Comparative Analysis of Seismic Response of Irregular Multi-Span Continuous Bridges

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SUMMARY:
EN 1998-2 prescribes conditions that must be fulfilled in the design of bridges for earthquake resistance. In addition to the strength capacity, it is necessary to provide the appropriate ductility in critical regions of the structural elements, i.e. the ability of sections to resist required plastic deformation without significant loss of capacity. Bridges of ductile behaviour shall be designed so that a dependably stable partial or full mechanism can form in the structure through the formation of flexural plastic hinges, usually in the piers, while the bridge deck shall remain within the elastic range. For the example of irregular RC girder bridge over the ten spans with different lengths, the results of linear and nonlinear analysis performed to determine the seismic effects in the longitudinal and transverse directions of bridge designed according to EN 1998-2 are shown.

Keywords: seismic demands, bridge, irregular structure, pushover analysis, nonlinear time history analysis

1. INTRODUCTION

The basic design concept of reinforced concrete bridges in seismically active regions, provided in EN 1998, is reflected in the reduction of the expected seismic forces obtained from the elastic response spectrum, within the linear analysis. The reduction level of seismic actions depends on the expected structural behaviour and the level of structural damage that can be allowed. According to that, a decision is made, whether the structure is going to be designed as ductile or limited ductile, but regardless to the level of acceptable damage, it should not collapse. If a ductile structural behaviour is adopted, it is necessary to apply a special design method, using the method of programmed behaviour (known as "Capacity Design Method"), with the appropriate designing and detailing of such structural elements in which the formation of plastic hinges and the appearance of nonlinear deformations is predicted.

The potential damage and the appearance of inelastic deformations in reinforced concrete (RC) bridges is allowed in the columns, while the behaviour of the bridge beams should remain in the elastic region. The level of reduction of seismic forces is defined by the behaviour factor q. According to EN 1998-2, in the case of ductile and limited ductile behaviour the value of q factor cannot be greater than 3.50 and 1.50, respectively. The ductility of critical sections of structural elements in general is primarily achieved with transverse reinforcement for cross-sections confinement. The EN 1998-2 prescribes required conditions for confinement reinforcement, such as quantity and arrangement of reinforcement as well as its distance in the longitudinal direction of the element. In terms of securing the necessary ductility, a form of column cross-section can be more or less favourable, as the compact and box shaped are better than the heterogeneous cross-sections.

For the seismic analysis of civil engineering structures, an appropriate dynamic model needs to be adopted and the input data must be defined depending on representation of the seismic actions. The bridge mass in a dynamic model is determined in accordance with Eurocode EN 1990, Annex A2, depending on the type of bridge and traffic intensity. Dynamic characteristics of structures are determined using the effective stiffness of RC element cross-sections, equal to the secant stiffness corresponding to the theoretical analysis or estimated in accordance with the recommendations given in EN 1998-2, Annex C. The behaviour regularity of reinforced concrete bridges with ductile behaviour is determined in accordance with EN 1998-2, and in the case of bridges, this control may result in the reduction of the behaviour factor for the given direction, which must not be less than 1.0.

2. BRIDGE STRUCTURE DESCRIPTION

The reinforced concrete bridge having a beam of continual static system (Fig. 1) with single-cell box cross section, with the upper and lower slab thickness of 35 cm. The webs are approximately 50 cm thick and inclined in relation to the vertical plain for 20º. In the zone of beams being supported by bridge piers and side elements, beam cross section webs are strengthened by increasing their thickness to 1.9 m. Beam cross section height is 2.6 m, upper slab width is 14 m, and lower flange width is 6.5 m (Fig. 2). The beam bridge is at its ends freely supported via side elements rigidly fixed into piles. Bearings between beams and side elements permit displacement in longitudinal direction, and prohibit displacement in vertical and transverse direction. The bridge beam lies over bridge pier tips, and it is freely supported by them. The bridge has ten fields: the first and last field have the span of 26 m, while the internal eight fields have the span of 33 m. Bearings between beams and pier tips equalize displacements in transverse and vertical direction, and only three piers (piers 1, 2 and 3 presented in Fig. 1), together with the one side element on the joint with the beam, have bearings that equalize displacements in transverse, longitudinal and vertical direction of the bridge. Total bridge beam length is 320 m with the longitudinal inclination of 0.055% and straight line axis. Piers (there are nine of them) have “I” cross section (Fig. 2) with the flanges of rectangular cross section with the dimensions 220×145 cm and the web 30×350 cm. Total bridge pier cross section is 650 cm, and the largest width is 200 cm. Bridge piers are fixed into pile elemets rigidly supported onto piles and having diverse lengths in the range from 5.4 m to 33.0 m. Bridge structure concrete quality is C25/30, and reinforcement is S400 (class C)

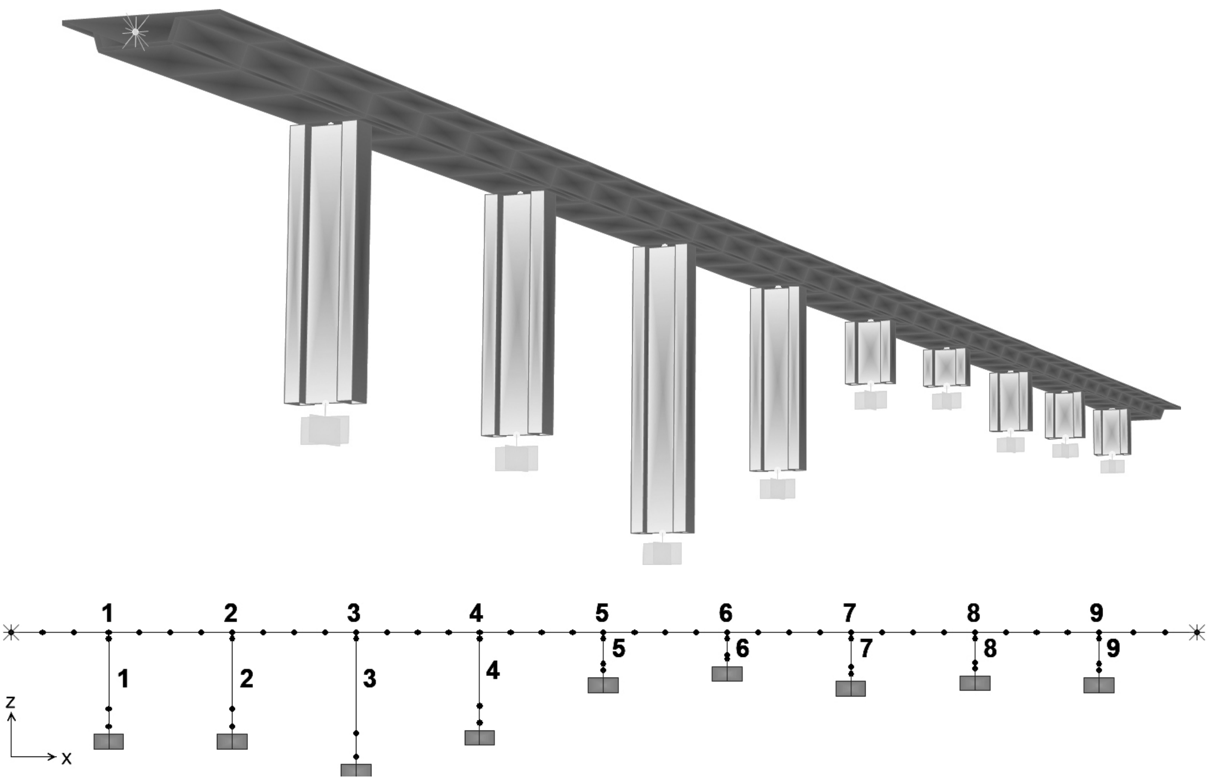

Figure 1. Structural model of RC girder bridge

The geometrical characteristics of columns and bridge beam cross-sections are shown in Fig. 2.

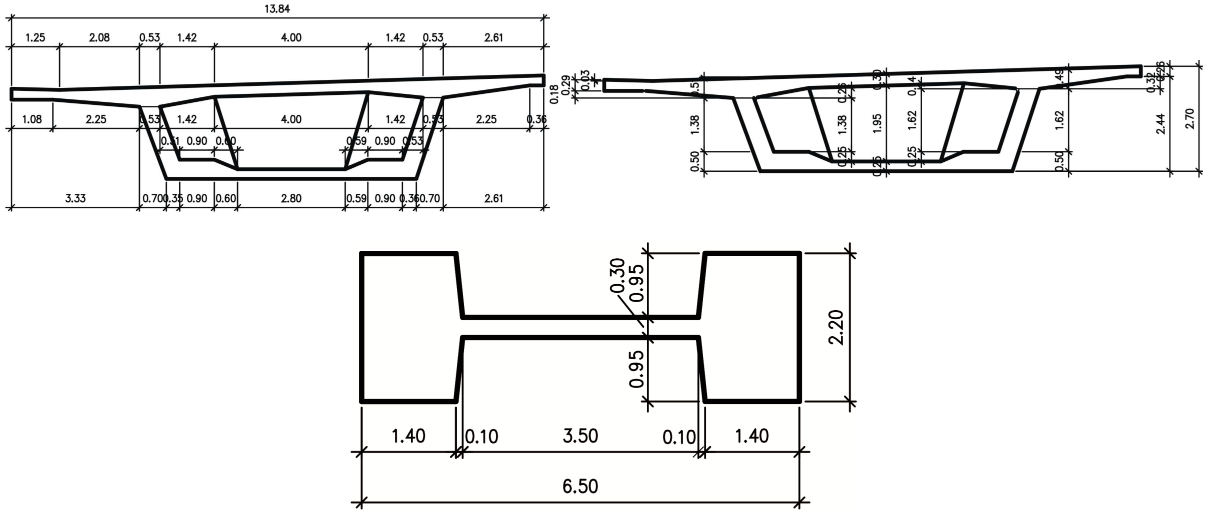

Figure 2. The geometrical characteristics of columns and beam cross-sections

3. BRIDGE DESIGN MODEL

Dynamic analysis of the bridge is carried out on a 3D structural model, with six degrees of freedom acquired for each node. Free vibration analysis was carried out first, and then a preliminary seismic analysis for the dimensioning and nonlinear modelling. Total weight of the bridge beam is $W_b = 107582$ kN, and of the columns $W_c = 26518$ kN, so the total weight of the bridge is $W_s = 134100$ kN. Bridge mass ($m_s = 13670$ t) is concentrated in the beam nodes in proportion to the length of the segments. Vibration period of the first mode for longitudinal and lateral direction is $T_{1,x} = 3.19$ s and $T_{1,y} = 0.47$ s, respectively. The main vibration shapes of the first mode for longitudinal and lateral direction are given in Fig. 3.

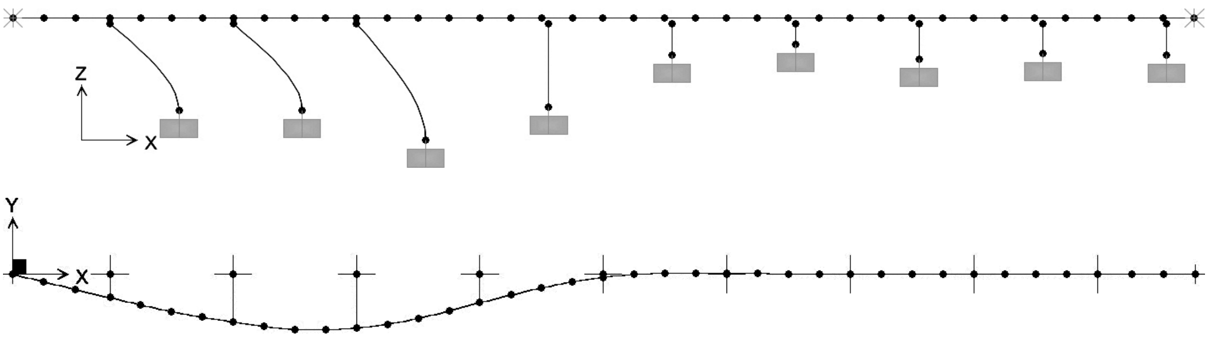

Figure 3. The fundamental modes of vibration in the longitudinal (top) and transverse direction (down)

Multimodal analysis was conducted to determine the seismic design effects for the purpose of dimensioning the bridge columns. Two types of seismic behaviour are adopted, limited ductile ($q = 1.5$) for transverse direction and ductile behaviour ($q = 3.5$) for longitudinal direction. To satisfy the requirement specified in EN 1998-2, the total number of modes that was taken into account was nine, so that the sum of effective modal masses is greater than 90% of the total mass of the bridge. The torsion effects were not taken into account as the bridge is straight, and also the required ratio of width and length of the plate $B/L = 0.044 < 2.0$, was satisfied.

The longitudinal reinforcement for columns 1, 2 and 3 (Fig. 4 – left), that are biaxial loaded, was adopted through dimensioning, while the other columns are uniaxial loaded (Fig. 4 – right).

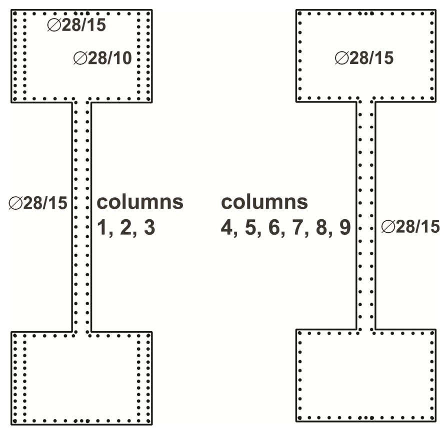

Figure 4. Adopted reinforcement of columns

Control of the adopted longitudinal reinforcement for the specific column 2 (biaxial loaded column) is shown on Fig. 5, and for the other columns (uniaxial loaded columns) in Fig. 6.

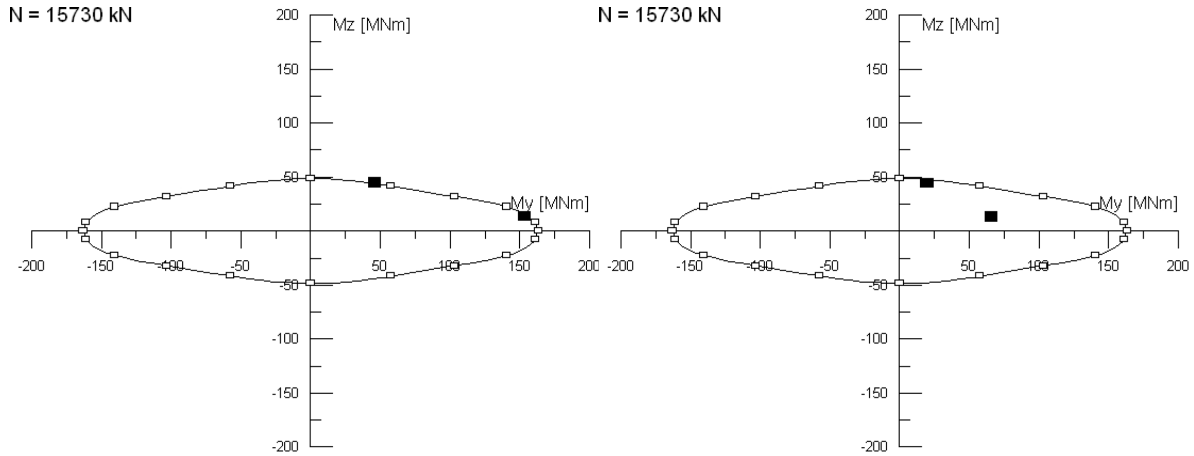

Figure 5. Checking of adopted reinforcement for column 2 (left – q = 1.5; right – q = 3.5)

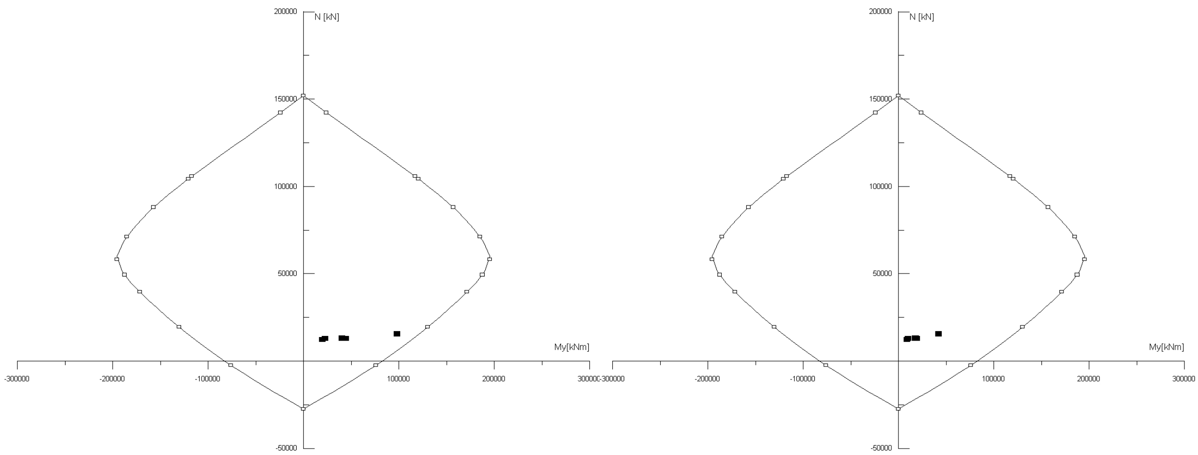

Figure 6. Checking of adopted reinforcement for piers 4, 5, 6, 7, 8 and 9 (left – q = 1.5; right – q = 3.5)

The structure has irregular behaviour in the lateral direction, as shown through the control of the behaviour regularity, and the maximum value of the behaviour factor for that direction is $q_r \cong 1.5$. Confinement of the cross-section was performed for column flanges within the potential plastic hinge regions, at the constraint zone, with the adopted reinforcement ∅10/7.5 cm, as shown in Fig. 7.

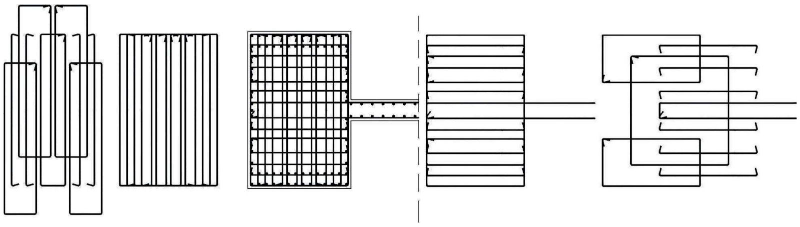

Figure 7. Adopted confinement reinforcement of column in critical regions

Two models are formed for the purpose of nonlinear static pushover analysis (Fig. 8). The first with plastic hinges defined for the column cross-section loaded with calculated axial force and bending moment by the moment-curve relation, and the second model with a plastic fiber hinges formed by the geometrical cross-sectional characteristics and adopted reinforcement. Relationship between stress and strain was adopted, especially for unconfined (protective concrete layer for reinforcement and web of the column) and confined concrete (core of the column flanges), and for reinforcement, as well. At the critical column sections (column constraint zone and top of the piles), the plastic hinges with assigned length according to Eurocode 8 are applied. The control node was adopted in the beam above the column 3 (Fig. 8), which has the largest movement in the longitudinal and lateral direction according to nonlinear static analysis.

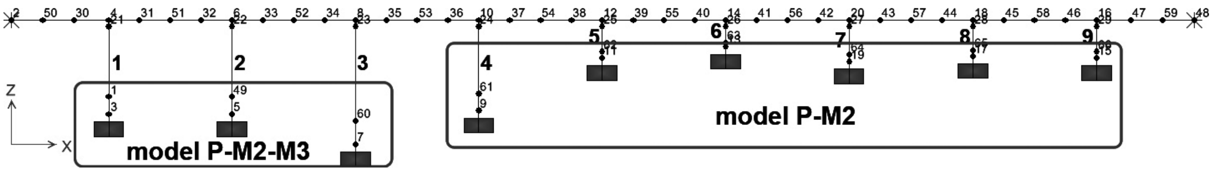

Figure 8. Structural model for nonlinear analysis

All the nonlinear analyses include the calculation of the geometric nonlinearity through the *P*-Δ effects. The material nonlinearity is introduced through the plastic hinge defined by the moment-curvature relationship (Fig. 9) and fiber model of the plastic hinge that allow taking into design the interaction between axial force and bending moments around two axes. Fiber model is implemented in three-dimensional pushover analysis and dynamic (time history) three-dimensional analysis. Model with plastic hinges is implemented in three-dimensional pushover analysis. The calculation includes corresponding nonlinear stress and strain relationships for confined concrete (Mander's model in accordance with the recommendations in EN 1998-2) and unconfined concrete according to EN 1992-1-1. "Rebar-uni-axial" model was used for reinforcement steel with specific values determined according to EN 1992-1-1. Hysteretic concrete behaviour was taken into design through the "Takeda" model, and the "kinematic" model for steel was used. All calculations were conducted using the software SAP2000.

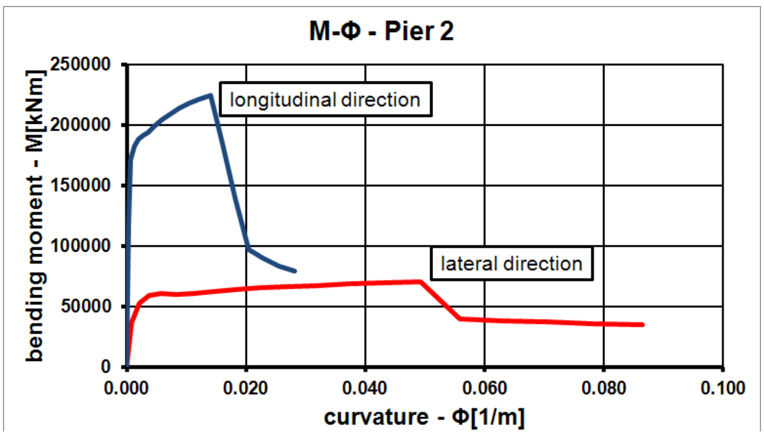

Figure 9. Moment-curvature relationship for column 2

4. NUMERICAL ANALYSIS RESULTS

Fig. 10 and Fig. 11 show the provided pushover curves for the previously described models for longitudinal and lateral direction. Pushover curve describes the nonlinear relationship between the total force and displacement of the controlled node (the top of the third column), so that the bearing and deformation capacity of the structure can be estimated.

To assess the nonlinear dynamic response, the previously described model with plastic fiber hinges was used, and the analysis was conducted using three different accelerograms of Loma Prieta earthquake ($a_{h,\max} = 0.21g$, $a_{v,\max} = 0.10g$), which are given as a seismic action. Full time history analysis of seismic response was conducted by linear and nonlinear analysis in form of a movement of the top of the third column (Fig. 12 and Fig. 13).

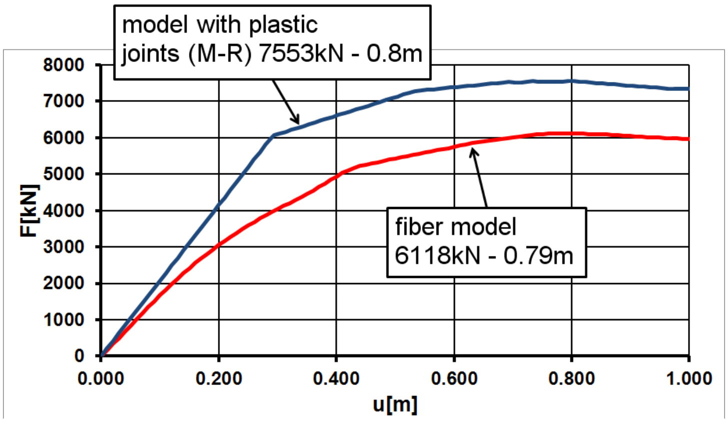

Figure 10. Pushover curves for longitudinal direction

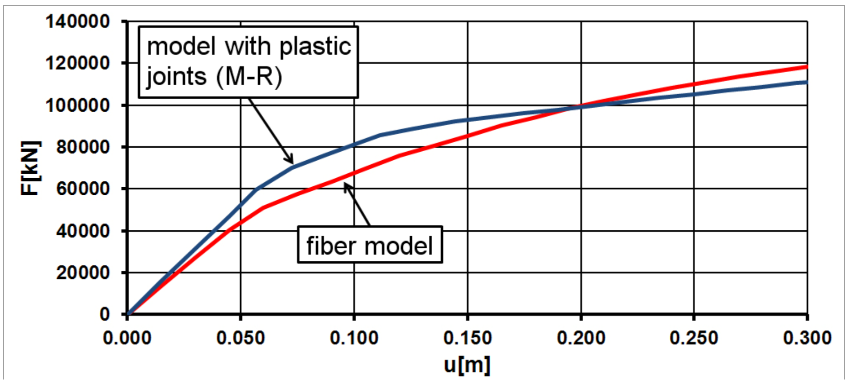

Figure 11. Pushover curves for lateral direction

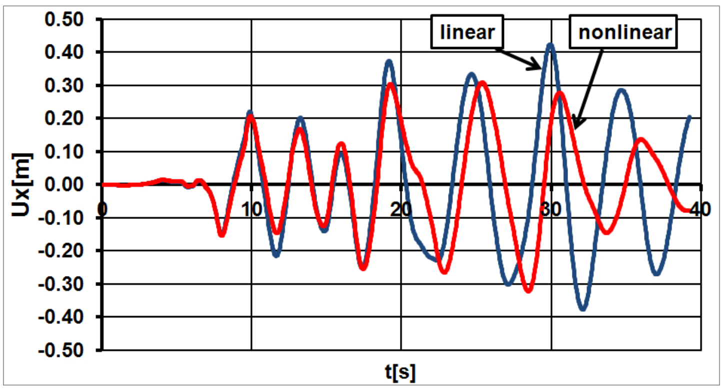

Figure 12. Horizontal displacements at the top of column 3 in *x* (longitudinal) direction

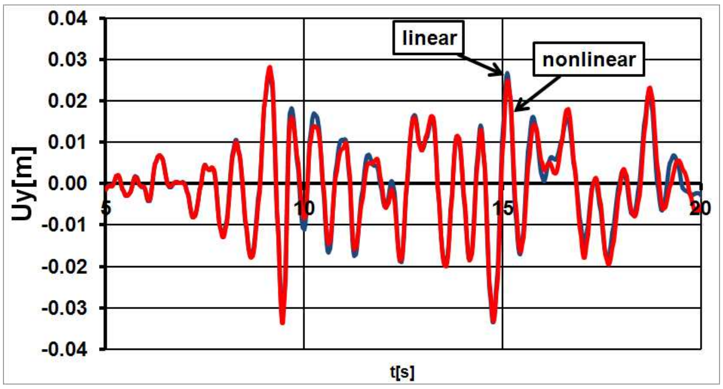

Figure 13. Horizontal displacements at the top of column 3 in *y* (lateral) direction

Relationship between the moment and rotation of the plastic joint cross-section for column 2 is shown on Fig. 13.

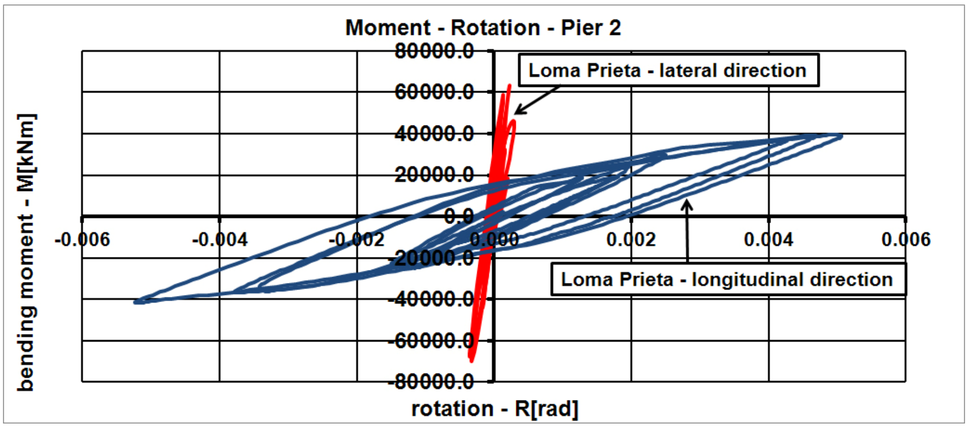

Figure 14. Moment-rotation relationship of fiber plastic hinge of column 2

Conducted analysis shows that the structure response for the considered earthquake in longitudinal direction is highly nonlinear, but the size of the movements and nonlinear deformations is in the allowed limits. The structure behaves almost linear elastic in the transverse direction for specified seismic action, with very small nonlinear effects, which is in accordance with the applied philosophy in designing the bridge (limited ductile behaviour). This is due to the large bearing capacity of the columns, arising from the application of small behavioural factors (q = 1.5).

4. FINAL NOTES

The procedures used for seismic design are mainly based on the methods of linear-elastic analysis. Calculation using the linear method of analysis gives a good estimation of the seismic forces that occur in the structure during an earthquake, but not a good assessment of displacement and deformation values. With the development of computers and software programs, methods of analysis that are based on the nonlinear behaviour of structures are gradually being introduced. The occurrence of inelastic deformations in structural elements during earthquakes leads to dissipation of seismic energy input into the structure. Due to the fact that during strong earthquakes occurrence of inelastic deformation is expected, it is necessary to determine their size. It is, therefore, of interest to develop methods of analysis and procedures to assess the seismic requirements in terms of stiffness, strength and ductility, while they may not be too complicated for use in routine engineering practice.

Linear static and dynamic analysis is now used in everyday engineering practice, and includes regulations for the design of buildings in seismically active regions. The EN 1998-2 provides the determination of seismic effects on the basis of linear elastic structural behaviour, or by using equivalent static or modal analysis in combination with the response spectrum method, where the elastic response spectrum is practically reduced by behaviour factor. The possibility of structural inelastic deformation, and the level of damage which will be allowed in the structure, without its collapse, is defined. On the basis of this particular approach to seismic forces, structures are to be dimensioned with particular attention focused to structural design of critical sections in which the appearance of plastic deformations is allowed. If there is a necessity of considering the behaviour of the structure after the occurrence of damage during strong earthquakes, the usage of nonlinear static and dynamic analysis is suggested in EN 1998-2.

The best insight into the dynamic response of the structure is obtained by means of nonlinear dynamic analysis, but it is a complex, time-consuming and impractical for everyday engineering practice. This is a consequence of insufficient degree of development of computers and computer programs used for this purpose. Also, the existence of a large number of parameters, which small changes could substantially alter the result, requires a high level of skill and knowledge necessary for this analysis in order to obtain quality results. This is why this method of analysis is commonly used only when dealing with structures of high importance or for research purposes. Lately, in practical engineering design, nonlinear static pushover analysis is increasingly used. This method can not fully replace the nonlinear time history analysis, especially in irregular structures of complex geometry, but is much simpler and faster. This method can in many cases be used successfully in the assessment of structural behaviour after the occurrence of nonlinear deformation during strong earthquakes. The EN 1998-2 provides that, in accordance with the requirements, both above mentioned methods of nonlinear analysis of structures in seismically active areas can be applied.

Seismic effects, as well as the effects from other loads that are included in seismic design situations in engineering practice, are determined by linear elastic behaviour of the structure. Referential method for determination of the seismic design effects in EN 1998 is multimodal analysis combining with the response spectrum method, which uses a linear elastic structural model and reduced seismic actions. However, to evaluate the seismic response, nonlinear methods can also be used, either static or dynamic. The use of nonlinear analysis methods is provided for irregular bridges in EN 1998-2 if used with standard analysis based on response spectrum method, in order to provide insight into post-elastic behaviour and comparisons of required and available local ductility. In general, the results of nonlinear analysis should not be used for relaxation of seismic requirements arising from the analysis of response spectrum method. However, in the case of irregular bridges, smaller seismic effects of the non-linear analysis can replace the results of the analysis based on the response spectrum method, especially rigorous nonlinear dynamic analysis which determines the complete time history of seismic response.

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REFERENCES

Ady Avram, Kevin R. Mackie, Božidar Stojadinović (2008): Pacifik Earthquake Engineering Research Center - Guidelines for Nonlinear Analysis of Bridge Structures in California - PEER 2008/03.

EN 1998-1: Eurocode 8: Design of structures for earthquake resistantce. Part 1 (2009): General rules, seismic actions and rules for buildings, 2004. Prevod – Građevinski fakultet, Beograd.

EN 1998-2, Design of Structures for Earthquake Resistance, Part 2 (2004): Bridges, European Committee for Standardization, Brussels.

SAP2000 (2009): Linear and Nonlinear Static and Dynamic Analysis and Design of Three-Dimensional Structures, CSI – Computers and Structures, Inc. Berkeley, California.