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SEISMIC RESPONSE OF VIADUCTS SUPPORTED BY HOLLOW BOX COLUMNS WITH NONSTANDARD REINFORCING DETAILS

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ABSTRACT

Many viaducts, built in central Europe several decades ago, are supported with hollow box columns, comprising deficient structural details: 1) the diameter of transverse reinforcing bars is reduced from the bottom to the top of the columns, while their relatively large distance is kept constant along the whole column, 2) the lap splices are constructed in the critical region near the column foundations, and 3) transverse bars are located at the inner side of the longitudinal reinforcement. Due to the poorly constructed shear reinforcement, these columns were expected to fail in shear. Due to the poor lateral supports of longitudinal bars and insufficient confinement of the cross section, buckling of the longitudinal bars was also expected.

A typical existing 600 m long bridge, supported by 16 single column bents was analyzed. Contrary to the assumptions, it was found that the seismic resistance of the bridge was adequate since the intensity of the load at the original site was relatively low, and the capacity was larger than expected. The relatively large flexural and shear capacity was demonstrated by the experiment of a typical short and long column.

The observed shear strength was considerably larger than that predicted by the Eurocode 2 and Eurocode 8/2. The columns yielded before the shear failure. Moderate displacement ductility was observed due to the low demand in the compression zone (favorable hollow box cross section, with large compression zone, low axial forces and high strength of concrete).

Introduction

Relatively large number of viaducts in central Europe, which were constructed before the modern principle of the seismic design were established, comprise poor structural details, which are not appropriate for the seismic regions. The typical inadequate details are mostly related to the transverse reinforcement in columns. The shear reinforcement is usually improperly constructed. The bars of the transverse reinforcement are located at the inner side of longitudinal bars. Therefore, the longitudinal bars are not laterally supported by transverse reinforcement and there is no confinement of the cross-section. Additionally, the lap splices of the longitudinal reinforcement are typically constructed in the critical region near the column foundations.

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The inappropriate construction details however are not limited only to columns. The elastomeric bearings, constructed between the superstructure and columns cannot manage even the displacement caused by the seismic load only, not to mention additional effects of the temperature. This can make the superstructure to fall down, since it is also poorly constructed as the series of simple supported girders continued with the top slab.

In Slovenia, the need for seismic strengthening of such bridges has been recognized and the related project funded by the National highway agency (DARS) started. The extensive analytical as well as experimental studies have been performed at the University of Ljubljana and Slovenian National Building and Civil Engineering Institute (ZAG), respectively. The first phase of the project was related mostly to the estimation of the seismic performances of existing bridges, particularly the seismic response of the hollow box columns.

At the time when the project started, a limited number of references related to the behavior of the hollow box columns with insufficient details was available (e.g. Pinto et. al 2001, Mo et. al 2004). Moreover, the reported results were related only to columns with transverse bars properly constructed outside the longitudinal reinforcement. Since the longitudinal bars in the investigated column were not supported by transverse reinforcement, there were some doubts that buckling of the longitudinal reinforcement could occur prior to their yielding. Therefore, in the first phase of the project the seismic performances of columns were estimated experimentally (Bevc 2006). The cyclic test on two models was performed. The response of typical short and long column was investigated. The short description of the experiment is presented in the second section of the paper.

The experimental data were used to verify the analytical models (see third and fourth section), used in extensive analytical investigation of a typical existing 600 m long viaduct (see description in the fifth section). The viaduct comprises all previously described construction deficiencies. Its seismic response was estimated using different linear and nonlinear analytical methods. For the purpose of the nonlinear time-history analysis accelerograms, specific for the site of the bridge, were generated. The description of the study and the related results are presented in fifth and sixth section, respectively.

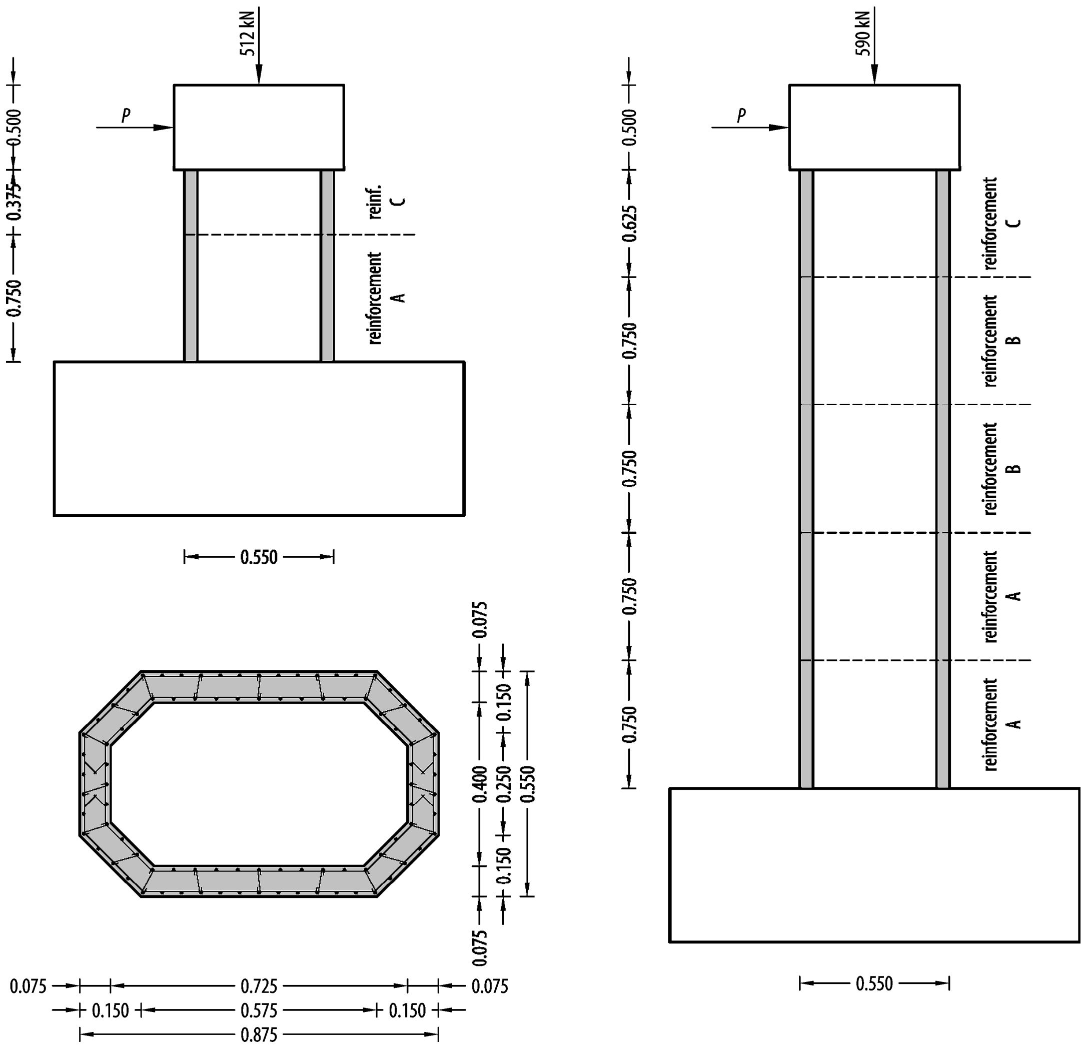
Description of the Experiment

The cyclic response of typical short and long column subjected to horizontal cyclic load was tested experimentally. The main properties of 1:4 scaled models are presented in Fig. 1.

Both specimens had hollow box cross-section with the cross-section area of 0.169 m^2 and moment of inertia of 0.0136 m^4. The compression strength of the concrete was 41.6 MPa and 49.4 MPa for the short and long column, respectively. The yield stress of steel was 324 MPa and 240 MPa for longitudinal and transverse reinforcement, respectively.

At the bottom region both model columns were reinforced with 1.5% of longitudinal reinforcement. In the upper parts the amount of longitudinal reinforcement was gradually reduced. In the bottom part the transverse reinforcing bars had diameter of 4 mm and they were spaced at the distance of 5 cm. In the upper parts of columns the diameter of transverse reinforcement was gradually reduced to 2.5 mm, while the distance was kept the same as in the bottom regions (5 cm). The shear reinforcement was constructed inside the longitudinal reinforcing bars. The lap splices were constructed in the region close to the column foundations.

At the top of the columns the axial load was applied and kept constant during the whole experiment. A level of the normalized axial forces was about 7% of the compression strength of concrete. The horizontal displacements were cyclically imposed in the middle of the column cap. Their absolute values were increasing each time the three full cycles were completed. The columns were loaded up to the failure. Other basic data about the tested specimens could be found in Fig. 1.



reinforcement A: 90 ϕ 6mm, reinforcement B: 90 ϕ 4.2 mm, reinforcement C: 90 ϕ 3.4 mm.

Figure 1. Experimental models of short and long column subjected to the cyclic load.

Analytical Models of the Columns

The nonlinear analysis was performed to estimate the response of columns subjected to the same horizontal cyclic load as in the experiment. Conventional analytical column models (see Fig. 2) typical for columns comprising standard details were used. Experimentally obtained results were used to verify these analytical models, since it was not clear if they are suitable for modeling the investigated columns too.

Both columns were modeled as cantilever beams using macro models with concentrated plasticity at the bottom of the columns. The cyclic behavior was modeled using the Takeda hysteretic rules (Powell 1978). The properties of the analytical models were defined based on the first principle. Three-linear moment-rotation envelope was defined based on the moment-curvature analysis of the bottom cross-section. The axial force of the 590 kN and 512 kN was taken into account in the long and short column, respectively. The model of unconfined concrete, defined in the standard Eurocode 2 (EC2 2004) was used. It was represented by the parabolic stress-strain relationship, supposing that the maximum compressive stresses (41,6 MPa and 49,4 MPa in short and long column, respectively) corresponded to the strain of 0,23 %. Bilinear stress-strain relationship was used to model the steel, taking into account the yielding stress of 324 MPa. The moment-rotation relationships were defined, taking into account recommendations of the standard Eurocode 8/2 (EC8/2 2004). The properties of plastic hinges, used in the nonlinear analysis of both columns are summarized in the Table 1. Part of the column between plastic hinge and top of the column was modeled with column elastic properties.

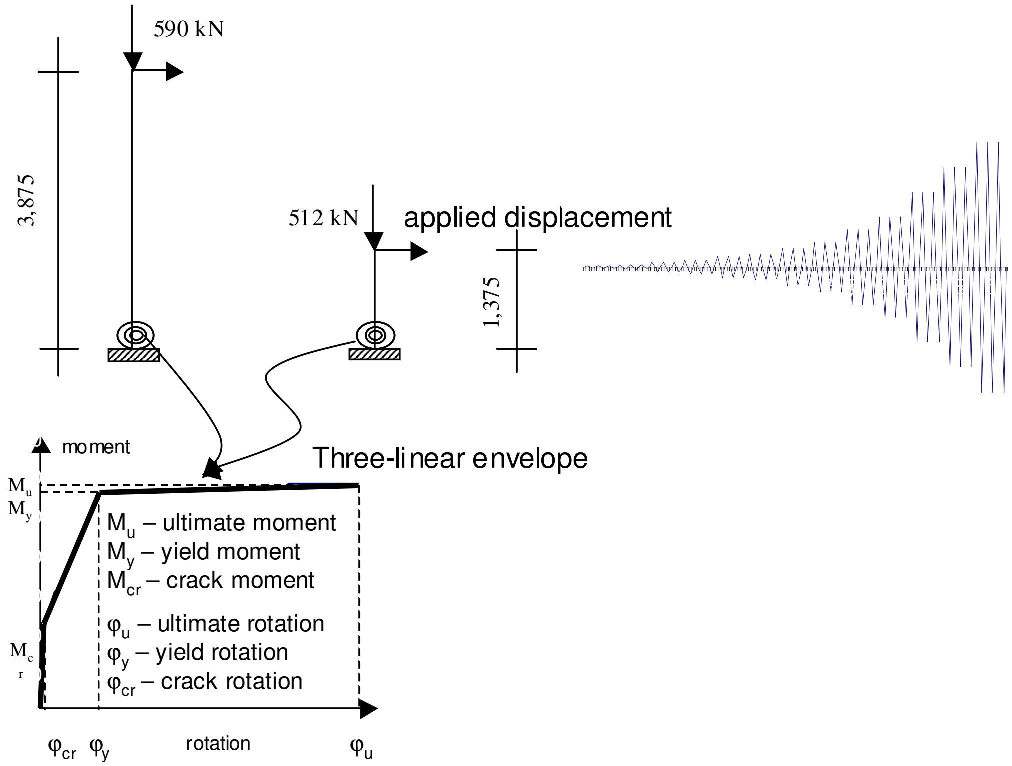


Figure 2. Analytical models of short and long column.

Table 1. Properties of the column's plastic hinges.

Column	M_{cr} [kNm]	φ_{cr}x10^{-4}[rad]	M_y [kNm]	φ_yx10^{-3}[rad]	M_u [kNm]	φ_ux10^{-2}[rad]
Short	203	2.14	500	2.24	535	0.65
Long	217	5.94	530	5.00	570	1.70

Analytical and Experimental Results

The damage of the tested columns in the end of the experiment is presented in Fig 3. The cyclic response obtained experimentally and analytically is compared in Fig. 4. Solid thin line represents the experimental results and the dashed bold line represents the analytical values. In both, short and long column the analytical and experimental results match quite good. Some differences were observed in the short column, only in the region close to the column failure. The standard analytical model was not able to take into account column strength degradation. However, this was irrelevant for the further analysis, since the seismic demand was far from this region (see next chapters). Therefore it can be concluded, that the experiment proved that the standard flexural models, supposing the uniaxial stress-strain relationship for concrete could be used in the investigated columns, too. Note that suitable analytical models were not known in advance because the columns include several nonstandard details.

a) short column b) long column

Figure 3. Damage of the columns in the end of the experiment (Bevc et. al 2006).

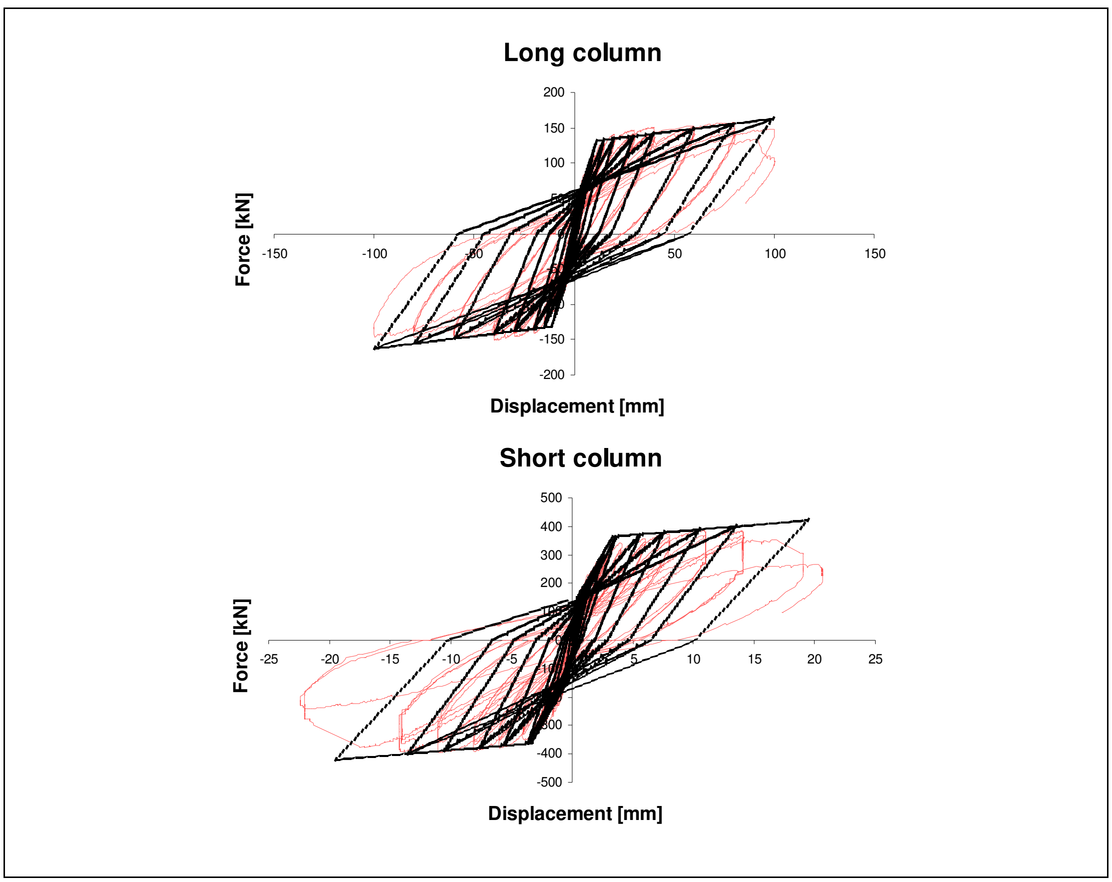


Figure 4. Analytically and experimentally determined cyclic behavior of columns.

The type of the failure of the short column was combined shear-flexural (see Figs. 3 and 4). Due to the relatively high shear strength (see the discussion in the next paragraph) the failure occurs after yielding of the longitudinal bars. Considering all inadequate construction details, the displacement ductility capacity was relatively high (about 3.5).

The shear strength of short column was analytically estimated according to the standard EC8/2 and EC2, first. The estimated value of 171 kN was very conservative comparing to the measured value of 380 kN, since, the contribution of the shear strength of concrete was, according to the standards neglected. When the shear strength of the concrete was taken into account, the value of the shear strength (318 kN) was still lower than the measured value. The more realistic analytical estimation of the shear strength was obtained using procedures defined by Priestley (Priestley et. al 1996), and that proposed in the standard Eurocode 8/3 (EC8/3 2004). The shear strength of 370 kN and 372 kN was obtained by the first and second procedure, respectively.

The failure of the long column was flexural. The experimental as well as the analytical studies proved that the shear strength of this column is considerably larger than the flexural strength. In spite of the insufficient construction details, the displacement ductility was relatively high (about 4).

The response of the tested columns was relatively satisfactory because of their favorable hollow box cross section, with large compression zone, because of relatively low axial forces, which reduced the level of compression stresses and because of the high strength of the concrete. Due to the low compression stresses the relatively satisfactory flexural behavior was obtained in spite of the lack of cross-section confinement.

Description of the Typical Bridge and Related Analytical Studies

The analyzed typical bridge (see Fig. 5) comprises all deficient details, described in the previous sections. It is 600 m long with individual span lengths between 33 m and 37 m. It is supported by sixteen single column bents. Column heights vary between 6.5 m and 34.5 m. They are supported by spot foundations and constructed in the same way as the experimentally tested specimens. The superstructure is build as the series of simple supported beams, which are continued by the deck slab. Each beam consists of four I shape girders, which are supported mostly by elastomeric bearings. In the longitudinal direction the bridge is divided into four substructures. Between substructures the dilatations are constructed. In the end of each substructure the superstructure girders are supported by PTFE sliding bearings. To limit the displacements of the superstructure in the transverse direction of the bridge, the shear keys are constructed between bearings. Bearings can freely move in the longitudinal direction of the bridge. In the original project, the earthquake load was not taken into account in the bearing design.

The bridge is constructed at the site where the maximum peak ground acceleration of 0,23g is expected. Five accelerograms corresponding to the site were generated, taking into account that the analyzed bridge is constructed on the rock.

The bridge was analyzed using different elastic and inelastic methods. The analysis was performed in the transverse and longitudinal direction separately. In the paper only the results of the study in the transverse direction (more critical) are presented.

Analytical models of different levels of sophistication were employed in the study. The basic, most simple model is presented in Fig. 6. In this model the extreme case was taken into account. It was supposed that the displacements of the superstructure were so large that the superstructure was leaned to the shear keys. Accordingly, the links between columns and the superstructure were modeled with flexural hinges. Columns were modeled in the same way as it was described in the previous sections. For the further analysis more sophisticated models were used. The models of the elastomeric and PTFE sliding bearings, shear keys and possible impacts were included in the model.

Figure 5. Typical bridge.

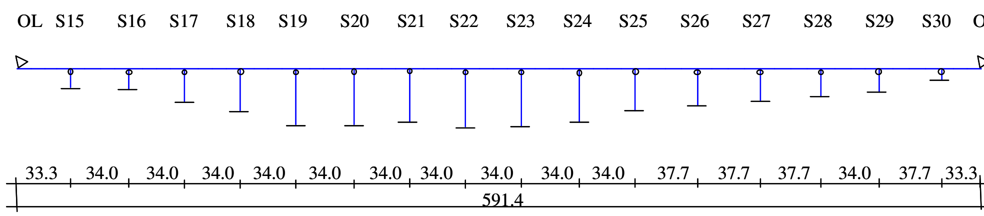


Figure 6. Basic, simplest analytical model of the bridge, used for the inelastic time history analysis in the transverse direction of the bridge.

Seismic Response of the Typical Bridge

The seismic response of typical bridge was estimated based on the displacement, bending moment and shear forces demand. The envelopes and mean values of five nonlinear analyses using accelerograms, generated for the site of the viaduct, are presented in Figs. 7 - 9. The presented results are obtained using the simplest analytical model, with the exception of the Fig. 9, where the results corresponding to more sophisticated models are also presented.

The first period of the structure was relatively high (1.67 s), however the displacement demand was low, since the bridge is constructed on the rock. The drift of the columns did not exceed the value of 0.4%.

Values of the bending moments, obtained by two different models, described in the previous section, were compared with the values of the first yielding moment M_{y1} (bending moment corresponding to yielding of the first reinforcement layer), yielding moment M_y and ultimate moment M_u (see Fig. 9.). The seismic demand is far below the strength of the columns. In more than half number of columns only the cracking of the columns was observed, and the demand was below the first yielding moment.
The shear strength of short columns was estimated to 6000 kN based on the experimental results. This

value was reduced according to the EC8/3 by factor 1.35 taking into account different uncertainties related to the materials and the existing damage of the columns, due to very aggressive environmental conditions at the site of the viaduct. The shear demand was increased by the factor 1.25 according to the standard EC8/2. In the majority of the columns the shear demand was far below their shear strength.

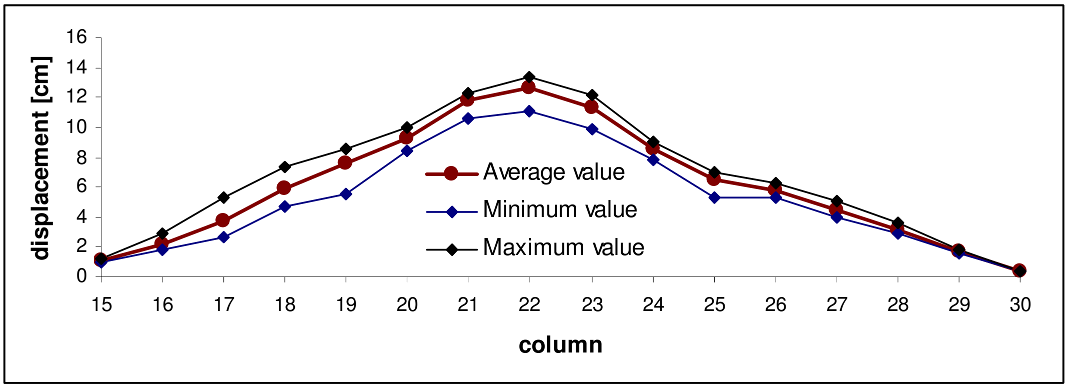


Figure 7. Displacements in the transverse direction of the bridge.

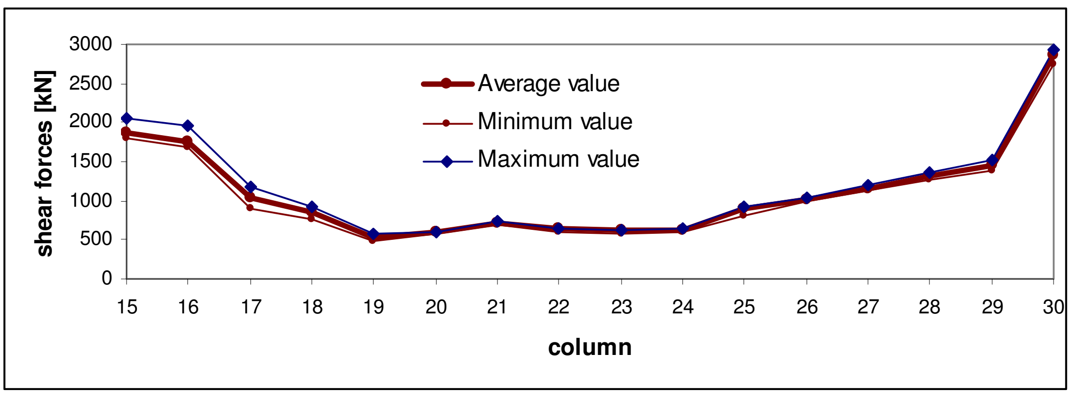


Figure 8. Shear forces in the transverse direction of the bridge.

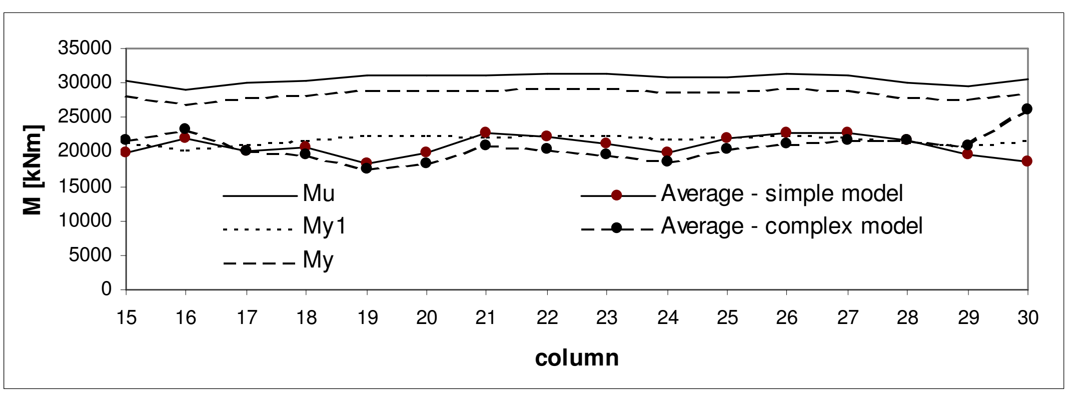


Figure 9. Bending moments in the transverse direction of the bridge.

Conclusions

The response of a typical bridge supported by hollow box columns with nonstandard construction details was investigated. The observed seismic response was acceptable because the displacement ductility capacity and the strength of columns was relatively high, compared to the seismic demand at the location of the bridge, which was relatively low.

The ductility capacity and the strength of the columns to with nonstandard details were estimated based on the experimental and analytical studies. Typical long and short columns were analyzed. Due to the low demand in the compression zone, columns had moderate displacement ductility capacity between 3 and 4. This moderate ductility was enabled by favorable hollow box cross section with large compression zone, by low axial forces, and by the high strength of concrete. Since the compression stresses were low, the buckling of the longitudinal bars did not occur before to column yielding. Consequently, in the region of moderate ductility, the standard analytical models, defined based on the properties of unconfined concrete, could be used to realistically estimate the column response.

The experiment proved that the shear strength of long columns was relatively high compared to their flexural strength. The short columns yielded before the shear failure. The shear strength, obtained during the cyclic tests, was considerably larger than the analytical values, calculated according to the standards Eurocode 2 and Eurocode 8/2. More realistic analytical estimations of the shear strength were obtained using the procedures proposed in the Eurocode 8/3 and by Priestley et. al. (1996).

The bridge is constructed on rock at the site, where the maximum peak ground acceleration was 0.23g. Due to the favorable soil conditions and relatively low seismicity, the seismic response of the bridge was essentially elastic. The seismic demand in the analyzed bridge was considerably lower than the strength of the columns.

Acknowledgments

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References

Bevc, L., Tomaževič, M., Bohinc U., 2006. Experimental and analytical study of seismic vulnerability and seismic strengthening of viaduct Ravbarkomanda and other similar viaducts: experimental part. (in Slovenian). *Report*. Slovenian National Building and Civil Engineering Institute.

EC2. 2004. *Eurocode 2: Design of concrete structures Part 1-1: General rules and rules for buildings*. EN 1992-1-1. European Comitee for Standardization, Brussels.

EC8/2. 2004. Eurocode 8: *Design of structures for earthquake resistance Part 2: Bridges*, prEN 1998-2, European Comitee for Standardization, Brussels.

EC8/3. 2004. *Eurocode 8: Design of structures for erathquake resistance Part 3: Assesment and retrofitting of buildings*. prEN 1998-3, European Comitee for Standardization, Brussels.

Mo, Y. L., Yeh, Y. K., Cheng, C. T., Tsai, I. C. in Kao, C. C., 2004. Seismic Performance and Retrofit of Hollow Bridge Columns. *Earthquake Engineering and Engineering Seismology*, 3(1), 59-66.

Pinto, A., Molina, J, Tsionis, G., 2001. Cyclic Test on a Large-Scale Model of an Existing Tall Bridge Pier (Warth Bridge – Pier A40). *Report*. Ispra: The European Laboratory for Structural Assessment (ELSA).

Pinto, A., Molina, J. in Tsionis, G., 2001. Cyclic Test on a Large-Scale Model of an Existing Short Bridge Pier (Warth Bridge – Pier A40). *Report*. Ispra: The European Laboratory for Structural Assessment (ELSA).

Powell., G. H., 1978. Mathematical Modeling for Inelastic Seismic Response of Buildings. Closing symposium on research in the field of earthquake resistant design of structures, Dubrovnik, Yugoslavia.

Priestley, M. J. N., Seible, F. in Calvi, G. M., 1996. *Seismic Design and Retrofit of Bridges.* New York, NY: John Wiley & Sons, Inc.