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CYCLIC RESPONSE OF I-SHAPE BRIDGE COLUMNS PRONE TO BUCKLING OF THE LONGITUDINAL REINFORCEMENT

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ABSTRACT

Many newer bridges in central Europe (Slovenia) are supported by I shape reinforced concrete columns. Although these columns usually comprise an adequate amount of the shear reinforcement, the lateral reinforcement is quite often insufficient to ensure the necessary confinement of the concrete core and to prevent the buckling of the longitudinal bars. The ductility capacity of such columns is therefore considerably reduced and their brittle failure can be expected when they are subjected to strong seismic excitation. Such columns were tested experimentally in order to define how the substandard lateral reinforcement influences their seismic response. Based on the results of this experiment an appropriate technique for their seismic strengthening was investigated. The CFRP strips combined with steel anchors were used in order to increase their ductility capacity.

Numerical analyses of the as-built and strengthened column were also performed. The feasibility of standard macro-models was investigated. It has been found that investigated models could describe the cyclic response of as-built and strengthened column with reasonable accuracy. The strength degradation between different cycles was possible to take into account using appropriate models for steel. The standard numerical models could not adequately simulate the in-cycle strength degradation.

INTRODUCTION

The new standards, which are based on the modern principles of the seismic design (e.g. CEN 2005a) introduced several novelties into the seismic design practice of bridges in central Europe. These changes are particularly related to the design of RC piers (columns), more specifically to their lateral reinforcement.

Before the new standards were introduced to the design practice, the lateral reinforcement in columns was usually designed taking into account only the shear demand. The other two functions of this reinforcement, confinement of the concrete core and prevention of the buckling of the longitudinal (flexural) reinforcement, were often inadequately considered or even neglected. This was the design practice even in many newer bridges. Therefore, in many existing bridges, the lateral reinforcement in columns is inadequate for the seismic regions. This can substantially reduce their ductility capacity, particularly when the cross-section of the columns has an unfavourable shape.

Typical examples are “I” shape columns, which are broadly used in the central Europe, since their construction is quite simple and more economical comparing to other columns’ shapes, which are more appropriate for the seismic regions. Their response is particularly critical in their weak direction,

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where relatively large amount of lateral reinforcement is necessary to provide an adequate ductility capacity.

To investigate this problem, an "I" shape column, which is typical for bridges in central Europe was investigated. It is reinforced by an adequate amount of the shear reinforcement (defined based on the capacity design procedure), however this reinforcement was still substantially smaller than that, which is required by the standard Eurocode 8/2 (CEN 2005a) to ensure the adequate confinement of the core of the column and to prevent the buckling of the longitudinal reinforcement. Moreover, the minimum requirements related to the construction details of the lateral reinforcement were not properly addressed.

In the paper cyclic experiment of such column is presented first. Then the experimental investigation of the strengthened column is described. While a number of different strengthening solutions for circular, rectangular, diamond shape or even hollow-box bridge columns are broadly available (e.g. Saiidi et al., 2001, Kawashima et al., 2001, Mo et al., 2004, Calvi et al., 2005, Isaković et al., 2012) the appropriate solutions for I shape columns are quite rare. In the investigated case, the CFRP wrapping was used, since this way of strenghtning did not increase the seismic demand in foundations and its construction was less complicated compared to other available solutions. However, even for this type of strengthening, the construction was not straightforward. Due to the unfavorable shape of the cross-section of the column, the appropriate solution of anchoring of the CFRP strips to the column had to be found. Two solutions were considered: carbon fiber anchor and steel bolts.

Finally numerical analyses of both types of columns, as-built and strenghtened, was performed. The feasibility of standard macro-models was investigated. Two types of these models were analysed: a) force-based fiber model – designated as "Beam with hinges" (Scott and Fenves, 2006) and the beam-column model with lumped plasticity (using the Takeda's hysteretic rules) in programme system OpenSees (McKenna et al., 2008).

EXPERIMENTAL INVESTIGATION OF THE AS-BUILT COLUMN

Deficiencies of the as-built column

The properties of the typical as-built column were identified based on the special study of existing viaducts supported by I-shape columns. It was found that in such columns typical normalized axial compression forces varied between 8% and 11% of the characteristic compression strength of concrete. In such cases lateral reinforcement should fulfil all the requirements of the standard Eurocode 8/2 (CEN 2005a) related to the confinement and the buckling of the longitudinal bars.

The required minimum mechanical reinforcement ratio of confinement reinforcement in columns of rectangular cross-section is $\omega_{w,min}$ = 0.12 (DCH structures). This means that a minimum amount of lateral reinforcement is ρ_w = 0.552 % as far as the column is reinforced using steel S500 and concrete C30/37. In columns, included into the study, this amount was in the range 0.21 % to 0.40 %.

The maximum allowed distance between stirrups along the column depends on the diameter of the longitudinal bars and dimensions of the column. For most of the investigated columns the maximum allowed distance was about 15 cm. However, in most of the analysed cases it was exceeded. The typical value was about 20 cm.

According to the EC8/2 (CEN 2005a), the maximum distance between engaged longitudinal bars is also limited. This distance cannot be larger than 20 cm. In some of the investigated columns this value was substantially exceeded.

The layouts of the lateral reinforcement, which are typically used in I-shape columns is presented in Figure 1. The outer stirrups are constructed as it is shown in Figure 1(a) and 1(b). The later was considered in the presented study. The typical shape of inner lateral reinforcing bars is presented in Figure 1(c).

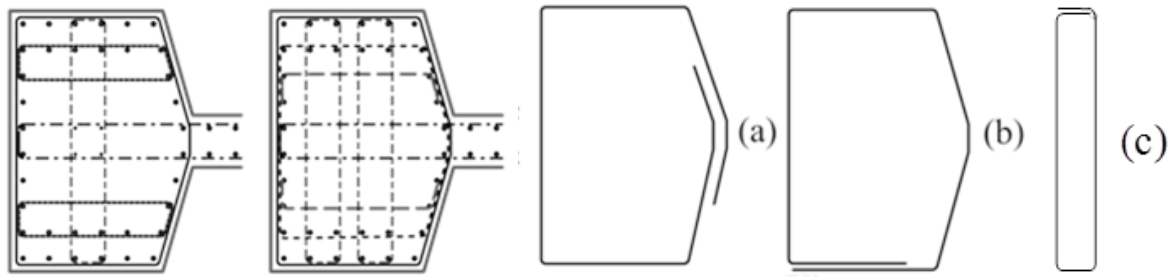


Figure 1. Typical layouts of the lateral reinforcement

Overview of the experiment

The cyclic response of the typical I-shape column (see Fig. 2) was tested. The main properties of the 1:4 scale model are presented in Figure 3. The test was performed in weak column direction. The height of the column was 2.9 m. The column was loaded at the height of 2.5 m. This corresponds to the average height of the prototype column of 10 m. The model comprised all the construction deficiencies described in the previous section.

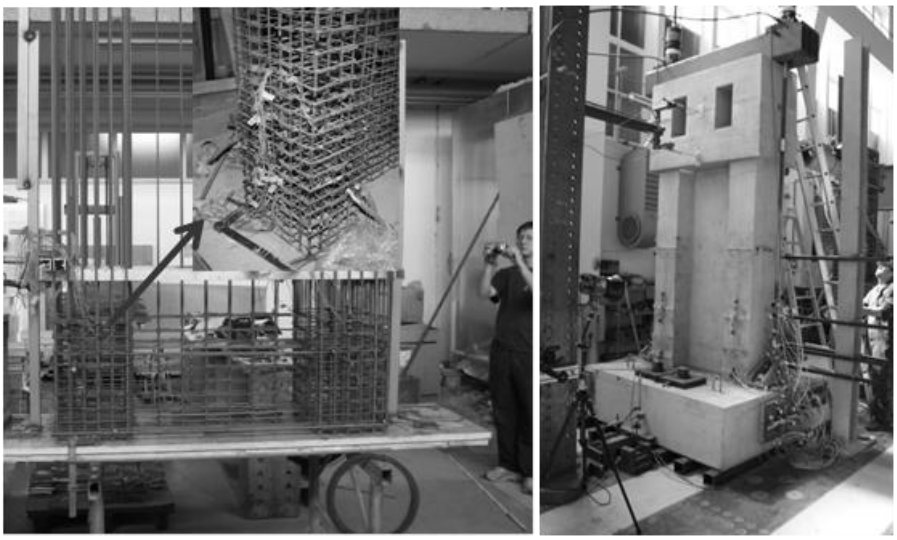


Figure 2. A tested specimen (as-built column)

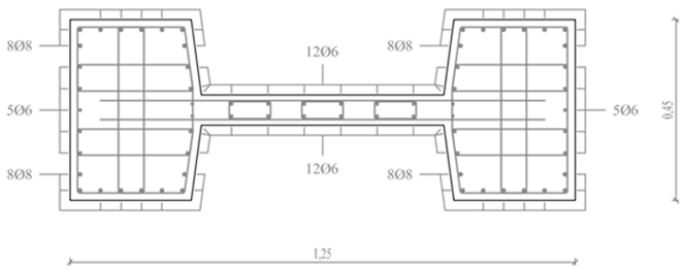


Figure 3. The reinforcement of the as-built column

The plain bars (steel S240) were used for lateral reinforcement. The diameter of the bars was 4.2 mm. The distance between them was 5 cm. In this way the same mechanical reinforcement ratio of 0.038 as in the typical prototype column was provided. It was substantially smaller than the minimum

amount of confinement reinforcement which is required by the standard EC8/2 (CEN 2005a). Nevertheless, it provided sufficient shear strength of column.

Cyclic response of the as-built column

In the beginning of the test there was no substantial damage of the column. Only some thin cracks were noticed at the bottom of the column at the region of the potential plastic hinge. The yielding of the longitudinal bars was observed at 1.4% drift. In the following cycles the maximum strength of the column of 200 kN was reached (see Fig. 4). It corresponded to 50 mm top displacement (2% drift).

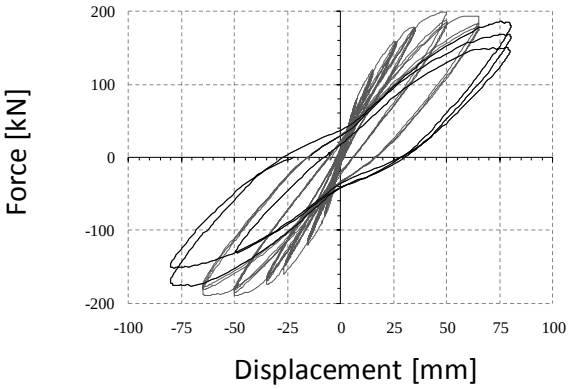


Figure 4. Cyclic response of the as-built column

In the next cycles the spalling of the cover concrete was initiated (Fig. 5(a)). It failed when the top displacement of 80 mm (3.2% drift) was reached (Fig. 5(b)). After that the buckling of the longitudinal bars was observed. The buckling of the longitudinal bars is presented in Figures 5(c) and 5(d) on the side where the lateral reinforcement was overlapped and on the opposite side, respectively. The experiment was terminated at 3.2% drift to prevent sudden brittle failure, since the strength was progressively deteriorated due to the buckling and the rupture of the longitudinal bars.

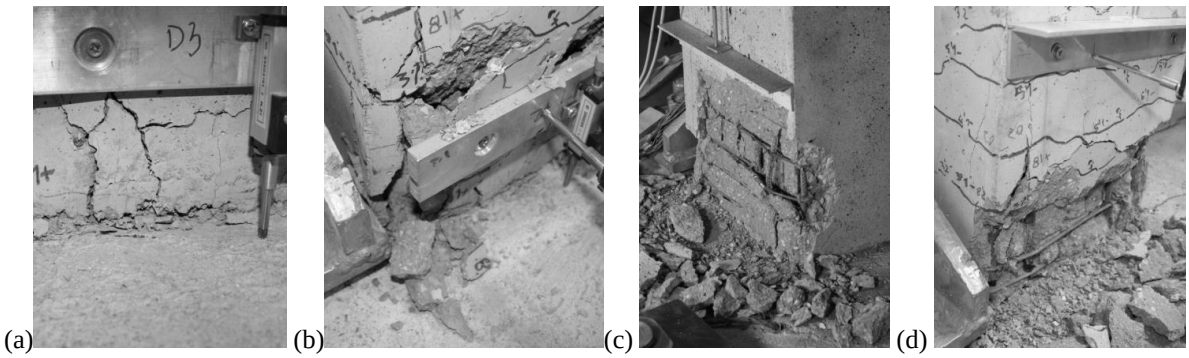


Figure 5. (a) The damage of the concrete cover was initiated at 2 % drift, (b) Concrete cover spalled at 3.2 % drift, (c) then the buckling of the longitudinal bars occurred, first at the side where the lateral bars were overlapped, (d) then on the opposite side

There were several reasons for buckling of the longitudinal bars. The first, most obvious reason was inadequate layout of the stirrups, which were constructed without any hooks. After spalling of the cover concrete, the efficiency of the lateral bars was therefore considerably reduced, because of their splitting.

The buckling of the longitudinal bars was also due to the insufficient amount of lateral reinforcement and due to the too large distance between bars along the column. This is illustrated in Figure 6, where two types of buckling can be observed. In the corners, the buckling of the longitudinal bars occurred between two consecutive lateral bars. That is an indication that the distance between stirrups was too large. The longitudinal bars more distant from the corners were buckled over several

stirrups. This is the consequence of the inadequate layout of the lateral bars and insufficient amount of transverse reinforcement.

It is evident form Figure 4 that yielding of the longitudinal bars was observed when the column top displacement of 35 mm was reached. Therefore it can be concluded that the column had relatively low displacement ductility capacity of $\mu_D = 80/35 = 2.3$.

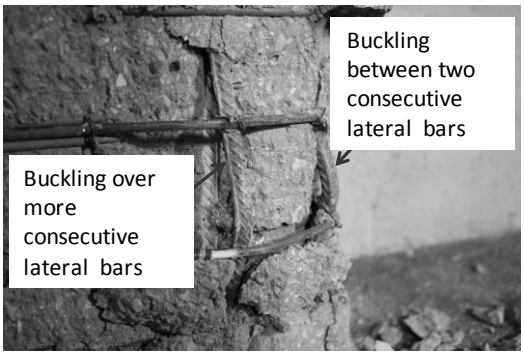


Figure 6. Different ways of buckling of the longitudinal bars

EXPERIMENTAL INVESTIGATION OF THE STRENGTHENED COLUMN

CFRP wrapping and anchoring of the strips to the column

The main purpose of the strengthening was to increase the ductility capacity of the column. The column was wrapped by CFRP sheets, since this way of strengthening is simpler and quicker than other available solutions. Considering the main purpose of the strengthening, the CFRP fibers were provided only in the horizontal direction. In this way the confinement and lateral support of the longitudinal bars were improved, but the stiffness and the flexural strength of the column were kept practically the same as in the as-built column. This was quite important in the investigated case; since the increased stiffness and strength of column could considerably increase the demand in the foundations (in that case the foundations should be strengthened as well). That is one of the main advantages of the CFRP wrapping against other possible solutions, which usually increase the stiffness as well as the strength.

The construction of CFRP strengthening, however, was not straightforward. It was not possible to wrap the complete column cross-section because of its unfavourable shape. Therefore the wrapping was provided only partly, around the flanges (see Fig. 7). The strengthening was provided only in the region of the potential plastic hinge (60 cm from the footing). CFRP sheets were anchored to the column flanges using carbon fiber anchors (see Fig. 7). Four CFRP anchors, at distance 12 cm between each other, were used to connect the CFRP wrap with the column. This way of anchoring was tested on the wrapped damaged column. The purpose of this test was not to improve the response of the damaged column (since the brittle failure was obtained and consequently the response of the damaged column was quite difficult to improve), but to test the efficiency of the anchorage of the CFRP strips. Therefore, the response of the wrapped damaged column is not provided in the paper, except the response of the anchors.

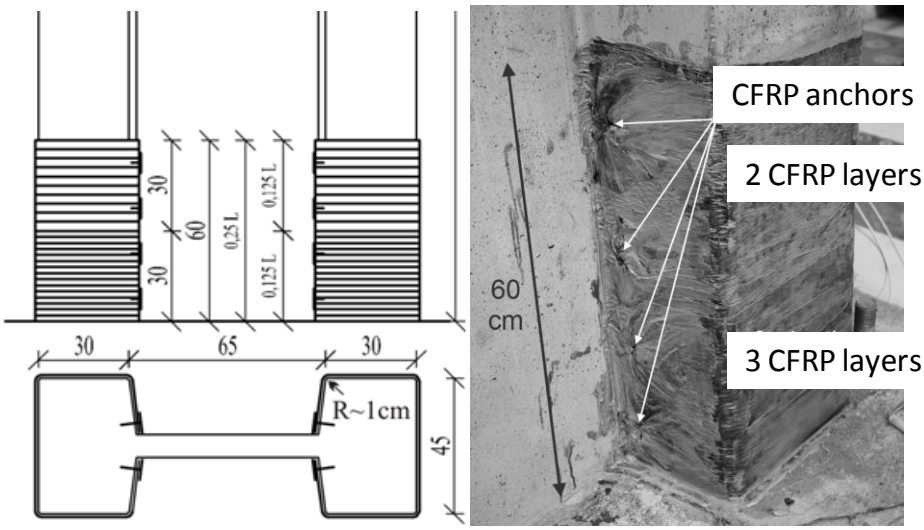


Figure 7. Seismic strengthening of the I shape column using CFRP strips attached to the columns using CFRP anchors

The CFRP anchors did not provide the adequate connection between wrap and the column flanges. When the larger flexural deformations were occurred in the critical region near the foundations, the polymer matrix was cracked below the most bottom anchor. In between the anchor and the foundation (at the most critical and most loaded part of the column) the CFRP sheets and the column were attached only as long as the epoxy resin between them provided sufficient resistance. Since it was not sufficient (comparing to the demand in this critical region) the CFRP sheets were de-bonded from the column. Considering the results of this test it was decided to change the way of anchoring in order to provide the more uniform connection of the CFRP sheets and the column. It was provided by the steel plate, which was bolted to CFRP sheets and the column as it is shown in Fig. 8.

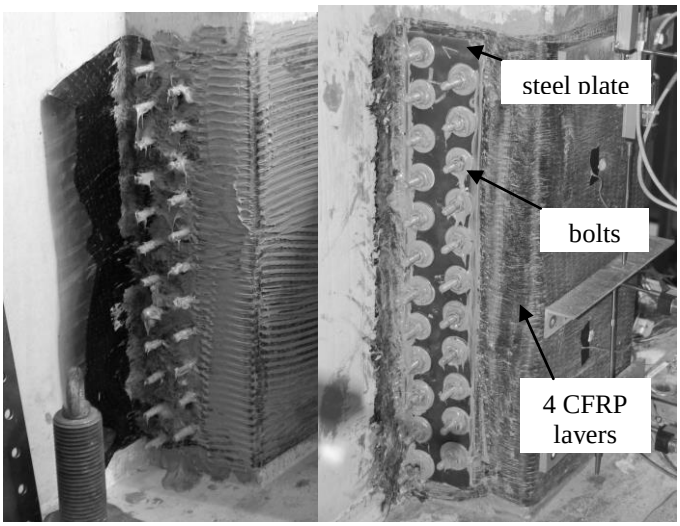


Figure 8. Anchoring of CFRP wrap by bolted steel plate; scheme of the steel plate and bolts

Cyclic test of the strengthened column

The strengthening by 4 CFRP layers was provided at the bottom 60 cm of the column. CFRP sheets were anchored as it was described in the previous subsection. In the first cycles the response of the strengthen column was similar to that of the as-built column (see Fig. 9). The yielding of the longitudinal bars corresponded to 1.4% drift (35 mm of the column top displacement). The damage of

the column was spread uniformly over the plastic hinge region. The maximum strength of 195 kN was achieved approximately at the same drift as in the as-built column (2%). The strength was then gradually decreased, and at the 5% drift (125 mm) was reduced to 72% of the maximum value. This was characterized as the column failure, although the column was still capable to resist 6% drift, when the strength was reduced to 50%. The failure occurred gradually due to the considerably damaged concrete core and the rupture of the longitudinal bars. The displacement ductility of 3.6 was achieved.

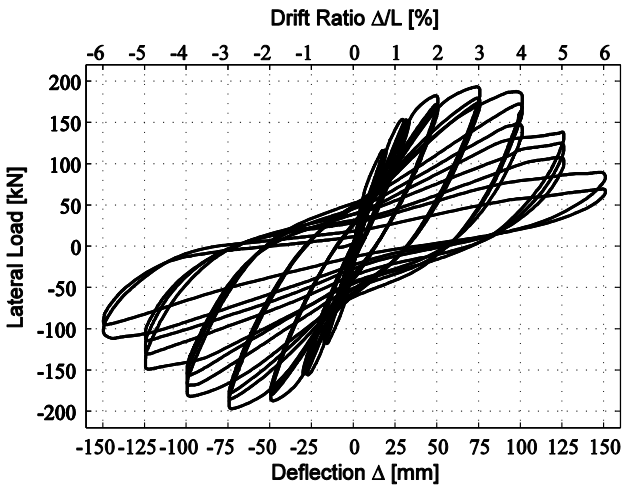
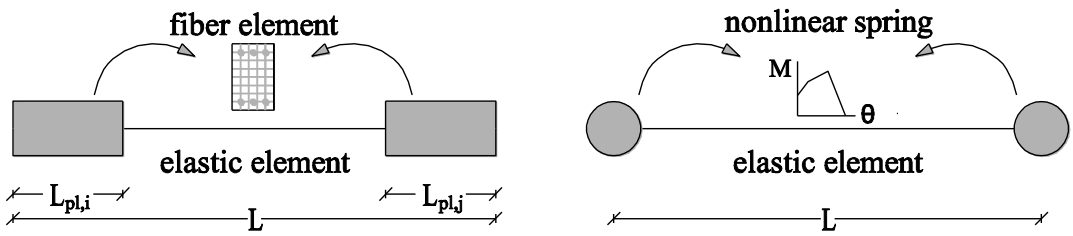


Figure 9. The cyclic response of strengthened column

NUMERICAL MODELLING OF TESTED COLUMNS

Two standard numerical models were used (see Fig. 10) to model the as-built and the strengthened column: a) force-based fiber model – designated as “Beam with hinges” (Scott and Fenves, 2006) and b) the beam-column model with lumped plasticity (using the Takeda’s hysteretic rules) in programme system OpenSees (McKenna et al., 2008).



(a) (b)

Figure 10. Numerical models: fiber model (a) and lumped plasticity model (b)

Fiber model

Fiber element "Beam with hinges" (BWH) integrated in OpenSees platform (McKenna et al., 2008) was used to model the nonlinear cyclic response of specimens. The plasticity was distributed over the defined length of the plastic hinge L_{pl} at the column base. It was assumed that the rest of the column would respond elastically. The fiber mesh was defined based on the positions of longitudinal reinforcement and dimensions of the specimen. Three types of uniaxial stress-strain relationships for concrete were considered. The response of unconfined concrete was defined using the standard stress-strain diagram. The Mander's model (Mander et al., 1988) was used to define the stress-strain relationship of the confined concrete in as-built specimen. The stress-strain model defined by Lam and Tang (2003) was used for confined concrete in the strengthened column. Due to the specific shape of

the cross-section of investigated column, the effective area, originally defined for rectangular columns, was modified as it is presented in Figure 11. This area was increased for about 30% due to the favourable effect of the cross-section web to the confinement of the concrete in the flange (see the area denoted as increased confinement in Figure 11).

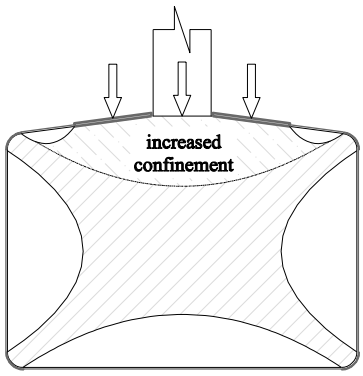
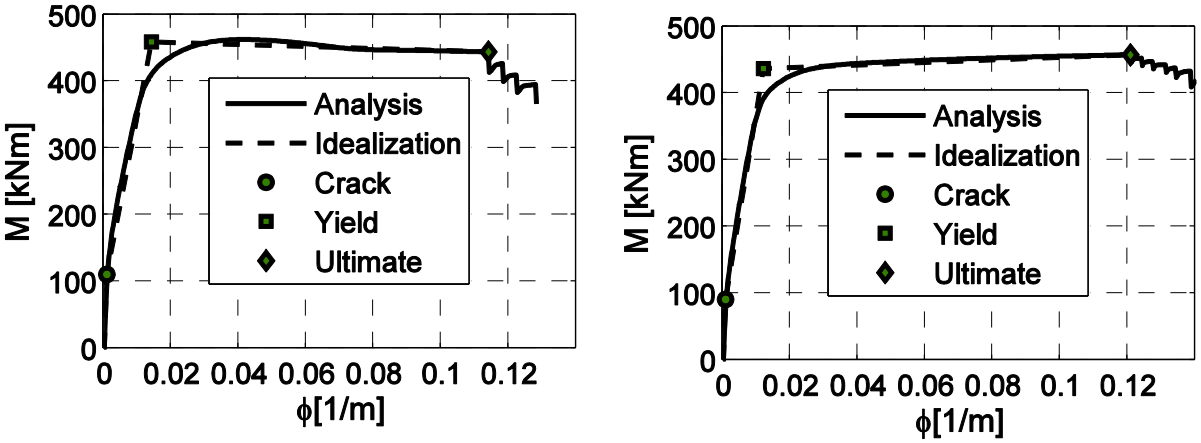


Figure 11. Effective confinement area of the strengthened specimen

The advanced material model for steel "ReinforcingSteel" (Kunnath et al., 2009) was used to model the stress-strain response of longitudinal reinforcement. The Coffin-Manson fatigue behaviour was taken into account utilizing the damage factors a=0.506 and C_f=0.26. The strength reduction constant of magnitude C_d=0.30 was also considered. A special attention was devoted to the buckling of longitudinal reinforcement. İn the investigated case it was considered using the Dhakal and Maekawa (2002) model, where the response of the steel (longitudinal bars) in compression is defined taking into account coefficient α, which is in the range between 0.75 and 1.0 (these values are proposed by the authors). In the investigated case the value of 0.8 was used. The slenderness ratio of 6.3 and 8.3 was considered for bars $\phi 8$ and $\phi 6$, respectively. The moment-curvature relationship obtained using previously described models for as-built and strengthened specimen is presented in Fig. 12.



(a) (b)

Figure 12. Moment-curvature of as-built (a) and strengthened (b) specimen cross-section

The length of plastic hinge L_{pl} was determined according to Eurocode 8/2 (CEN 2005a). It was 33 cm and 31 cm in as built and strengthened column, respectively. Effective moment of inertia of the elastic part of the element was reduced to take into account cracking of the concrete. It was estimated according to Annex C of Eurocode 8/2 (CEN 2005a). Effective moment of inertia was reduced to

31% and 35 % of the moment of inertia of gross cross-section for as-built and strengthened column, respectively.

Lumped plasticity model

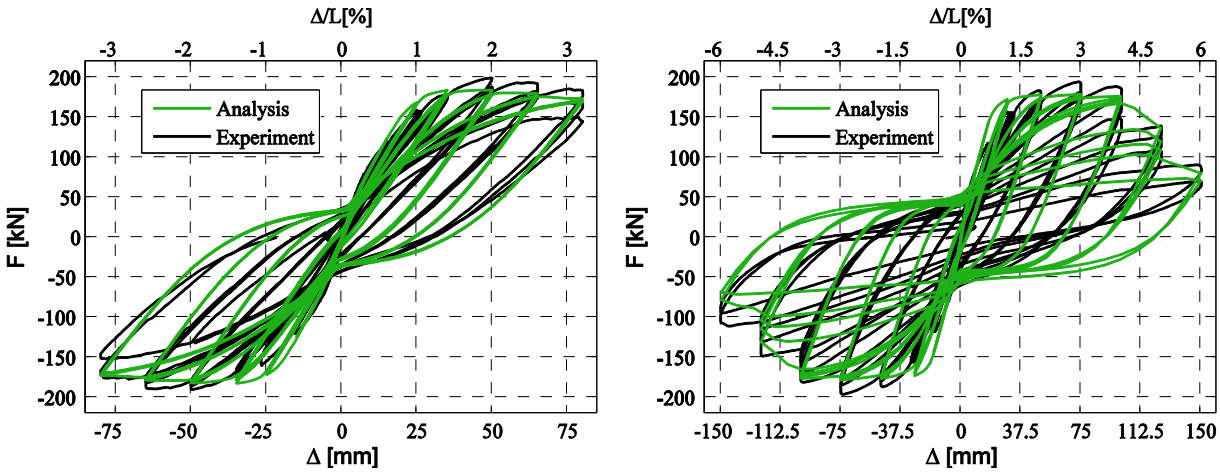
When the lumped plasticity model was used to analyse both investigated columns, the non-linear cyclic response was defined using non-linear moment-rotation spring at the column base. The behaviour of this spring was defined using Takeda hysteretic rules (Takeda et al., 1970). The post-capping stiffness was also considered (Takahashi, 2009). The three-linear moment-rotation envelope was defined based on the moment curvature analysis (see Fig. 12), where the actual moment-curvature relationship was idealized as it is shown in Fig. 12. The ultimate and yield chord rotations were defined according to Eurocode 8-3 (CEN 2005b) using the equations A.4 and A.10a. The length of the plastic hinge was the same as in the case of the fiber element.

Comparison of experimental and numerical results

When the analysis of the as-built column was performed using the fiber model, quite good agreement with the experiment was obtained as long as the buckling of the longitudinal bars did not occur (Fig. 13a). The buckling was not modelled properly. In spite of the used advanced material model for steel, it was not possible to take into account the substandard details of transverse reinforcement (the transverse reinforcement was without hooks) properly. Consequently, the ultimate strain of the confined concrete was overestimated and the occurrence of the buckling of the longitudinal bars was shifted. This indicates that concrete material model needs some modifications.

In the case of the strengthened column the agreement of the analysis and experiment was better (Fig. 13b). The ultimate strain defined by Lam and Teng (2003) (taking into account increased effective confined area – see Fig. 11 for more details) was somewhat more accurate. Thus the calculated post-capping response was correlated somewhat better with the experiment (comparing to the as-built column).

The assumed damage parameters for steel, used in the fiber model, were not able to identify the failure of longitudinal reinforcement in both investigated columns. In the experiments the fracture of the bars was accelerated due to the buckling phenomena, which was not properly taken into account in the analysis. Therefore further investigation is needed to adequately model the low-cycle fatigue.



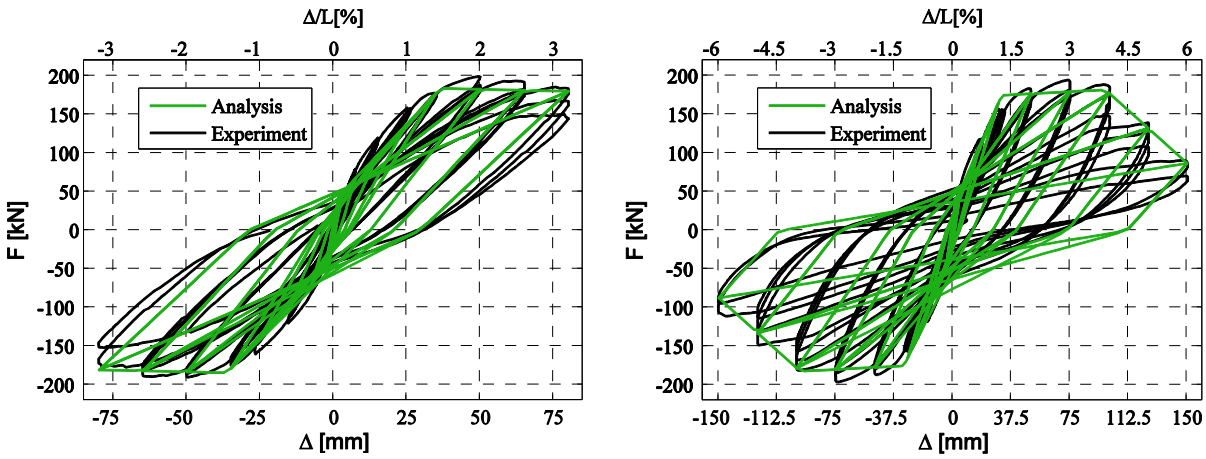
(a) (b)

Figure 13. Cyclic response of (a) the as-built and (b) the strengthened column – Comparison of the fiber model and the experiment

The agreement of the experiment and analysis using the lumped plasticity model was similar as in the case of the fiber model. The best agreement was obtained when parameter $\alpha = 0.7$ and $\alpha = 0.4$ (which defines the unloading stiffness) was considered for as-built and

strengthened column, respectively (Fig. 14). The lumped plasticity model exhibited similar deficiencies as the fiber model. It was not able to describe the in-cycle strength degradation of the as-built column and its progressive brittle failure due to the buckling of the longitudinal bars.

In the case of the strengthened column the accuracy of the lumped plasticity model was also similar to that of the fiber model (see Fig. 14(b)). The strength degradation between cycles was modelled properly, but the in-cycle strength degradation was not possible to take into account.



(a) (b)

Figure 14. Cyclic response of (a)the as-built and (b) the strengthened column – Comparison of the lumped plasticity model and the experiment

It can be concluded that both standard numerical models are capable to describe the envelopes of the cyclic response of as-built and strengthened column with reasonable accuracy (Fig. 13 and 14). The strength degradation between different cycles was possible to take into account using appropriate models for steel. However, these models were not able to adequately simulate the in-cycle strength degradation, which was caused by progressive brittle failure due to the buckling of the longitudinal bars.

CONCLUSIONS

Cyclic response of I shape bridge column, which is typically used to support newer bridges in Central Europe was experimentally analyzed. The typical column was reinforced by inadequate amount of the lateral reinforcement (according to the Eurocode 8 standard), which was not properly constructed. The column was tested up to the failure. The progressive and brittle failure was caused by the buckling of the longitudinal bars. This considerably affected the column displacement ductility capacity.

The seismic strengthening of the investigated typical column using CFRP strips was also investigated. The construction of the wrap was not straightforward. Due to the unfavourable shape of the column cross-section, an appropriate solution for anchoring of the CFRP strips to the column had to be found. Two solutions were considered: carbon fiber anchor and steel bolts. Carbon fiber anchors were found to be inadequate, since they did not provide the uniform connection of the CFRP strips and column. Steel plate and steel bolts were more efficient since they successfully prevented splitting of the wrap and the column.

Finally numerical analyses of both types of columns, as-built and strenghtened, was performed. The feasibility of standard macro-models was investigated. Two types of these models were analysed: a) the force-based fiber model and b) the beam-column model with lumped plasticity. It was found that both standard numerical models were capable to describe the envelopes of the cyclic response of as-built and strengthened column with reasonable accuracy. The strength degradation between different cycles was possible to take into account using appropriate models for steel. However, these models were not capable to adequately simulate the in-cycle strength degradation, which was caused by progressive brittle failure due to the buckling of the longitudinal bars.

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