CYCLIC RESPONSE OF RC HOLLOW BOX BRIDGE COLUMNS STRENGTHENED USING CFRP AND RC JACKETING

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Abstract

An experimental investigation of the cyclic response of hollow box bridge columns that comprise substandard construction details is presented. Two techniques were used to increase their insufficient shear strength: 1) wrapping with CFRP strips and 2) concrete jacketing. Relatively small amount of CFRP strips and concrete jacketing efficiently increased the shear strength. In both cases the ductility capacity was not substantially improved, since the minimum amount of jacketing was not able to prevent other unfavourable failure mechanisms induced by other substandard construction details. Particularly critical was buckling and rupture of the longitudinal bars as well as the bar slip at the column base. Different analytical methods were used to estimate the shear strength of as-built and strengthened columns. Based on the large differences between the methods it can be concluded that the problem of shear strength and deformability is, in general, not adequately solved. It may be appropriate to reconsider this problem in the future developments of the EC standards.

Keywords: CFRP jacketing; Cyclic response; experiment; hollow box bridge columns; RC jacketing; seismic response; shear strengthening.

1. Introduction

A relatively large number of viaducts in central Europe, which were constructed before the modern principles of seismic design had been established, have non-standard construction details, which are nowadays considered inappropriate for seismic regions. An example of such bridge is shown in Figure 1. This bridge was constructed on one of the main highways in Slovenia. It is a multi-span simply-supported bridge, whose superstructure beams are connected together by means of a continuous deck slab. The superstructure is supported by elastomeric and teflon bearings, located at the top of the single-column piers. The columns have a hollow box cross-section, and are supported by spread footings.

There have been several concerns regarding the seismic safety of this bridge. Only those issues which are related to the non-standard reinforcement details in the columns (see Figure 2) are summarized below:

1) The lap splices were constructed near the column foundation, in the region of potential plastic hinges. It was believed that this could considerably reduce the flexural strength of the columns.

2) Additional doubts regarding the strength of the columns were caused by the transverse reinforcement which was placed on the inside of the longitudinal bars. Thus there were concerns that column strength could be considerably reduced due to buckling of the longitudinal bars, which could occur prior to their yielding.

3) The amount of the transverse reinforcement gradually reduces from the base to the top of the column. Since the shear force due to seismic loads is constant along the column, the possibility of shear failure at the top of the columns was also considered.

4) Plain bars were used for the longitudinal as well as for the transverse reinforcement. This can reduce the strength as well as the ductility of columns.

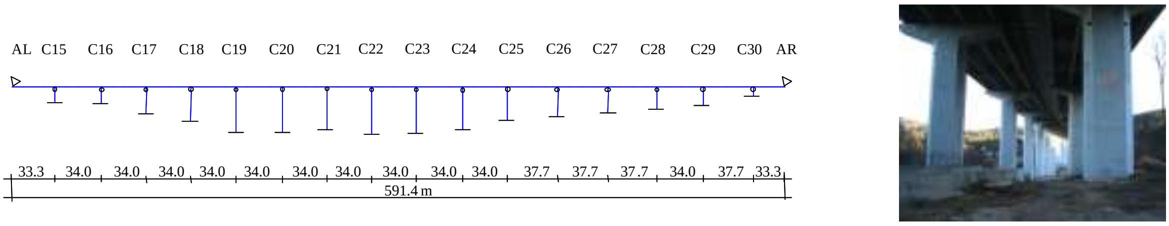

Figure 1. Viaduct, supported by columns that include substandard construction details

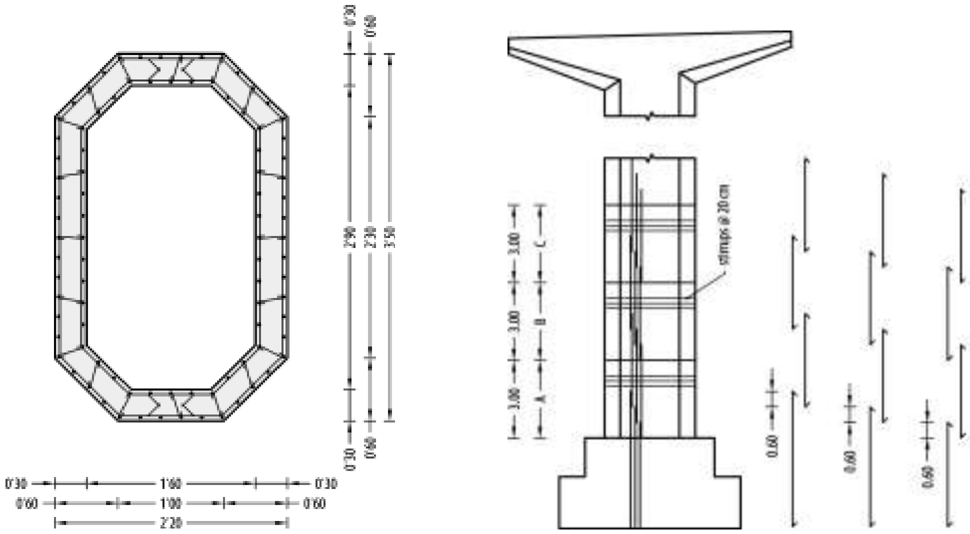

Figure 2. Construction details of the typical column

To investigate the seismic response of such columns, several cyclic tests were performed, investigating columns of different heights. Only the response of short columns is analyzed in this paper. The short description of the experimental investigation of its cyclic response can be found in Section 2.

This experiment exhibited that the shear failure was the most critical failure mechanism of investigated short as-built column. Therefore its shear strengthening was performed. Two strengthening techniques: CFRP and RC jacketing were applied and experimentally analyzed. A short description of the cyclic experiments, performed on strengthened columns is presented in Section 2. Their cyclic response is presented and compared with the response of the as-built column in Section 3.

The shear strength of as-built and jacketed columns was also estimated using different available analytical procedures. Quite different results were obtained. An overview of this study and comparison with the experiment is presented in Section 4.

2. The short overview of the experiments

2.1 The as-built column

The typical column, with an aspect ratio of 1.86, was chosen to be examined experimentally. The main properties of the 1:4 scale model are presented in Figure 3.

The longitudinal reinforcement ratio at the base was 1.5% of the gross cross-sectional area. At the top of the columns the amount of longitudinal reinforcement was reduced to 0.5%. At the base the transverse reinforcing bars had a diameter of 4 mm (16 mm in the prototype), and they were spaced at a distance of 5 cm (20 cm in the prototype). At the top the diameter of the transverse reinforcement was reduced to 2.5 mm (10 mm in the prototype), but the distance between the bars was kept the same as in the column base (5 cm; 20 cm in the prototype). The shear reinforcement was placed inside the longitudinal reinforcing bars. The lap splices were constructed close to the column foundations (see the construction details in the prototype column, shown in Figure 3). The compressive strength of the concrete was 41.6 MPa. The yield stress of the steel was 324 MPa and 240 MPa for the longitudinal and transverse reinforcement, respectively.

The model was subjected to a horizontal cyclic load. During the test, horizontal displacements were imposed cyclically at the middle of the column cap. Their absolute values were increased each time three full cycles had been completed. The axial load was applied at the top of the column, and was kept constant during the whole experiment. The level of the normalized axial forces was equal to about 7% of the compressive strength of the concrete. The column was loaded up to failure. Other data about the specimen can be found in [1].

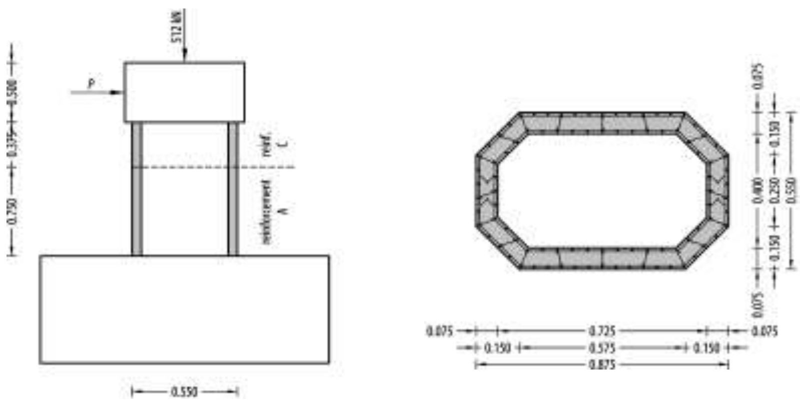

Figure 3. The 1:4 scale model of the as-built column

2.2 The strengthened columns

During the cyclic test, described in the previous subsection, a mixed shear-flexural failure of the as-built column was observed (see Section 3 for more details). Therefore its shear strengthening was performed. The necessary amount of strengthening was defined taking into consideration its available shear strength and the shear demand at the locations of the viaducts, presented in Figure1. Two strengthening techniques were investigated: CFRP and reinforced concrete jacketing.

The design of appropriate strengthening was not a trivial task, since the jacket was possible to construct only on the outer side of the column (the strengthening of the inner side would be quite demanding). The outer jacket had to provide the sufficient shear strength, but it could not be to strong, because it could worsen the response of the inner parts of the column. Namely, very strong jacket can substantially increase the maximum possible compression deformation on the outer column edges but at the same time it can also increase the deformation on inner non-strengthened edges. This can cause the spalling of the concrete cover at the inner side of the column and buckling of the longitudinal bars.

Therefore it was necessary to design the jackets which would be able to provide sufficient shear strength but which would not be too strong. Since the shear demand was not considerably larger than the shear capacity of the investigated column, the minimum amount of jacketing was provided (see next two sub-sections) in both investigated cases.

2.2.1 The column, strengthened using CFRP strips

The column was jacketed using an absolute minimum amount of CFRP strips. The one layer, 7.5 cm (30 cm in the prototype) wide strips, which were placed at the distance of 10 cm (40 cm in the prototype), and which were overlapped for 20 cm were used (see Figure 4). Carbon fibres were oriented only in the horizontal direction, perpendicularly to the vertical axis of the column. The column was wrapped along its total length. The layout of the strengthened column is presented in Figure 4.

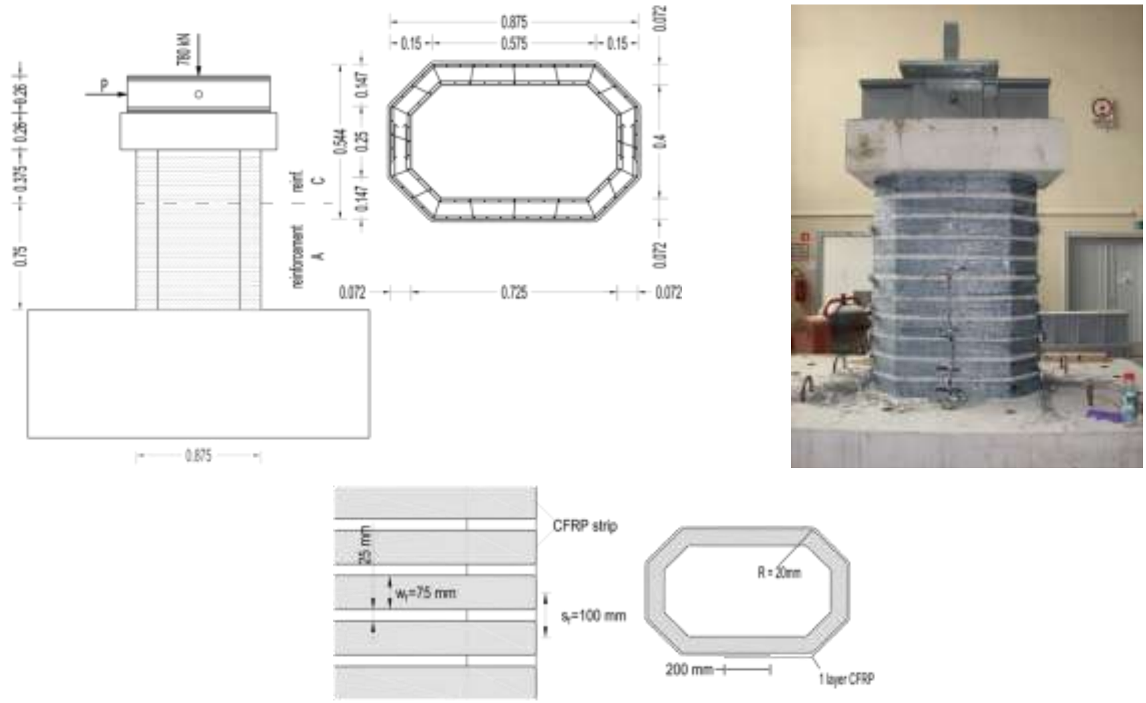

Figure 4. The 1:4 scale model of the column, strengthened using CFRP strips

The strengthened column was tested cyclically in a similar way as the as-built column. The absolute values of displacements were increased each time three full cycles had been completed. The axial force was somewhat increased (to 780 kN) (compared to the as-built column) to obtain approximately the same compression stresses (normalized stresses of 0.079 f_{ck}, where f_{ck} = 58 MPa was the characteristic cylindrical strength of concrete of jacketed column) as in the as-built column (normalized stress was 0.074)..

2.2.2 The column, strengthened using reinforced concrete jacket

The alternative way of strengthening of the investigated column using reinforced concrete jacketing was also analyzed analytically and experimentally. The scale of the specimen used in the experiment was kept the same as in the previous cases, as well as the cross-section dimensions and reinforcement details. The axial force was the same as in the case of CFRP jacketing (780 kN).

The layout of the strengthen specimen is presented in Figure 5. The column was jacketed using an outer layer of concrete. The thickness of this layer was 2 cm (8cm in the prototype). It was reinforced by longitudinal plain bars ϕ3.4 mm (ϕ14 in the prototype), which were placed at distance of 2.5 cm (10 cm in the prototype). The transverse reinforcement of the concrete jacket consisted of plain bars ϕ3.4/2.5 cm (ϕ14/10 cm in the protype). The quality of the steel of all reinforcing bars was S240.

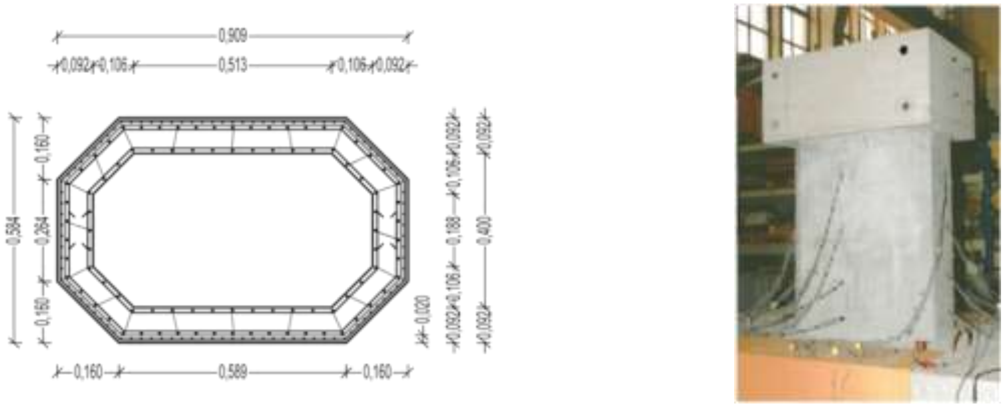

Figure 5. The 1:4 scale model of the column, strengthened using RC jacket

3. An overview of the cyclic response of as-built and strengthened columns

Although the as-built column included several sub-standard construction details that could induce the buckling of the longitudinal bars prior to their yielding, the column failed due to the insufficient shear strength after yielding of the longitudinal bars (see Figure 6). Considering poor construction details a relatively large displacement ductility capacity of 4 was obtained (see Figure 7). It was provided by the favourable hollow box cross-section with large compression zone, by the low axial forces, and by the relatively high strength of the concrete.

The response of both strengthened columns was, in general, similar. Both strengthening techniques efficiently prevented the shear failure of the column (see Figures 8 and 9). The cyclic response and the type of failure were similar. The spalling of the concrete cover at the outer and inner edges was firstly observed. It was followed by buckling and the rupture of the longitudinal bars. The pull-out of some of the longitudinal bars was also observed. In both strengthened columns pronounced horizontal crack was observed at the bottom of the column near the footing. The rupture of the longitudinal bars was followed by substantial rocking of the column. After successful shear strengthening of the column, the other unfavourable types of failure induced by the other construction deficiencies (see Introduction) were activated.

Nevertheless the response of both strengthened columns was in general similar, the concrete strengthening was more efficient, due to the additional layer of the concrete, which prolong the buckling of the longitudinal bars (comparing to the CFRP jacketing). The deterioration of the column strength was less pronounced and the energy dissipation capacity was better.

This can be observed from Figure 10, where the cyclic response of both columns is compared. The maximum achieved strength of both strengthened columns was similar (470 kN and 450 kN for concrete and CFRP jacketing, respectively). In columns strengthened by CFRP strips, the strength was decreased more rapidly, and the energy dissipation capacity was obviously lower than in the case of concrete jacketing. However, it should be noted that some of the longitudinal bars in the column, strengthened by CFRP strips were corroded prior to the test. This should be taken into account when analyzing the results, since the strength and the ductility of these bars can be substantially affected by the corrosion, reducing the energy dissipation capacity of the column.

Figure 6. The investigated as-built column after the failure

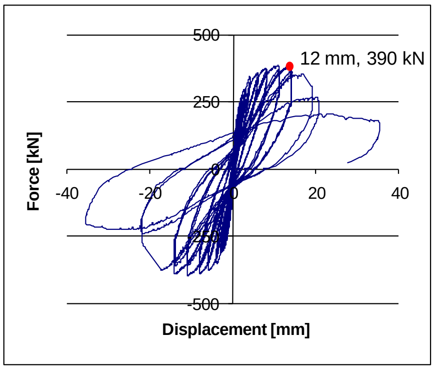

Figure 7. Cyclic response of as-built column

Figure 8. Column, strengthened by CFRP strips after the experiment

Figure 9. Column, strengthened using RC jacket after the experiment

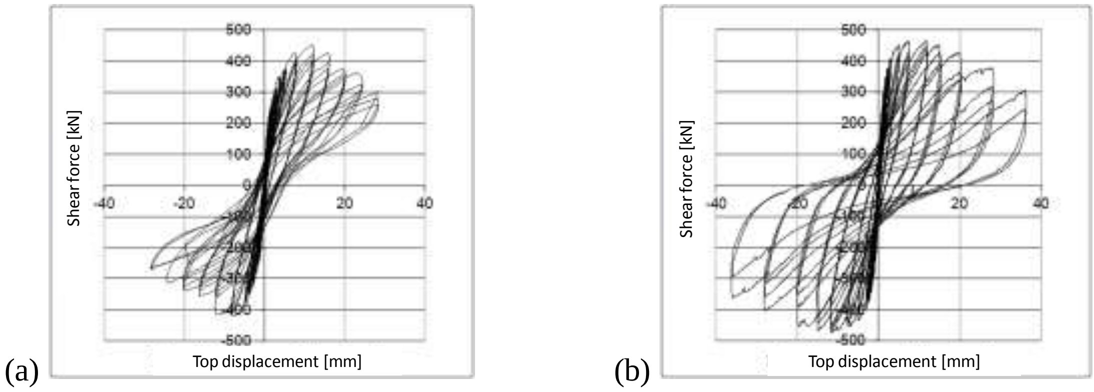

Figure 10. Cyclic response of (a) column strengthened using CFRP strips, (b) column strengthened using concrete jacket

4. Analytical estimation of the shear strength

An analytical prediction of the shear strength of all investigated columns was quite complicated and uncertain. Shear strength was calculated using three different procedures: 1) the procedure from Eurocode 2 [2], 2) the procedure proposed in the standard Eurocode 8/3 (EC8/3) [3] and 3) the procedure proposed at University of California, San Diego - USCD [4].

In general, all these procedures define the shear strength in the same way, taking into account: 1) the shear strength of a concrete without shear reinforcement (V_c), 2) the contribution of the compressive stresses to the increase in shear strength (V_N), and 3) the contribution of the shear reinforcement (V_w). However, the ways in which these mechanisms are considered are quite different. In the investigated case the estimated values of shear strength, particularly the contributions of the concrete, were significantly different. All these as well as the total predicted value of the shear strength V_{tot} at the base of the investigated column at the moment of its failure are summarized in Table 1. The total predicted shear strength V_{tot} is compared with the experimentally observed value V_{exp} in the last column of Table 1.

Table 1. Total shear strength and the contribution of different shear mechanisms, estimated using different analytical methods

METHOD	DISPL DUCTILITY	V_C [kN]	V_N [kN]	V_W [kN]	V_{tot} [kN]	V_{tot}/V_{exp}
EC2(EC8/2)	-	93	54	171	318 (171)	82% (44%)
EC8/3	3.9	117	110	146	373	96%
UCSD	3.9	83	110	171	364	93%

The values of the shear strength, determined according to UCSD and EC8/3, matched the experimental data quite well. An estimation of the column's shear strength according to EC2 was less accurate. The value of V_{tot} is significantly underestimated, since the EC2 neglects the contribution of concrete and compressive stresses when the demand exceeds the sum of these two contributions. In the investigated case this is evidently too conservative (values presented in the parentheses), since these two mechanisms contribute approximately half of the total shear strength. When they were taken into account the estimated shear strength was much larger and similar to the values predicted by the other two methods. Based on this and some other observations in similar columns, it can be concluded that current EC2 requirements are not adequate for estimation of the shear strength in hollow box bridge columns and similar structural elements (e.g. RC walls).

The shear strength was estimated also at the upper part of the column. Both of the standards EC8/3 and EC2 were less accurate. They predicted that the column would fail in its upper part, and that failure would occur prior to yielding of the longitudinal bars. This was not demonstrated by the experiment. Therefore it can be concluded that the UCSD method was the most accurate in the investigated case. More details about this analysis can be found elsewhere [5].

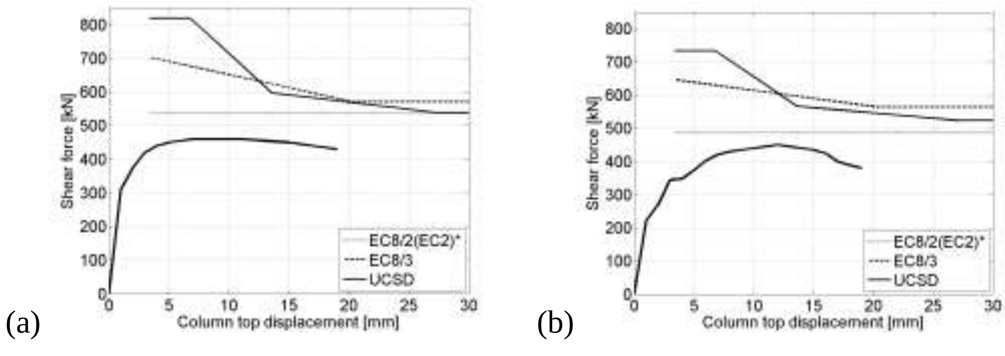

Figure 10. Shear strength of (a) column strengthened using CFRP strips, (b) column strengthened using concrete jacket

The shear strength of the strengthened columns was estimated in a similar way as for the as-built columns using the same analytical procedures. The contribution of the RC jacket and FRP wrap to the shear strength was added to contributions of other three mechanisms (V_C, V_N and V_W). The shear strength of CFRP wrap was estimated according to the [3] and [4]. In the case of RC strengthening the contribution of the jacket to the shear strength was determined taking into account its shear reinforcement in the same way as the V_w was defined. The shear strength in the case of the CFRP and RC jacketing is summarized and compared with the shear demand in Figure 11. All considered methods predicted that the used amount of jacketing was sufficient to prevent the shear failure, as it was later proved by the experiment.

5. Conclusions

The experimental and analytical studies of the cyclic response of reinforced concrete hollow box columns, which comprises several construction details, which are inappropriate for seismic regions are presented. The experiment of the as-built column demonstrated that in spite of the poor construction details it had relatively good displacement ductility capacity, which was acceptable for moderate seismic demand. This ductility was provided by the favourable hollow box cross-section with its large compression zone, by the low axial forces, and by the relatively high strength of the concrete.

The column was strengthened using concrete and CFRP jacketing. Both strengthening techniques successfully increased the shear strength of the column with minimum amount of wrapping. When the sufficient shear strength was provided, the other unfavourable types of failure induced by the other construction deficiencies were activated in both strengthened columns. Particularly critical was the buckling and the rupture of the longitudinal bars as well as the bar slip at the column base. The concrete strengthening was somewhat more efficient in improving the column ductility capacity and energy dissipation capacity. To improve the ductility capacity of the investigated type of column using CFRP wrapping, larger amount of CFRP strips was needed (note that the absolute minimum amount of CFRP strips was used).

The shear strength of columns was estimated analytically by using standard methods, which yielded quite different results. The best estimates of the type and location of failures were obtained by using the UCSD method. Based on the large differences between the considered procedures, as well as other results presented elsewhere, it can be concluded that the problem of shear strength and deformability is, in general, not adequately solved and it demands further studies. It may be appropriate to reconsider this problem in the future developments of the Eurocode standards.

6. Acknowledgements

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7. References

[1] ISAKOVIC T., BEVC L, FISCHINGER M., "Modeling the Cyclic Flexural and Shear response of the R. C. Hollow Box Columns of an Existing Viaduct", *Journal of Earthquake Engineering*, Vol. 12, No. 7, pp. 1120-1138.

[2] CEN, *Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings*. EN 1992-1-1. Comité Européen de Normalisation, Brussels, 2004.

[3] CEN, *Eurocode 8: Design of structures for earthquake resistance. Part 3: Strengthening and repair of buildings*. EN 1998-3, Euro Commit for Stand, Brussels, 2005

[4] PRIESTLEY M.J.N., SEIBLE F., CALVI G.M., *Seismic Design and Retorfit of Bridges*, John Wiley & Sons, 1997.