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Interface treatment in shotcrete jacketing of reinforced concrete columns to improve seismic performance

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Abstract. An investigation of the effectiveness of the interface treatment when column concrete jacketing is performed is presented. Alternative methods of interface connection were used in order to investigate the performance of strengthened concrete columns. These connecting techniques involved roughening the surface of the original column, embedding steel dowels into the original column and a combination of these two techniques. The experimental program included three strengthened specimens, one original specimen (unstrengthened) and one as-built specimen (monolithic). The specimens represented half height full-scale old Greek Code (1950's) designed ground floor columns of a typical concrete frame building. The jackets of the strengthened specimens were constructed with shotcrete. All specimens were subjected to displacement controlled earthquake simulation loading. The seismic performance of the strengthened specimens is compared to both the original and the monolithic specimens. The comparison was performed in terms of strength, stiffness and hysteretic response. The results demonstrate the effectiveness of the strengthening methods and indicate that the proper construction of a jacket can improve the behaviour of the specimens up to a level comparable to monolithic behaviour. It was found that different methods of interface treatment could influence the failure mechanism and the crack patterns of the specimens. It was also found that the specimen that combined roughening with dowel placement performed the best and all strengthened columns were better at dissipating energy than the monolithic specimen.

Keywords: concrete columns; strengthening; shotcrete; jacketing; interface connection; roughening; dowels; seismic performance.

1. Introduction

Structures designed to older codes may prove to be inadequate to carry seismic loads. This is because the older codes underestimated seismic actions and were based on a lower level of knowledge when compared to the present day level of knowledge. Jacketing is a popular worldwide method to strengthen structural elements that have been designed using older codes. The method can be performed using concrete, steel elements such as plates, rings, angles or stirrups and fibre reinforced polymer fabrics, plates or rings. By using a variety of techniques, concrete jacketing has been carried out on beams, columns and joints. With regard to concrete columns, this is one of the oldest intervention methods. The method has been experimentally investigated in the past (Bett *et al.* 1988, Ersoy *et al.* 1993, Chronopoulos *et al.* 1994, Rodriguez and Park 1994, Dritsos *et al.* 1997,

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1998, Gomes and Appleton 1998, Tsonos 1999) and it has been proved that the bending, the shear capacity, the stiffness and the ductility of columns can be improved. The method can also improve the axial load carrying capacity of columns and may alleviate problems caused by inadequate lap splice lengths (Bousias *et al.* 2004). Although concrete jacketing is a common retrofit technique for existing columns, there are a very limited number of papers on this subject. Therefore, any obtained experimental data can be considered valuable. It has to be noted that, since concrete jacketing of individual columns increases the stiffness of the strengthened columns, the seismic behaviour of the structure may alter significantly. That is, seismic actions on jacketed columns are increased while seismic actions on unstrengthened elements are reduced. Moreover, the influence of jacketing on the foundation of the existing structure must be taken into account. Obviously, when jackets have been placed around some or all of the columns of a structure, the whole structure should be considered in the redesign procedure.

2. Research significance

Concrete column jacketing has been performed in practice using alternative procedures for the construction of the jacket. To date, no reported experimental work has been presented that compares the alternative construction methods. Parameters such as the behaviour at the interface between the original column and the jacket, the influence of the slip between the jacket and the column and the load carrying mechanism of dowels have not been experimentally investigated and compared. This has resulted in the unreasonable use of the method, high costs and, at times, ineffective results. The aim of the present work was to evaluate the different procedures used to connect at the interface between the column and the jacket. In this work, the roughening of the surface of the original column, the placement of dowels perpendicular to the interface between the old and the new concrete and combinations of roughening and dowel placement are examined.

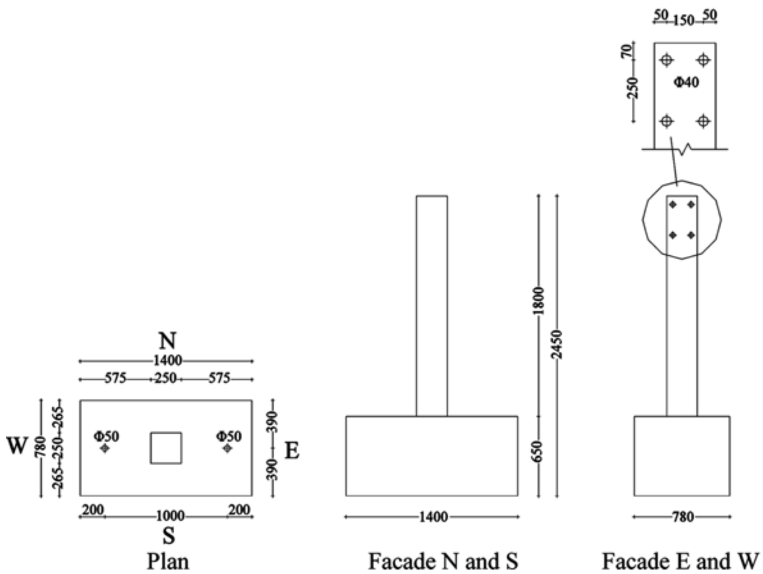

Fig. 1 Original column (dimensions in mm)

3. Experimental work

Three strengthened specimens (denoted as R for roughening the surface of the original column, D for dowel placement and RD for combined roughening and dowel placement) were constructed and represented a half height full-scale ground floor column. Each specimen initially consisted of the original column on a strong footing, as shown in Fig. 1. The column and the footing of every specimen were cast at the same time. The design details simulated usual detailing deficiencies such as mild steel longitudinal reinforcement and widely spaced mild steel transverse reinforcement with insufficient hooks at the ends of the reinforcement.

The cross sectional dimensions of the original column for the strengthened specimens were 250 mm by 250 mm. The longitudinal reinforcement consisted of four 14 mm diameter grade S220 bars, anchored into the footing using 180^{o} hooks. The transverse reinforcement comprised of 8 mm diameter grade S220 stirrups spaced at every 200 mm. The stirrup ends were 90^{o} hooks and the concrete cover was 10 mm. The top of the original column, where the horizontal load was to be applied, was locally strengthened using four 18 mm diameter grade S500 bars and 8 mm diameter grade S220 stirrups at 100 mm spacing. Additionally, a 10 mm diameter grade S500 U-shaped bar was placed parallel to each side, near the centre of the specimen.

These details are presented in Fig. 2. Fig. 2 also shows the 40 mm plastic pipes that were used to form holes in order to anchor the horizontal displacement actuator. The unstrengthened specimen (denoted as O) was constructed in exactly the same way as the original columns.

For the strengthened specimens, the jacket thickness was 75 mm, the jacket was reinforced with four 20 mm diameter (grade S500) longitudinal bars and 10 mm diameter (grade S500) stirrups spaced at every 100 mm. The jacket stirrup ends were bent in towards the concrete core as far as

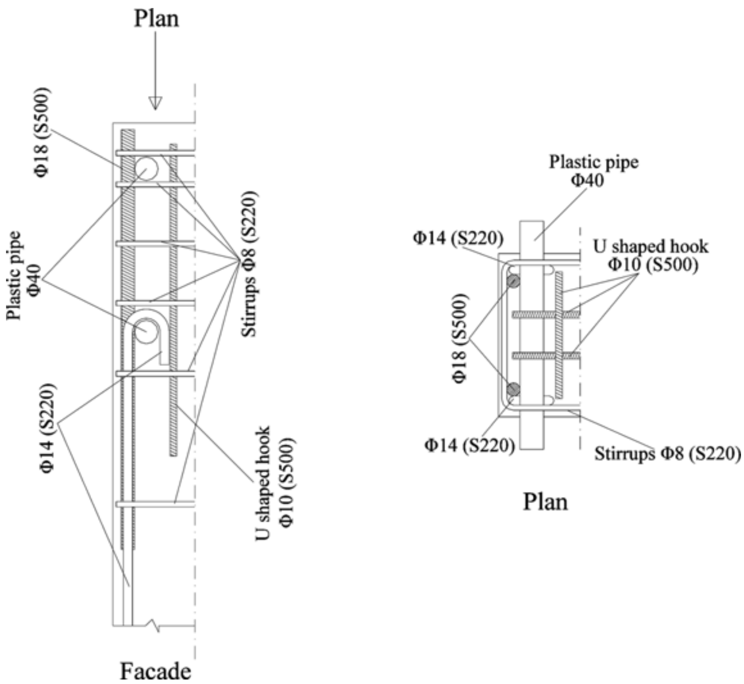

Fig. 2 Local strengthening at top of column

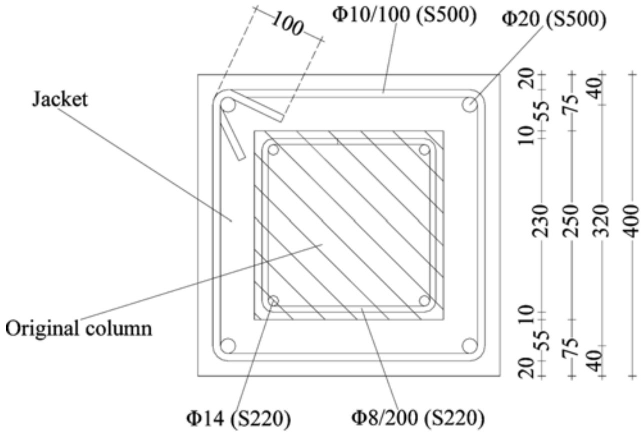

Fig. 3 Cross sectional detailing and geometrical characteristics of the strengthened elements (dimensions in mm)

possible. However, the angle achieved was less than code requirements of 135° hooks. The jacket concrete cover was 20 mm. The longitudinal bars for the jacket were placed from the beginning and were anchored in the footing by using 200 mm long 90° hooks. The height of the bars and the jacket was 1300 mm above the top of the footing. Placing the jacket longitudinal reinforcement before casting the concrete of the original column does not reflect actual practice, which is to use an adhesive to place anchor dowels into drilled holes. As the aim of the present work was to evaluate the influence of the interface treatment on the behaviour of the specimen, parameters such as bond-slip behaviour of the longitudinal reinforcement of the jacket within the footing had to be eliminated. The overall dimensions of the cross section, after casting the jacket, were 400 mm by 400 mm, as shown in Fig. 3. One month after casting the original column, the transverse reinforcement was placed and the jacket was constructed using shotcrete.

The cross sectional dimensions of the monolithic specimen (denoted as M) were 400 mm by 400 mm and the specimen had the same longitudinal and transverse reinforcement as that of the strengthened specimens (Fig. 3). The 1400 mm by 780 mm by 650 mm footing of all the specimens (Fig. 1 above) was heavily reinforced using 16 mm diameter (grade S500) bars spaced at every 150 mm in three directions.

The mean value of the cylindrical concrete strength on the day of testing of the monolithic specimen was 24.7 MPa. For the unstrengthened specimen (O) and the original column of specimens D, R and RD, the concrete strength was 27.0 MPa. The jacket strength was 55.8 MPa. The characteristics of the steel used are presented in Table 1.

Table 1 Steel characteristics

Element		Steel grade	Bar diameter	Yield stress (MPa)	Ultimate stress (MPa)
Original column	longitudinal reinforcement	S220	14	313.0	441.7
	stirrups	S220	8	425.4	596.3
Jacket	longitudinal reinforcement	S500	20	487.1	657.0
	stirrups	S500	10	599.2	677.2

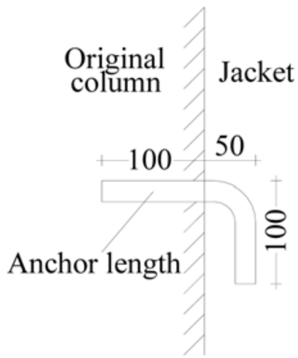

Fig. 4 Dowel geometry

The preparation of the interface between the original column and the jacket was different for each strengthened specimen. For specimen R, the surface of the original column was roughened to a depth of 6 mm by using a mechanical scabbler. Compressed air was then used to clean the surface. For specimen D, a total of 16 dowels were placed after the jacket stirrups had been positioned. That is, four dowels were embedded into every side of the original column at heights of 150 mm, 480 mm, 810 mm and 1140 mm above the footing. In practice, it is normally preferred to place dowels at different heights. For the present work, it was decided to place the dowels at the same height on all sides of the specimen to guarantee the same influence of the position of the dowels in both the pulling and the pushing directions and to eliminate any effects due to a lack of symmetry. The dowels were 20 mm diameter (grade S500) reinforcement bars, as shown in Fig. 4. Holes of 22 mm diameter were drilled and special resin was injected into each hole in order to anchor the dowels. The resin used was a typical anchor resin and had properties of 12.0 MPa in tension and 60.0 MPa in compression. As also shown in Fig 4, the anchor length for the dowels was 100 mm, which was 5 times the dowel diameter. It must be noted that, although in practice the anchor length used is 10 times the dowel diameter, the actual anchor length required to develop the full dowel action is less than 5 times the dowel diameter (CEB Bul. N$^{o.}$ 162, 1983, Tassios 2004). For specimen RD, the preparation of the surface of the original column was a combination of the connecting procedures used for specimens R and D. That is, the surface of the original column was roughened as for specimen R and the same amount of steel dowels as for specimen D were placed.

4. Test procedure

The test procedure was the same for all specimens and the test set up is presented in Fig. 5. Each specimen was moved to the testing area and an initial constant axial load was applied. Next, a displacement controlled horizontal cyclic load was applied to the top of the unjacketed part of the column at a height of 1600 mm above the top of the footing. Lateral displacement was also measured at this level. The horizontal displacement actuator that applied the lateral displacement is also shown in Fig. 5. The amplitude of the imposed displacement of the first cycle was 5 mm and it was increased by 5 mm for every additional cycle. Throughout this paper, results are presented in terms of drift ratio.

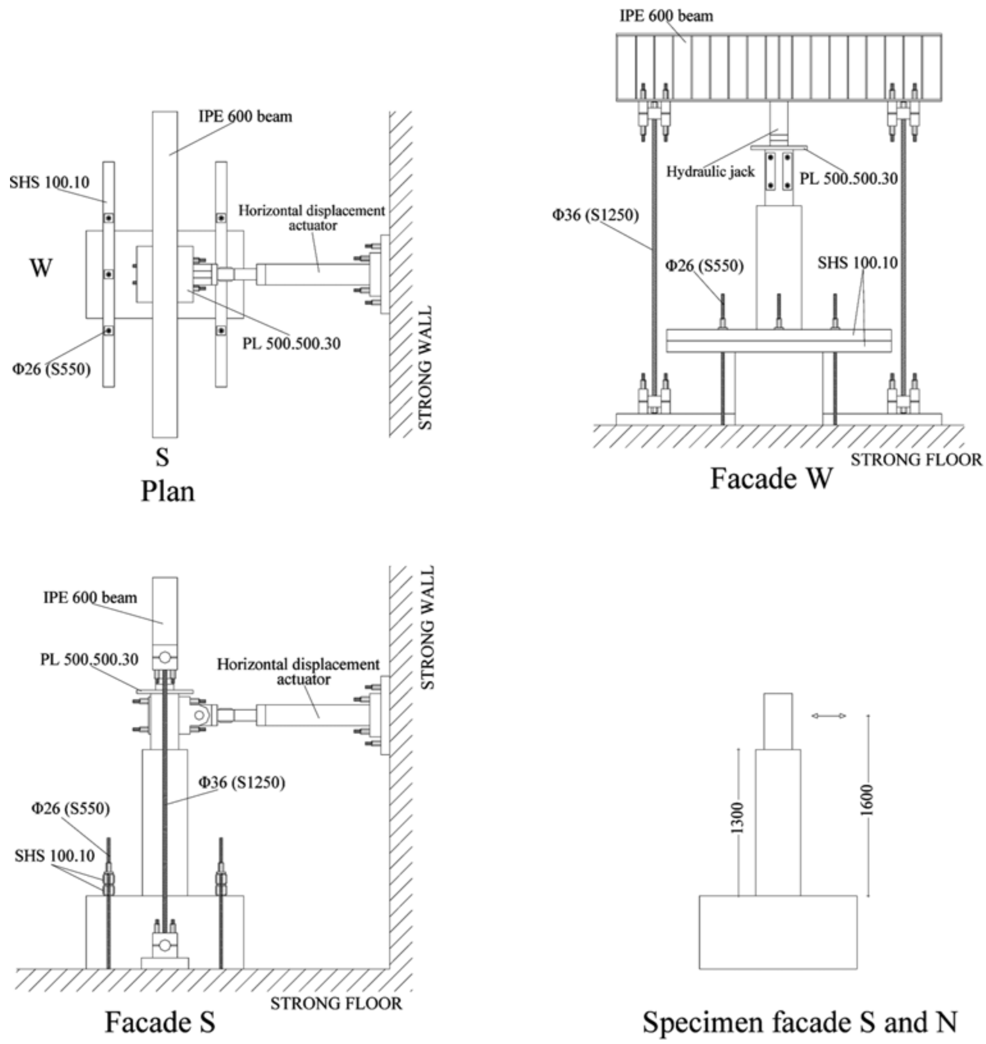

Fig. 5 Test set up

The drift ratio, expressed in percentage terms, is defined as the ratio of the measured displacement at the top of the sample to the height above the footing at which the applied load was applied (1600 mm). Therefore, an imposed displacement of 5 mm corresponds to a drift ratio of 0.31%. The loading sequence is shown in Fig. 6.

As shown in Fig. 5, an axial load was applied by using a hydraulic jack that was placed between the top of each specimen and an IPE 600 steel beam. An initial axial load of 800 kN was applied manually to all the test specimens. Test on specimen O was performed in the past by other researchers (Bousias *et al.* 2004) and the initial applied load was 680.0 kN. The applied axial load was not constant during testing and it increased and decreased throughout the test. This was due to the cyclic loading and the swaying of the specimens, which, as the horizontal displacement increased, imposed an increasing additional axial load via the tendons that held the IPE 600 beam in place. The actual axial loads that were activated were measured throughout each test and ranged

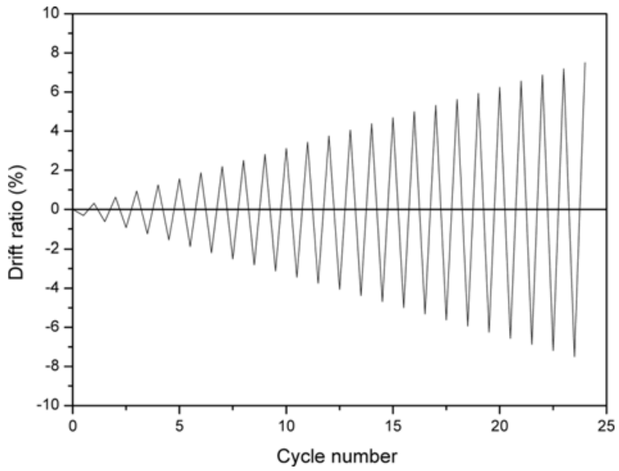

Fig. 6 Typical displacement loading pattern

from 800.0 to 930.0 kN for specimen R, 800.0 to 920.0 kN for specimen D, 800.0 to 970 kN for specimen RD, 800.0 to 1050 kN for specimen M and 680 to 690 kN for specimen O. Maximum axial load values occurred at the maximum load stage.

The normalized axial load ratio (ν_i) for the strengthened specimens, was calculated from the following equation:

$$\nu_i = \frac{N_i}{(A_{co} \cdot f_{co}) + (A_{cj} \cdot f_{cj})}$$

while for specimens O and M, ν_i was calculated from the following equation:

$$\nu_i = \frac{N_i}{A_{co} \cdot f_{co}}$$

where:

N_i : is the maximum applied axial load,

A_{co} : is the cross sectional area of original concrete (400×400 mm^2 for specimen M and 250×250 mm^2 for specimens R, D, RD and O),

A_{cj} : is the cross sectional area of the jacket concrete ((400×400)-(250×250) mm^2 for specimens R, D and RD),

f_{co} : is the concrete strength of the original or the monolithic column on the day of testing and

f_{cj} : is the concrete strength of the jacket on the day of testing for the strengthened specimens R, D and RD

The theoretical maximum flexural strength of the strengthened specimens, calculated according to EC 2 (2004) and if considered as monolithic, was found to be 311.4 kNm, which corresponds to a lateral force of 194.6 kN, and the theoretical shear strength was 372.5 kN. The maximum theoretical flexural strength of the monolithic specimen (M) was found to be 301.0 kNm, which corresponds to a lateral force of 188.1 kN, and the theoretical shear strength was 368.7 kN. This means that a flexural failure of the strengthened specimens and the monolithic specimen would be expected.

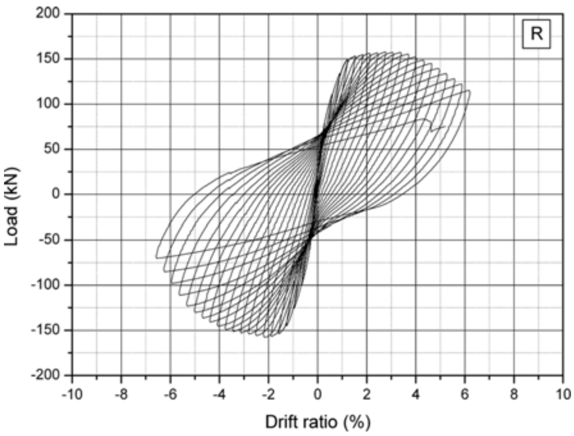

Fig. 7 Load against drift ratio curve for specimen R

Fig. 8 Bar buckling due to stirrup end opening

5. Test results

5.1 Strengthened specimens

The failure mechanism of the strengthened specimens was similar for all specimens. The typical damage sequence was as follows: At the beginning, horizontal cracks occurred just above the footing, then the cover spalled, the jacket stirrup hook ends opened, the jacket longitudinal bars buckled and, sometimes, these longitudinal bars fractured.

5.1.1 Specimen R

The load against drift ratio curve for specimen R is shown in Fig. 7. Horizontal cracks first appeared when the drift ratio was 1.25%. When the drift ratio was 2.19%, cracks occurred parallel to the jacket longitudinal bars, indicating a loss of bond between the longitudinal bars and the jacket. These cracks extended to the whole of the specimen height, and part of the jacket cover peeled off. At the same time, a horizontal crack was observed at the base of the jacket and it extended through the jacket thickness. When the drift ratio was 4.69%, the lowest stirrup in the NE corner started to open at its ends and the longitudinal bar buckled, as shown in Fig. 8.

The strength degradation was greater in the negative loading direction (pull), as shown in Fig. 7. This can be explained as the first bar to buckle was on that side (E) of the specimen. The maximum strength of the specimen was 158.0 kN. The test was terminated when the drift ratio was 6.56% and when a longitudinal bar had fractured. At this point, the strength of the specimen had degraded to 44% of the maximum. After the test, the jacket condition was good, with damage within the plastic hinge zone and some minor cracks occurring over the rest of the height of the jacket. The bond between the jacket and the column was very satisfactory due to the roughening of the original column, although a longitudinal crack occurred and a local disconnection had formed between the original column and top of the jacket in the SW corner.

The crack patterns observed on the sides of the specimen are presented in Fig. 9. The hatched areas represent areas where the concrete had spalled.

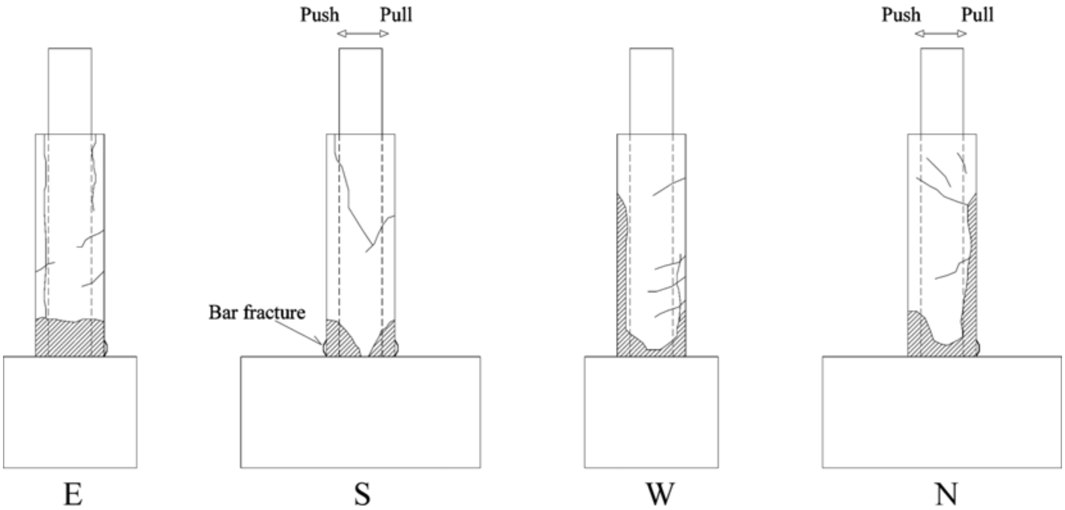

Fig. 9 Crack patterns for specimen R after testing

5.1.2 Specimen D

The load against drift ratio curve for specimen D is presented in Fig. 10. The first horizontal cracks were observed when the drift ratio was 0.94%, near the base of the specimen, 100 mm above the footing. During the next cycles, cracks appeared 300 mm above the footing. These horizontal cracks developed into inclined cracks on side N. A horizontal crack was observed at the base of the jacket and it extended through the jacket thickness. When the drift ratio was 4.69%, the concrete at the corners of the specimen crushed and the first bar buckled in the NW corner.

This bar buckled in the region between the two lower jacket stirrups and buckled parallel to the N-S direction. During the next cycle, the bars of the opposite side of the specimen buckled parallel to the E-W direction. The maximum measured load was 147.0 kN. Finally, the test was terminated, due to bar fracture, when the drift ratio was 5.94%. At that time, the strength of the specimen was

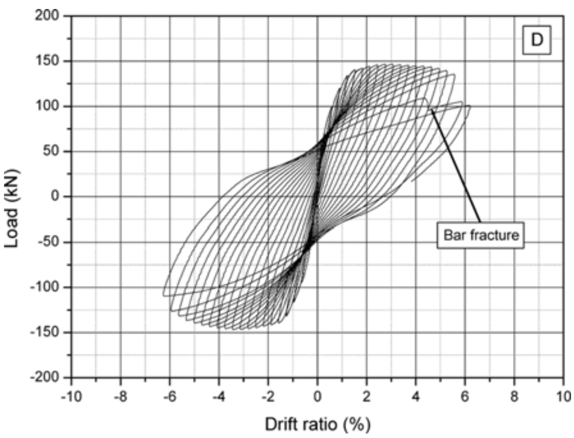

Fig. 10 Load against drift ratio curve for specimen D

Fig. 11 Jacket bar fracture

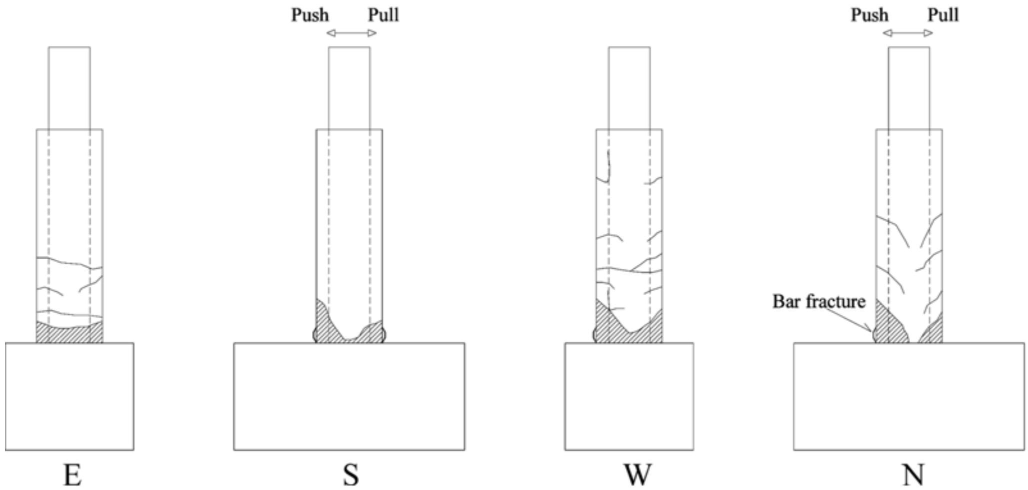

Fig. 12 Crack patterns for specimen D after testing

approximately 68% of the maximum load. The bar that fractured bar was in the SW corner and a detail of the SW corner of the specimen is shown in Fig. 11. Most of the damage was concentrated at the base of the jacket in the region 300 mm above the footing but some cracks were observed higher up the jacket. The cracks were not symmetrical on the sides parallel to the loading. At the end of the test, the jacket was in very good condition. The bond between the original column and the jacket had remained for the duration of the test. Fig. 12 presents the crack patterns observed on the four sides of the specimen.

5.1.3 Specimen RD

The plot of the lateral load against drift ratio for specimen RD is shown in Fig. 13. Cracks first appeared when the drift ratio was 1.56% and occurred on the sides parallel to the direction of

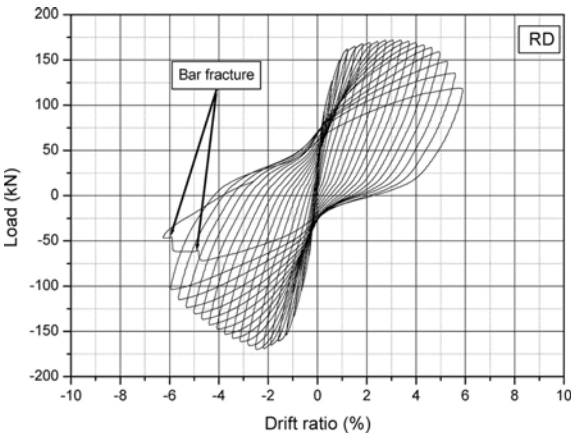

Fig. 13 Load against drift ratio curve for specimen RD

Fig. 14 Damaged region of specimen RD

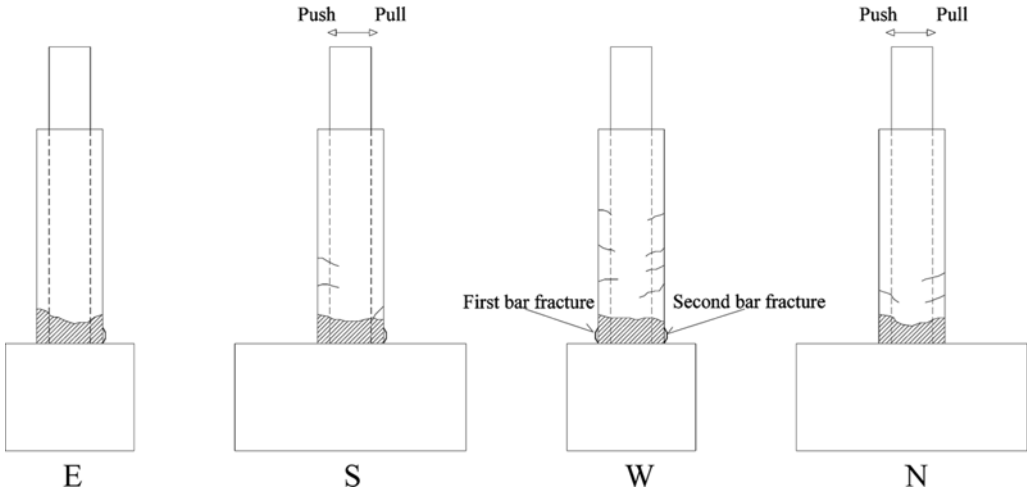

Fig. 15 Side crack patterns for specimen RD

loading (sides N and S). The width of a horizontal crack at the base of the specimen increased as the loading increased until its depth extended through the thickness of the jacket and the first jacket stirrup could be seen. At the same time, concrete crushing occurred on the compressive sides of the specimen (sides E and W). The first bar buckled in the NE corner, when the drift ratio was 4.37%. During the next cycle, the bars of side W buckled. The maximum strength of the specimen was 172.2 kN. The test was terminated when both bars of the W side had fractured and the drift ratio was 6.25%. At that time, the remaining strength of the specimen was 27% of the maximum. There was a very good bond between the jacket and the column and no disconnection between the jacket and the original column was observed.

The jacket condition was very good and all damage had occurred within the region 200 mm above the footing, as shown in Fig. 14. Even the cracks on the sides parallel to the loading direction were very limited. This was the only strengthened specimen where no cracks were observed parallel to the longitudinal reinforcement. The side crack pattern is presented in Fig. 15.

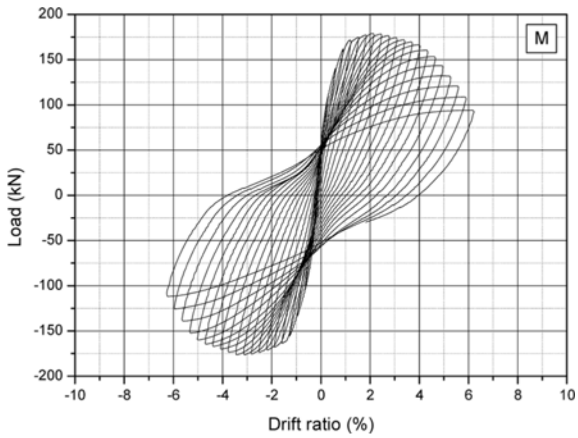

Fig. 16 Load against drift ratio curve for the monolithic specimen

5.2 Monolithic specimen

The load against drift ratio curve for the monolithic specimen is shown in Fig. 16.

As shown in Fig. 16, and as expected, the behaviour of the specimen was symmetrical in both loading directions. The specimen failed due to bending. Horizontal cracks were first observed just above the footing, when the drift ratio was 1.25%. When the drift ratio was 2.19%, a crack at the column-footing interface was wide enough for the first outer stirrup to be seen. The crack widths increased as the displacement increased and more cracks occurred at levels of 400 mm and 800 mm

Fig. 17 Bar buckling of the monolithic specimen

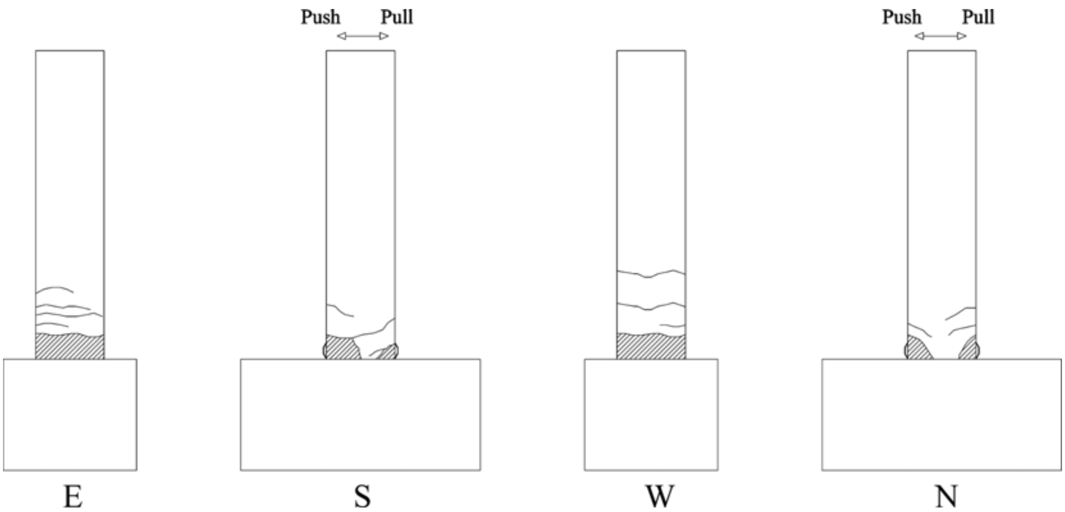

Fig. 18 Side crack patterns for specimen M

above the footing. At the same time, small width cracks occurred from the specimen edges to the middle of the sides, 500 mm above the footing. Concrete crushing was observed when the drift ratio was 2.19%. When the drift ratio was 4.69%, the longitudinal bar in the SW corner buckled, as shown in Fig. 17.

The maximum strength of the specimen was 179.0 kN. The test was terminated when the drift ratio was 6.25% because the strength of the specimen had significantly reduced, although none of the bars had fractured. At that time, the strength of the specimen was 56% of the maximum measured strength. At the end of the test, no stirrup damage was observed. The general behaviour of the specimen was very good and the damage was limited at the lower part of the column where a plastic hinge zone had occurred. For the initial cycles, the maximum strength was close to the ultimate strength for many cycles and the strength reduction was steady, indicating that the specimen had good ductility. The side crack pattern of the specimen is presented in Fig. 18.

5.3 Unstrengthened specimen

For comparison purposes, the load against drift ratio curve of the unstrengthened specimen (specimen O), taken from a previous study (Bousias *et al.* 2004), is shown in Fig. 19. More details about the behaviour of the unstrengthened specimen can be found elsewhere (Bousias *et al.* 2004).

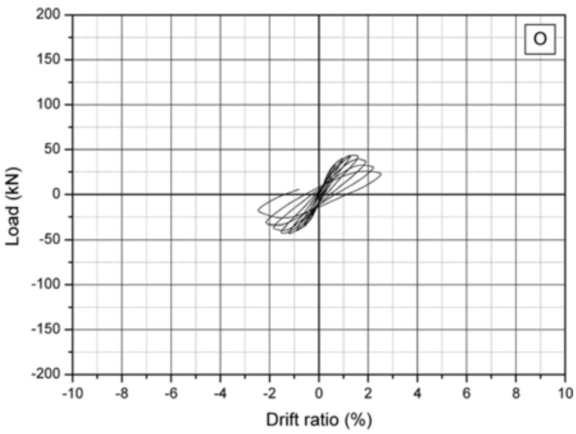

Fig. 19 Load against drift ratio curve for the unstrengthened specimen

Table 2 Test results

Specimen	P_y (kN)	d_y (%)	$P_{\max}$ (kN)	$d_{\max}$ (%)	P_u (kN)	d_u (%)
R	142.4	0.67	158.0	2.76	126.4	5.69
D	128.4	0.64	147.0	3.05	117.6	5.76
RD	147.9	0.45	172.2	3.07	137.8	5.57
M	148.4	0.39	179.0	2.08	143.2	4.97
O	32.5	0.59	43.5	1.23	34.8	2.04

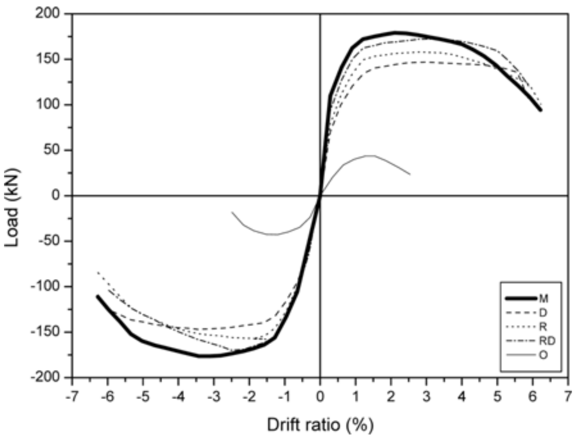

Fig. 20 Load against drift ratio envelopes for all specimens

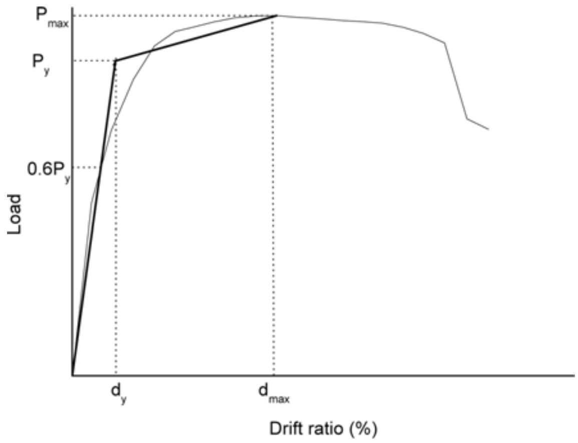

Fig. 21 Yield point definition

6. Discussion

The test results presented in the previous section are summarised in Table 2 and Fig. 20 presents the load against drift ratio envelopes for all specimens. From Fig. 20, it should be noted that the results in the pulling direction are almost the same as the results in the pushing direction. Small differences would be expected due to the degradation of the samples as each test progressed. Comments made in this section for specimen behaviour refer to the pushing direction. For the pulling direction, the results are similar with the exception of some minor differences for the strengthened specimens after the failure stage. Therefore, any comment made here for the pushing direction is also valid for the pulling direction. In order to obtain the values presented in Table 2, the yielding load, P_y, has been calculated by using an analogous procedure similar to the one described in ATC 40 (1996). The experimental load against drift ratio curve can be idealised by a bilinear curve, as shown in Fig. 21. The identical bilinear curve has to accomplish the following two conditions:

- The identical curve must cut the real experimental curve at the point that is 60% of the yield load and
- The area created by the real experimental curve outside the identical bilinear curve must equal the area created by the real experimental curve inside the identical bilinear curve.

In addition from Table 2, the maximum lateral load capacity, $P_{\max}$, and the corresponding drift ratio, $d_{\max}$, are the values that have been recorded during the tests. When comparing maximum values to theoretical values, it can be stated that all the experimentally obtained results were lower than the theoretical values and the biggest difference was with specimen D. The maximum sustainable load capacity, P_u, is defined as a load that is 20% less than the maximum achieved load and the maximum drift ratio, d_u, is the drift ratio that corresponds to P_u.

The significant improvement in strength and deformation capacity of the strengthened specimens, when compared to the unstrengthened specimen at all stages of loading, is clearly demonstrated in Fig. 20. Characteristically, Table 2 shows that the maximum strength of the strengthened specimens R, D and RD were 3.63, 3.37 and 3.95 times respectively greater than the maximum strength of the unstrengthened specimen.

The drift ratios of the strengthened specimens were more than two times greater than the corresponding drift ratio of the unstrengthened specimen.

In addition from Table 2, it can be noted that, at the yield load stage, the strength of the strengthened specimens R, D and RD were 4.04%, 13.5% and 0.34% respectively less than the strength of the monolithic specimen. At the maximum and the ultimate load stages, the differences were greater and the corresponding values were 11.7%, 17.9% and 3.80% respectively. These values demonstrate that, for strength at all stages of loading, the behaviour of specimen RD was close to monolithic and specimen D was the least effective strengthening procedure. The better behaviour of specimen RD after reaching the maximum load, especially in the pushing direction, can be attributed to a superior interface connection. Specimen RD performed better than specimen R, which in turn, performed better than specimen D. This demonstrates that roughening is more important than placing dowels since specimen D was the only specimen where roughening was not part of the strengthening procedure. This finding is the opposite of what is believed in practice. It can be concluded that, because of the almost monolithic behaviour of specimen RD, this interface connecting method would be the recommended procedure while the interface connecting method used for specimen D appears to be the least effective procedure.

When compared to the monolithic specimen, Table 2 shows that all the strengthened specimens had higher drift ratios at all stages of loading. Although differences in concrete strength and axial load may affect the results, the main reason for the above findings can be attributed to the existence of slippage at the interface between the jacket and the original column, which leads to an additional capacity to deform. Obviously, higher drift ratios for the strengthened elements in any loading stage indicate a lower stiffness. Furthermore, from Fig. 20, it can be seen that, for the same level of loading, the drift ratio of specimen D is greater than the drift ratio of specimen R and, in turn, the drift ratio of specimen R is greater than the drift ratio of specimen RD. That is, the worst interface connection gives the highest drift ratio and the lowest stiffness. Again, the differences between the strengthened elements can be attributed to slippage at the interface.

For reasons of completeness, the experimental results of the present study are compared with the results of a similar specimen that was constructed with no treatment at the interface before constructing a shotcrete jacket. This specimen comes from research work carried out in the past, at the same Laboratory as the present work, but the work was performed by another group of researchers (Bousias *et al.* 2004). In the following, this specimen is denoted as NT. The strengths and the drift ratios of specimen NT were reported to be 105.9 kN and 0.42%, 148.6 kN and 1.55%, 118.9 kN and 4.81% at the yield, maximum load and failure stages respectively.

Obviously, at all stages of loading, specimen NT had the lowest strength and ductility values. Without doubt, it can be concluded that the absence of any treatment at the interface gives the worst results.

Fig. 22 presents the stiffness degradation of the specimens. Results for specimen NT (Bousias *et al.* 2004) have been added for comparison purposes. From Fig. 22, the significant improvement in stiffness is obvious when comparing the curves of the strengthened specimens to the curve of the unstrengthened specimen. At all stages of testing, for the same imposed displacement, the stiffness of any strengthened specimen was 3 to 5 times greater than the stiffness of the unstrengthened specimen. When the drift ratio was 0.3%, the stiffnesses of the strengthened specimens were more than three times greater than the stiffness of the unstrengthened specimen and, when the drift ratio was 2.0%, the stiffnesses were more than four times greater.

In addition, from Fig. 22, it is obvious that the monolithic specimen had the higher initial stiffness.

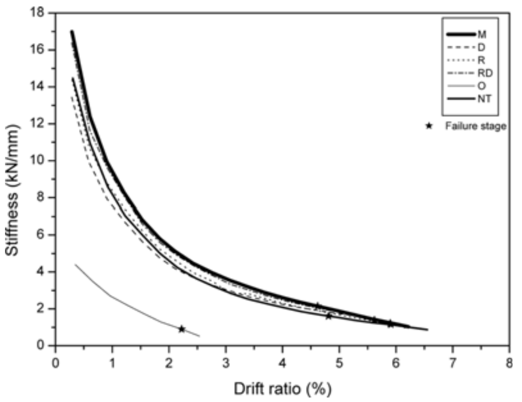

Fig. 22 Stiffness against displacement envelopes for all specimens

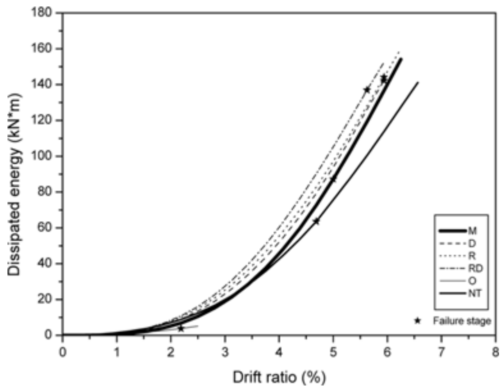

Fig. 23 Dissipated energy rate for all specimens

When comparing the strengthened specimens, specimen RD had the largest stiffness and was similar to the monolithic specimen. Specimen RD's initial stiffness was only 2.08% lower than that of the monolithic specimen. The stiffness of specimen R was higher than the stiffness of specimen D. The initial stiffness of specimens R and D were 15.6% and 20.4% respectively lower than that of the monolithic specimen. It can be stated that, in terms of stiffness, specimen RD had a higher stiffness than specimen R, which in turn had a higher stiffness than specimen D. In addition, the stiffness of specimen RD was almost identical to the stiffness of the monolithic specimen at all stages of loading.

When comparing the stiffness of the strengthened specimens with specimen NT, specimen NT had a slightly higher initial stiffness than specimens R and D and a slightly lower stiffness than specimen RD. Specimen NT experienced the fastest stiffness degradation during testing, resulting in the lowest stiffness of all the strengthened specimens. It can be observed that, during the initial stages of loading, the strengthened specimens had stiffnesses that were close to the initial stiffness of monolithic specimen. This can be attributed to there being no initial disconnection at the interface as was observed at this initial stage. As the loading increases, more slippage occurs at the interface and the stiffness degrades. The highest slippage occurs in the sample with the worst interface connection. As specimen NT had the worst interface connection of all the strengthened specimens, this specimen experienced the fastest stiffness degradation.

The dissipated energy rate for all specimens is presented in Fig. 23 and, for comparison purposes, the results for specimen NT (Bousias *et al.* 2004) have been added. From Fig. 23, it is clear that the strengthened specimens had significantly higher dissipated energy rates when compared to the unstrengthened specimen. At the failure stage, the total dissipated energy for any strengthened specimen was more than 30 times greater than the total dissipated energy for the unstrengthened specimen.

As also shown in Fig. 23, with the exception of specimen NT, all the strengthened specimens performed better than the monolithic specimen. When comparing the strengthened specimens, specimen RD was better at dissipating energy while specimen R was slightly better than specimen D. The higher energy dissipation rates for the strengthened specimens, when compared to the monolithic, can be attributed to the additional energy dissipation mechanisms of friction at the interface and dowel action when dowels are present. This is of significant practical importance as the objective is monolithic behaviour and, for jacketed columns, the capacity to dissipate energy surpasses monolithic behaviour.

The only strengthened specimen that experienced lower dissipated energy than the monolithic specimen was specimen NT. This result can be attributed to the lack of friction at the interface for this specimen.

In general, from Figs. 21 to 23, it can be observed that a better interface connection improves the behaviour of the specimens. It can be concluded that the absence of any treatment at the interface leads to the worst results because of a lack of friction at the interface. Placing dowels improves the mechanical characteristics because of the dowel action effect. Roughening the surface appears to produce better results than dowel placement, that is, friction at the interface offers more than dowel action. The best results are obtained when dowel placement is combined with roughening the surface due to the combined effects of friction and dowel action at the interface. Obviously, this technique should be recommended for practice since almost monolithic behaviour can be expected for jacketed columns and better than monolithic energy dissipation rates are likely.

When drawing comparisons to previous experimental work (Bett *et al.* 1988, Ersoy *et al.* 1993, Chronopoulos *et al.* 1994, Dritsos *et al.* 1997, 1998, Gomes and Appleton 1998), the findings of this investigation are in general agreement with regard to the improvement of the behaviour of the strengthened elements. In contrast, Rodriguez and Park (1994) reported that the repaired and/or strengthened specimens experienced higher strength and stiffness than the monolithic specimens. In the present research, the strength and the stiffness of the strengthened specimens were lower than that of the monolithic specimen and only the energy dissipation rate was higher.

7. Conclusions

An investigation of the effectiveness of using alternative methods of concrete jacketing in order to strengthen concrete columns has been presented. Three different interface treatment procedures that were used to connect the jacket to the original column have been examined. These were: (a) roughening the surface of the original column (specimen R), (b) use of steel dowels (specimen D) and (c) roughening the surface combined with steel dowel placement (specimen RD). In general, it can be concluded that strengthening always significantly improves the behaviour of the elements, no matter what technique is used to construct the jacket.

When comparing the strengthened specimens to the unstrengthened specimens, significant improvements in strength, deformation capacity, stiffness and dissipated energy rate have been clearly demonstrated. Characteristically, when comparing the unstrengthened specimen and the RD specimen, at all stages of testing, for the same imposed displacement, specimen RD achieved approximately 4 times higher strength, 3 to 5 times higher stiffness and was 36 times better at dissipating energy.

It was found that the strengths and the stiffnesses of strengthened specimens were slightly lower than that of the monolithic specimen but the drift ratios and the rates of energy dissipation were higher for the strengthened specimens at all stages of loading. The strengths of specimens R, D and RD at the yield point were 4.04%, 13.5% and 0.34% respectively less than that for the monolithic specimen. At the maximum and the ultimate load stages, these values were 11.7%, 17.9% and 3.80% respectively. The respective initial stiffnesses were found to be 20.4%, 15.6% and 2.08% lower than the initial stiffness of the monolithic specimen.

The finding that the dissipation energy capacities of the jacketed columns were higher than monolithic is of significant practical importance. The higher energy dissipation rates for the strengthened specimens, when compared to the monolithic, can be attributed to the additional energy dissipation mechanisms of friction at the interface and dowel action when dowels are present.

As far as strength, deformation capacity and dissipation energy capacity are considered, it was found that the absence of any treatment at the interface leads to the worst behaviour. Placing dowels improves the mechanical characteristics. Roughening the surface produces better results than dowel placement. The best behaviour was obtained when dowel placement was combined with roughening the surface. Without any doubt, this technique should be recommended for practical applications. By performing the above connecting procedure, an almost identical behaviour to monolithic can be expected and better than monolithic energy dissipation rates are likely. This last conclusion has practical significance for the engineer as, if dowels and roughening are combined as a strengthening technique, it is possible to assume monolithic behaviour for a strengthened element. This conclusion also justifies the obligation of the draft Greek Retrofitting Code (GRECO 2004) for roughening at the interface to be combined with the placement of a minimum amount of steel dowels, when connecting new concrete layers to existing concrete elements.

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