HIGH-PERFORMANCE FIBER-REINFORCED CONCRETE JACKETING IN A SEISMIC RETROFITTING APPLICATION

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In the seismic retrofitting of reinforced concrete (RC) elements, different techniques are usually proposed. Regarding the strengthening of existing columns, the possibility of using an RC jacket is usually considered, especially when the element is made of low-strength concrete. Traditional jacketing presents some inconvenience, as the jacket thickness is governed by the depth of concrete cover over the reinforcing steel (both external and internal). This often leads to a jacket thickness of at least 3 to 4 in. (70 to 100 mm) and a consequent increase of the section geometry. This additional thickness results in an increase in both mass and stiffness that can cause some problems with respect to seismic behavior. This aspect is particularly important when small columns, measuring 10 to 12 in. (250 to 300 mm) in width, are considered.

The traditional reinforcement in the jacket can be eliminated by using a thin high-performance fiber-reinforced concrete layer 1 to 1.6 in. (30 to 40 mm) thick. This technique has effectively strengthened existing columns, and it achieves the same results as other techniques, such as traditional jacketing or fiber-reinforced polymer (FRP) wrapping, particularly when a low-strength concrete is present in the existing structure.

Recent investigations on existing structures built in Italy around the 1960s and 1970s found an average concrete compressive strength of less than 2175 psi (15 MPa). These buildings not only have problems carrying the vertical design loads but also have to be strengthened significantly when a seismic retrofitting is required. In this case, the proposed strengthening technique can be easily adopted, and this strengthening could also result in an increase in the structure's performance.

The proposed technique was used in the following real case study. The project involved a school building located in a seismic area near Rome (Fig. 1), where tests showed that concrete with an average compressive strength of 1600 psi (11 MPa) had been used. Because of the low strength of the existing concrete, a complete retrofit of the building had to be undertaken to comply with new Italian seismic codes.

The columns were strengthened with a 1.6 in. (40 mm) jacket of high-performance fiber-reinforced concrete. Before the installation of the jackets, a full-scale test simulating the behavior of the existing columns was requested from the Italian Council for Public Works. This agency has to be consulted when a structural system that does not comply with

Fig. 1: Overall view of the outside Zagarolo School near Rome, Italy

existing codes is intended for use. The test was performed up to failure by applying cyclic loads of increasing amplitude. Once the results demonstrated the effectiveness of the proposed technique, the use of the strengthening jackets on the school building was authorized and eventually executed.

SPECIMEN PREPARATION AND TEST SETUP

A column with a 16 x 16 in. (400 x 400 mm) square cross section was tested (Fig. 2). The 10 ft (3 m) high element was cast on a 20 in. (500 mm) thick foundation. The reinforcement and concrete strength were typical of this kind of element in the 1960s and consisted of 0.6 in. (16 mm) diameter longitudinal reinforcement steel and 0.3 in. (8 mm) diameter stirrups spaced at 12 in. (300 mm) with a concrete strength of approximately 2900 psi (20 MPa) (Fig. 3).

After casting and a curing period of 14 days, the column surface was sandblasted to achieve a roughness of 0.04 to 0.08 in. (1 to 2 mm or ICRI Concrete Surface Profile [CSP] 3) to ensure a good adhesion between the new and old concrete.

The specimen was placed on the testing frame and an axial load equal to 38,220 lbf (170 kN) was applied by means of two hydraulic jacks (Fig. 4). This axial load was designed to reproduce the effect of the dead loads acting on the column at the time of the jacket application.

The strengthening jacket with a thickness of 1.6 in. (40 mm) was eventually cast (Fig. 5) with a self-consolidating high-performance fiber-reinforced concrete with a compressive strength of 18,855 psi (130 MPa) and a tensile strength of 870 psi (6 MPa). To connect the jacket to the column base, a pocket 2 in. (50 mm) deep was created in the foundation and a high-strength steel mesh 0.08 in. (2 mm)

Fig. 2: Column geometry and specimen construction

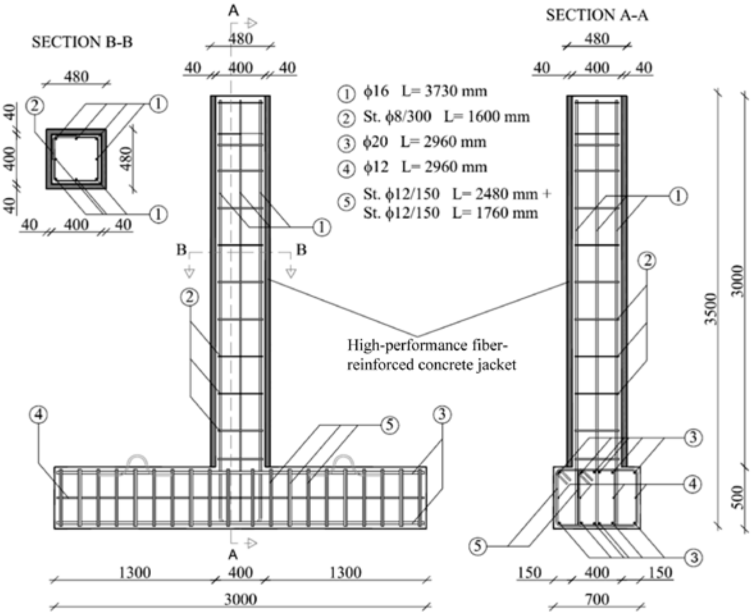

Fig. 3: Details of the specimen construction. (Note: 1 mm = 0.039 in.)

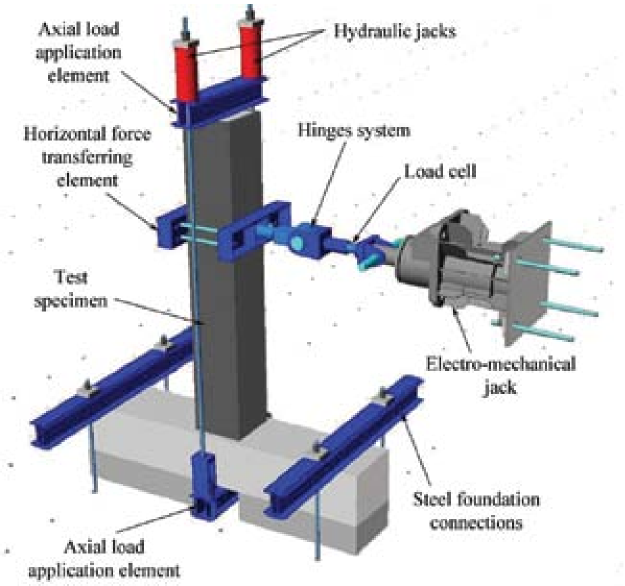

Fig. 4: Test setup

Fig. 5: High-performance jacket casting

diameter wire with a 0.8 in. (20 mm) grid was inserted in the jacket for the first 6 in. (150 mm) of the column. This solution has proven to be effective in other similar applications. The same mesh was then applied at midheight of the column, where a cast interruption was anticipated. After curing of the jacket, the column was tested.

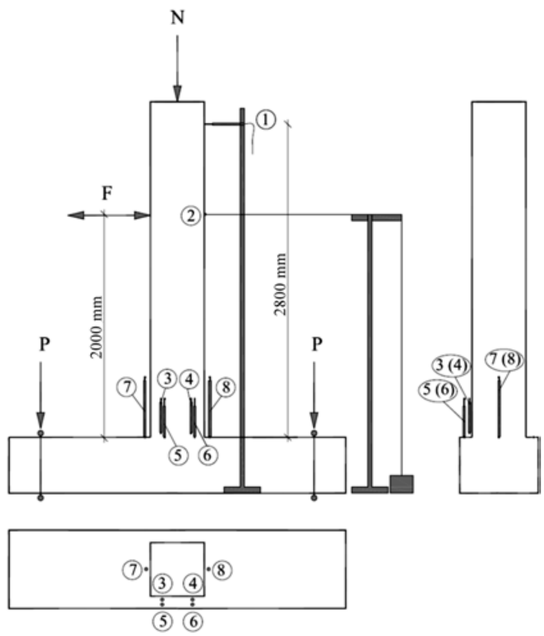

Fig. 6: Displacement measurement setup. (Note: 1 mm = 0.039 in.)

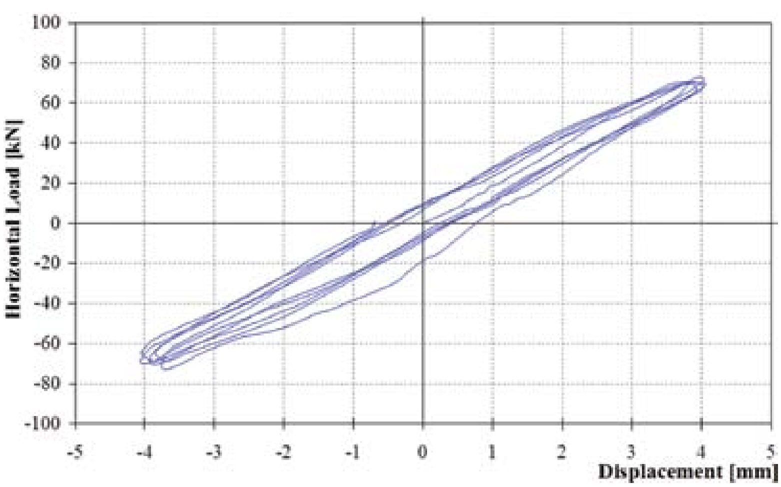

Fig. 7: Horizontal load versus displacement for the design load level. (Note: 1 mm = 0.039 in.; 1 kN = 225 lbf)

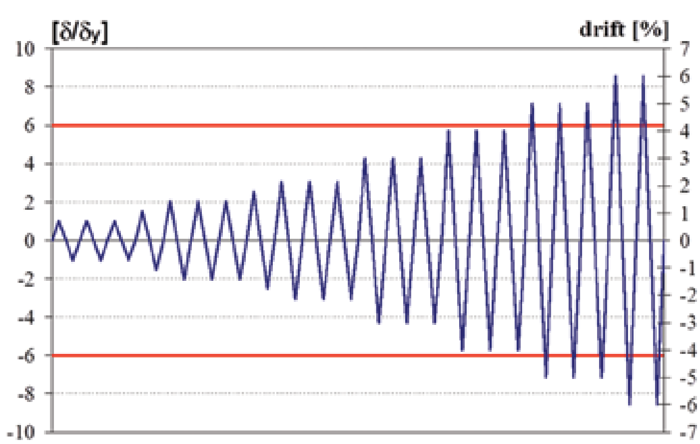

Fig. 8: Load history

The column foundation was anchored to the laboratory basement with four pretensioned high-strength reinforcing steel bars. The initial axial load was increased up to 145,000 lbf (645 kN) in accordance with the critical design load for the column in the building. Eventually, a horizontal cyclic load was applied by means of an electromechanical jack fixed to the reaction wall of the laboratory. The jack was linked to the column by means of a hinged bar system in which a load cell was placed. The horizontal force was applied at a height of 6.6 ft (2 m) from the column foundation connection to achieve the same moment-shear ratio at the critical section (column base section) obtained in the building design.

To measure the horizontal displacements, potentiometric transducers were placed on the column top (Position 1 in Fig. 6) and at the level of the load application (Position 2 in Fig. 6). The rotations at the column base were measured by means of a series of potentiometric transducers. The devices in Position 3-4-7-8 of Fig. 6 were placed on the column, whereas the devices in Position 5-6 of Fig. 6 were placed to measure the relative displacement between the column and foundation base.

TEST PROCEDURE

Initially, a cyclic horizontal force and a constant vertical force were applied to simulate the maximum design actions (axial force N = 145,000 lbf [645 kN], bending moment M = 106,210 ft-lb [144 kN-m], and shear force V = 16,186 lbf [72 kN]). In this testing phase, five cycles were performed by applying maximum design bending moment and shear action in both directions with a constant axial force. Under these actions, there did not appear to be any damage, and no cracks were detected on the strengthening jacket for the column. As observed in Fig. 7, where the horizontal force-versus-displacement curve is presented, the behavior is almost linear elastic and only the first cycle showed some settlement of the loading system.

To verify the effectiveness of the strengthening technique, the test was allowed to continue by applying the horizontal load with cycles that increased in amplitude up to failure. Cycles with a displacement amplitude double the initial one were applied to define the structural yielding point. This was determined to be a horizontal load equal to 25,850 lbf (115 kN), corresponding to a bending moment equal to 169,640 ft-lb (230 kN-m), which is almost 1.6 times the maximum design value.

The structural yielding occurred at a displacement δ_y equal to 0.04 in. (1 mm), measured at the load application point level. At this level, the yielding drift, defined as the ratio between the displacement δ_y and the lever arm of the horizontal load (6.6 ft [2 m]) with respect to the column base, was equal to 0.7%. The test continued by applying cycles with displacement amplitude proportional to the yielding drift. Initially, three cycles at a drift of ±0.7%, one cycle at ±1%, three cycles at ±1.5%, one cycle at ±1.75%, and three cycles at ±2% were applied. Eventually, three cycles for increments of drift equal to 1% were applied up to collapse. The applied load history is summarized in Fig. 8, where a ductility limit close to $6\delta_y$ is indicated. This value is associated with the behavior factor for high-ductility frame systems.

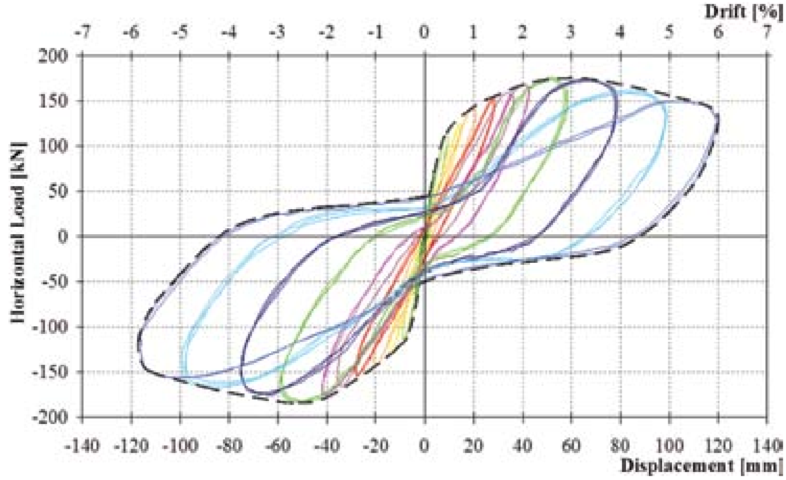

Fig. 9: Horizontal load versus displacement at the load application point. (Note: 1 mm = 0.039 in.; 1 kN = 225 lbf)

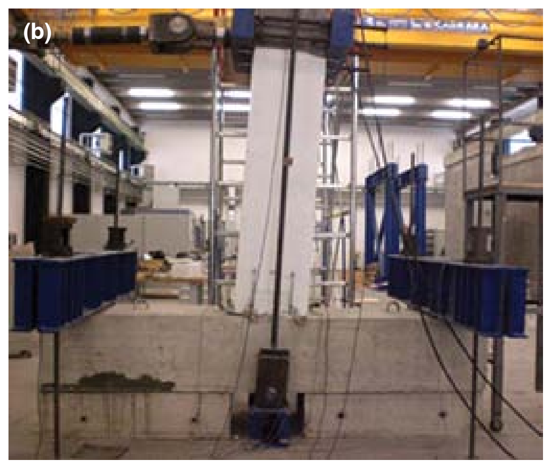

Fig. 10: (a) Cracking at the column base; and (b) column deformation at failure

RESULTS

The results of horizontal load versus displacement at the level of the load application point are shown in Fig. 9. The column reached collapse during the third cycle at a drift level equal to 6% (4.7 in. [120 mm]; $\delta/\delta_y = 8.6$). The collapse was due to the rupture of one of the longitudinal reinforcing steel bars.

After the onset of the flexural cracking at a drift equal to 1%, the behavior was stable up to failure with limited damage. The main crack was located near the end of the high-strength steel mesh that was intended to link the high-performance concrete jacket to the base foundation. Other cracks developed with a spacing of about 11.8 in. (300 mm), which was equal to the stirrup spacing.

Figure 10(a) shows the crack pattern at failure. Notice the localized damage in relation to the jacket-foundation interface. Figure 10(b) shows the column deformation at a 6% drift.

The envelope curve (dotted line) in Fig. 9 indicates strength degradation for drift levels higher than 3.5%. The maximum bearing capacity is equal to 39,340 lbf (175 kN), whereas the horizontal load at failure is equal to 32,600 lbf (145 kN)—83% of the maximum load. The load decrease can be justified because of a progressive slip of the jacket at the foundation base, as shown in Fig. 11.

Fig. 11: Slip between jacket and foundation base

This slip is confirmed in Fig. 12, where the moment-versus-curvature curves at the column base are drawn. In one curve, the curvature was determined by reading the displacement transducers placed on the column, whereas the other curve was determined by reading the transducers measuring the relative displacement between the column and foundation. As a result, the first curve

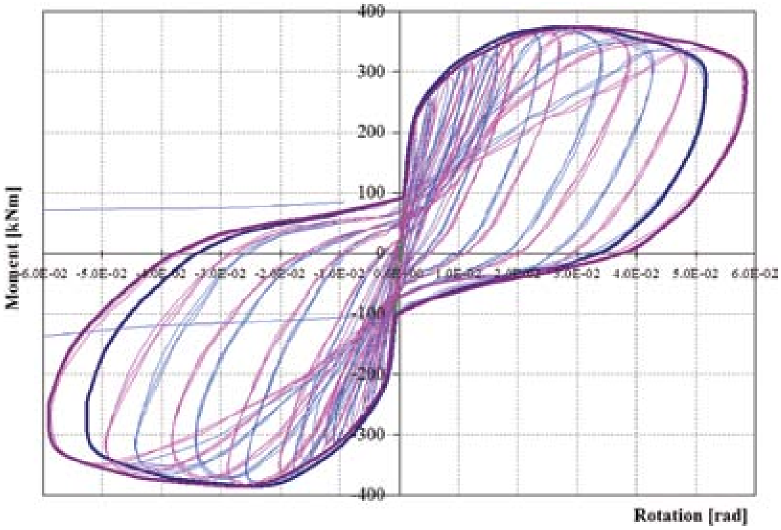

Fig. 12: Moment-versus-curvature curves with and without slip contribution. (Note: 1 mm = 0.039 in.; 1 kNm = 737.5 lbf)

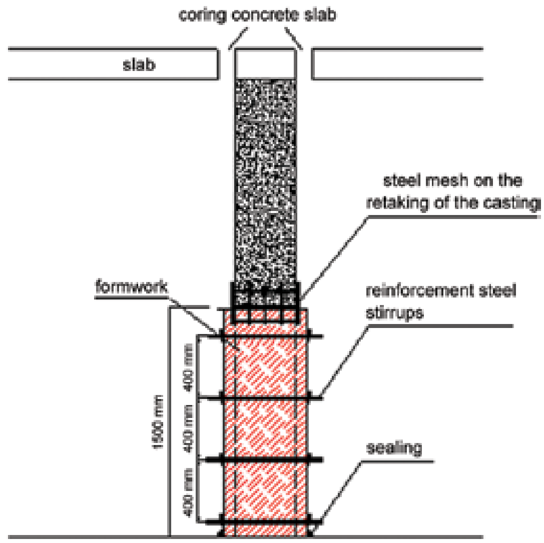

Fig. 13: Diagram of Zagarolo School column-strengthening techniques. (Note: 1 mm = 0.039 in.)

does not take into account the jacket slip as the second curve does. The two curves tend to diverge with the slip mechanism activation after the maximum moment (horizontal force) is reached. This aspect justifies the load decrease at 3.5% of drift.

CONCLUSIONS

This article illustrates the first application of a new strengthening technique based on the use of high-performance fiber-reinforced concrete. Given the favorable results obtained during this research, application of the new technique was conducted on columns at the job site in Zagarolo (Fig. 13 to 15).

The full-scale laboratory tests that were conducted demonstrated the effectiveness of the jacket application and showed a remarkable increase in terms of bearing capacity and ductility. The adoption of this technique has advantages with respect to traditional strengthening techniques. In particular, it is possible to limit the increase in the size of the column, thereby minimizing the weight and stiffness of the structure. The high viscosity of the self-consolidating material resulted in smooth surfaces.

In the application on the school building, it was not necessary to add a plaster layer (0.8 in. [20 mm] thick) that was previously present. Therefore, the change in the size of the columns was only 0.8 in. (20 mm) due to the fact that the jacket thickness was 1.6 in. (40 mm). The adopted technique appears particularly useful when the original concrete is in poor condition because the use of the high-performance concrete layer protects the internal column, thereby increasing its durability.

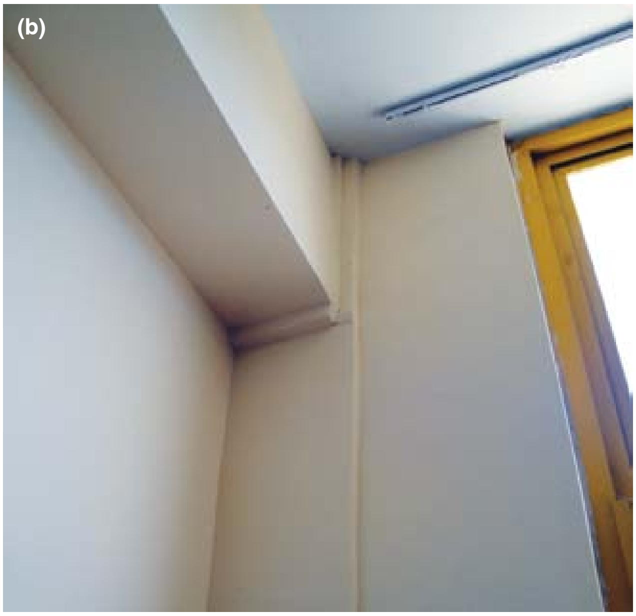

Fig. 14: (a) Strengthening repairs in progress; and (b) completed repairs

Fig. 15: Completed repairs

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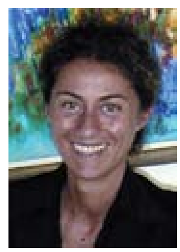

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