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SEISMIC STRENGTHENING OF RECTANGULAR RC COLUMNS USING PRESTRESSED CFRP WRAP

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Abstract

Although CFRP wrap has become a standard technique for retrofit and strengthening of different types of reinforced concrete (RC) structures, its application to rectangular RC columns with large cross-section aspect ratio, where the improvement of the confinement in the plastic hinge zone is needed, may be quite insufficient. Even though when the significant amount of CFRP layers are typically provided, the concrete is crushed along the longer side of the cross-section and the longitudinal bars are buckled in such cases.

Since the CFRP wrapping has many advantages comparing to e.g. concrete and steel strengthening, an attempt has been performed to increase its efficiency in cases when the enhancement of the confinement is needed in rectangular columns with large cross-section aspect ratio. To achieve this goal the CFRP fibres were pre-stressed and the cover concrete in the plastic hinge zone of the column was removed. The new technique of pre-stressing the fibres has been proposed.

The efficiency of the new strengthening technique was tested by experimental studies. Two rectangular reinforced concrete columns (non-strengthened and strengthened) with insufficient amount of lateral reinforcement and overlapping hoops were tested. The examined specimens were designed taking into account typical deficiencies that were identified in bridges with barbell cross-sections. Rectangular columns (b/h = 30/40 cm) were used to simulate the flanges of bridge columns in the scale 1:4. They were reinforced by stirrups (ϕ8/12cm), which did not provide sufficient support to prevent buckling of the longitudinal reinforcement (6ϕ16).

The examined column was strengthened by CFRP sheets. Upon the removal of the cover concrete, the rough concrete surface was smoothened with the layer of epoxy resin in the strengthened column. Before the installation of the jacket (3 layers of CFRP fabric) a special tensioning system was constructed. It consisted of two steel rods (h=500 mm, D =65mm) which were connected by bolts. The level of pre-stressing of CFRP fibres was about 20 % of the effective rupture strain of the CFRP.

The result of the cyclic tests under constant vertical load showed that pre-stressed wrapping was very efficient. The drift capacity was increased for more than 50 %. The CFRP wrap ruptured on both sides of the strengthened column, about 5 cm from the bottom of the column at the locations of buckled bars. Afterwards the rupture of longitudinal reinforcement occurred. Only local buckling (between two successive stirrups) was identified after the removal of the CFRP wrap, whilst in case of the non-strengthened column the global buckling (over three successive stirrups) has occurred. The energy dissipation capacity of strengthened column was significantly increased (for more than two times, comparing to the non-strengthened column).

Keywords: seismic retrofit; rectangular RC column; active confinement; prestressed CFRP wrap; removed concrete cover

1. Introduction

There are many bridges in central Europe that are supported by barbell shape columns, which often do not meet all requirements of the modern standards for the design of earthquake resistant structures. For example the transverse reinforcement in such columns often cannot prevent the buckling of the longitudinal reinforcement and cannot provide sufficient confinement of the concrete core. Typically the hooks of the stirrups are not properly constructed [1]. The strengthening of such columns is not straightforward due to the unfavourable aspect ratio of flanges dimensions. Traditional techniques of passive strengthening are often inefficient in such cases. Typically the passive strengthening cannot provide efficient confinement at longer edges of the flanges. The cover concrete between the wrap and the core of the cross-section is typically crashed at relatively small compression stresses.

In the paper a new approach for the seismic strengthening of such columns by means of CFRP wraps is presented. The main goal of the strengthening was to provide the sufficient confinement and to postpone the buckling of the longitudinal bars in the critical region in order to increase the column's drift (ductility) capacity. The CFRP sheets were prestressed to increase their efficiency. The cover concrete was removed and replaced by thin layer of the epoxy resin. The column's corners were rounded. The largest possible radius of the corners was provided.

The efficiency of the new strengthening technique was experimentally tested. Beside the strengthened column the non-strengthened column was also examined in order to be able to evaluate the improvements in the response of the strengthened column. These experiments are presented in Section 2 and Section 3 for non-strengthened and strengthened column, respectively.

Both specimens were tested cyclically under constant axial force provided by two vertical jacks. The displacements were imposed according to loading protocol (see Fig. 1 for details) proposed by FEMA 461 [2] with actuator of maximum load and displacement capacity of 250 kN and ±200 mm, respectively. Both specimens were loaded at the height 1.8 m from the column base.

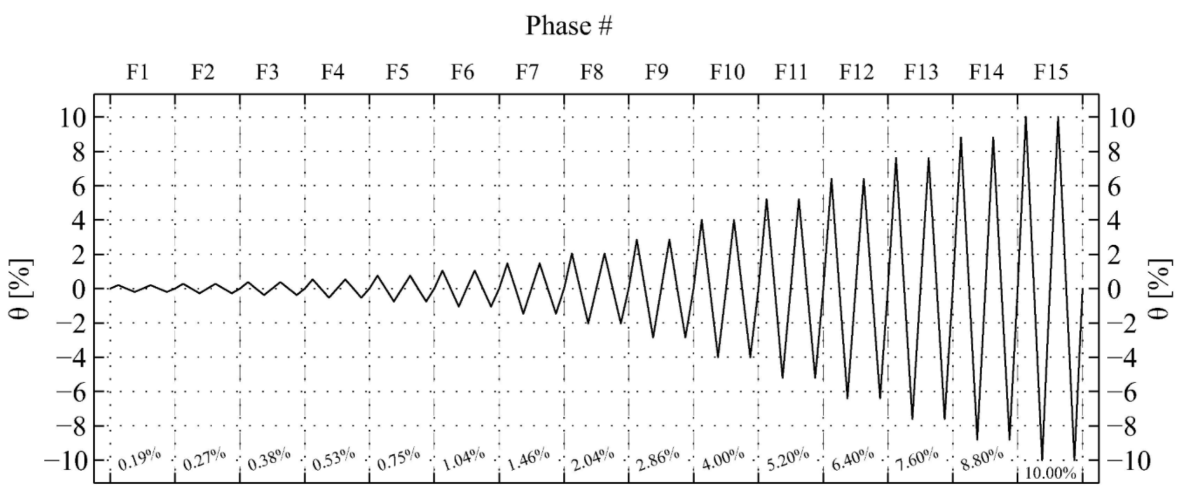

Fig. 1 – Loading protocol used for non-strengthened and strengthened column

2. Experimental investigation of the non-strengthened column

2.1 Overview of the experiment

The tested rectangular column (b/h=30/40 cm) represent the flange of the typical I-shaped bridge columns [3] in the scale 1:4. The specimen was designed taking into account typical deficiencies that were identified in bridges with barbell cross-sections in central Europe. The amount of mechanical reinforcement ratio provided in the column ($\phi 8$/12 cm) was about 5.5% in the weak direction (see Fig. 2). The distance between the stirrups s = 12 cm resulted in oversized slenderness ratio of s/d_{bL} = *12/1.6* =7.5. For example in EC8/2, the maximum value of 6 is allowed. Central longitudinal bars were not supported with cross ties (see Fig. 2). The stirrups were not terminated by hooks. Instead of that they were overlapped along the longer edge of the cross-section. The amount of longitudinal reinforcement was ρ_l=1% (6/ϕ16), which is typical for barbell bridge piers.

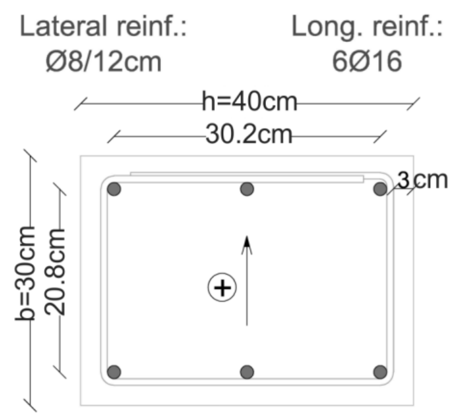

Fig. 2 – The cross-section of the non-strengthened column with the notation of the positive drift direction

The average concrete compressive strength f_{cm} measured on the cylinder on the day of the testing was 46.5 MPa. With the respect to the measured concrete strength the average level of normalized axial force was ν=10%. The mean characteristics of lateral and longitudinal reinforcement are provided in Table 1.

Table 1 – Mean material characteristics of lateral and longitudinal reinforcement in the non-strengthened column

Lateral reinforcement $\phi 8$		Longitudinal reinforcement $\phi 16$	
f_y [MPa]	495	f_y [MPa]	541
f_t [MPa]	577	f_t [MPa]	691
ε_y [%]	0.25	ε_y [%]	0.29
ε_u [%]	7.48	ε_u [%]	8.96

Notations:

f_y – mean yield strength of the reinforcement
f_t – mean tension strength of the reinforcement
ε_y – mean yield strain of the reinforcement
ε_u – mean strain at the tension strength of the reinforcement

2.2 Cyclic response

The relationship between the lateral force F and the rotation θ recorded during the experiment is presented in Fig. 3. The maximum lateral resistance was achieved at 2.9% drift. At that drift amplitude flexural cracks were detected at the heights 12 cm and 25 cm from the column base in the vicinity of lateral reinforcement.

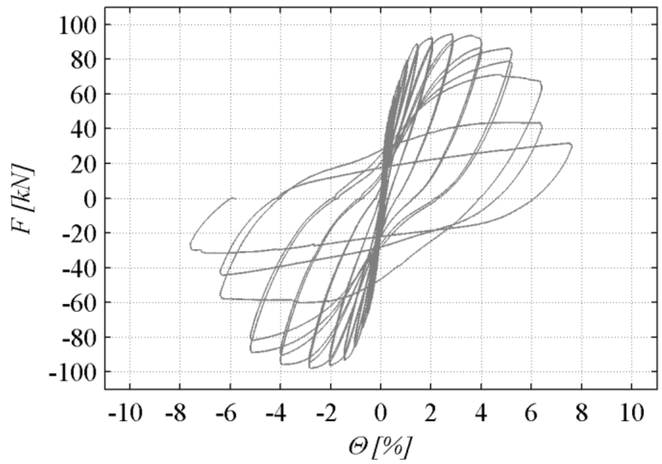

Fig. 3 – The cyclic response of the non-strengthened column

The spalling of the concrete was initiated at 4.0% drift amplitude at the height of 10 cm above the column base. The progressive spalling of the concrete was observed at 5.2% drift amplitude. The local buckling of the corner bars was initiated (see Fig. 4). Consequently the progressive reduction of the strength was initiated at this phase.

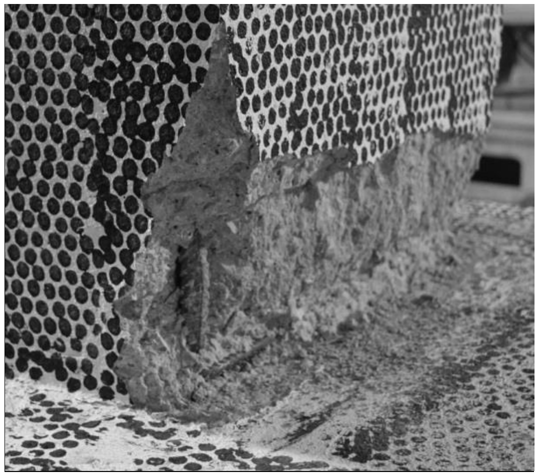

Fig. 4 – Local buckling of corner bar was initiated at 5.20% drift

Fig. 5 – Significant damage of the concrete core and global buckling of longitudinal reinforcement after the second cycle at 6.40% drift amplitude

In the first cycle at 6.4% drift amplitude the strength was considerably reduced to about 70% of the maximum observed strength. In the second cycle the strength was further considerably decreased (to about 60% of F_{max}), due to the considerable damage of the concrete core and buckling of the longitudinal bars (see Fig. 5). The strirrups without hooks were opened and were not able to support longitudinal bars.

Following the previously described observations it can be concluded that near collapse state was reached at about 5.2 % drift, while the failure occurred at 6.4% drift. This was confirmed in the last cycles of the test. At 7.6% drift the strength of the column was only 30% of the maximum value. The buckling of the longitudinal bars was significantly increased. The global buckling was observed and was more profound on the edge of the column were the stirrups were ovelapped (see Fig. 6). No rupture of longitudinal reinforcement was noticed at end the test.

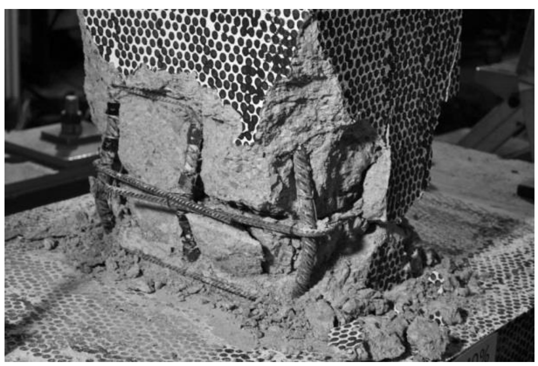

Fig. 6 – Extensive damage of the concrete core at the 7.60% drift amplitude; the global buckling of the longitudinal reinforcement

3. Experimental investigation of the strengthened column

3.1 Short description of the strengthening technique

To provide adequate confinement, which is required by the standard EC8/2, 3 layers of CFRP sheets were applied at the height of 50 cm above the column foundations. The thickness of the single CFRP layer *t* was 0.129 mm, with rupture strain ε_{rupt} of 1.7 % and elasticity modulus *E* of 230 GPa.

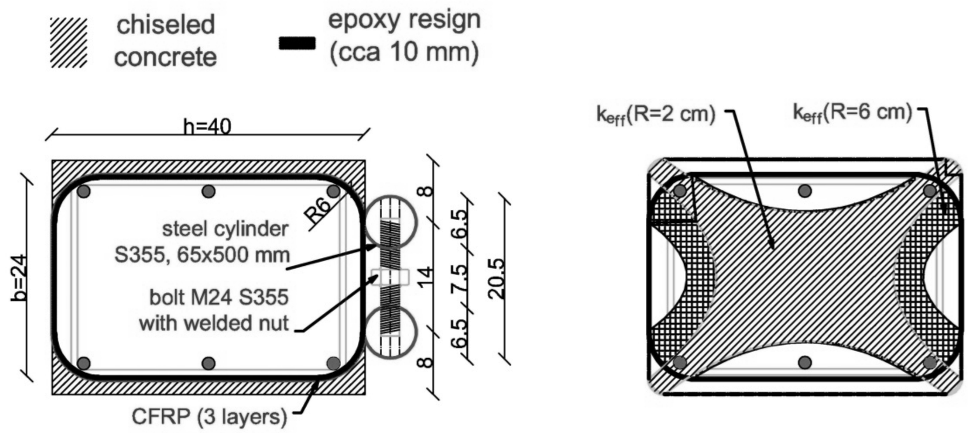

Fig. 7 – Plan view of the strengthened column with pretensioning system (left) and the comparison of effective confinement area contained by four parabolas in case of typical (R = 2 cm) and refined (R = 6 cm) level of corner radius with removed concrete cover (right)

In the non-strengthened column the inadequate confinement was provided along the longer edge of column due to the insufficient amount of stirrups, which were constructed without hooks. In the previous research it was observed that passive CFRP wrapping cannot considerably improve the confinement along this edge; their usage was for rectangular columns was generally not recommended [5]. Typically the cover concrete between the wrap and the concrete core was crashed at relatively small compression stresses. To avoid the damage of the cover concrete, which further reduce the effects of the wrapt the concrete cover was removed all over the height of 55 cm above the foundations (see Fig. 7-left). The rough concrete surface was smoothened with the epoxy resin (cca *d*=10 mm). The effectiveness of the wrap was further improved rounding the column corners with a radius of 60 mm. In this way the concrete core area, which is theoretically confined was increased for about 30%.

In order to improve the confinement and to prevent the buckling of the longitudinal bars a special system for the prestressing of the CFRP sheets was designed and applied. A tension force was applied to the CFRP sheets by means of two steel cylinders (ϕ65 mm, *h*=500 mm). They were connected by three fasteners with left and right thread of steel grade S355 with the pre-set torque wrench (see Fig. 7 and 8). The level of relative displacement between steel cylinders was monitored by LVDT and Vernier caliper during the prestressing and cyclic loading. The level of pretensioning was 20% of the effective rupture force of the CFRP sheets.

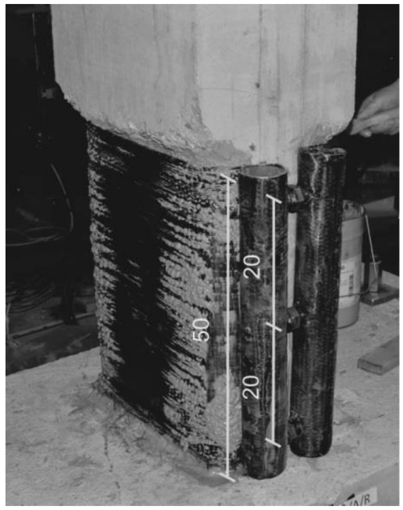

Fig. 8 – Side view of the strengthened column just after the tension stress was applied; significant leakage of the epoxy resin through CFRP jacket can be seen due to the pretension

The impregnation and application of the CFRP wrap was performed segmentally. Firstly, only the edges of the CFRP sheets were impregnated with 2-component epoxy resin. They were wrapped around the both steel cylinders at the length of one cylinder circumference. After that they were dried. Then, the remaining part of the CFRP was impregnated and applied to the plastic hinge region of the column together with the pretensioning system. Finally, the sheets were prestressed by using torque wrench prior the hardening of the epoxy (see Fig. 8).

3.2 Overview of the experiment

In the strengthened column the same amount and detailing of lateral and longitudinal reinforcement was used as in the non-strengthened column. The corresponding steel properties are reported in Table 2. The mean compressive concrete strength f_{cm} measured on the cylinder on the day of the experiment was 36.9 MPa (ν =13%). The loading protocol and level of normalized axial load with respect to the characteristic concrete C30/37 was the same as in the non-strengthened specimen.

Table 2 – Mean material properties of lateral and longitudinal reinforcement in the strengthened column

Lateral reinforcement $\phi 8$		Longitudinal reinforcement $\phi 16$	
f_y [MPa]	554	f_y [MPa]	600
f_t [MPa]	668	f_t [MPa]	709
ε_y [%]	0.30	ε_y [%]	0.30
ε_u [%]	10.12	ε_u [%]	7.56

For notations please see Table 1.

3.3 Cyclic response

The measured lateral force F rotation θ relationship is presented in Fig. 9. The maximum lateral resistance was obtained at 5.2% drift amplitude. It was about 15% smaller in the direction where the compression stresses were applied at the edge of the column with overlapped stirrups. At 6.4% drift the crashing of the concrete under the CFRP wrap was initiated all over the height of 10 cm above the foundation. At the same drift amplitude minor buckling of the CFRP jacket occurred in the vicinity of the central bars (see Fig. 10).

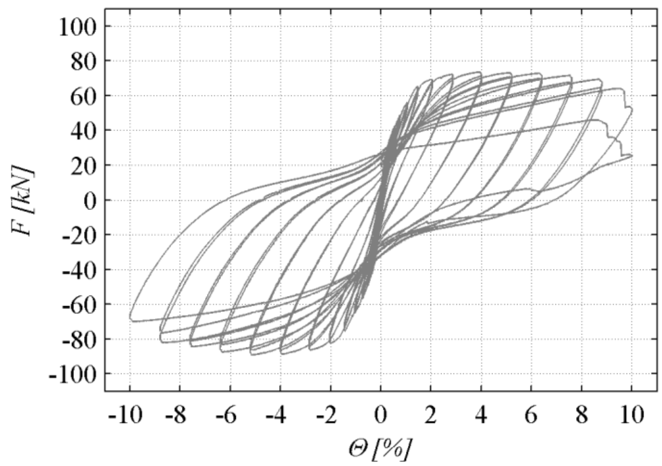

Fig. 9 – The cyclic response of the strengthened column

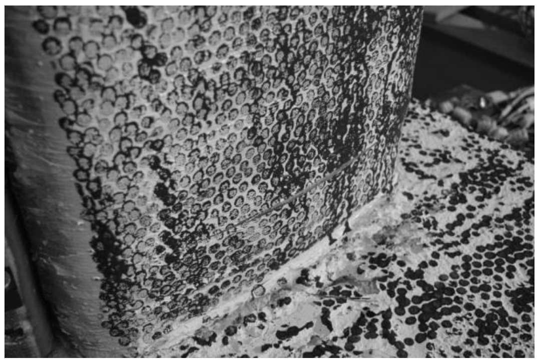

Fig. 10 – The onset of buckling of the CFRP jacket in the vicinity of the central longitudinal bar at 6.40% drift amplitude

At 7.60% drift amplitude deformations of the CFRP sheets were further increased and modest rupture occurred (see Fig. 11). However, this had no significant influence to the strength of the column. The CFRP sheets were ruptured and deboned from the column in the last phase of the test at 9.7% drift amplitude (see Fig. 12). Three longitudinal reinforcing bars were ruptured after the significant local buckling. The strength of the column before the failure was about 70% of the maximal observed strength. After the test the jacket was removed. Only local buckling of the longitudinal bars was observed on both sides of the column. The integrity of the concrete core was remarkably preserved (see Fig. 13).

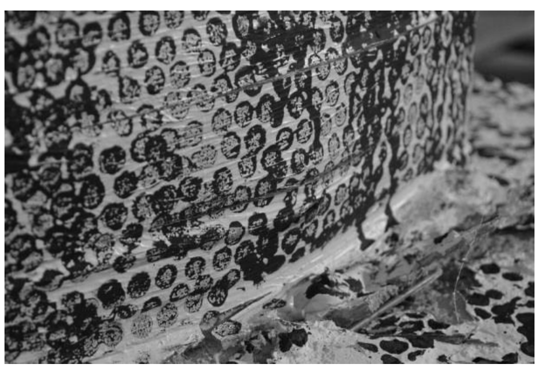

Fig. 11 – Modest rupture of CFRP wrap at 7.60% drift amplitude

Fig. 12 – Debonding of the CFRP wrap occurred in the last phase of the test

Fig. 13 – The damage of the specimen after the CFRP wrap was removed

4. Comparison of the response of non-strengthened and strengthened column

Based on the experiment, reported in Section 2 it can be concluded that near colapse stage in non-strengthened column occured at 5.2%, while the colapse was observed at 6.4% drift. The failure was brittle and occured due to the significant damage of the concrete core and global buckling of the longitudinal reinforcement. Note that none of the bars were ruptured.

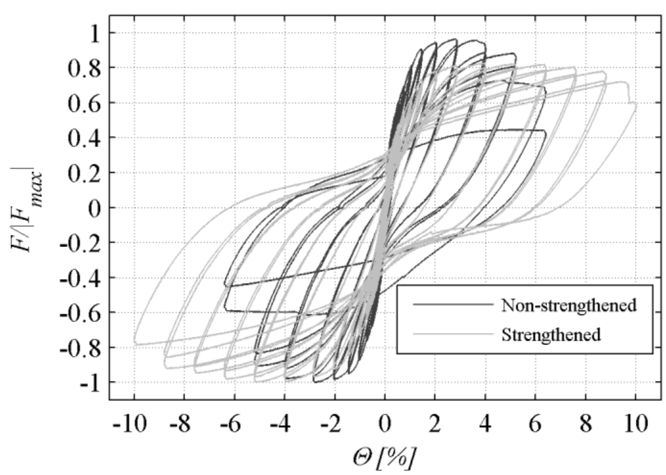

Fig. 14 – Comparison of normalized force F/F_{max} and rotation θ of the non-strengthened and strengthened column

The strengthening considerably improved the response (see Fig, 14, where the cyclic response of non-strengthened and strengthened column are compared). The maximum strength was observed at 5.2% drift. The failure occurred at 9.7 % drift due to the buckling and the rupture of the longitudinal reinforcement. The concrete core was only moderately damaged. The drift capacity was obviously considerably improved. .

5. Conclusions

The non-strengthened and strengthened rectangular cantilever columns with large aspect ratio of the cross-section were tested cyclically under constant axial load in scale 1:4. The non-strengthened column had inadequate amount of improperly constructed reinforcement, e.g. the distance between the stirrups was too large; longitudinal reinforcement was not properly supported by stirrups; the stirrups had no properly constructed hooks. These are the typical deficiencies, which can be found in the bridge columns with barbell cross-section in the central Europe.

New technique for the seismic strengthening of such columns by means of CFRP sheets has been designed. Concrete cover was removed all-over the plastic hinge region in order to improve the efficiency of the wrap. The corners of the cross-section were rounded considering larger radius than that, which is typically applied in the design practice. The CFRP sheets were prestressed by means of the specially designed pretensioning system.

The new strengthening technique was very efficient. The drift capacity of the column was considerably increased. The maximum strength of the strengthened column was observed at the same drift where the non-strengthened column was near the failure. The reduction of the strength in the strengthened column was negligible as long as the failure occurred. In the non-strengthen column this reduction was more pronounced. The maximum drift of about 10% was observed in the strengthen column.

4. Acknowledgements

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5. References

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