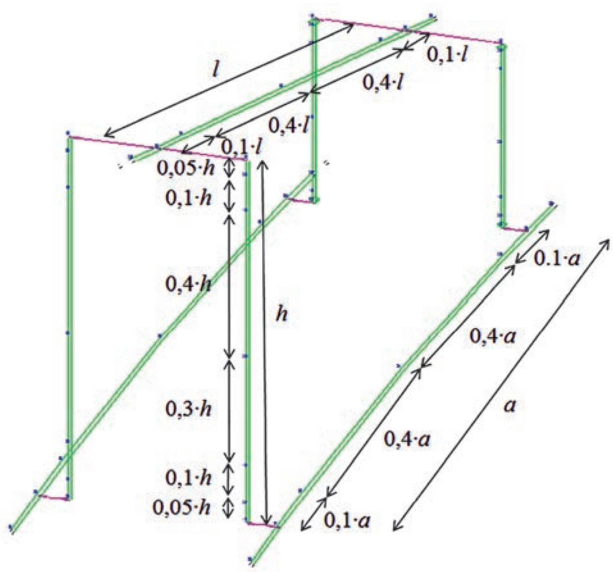

Figure 3. Discretization of an arch bridge numerical model

3.3.1. Effective stiffness of spandrel columns

In numerical model, cross sections of the bridge are to be defined with their actual as built reinforcement and with their effective stiffness. Cracked condition of concrete cross-sections at locations of potential plastic hinges is to be taken into the account by reducing the concrete stiffness [17]. According to EN 1998-2 [2], the effective stiffness may be estimated with Eq.(2) from the design ultimate moment M_{Rd} and the yield curvature Φ_y of the plastic hinge section [2]. The correction coefficient reflecting the stiffening effect of the un-cracked part of the pier amounts to ν =1,20.

$$E_c I_{eff} = \nu \cdot M_{Rd} / \Phi_y \tag{2}$$

This equation is applicable for bridges with relatively uniform column heights, which is not the case with spandrel columns on arch bridges. During inelastic response of arch bridge due to initial seismic stroke, the greatest deformation demands affect the shortest columns, which results in their excessive cracking and, finally, after a damage causing earthquake, in the need for their repair or retrofit [19, 20]. Upon the cracking of shortest columns and appurtenant stiffness reduction, deformation requirements are moved from the crown to coastal columns, which results in their degradation as well. That excessive cracking should be adequately taken into account with effective stiffness of column cross-sections. Partly based on analogy with bridges with the short central column [21, 22], and according to authors researching this issue [11], the initial assumption of effective stiffness of columns on arch bridges is derived in the following way. For each pair of spandrel and coastal columns, symmetric in relation to the arch crown, the coefficient α_i is calculated using (Eq.3) for the transverse direction (y) and the longitudinal (x) direction of the bridge. It depends on the spacing between considered pair of columns a_i, their mean height h_i, and the sum of heights of all columns Σh_i. (Figure 4).

$$\alpha_{i,y} = \frac{a_i}{\dfrac{1/h_i}{1/\sum h_i}}; \quad \alpha_{i,x} = \frac{1}{\dfrac{1/h_i}{1/\sum h_i}} \tag{3}$$

The coefficient α_i is scaled with $\alpha_{i,max}$, which generally corresponds to the highest column, and the scale factor for each pair of columns is:

$$SF = \alpha_i / \alpha_{i,max} \tag{4}$$

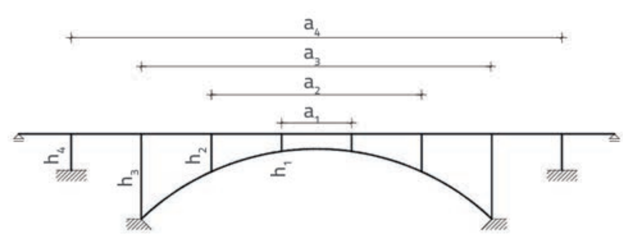

Figure 4. Columns heights and spacing between them necessary for calculation of coefficient α_i

Due to its flexibility, that highest column will most probably remain elastic during the earthquake, and it is assigned with the effective stiffness of $E_c I_{eff,in,i} = 2E_c I_{eff}$ ($E_c I_{eff}$ from Eq.(2)). The stiffness of other columns is scaled according to the previously calculated scale factor $E_c I_{eff,in,i} = SF \times 2E_c I_{eff}$. The initial effective stiffness calculated in this way provides results that are quite close to those obtained in papers [23, 24]. The assumed effective stiffness can be checked and iteratively corrected in the following way. The column global stiffness $K_{eff,i}$ is calculated from the shear force V_i in it and the displacement between the bottom and top of the pier Δ_i.

$$K_{eff,i} = \frac{V_i}{\Delta_i} \tag{5}$$

The resulting effective stiffness of cross section is obtained from the column global stiffness $K_{eff,i}$:

$$E_c I_{eff,res,i} = \frac{K_{eff,i} \cdot h_i^3}{12} \tag{6}$$

The stiffness ratio $E_c I_{eff,in,i}$ / $E_c I_{gross,i}$ of cracked and un-cracked column cross-sections in regions of potential plastic hinges of analysed bridges is shown in terms of percentages in Figure 9. Therefore, the authors suggest that the linear multimodal spectral analysis employing effective stiffness of columns according to the equation (2) should first be conducted in the scope of assessment of the existing arch bridges. After that, the effective stiffness should be calculated according to expressions (3) to (6) and then it should be used in the repeated linear multimodal spectral analysis with possible further iterations. Finally, the requirements on certain columns, resulting from initial and final multimodal analyses, are to be compared, and the larger values should be selected as relevant for the assessment. The response of the bridge and the degradation of its elements over time will thus be described.

3.3.2. Verification by time history analysis

The verification of the application of effective stiffness of columns, as proposed by the authors, is given by comparing results of the iterative linear multimodal spectral analysis with the maximum effect of the nonlinear time history analysis using three seismic records. Based on research presented in papers [25, 26], where it is emphasised that seismic records with higher peak ground velocities (PGV) are more relevant than those with higher peak ground acceleration (PGA) for seismic analysis of reinforced concrete arch bridges, the authors selected the following seismic records for this verification: Imperial Valey, 1940, a_g= 0.313 g, v_{max} = 33.4 cm/s; Loma Prieta, 1989, a_g= 0.363 g, v_{max} = 32.9 cm/s, and Northridge, 1994, a_g= 0.56 g, v_{max} = 52.0 cm/s.
Original time records of selected earthquakes are adjusted to the type 1 response spectrum with the peak acceleration a_g at the type A of soil according to nHRN EN 1998-1. 2011/NA [16, 14] using the software program Seismomatch V.2.0.0.

with the response spectra matching using the wavelets algorithm. [27, 28].

It can be seen from Figure 5 that the envelope of internal forces (CQC envelope) obtained based on the first assessment step from two different linear multimodal spectral analyses, the first one with the effective stiffness of columns according to EN 1998-2, equation (2), (CQC_EC8), and the second one with the effective stiffness of columns according to expressions (3) to (6), (CQC_eff), as originally suggested by the authors, generally corresponds well with the results of the time history analysis.

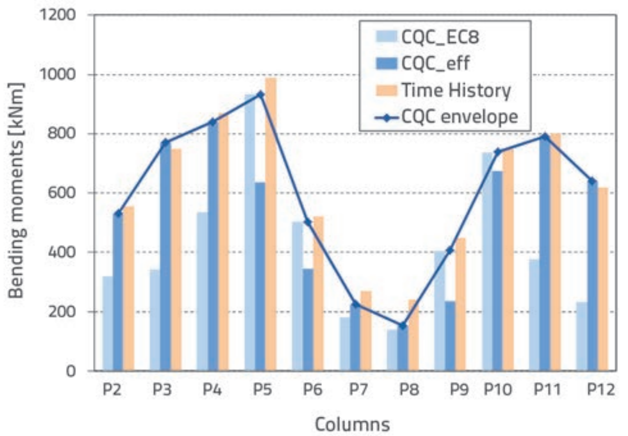

Figure 5. Bending moments in piers of the bridge example (transversal direction): comparison of multimodal spectral analysis CQC and time history analysis TH

3.4. Reference point and target displacement

The non-linear static analysis is to be carried out in two horizontal directions until target displacements d_{Tx} and d_{Ty} are reached at reference points. If conditions are met for application of the N2 analysis in the longitudinal and transverse directions, the reference points will be in the middle of the bridge at the bridge deck level (Figure 6, top, Figure 7, left). When the MPA needs to be carried out, reference points in the longitudinal direction are commonly: anywhere at the bridge deck (significant translation of bridge deck, Figure 6, top), the node at the quarter point of the arch span (longitudinal arch translation where one half of the arch decays and the other half rises, Figure 6, bottom), and the point in the middle of the highest abutment pier (local oscillation of the highest pier). Reference points in transverse direction are commonly: in the middle of the bridge at the bridge deck level (parabolic mode shape, Figure 7, left), and at the top parts of the highest abutment piers at the bridge deck level (diagonally symmetric mode shape, Figure 7, right).

The target displacement of the arch bridge, as a multimodal degree of freedom MDOF system in the observed reference point d_T, is to be calculated from the target displacement of the idealized equivalent single degree of freedom SDOF system d_T^* multiplied by the transformation factor Γ according to the step by step implementation of the N2 method for bridges [29, 3]. When the MPA needs to be carried out, the displacement demand is to be calculated by combining each separate pushover analysis following the significant mode shape and adequate reference point.

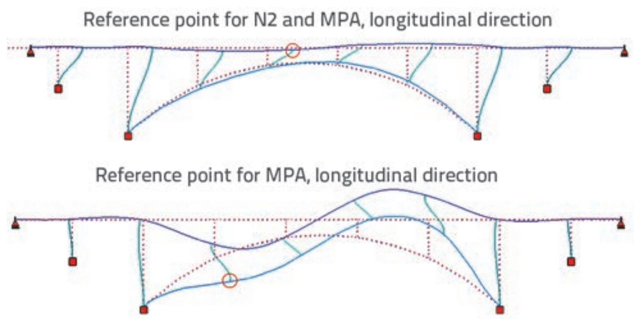

Figure 6. Common significant mode shapes of an arch bridge in longitudinal direction, and reference points for different pushover analyses

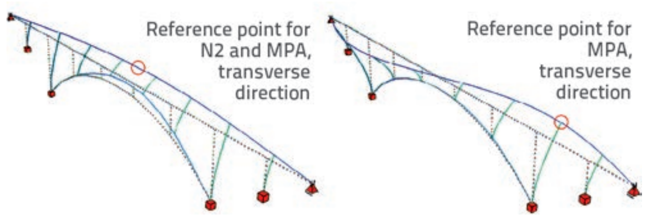

Figure 7. Common significant mode shapes of an arch bridge in transverse direction, and reference points for different pushover analyses

3.5. Checks based on linear multimodal analysis

The linear multimodal spectral analysis is performed on a bridge model with mean values of material properties [1]. The comparison of longitudinal and transverse displacements of the bridge deck at abutments under seismic load (d_E), with displacements that are actually allowable at these locations (d_{allow}), is to be performed as follows:

Check 1.1: $$d_{allow} \geq d_E \quad (7)$$

This check is done because displacements under seismic loads can be excessive, and can result in deck pounding into the abutment back wall. Based on displacement checks, the assessor can make decisions for limitation of bridge deck displacements with installation of seismic action restraining or isolating devices at abutments. If retrofit measures are to be taken, it is important to apply this same procedure once again on the model of the retrofitted bridge, and evaluate the results in the same way.

For retrieving seismic demands on structural elements for the interaction of axial force and bending moment $f(N_E, M_E)$, the bridge is modelled using mean values of material properties, and for retrieving the seismic shear demand V_E the bridge is modelled using mean values of material properties $f_{i,m}$ multiplied with the confidence factor CF. From the actual as-built reinforcement, design resistances for the interaction of axial force and bending moment $f(N_{Rd}, M_{Rd})$ are based on mean values of material properties $f_{i,m}$ while the shear resistance V_{Rd} is based on mean values of material properties divided by the CF and by the partial factor γ ($\gamma_{c,acc}$ = 1.2 for concrete, and $\gamma_{s,acc}$ = 1.0 for reinforcement). Internal force checks are conducted if the following expressions are valid:

Check 1.2: $$f(N_{Rd}, M_{Rd}) \geq f(N_E, M_E) \quad (8)$$

Check 1.3: $$V_{Bd,1} = \frac{V_{Rd}}{\gamma_{Bd,1}} \geq V_E \quad (9)$$

where $f(N_{Rd}, M_{Rd})$ represents the interaction resistance to bending moment and axial force, and $V_{Bd,1}$ represents the shear force resistance V_{Rd} from EN 1998-3 [1], additionally divided by the safety factor γ_{Bd1} = 1.25 against brittle failure.

Internal force checks are generally conducted for the arch and bridge deck and, for these elements, the acceptable performance under a seismic design situation will be proven already at this level. However, this is not the case for spandrel columns where the second level of assessment is normally required.

3.6. Checks based on non-linear static analysis

In this assessment, step bridge elements are modelled with mean values of material properties [1].

The curves retrieved from each pushover analysis represent correlation between the intensity of seismic load F (total base share) and the displacement d at the reference point. Curves are to be transformed to a single degree of freedom model (idealized for the yield point and possible ductility factor) and compared with seismic demand determined using the response spectra (both in spectral acceleration–spectral displacement, S_a–S_d, format). Possible idealised shapes of pushover curves of equivalent SDOF, with the corresponding target displacement, are presented in Figure 8.

The left-side figure corresponds to the bridge with ductile behaviour, and the right-side figure to the bridge with the limited ductile (almost elastic) behaviour, which is often the case for the existing reinforced concrete arch bridges. In the N2 method, this evaluation will be performed for a dominant mode shape in both longitudinal and transverse directions, while evaluation for the MPA method will be performed for each important mode shape.

An additional analysis of the deformed structure under seismic load is made through the following checks. When the N2 method can be applied, these checks are performed for the target displacement of the MDOF system and, when the MPA needs to be carried out, these checks are performed for the final displacement demand calculated by combining individual pushover analysis results.

Rotations at potential plastic hinge locations are evaluated. The verification is performed in such a way that the plastic hinge rotation demands $\theta_{p,E}$ are safely lower than the limit state defined chord rotation capacity θ_{ls}.

Check 2.1: $$\theta_{ls} \geq \theta_{p,E} \quad (10)$$

For the limit state of the bridge near collapse, the chord rotation capacity θ_{ls} is the total chord rotation capacity θ_{um} consisting of both the elastic and inelastic parts. For the limit state of the significant bridge damage, the chord rotation capacity θ_{ls} is $3/4\theta_{um}$. For the limit state of the bridge damage limitation, the chord rotation capacity θ_{ls} is equal to the chord rotation at yield, θ_y, [1]. In this research, the first two limit states were expected to be relevant for bridge assessment.

An accurate evaluation of the ultimate rotational capacity of reinforced concrete members may only be based on experimental data [30], which is due to numerous geometrical

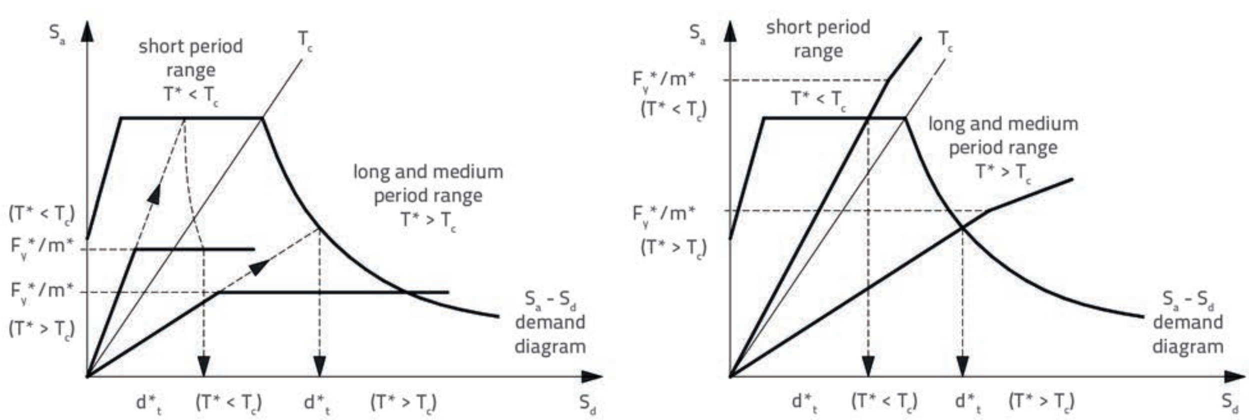

Figure 8. Possible idealised shapes of pushover curves of equivalent SDOF with the corresponding target displacement, left: ductile behaviour; right: limited ductile (almost elastic) behaviour

and mechanical parameters and uncertainties involved (load type: cyclic or monotonic, seismic detailing, concrete confinement, spalling of concrete cover, ribbed or smooth bars, overlapping length, plastic hinge length, bending contribution, height of the section, etc.). EN 1998-3 [1, 29] provides expressions for rectangular sections of elements with ribbed bars, seismically detailed, without lapping of longitudinal bars in the vicinity of the plastic hinge region, and correction coefficients for mentioned deficiencies. Unusual sections (see Figure 12 for spandrel columns sections) are to be conservatively approximated with rectangular section. This topic generally requires some additional research, which is beyond the planned scope of this paper.

Stresses of constitutive materials for the unconfined concrete $\sigma_{c,E}$ (arch, bridge deck, columns outside plastic hinge regions), confined concrete in plastic hinge regions $\sigma_{c,E}^{pl.hinge}$, and reinforcement $\sigma_{y,E}$, should each be lower than the mean material strength values divided by the confidence factor CF and the partial factor $\gamma_{c,acc}$ for concrete and $\gamma_{s,acc}$ for reinforcement:

Check 2.2.a: $$\frac{f_{cm}}{CF \cdot \gamma_{c,acc}} \geq \sigma_{c,E} \quad (11)$$

Check 2.2.b: $$\frac{f_{cm,c}}{CF \cdot \gamma_{c,acc}} \geq \sigma_{c,E}^{pl.hinge} \quad (12)$$

Check 2.3: $$\frac{f_{ym}}{CF \cdot \gamma_{s,acc}} \geq \sigma_{y,E} \quad (13)$$

The verification of members against non-ductile failure modes is conducted through the shear force check in all elements and joints adjacent to plastic hinges, taking into the account the additional safety factor against brittle failure ($\gamma_{Bd,1}$ = 1.25), where the shear resistance V_{Rd} of elements is based on mean values of material properties $f_{i,m}$ divided by the CF and by the partial factor γ (for concrete $\gamma_{c,acc}$ = 1.2 and for reinforcement $\gamma_{s,acc}$ = 1,0).

Check 2.4: $$V_{Bd,1} = \frac{V_{Rd}}{\gamma_{Bd,1}} \geq V_E \quad (14)$$

The possibility of outward buckling of the longitudinal compression reinforcement A_s between transverse ties A_t at spacing s_T is evaluated along the potential hinge area by satisfying the requirement:

Check 2.5: $$\frac{A_{t,built}}{s_{T,built}} \geq \min\left(\frac{A_t}{s_T}\right) = \frac{\sum A_s}{1,6} \times \frac{f_{ys}}{f_{yt}} \quad (15)$$

where f_{yt} is the yield strength of the tie and f_{ys} is the yield strength of the longitudinal reinforcement. The tag "built" refers to the actual built-in transverse reinforcement.

If all checks are positive, then the bridge performance under a seismic design situation is proven to be acceptable. If this is not the case, the assessor may decide to strengthen unsatisfactory bridge elements in concert with the owner of the bridge. If retrofit measures are to be taken, it is important to apply this very procedure once again on the model of the retrofitted bridge, and evaluate the results in the same way.

4. Application during assessment of three existing arch bridges in Croatia

All assessed bridges were designed and constructed according to design codes prevailing in 1960s, and no seismic actions were taken into account. In addition, smooth reinforcement was used, and no seismic detailing was made. The bridges are located in the zones of moderate seismicity according to the current European seismic design demands. They were assessed for seismic actions utilizing the 1st level (linear dynamic response spectrum analysis) and the 2nd level (nonlinear static pushover method) of the proposed assessment procedure, and the results obtained were evaluated in accordance with the demands defined by the current European seismic design codes. It should be noted that the foundation soil failure verification was not carried out in this research, as all Croatian arch bridges assessed in this study are founded on the sound rock. If this were not the case, the foundation failure check should have been incorporated in the proposed assessment procedure.

The Šibenik Bridge (Fig. 9, top), built in 1966, is a reinforced concrete three-cell box arch with the superstructure made of simply supported grillages consisting of four precast prestressed concrete girders. All columns are rigidly connected to the superstructure cross beams, arch, or their foundations.

The second structure is the reinforced concrete arch bridge across the Slunjčica River in Slunj, and it was built in 1961 (Fig. 9, central part). The main structural element of this bridge is a twin reinforced concrete arch structure with solid cross-sections of variable depth. The bridge superstructure is a standard solid reinforced-concrete slab. All columns are rigidly connected to the deck, arches, or their foundations.

The third one is the reinforced concrete arch bridge built in 1962 across the Korana River in Selište (Figure 9, bottom). The arch consists of two vaults whose depth gradually increases from the crown to abutments. At the crown, the arch connects with the solid reinforced-concrete superstructure. All columns are of circular cross-section and are rigidly connected to the deck, arches, or their foundations. According to the current Croatian seismic hazard map, the peak ground acceleration at the location of the bridges is 0.2 g, 0.12 g, and 0.12 g, respectively.

If no significant refurbishment measures are taken, an optimum remaining life t_L of all three assessed bridges is estimated at 40 years. In accordance with Eq. (1), that leads to the shorter return period $T_{R,red}$ = 380 years and the reduced value of peak ground acceleration for seismic assessment of $a_{g,red} = 0.92 \cdot a_{g,R}$.

The results of the first two levels of assessment, with the focus on deficient (the most critical) elements in seismic response of the assessed bridges, will be overviewed in the following text. Table 2 shows the conduct of assessment

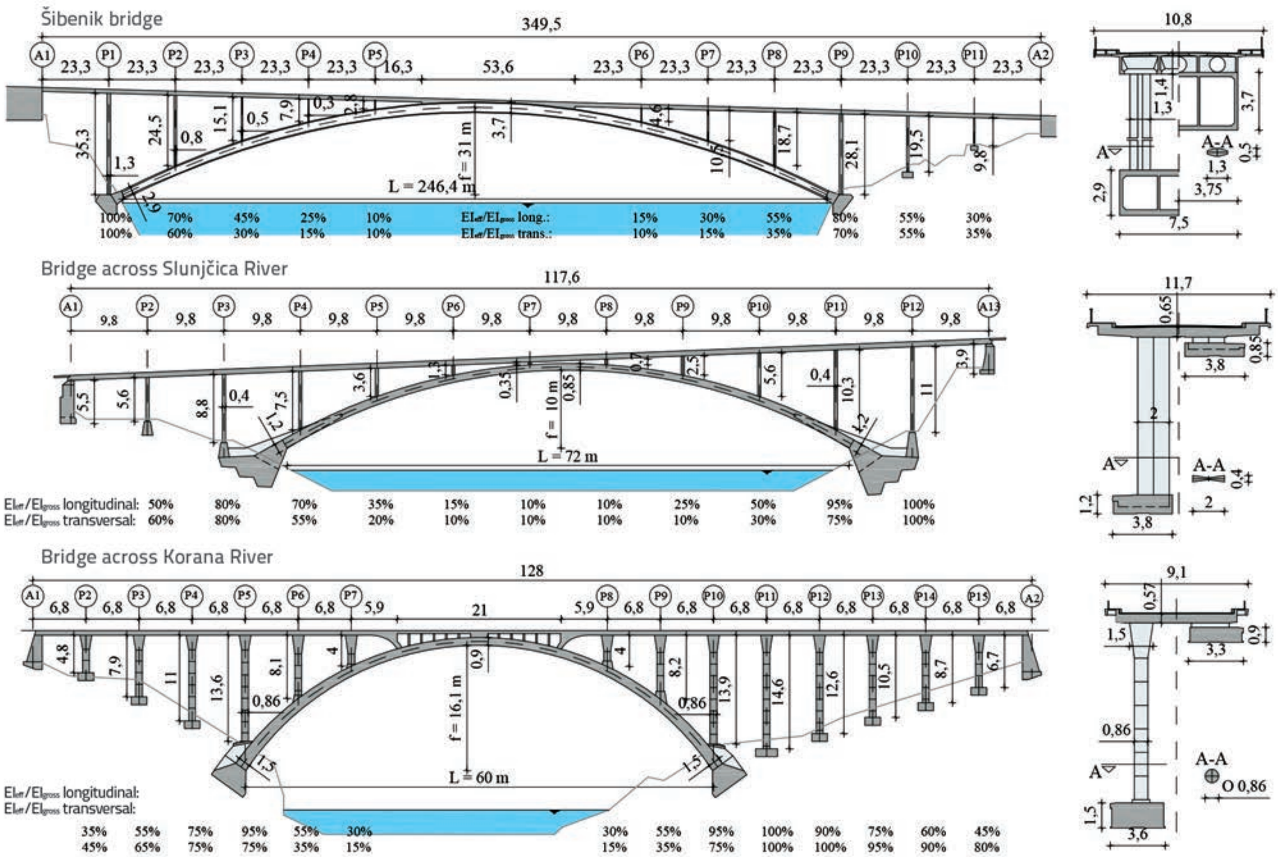

Figure 9. Longitudinal and cross sections of assessed bridges, (Units in m)

checks for each bridge with notification (*in italic*) of bridge elements which do not fulfil the requirements.

Table 2. Results of assessment checks on existing arch bridges in Croatia

Assessment checks		Šibenik Bridge	Slunjčica Bridge	Korana Bridge
1.1	$d_{allow} \geq d_t$	YES	YES	YES
1.2	$f(N_{Rd}, M_{Rd}) \geq f(N_E, M_E)$	NO P1, P5, P6, P9	NO P6, P7, P8, P9	NO P2, P3, P6 - P9, P14, P15
1.3	$V_{Bd,1} \geq V_E$	NO P5, P6	NO P5, P7, P8, P10	NO P2, P7, P8
2.1	$\theta_{ls} \geq \theta_{p,E}$	YES	YES	YES
2.2	$f_{c,i} \geq \sigma_{c,i}$	NO P5, P6	NO P6, P7, P8, P9	YES
2.3	$f_{y,i} \geq \sigma_{y,i}$	NO P5, P6	NO P6, P7, P8, P9	YES
2.4	$V_{Bd,1} \geq V_E$	NO P5, P6	NO P5, P7, P8, P10	NO P2, P7, P8
2.5	$A_{t,built}/s_{T,built} \geq \min(A_t/s_T)$	NO P1, P2, P3, P7, P8, P9, P10	NO all columns	YES

Displacement requirements for the first assessment level, considering movements of the deck at abutments, are fulfilled for all three bridges. The internal force vs. resistance requirements are fulfilled for the arches and decks, but are not fulfilled for all piers of the assessed bridges. Šibenik Bridge piers P1, P5, P6, and P9, do not have sufficient bending resistances, and piers P5 and P6 also fail to meet requirements for shear resistance. For the bridge over the Slunjčica River, the most critical elements are spandrel columns P6, P7, P8, and P9, with an insufficient resistance to bending, while columns P5, P7, P8, and P10 do not have a sufficient resistance to shear. For the bridge over the Korana River, the deficient elements are piers P2, P3, P6, P7, P8, P9, P14, and P15 with regard to bending resistance, while the piers P2, P7, and P8 fail to meet shear resistance requirements. At this level, it is advisable to go to the second level of assessment due to possible favourable force redistributions resulting from nonlinear pushover analysis, rather than to make deficient element strengthening decisions.

Figure 10 shows capacity curves of equivalent single degree of freedom (SDOF) systems versus seismic demand spectra based on modal pushover analysis (MPA) in transverse direction of the assessed bridges. The multimodal analysis of bridges in the first assessment step shows that only two dominant mode shapes with largest participation of effective mass need to be analysed to reach the required amount of 80 % of the total mass for the MPA in transverse direction. The dominant parabolic mode shape induces approx. 60 % of the total mass, and diagonally symmetric (S-shaped) mode induces approx. 25 % of the total mass. In the longitudinal direction, 3 or 4 mode shapes need to be analysed to reach the required 80 % of the total mass.

Capacity curves of analysed bridges are in general straight lines, which shows that seismic performance of the existing arch bridges is highly elastic (linear), and so the bilinear idealisation was in general unnecessary. Also, as we can not rely on ductility capacities of the existing bridge elements due to absence of seismic detailing of such elements, the capacity curves were compared with elastic demand spectra. For the 3rd S mode shape, the Šibenik Bridge can not reach the target displacement, while the response of both Slunjčica and Korana bridges is elastic for mode shapes with the short period $T^* < T_c$, and so the target displacement of the SDOF system is equal to the unlimited elastic one.

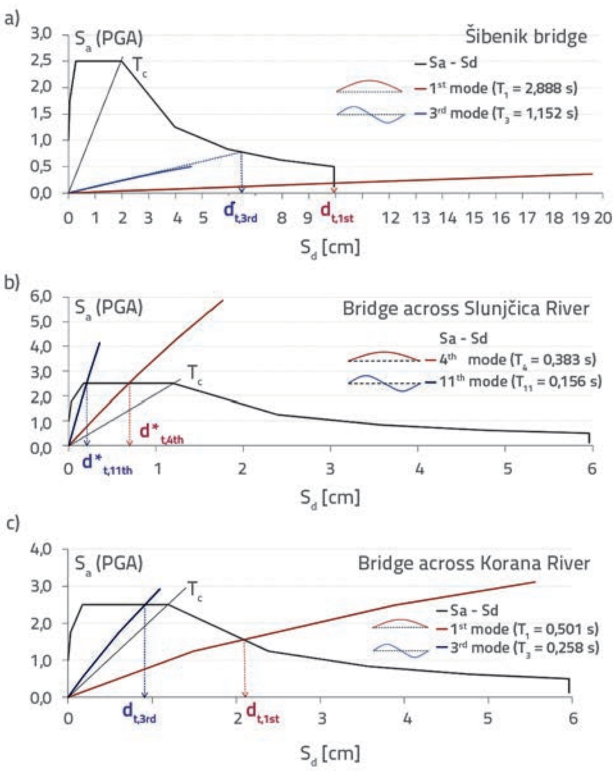

Figure 10. Capacity curves of the equivalent SDOF system for dominant mode shapes of assessed bridges in transverse direction versus seismic demand spectra analysed in MPA

At the second level of assessment, rotation seismic demands at the locations of potential plastic hinges (end parts of columns) are safely lower than the rotation capacities. Instead of checking resistance to bending moment and axial force like in the first level, which resulted in deficient resistances for all three bridges, stresses of concrete and reinforcement are checked at the second level of assessment, and the results were satisfactory for all elements of the bridge across the Korana River. For the Šibenik Bridge and the bridge across the Slunjčica River, the most critical bridge details, assessed through the concrete and reinforcement stress analysis, are short columns P5 and P6, and P5, P7, P8, and P9, respectively. For the bridge across the Slunjčica River, the yielding of reinforcement occurs under the horizontal seismic load of 0,15·g at the connection between the pier P7 and the arch. This conclusion is also partly confirmed by spalling of concrete cover at that location, as observed during visual inspection of the bridge [8] (Figure 11).

Figure 11. Spalling of concrete cover at the most critical detail of the Slunjčica River bridge - pier P7 to arch connection

For all assessed bridges, resistances to shear force are once again insufficient at the same critical piers as in the first level of assessment. However, it can be seen in Figure 12 that seismic demands gained at the second level are lower than those from the first level. That difference between the shear resistance and demand, ΔV, can be compensated by appropriate element strengthening (retrofitting) measures. It can be concluded from the same figure that the second level of assessment, which requires a somewhat greater computational effort, results in more economical retrofitting measures when compared to those that would be undertaken based on the first level results. Piers P2, P7, and P8 of the bridge across the Korana River can be strengthened with steel or FRP jacketing in critical zones, as shown in paper [31]. The shear demand at piers P5 and P6 of the Šibenik Bridge and at piers P7 and P8 of the bridge over the Slunjčica River is still excessive, so that a more appropriate retrofitting solution would be to transfer seismic forces along the deck from piers to abutments by installing seismic dampers or shock transmission units at bridge abutments, rather than by greatly changing the critical pier cross-sectional dimensions.

Checks focusing on the possibility of buckling of the longitudinal compression reinforcement between transverse ties along potential plastic hinge areas show that the spacing of ties is too large (except for the Korana bridge) according to modern seismic design criteria, and so an appropriate strengthening measure would be to confine areas of potential plastic hinges (5-10 % of end parts of piers) with steel or FRP jackets.

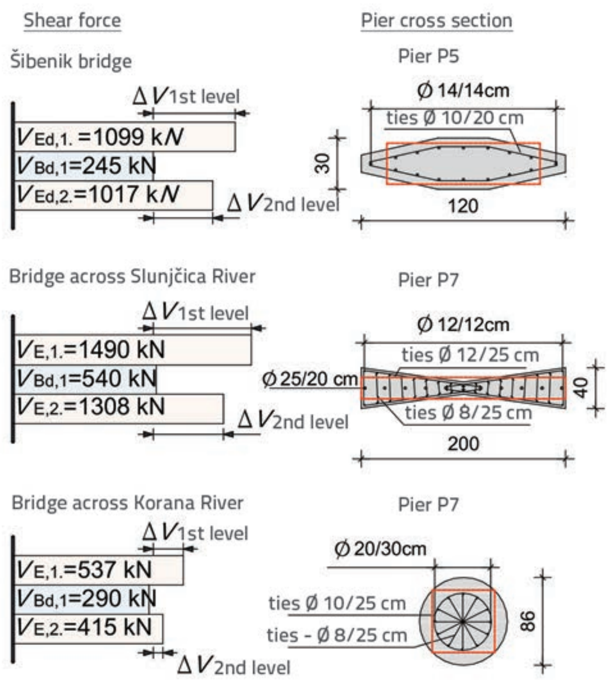

Figure 12. Comparison of shear seismic demands on the most critical piers retrieved from the 1st and the 2nd level of assessment and their cross sections

5. Conclusion

Based on the research of seismic performance of the existing reinforced concrete arch bridges, available methods for seismic assessment of structures have been further developed and improved, and properly incorporated into a new procedure suitable for the seismic assessment of reinforced concrete arch bridges (Figure 1). The procedure, running through levels of assessment, is applicable for the entire arch bridge structure, and it points to the most critical bridge details and elements, with regard to seismic response. It consists of two levels, with several evaluation checks at each assessment level. Each evaluation check provides an answer on whether the specified requirements have been fulfilled or not. These answers enable a relatively accurate definition of guidelines for seismic retrofitting of arch bridges, which can be presented to the bridge owner who will make the final decision about the proposed retrofitting.

The first level of assessment results in a more conservative estimate of the bridge seismic response than the second level. Therefore the second level of assessment must be conducted for bridges that do not fulfil the criteria checked at the first level. As reinforced concrete arch bridges are rather specific due to their robustness, it was established that the performance of arches under a seismic design situation can be proven already at the first level using the linear multimodal analysis. However, the second level of assessment based on the non-linear pushover analysis is needed for spandrel columns (particularly for the short ones near the arch crown). Although the second level requires a greater numerical and computational effort, it results in a less conservative estimate of bridge state than the first one, and thus in economically favourable retrofitting measures. If retrofitting measures are to be taken, it is important to apply this same procedure once again on the retrofitted bridge model, and evaluate the results by applying the same steps.

Additionally, certain aspects of available seismic assessment methods have been improved and developed to increase the arch bridge assessment accuracy. These are: adequate stiffness distribution of spandrel columns, required participation of effective modal masses, establishment of reference point for defining the capacity curve of arch bridges, and guidelines for data collection on arch bridges to reach the required knowledge level.

The authors conclude that, due to its straightforwardness, the presented seismic assessment procedure could easily find its place as an everyday tool in arch bridge weakness detection, retrofit decision making, and seismic retrofit design.

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