EXPERIMENTAL RESPONSE OF A NON-DUCTILE R/C BUILDING REHABILITATED BY MEANS OF FIBRE REINFORCED POLYMERS

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SUMMARY

A two-bay, two-storey R/C frame (Fig. 1), had been originally designed according to old design codes to withstand a 0.3 g peak ground acceleration earthquake. The resulting design details comply with most of the Eurocode 8 provisions for medium ductility frames. The 0.3 g peak acceleration on the prototype corresponds to 0.45 g peak acceleration on the experimental model, 2/3 scaled in geometry. At the ELSA Laboratory, it was exposed to an earthquake-type excitation of this intensity. The damage shown after the experiment was consistent to the design conception. Slip of the reinforcing bars of the beams has been observed around the joint, and some cracks in consequence.

The structure has then been repaired and strengthened by means of carbon fibre reinforced polymers (CFRP), Fig. 2. In order to distinctly classify the capability of CFRP, no additional reinforcing bar has been placed upon, as it would be suitable in order to provide anchoring both to the existing bars and to the lamina, laid on as part of the intervention. After this intervention, the structure has been subjected to the same earthquake excitation. No further damage has been observed. Then the frame was subjected to cycles of displacements of increasing amplitude, up to a peak displacement of 245 mm. Only at this level of intensity, collapse of bars and lamina detachment had been observed. No significant drop in the lateral strength took place up to story drifts as large as 1/20.

In theory, the exercise accomplished by CFRP sheets may be associated to that exerted by spirals and stirrups. The way they have been designed is in fact derived by the design criterion of spirals and stirrups. However, tests have shown the great advantage of CFRP in avoiding bars buckling. The paper is devoted to the discussion of this capability of CFRP membranes.

INTRODUCTION

There is an increasing evidence of the capability of FRP jacket, (or wrap, or membranes), to provide ductility in columns and beams. The idea is that they are able to furnish substantial confinement to the concrete. In the present research the capacity to prevent bar buckling under severe earthquake conditions has been proved. This capability can provide a further contribution to the ultimate resistance under seismic conditions. In order to introduce this contribution, the effect of stirrups under cyclic loading, beyond the plain concrete resistance, is briefly reviewed. For structures of ductility class H in Eurocode 8, in the critical region, the distance s between column stirrups shall be less than:

$$s = \min\{b_0 / 4; 100mm; 5d_{bL}\} \tag{1}$$

where b_0 is the minimal dimension of the column core, and d_{bl} is the diameter of the longitudinal bars.

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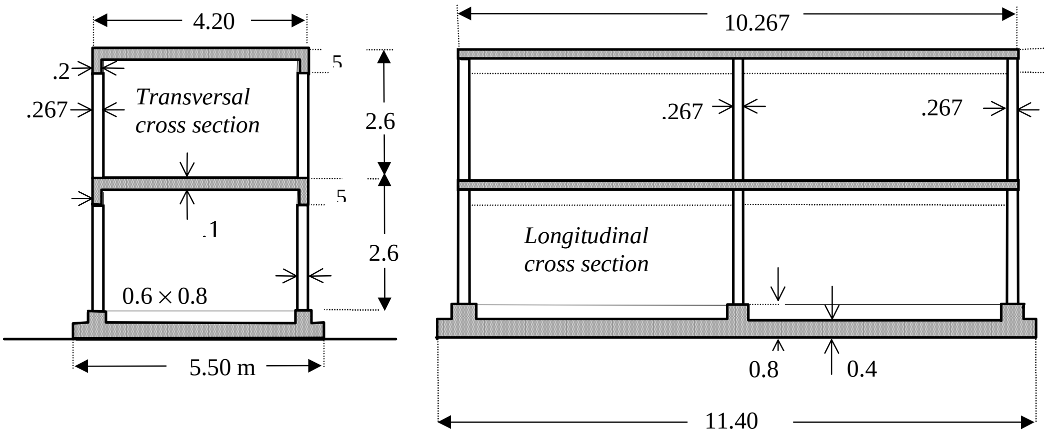
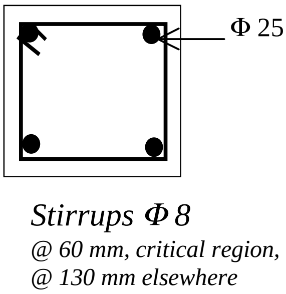


Fig. 1: Layout of the specimen.



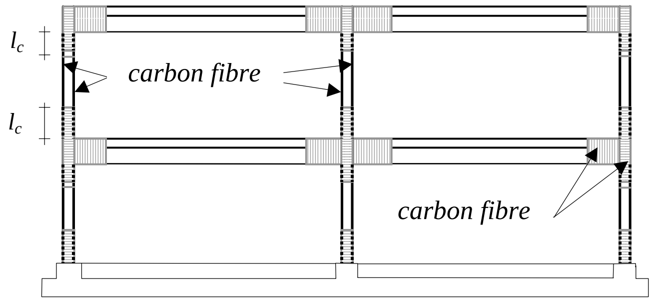


Fig. 3: Column cross section.

Fig. 2: Strengthening of the frame.

For the columns of the model under test, Eq. 1 provides s = 46 mm. In the original design the distance between stirrups is 60 mm, and thus it does not comply with the minimum distance requirement to classify the structure in the ductility class H. In general experiments single out a qualitatively different behaviour of models respecting the requirement of Eq. 1 and models which do not conform to it.

When cycles of axial or lateral loading of increasing amplitude are applied to a column, three stages of deformation may be identified.

1) At low level of deformations, the concrete cover is effective, and stirrups are practically unstressed.

2) At higher deformation amplitudes, microcracks open in the concrete cover, propagate and are capable to split sheets of cover from the concrete core. The cover starts to separate, in sheets of various dimensions, larger when axial loads are prevalent. At this moment tension develops in stirrups, even reaching the yield strength, depending upon the axial deformation amplitude.

3a) If the distance s between stirrup and stirrups is that currently employed in non-seismic conditions, i.e. fairly larger than that of Eq. 1, buckling of longitudinal bars occurs around 0.2 %, and concrete core in the bar vicinity crashes in compression. The column stiffness at this stage is practically null.

3b) If the distance is around that specified by Eq. 1, the bar buckling occurs at an axial deformation substantially higher than 0.2%. The closer is the distance between stirrups, the higher is the deformation at which buckling occurs. A few cycles after buckling, the ultimate resistance of the column may coincide with either the core crushing, or the brittle failure of the bar in tension, as Fig. 4 and 5 shows. This last mechanism, which occurs if the bar spacing is fairly less than that specified in Eq. 1, is substantiated by two circumstances:

When a steel bar is subjected to cycles of loading beyond the hardening limit, a few load reversals are sufficient to induce a brittle failure, that is, a failure without necking.

For the same axial deformation, buckling induces further strain in the most compressed fibre of the rebar, so that if a rebar buckles, a few cycles might promote brittle failure of the rebar, due to the combined effects of axial and bending strains.

The presence of FRP membrane is expected to induce a different behaviour in the critical region. In absence of it, layers of cover are likely to split from the core. If the membrane is placed correctly, it holds the layer of cover in their position, keeping it solid with the column core. In general, in these conditions there is no evidence of concrete crushing. Besides, buckling of the bars is avoided, and the state of collapse is reached when the bar deformation attains the hardening region. In these conditions, a few cycles are sufficient to provoke a brittle fracture. In the present investigation this fracture was not attained under earthquake conditions, but under imposed displacements of 245 mm amplitude.

DESIGN CRITERIA FOR THE ORIGINAL STRUCTURE

Design specifications:

- Pertinent codes: Eurocode 2 (Concrete structures), and Eurocode 8 (Constructions in seismic zones);
- steel yield strength fyk = 500 MPa; concrete strength Rbk = 25 MPa;
- permanent load, besides the weight of structures: 2000 N/m^2;
- live loads: 2500 N/m^2;
- earthquake load: the q-reduced, design-response spectrum of Eurocode 8;
- maximum ground acceleration: 0.3 g, soil type B. (Due to scale factors, it becomes 0.45 g for the experimental model); category: “medium ductility”, and “high regularity”; behaviour factor: q = 3.75.

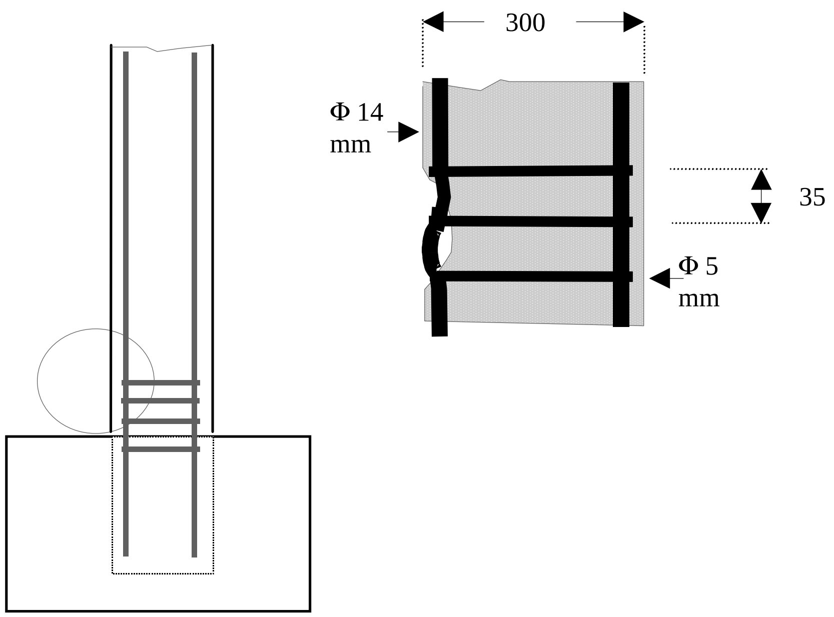


Fig. 4: buckling of longitudinal reinforcement (from [Saisi and Toniolo, 1998]).

In the following, attention is focused on the physical model only, Fig. 1. With respect to the prototype, the scale factor is 2/3 on geometrical dimensions, 1 for stress and 3/2 for gravity acceleration. In the physical model the density scale has been achieved by additional masses. The time history of the ground acceleration has been modified according to the time scale St=2/3, and the acceleration scale Sa = 3/2.

The structure has been subjected to the seismic excitation described above by means of a pseudodynamic test. The experimental procedure, to represent inertial forces during a ground motion, is explained in the paper by [Negro et al. 1996]. After the test, the structure showed a more or less uniformly distributed damage. In the columns, cracks opened and closed during cycles, so that only thin cracks, about 0.1÷0.2 mm wide, appeared after the excitation. Damage in the form of crack-through appeared in the slab, 0.3 ÷0.5 mm wide, near all joints.

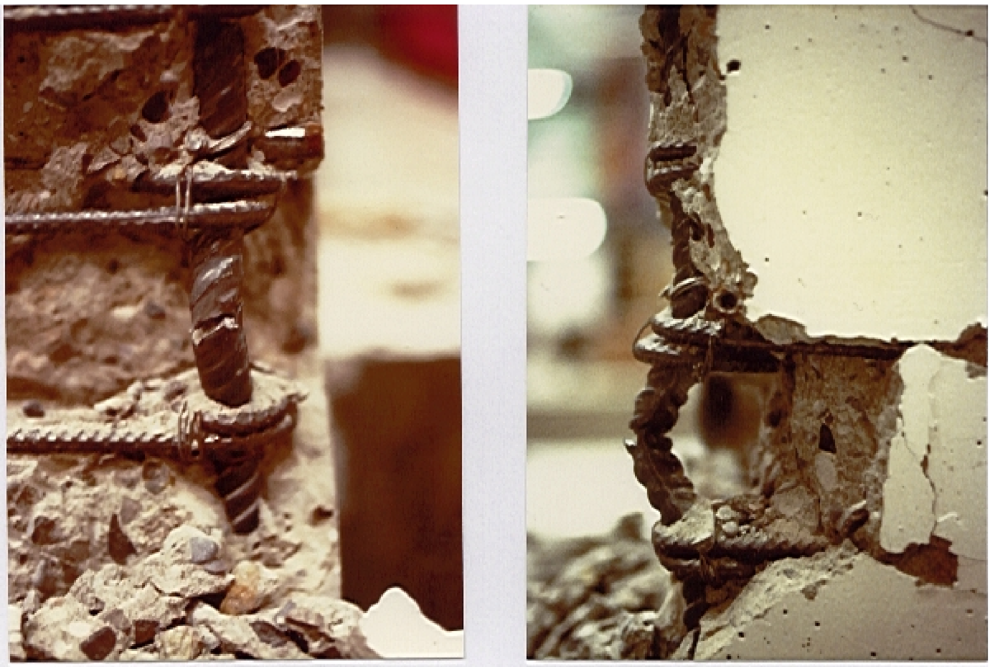


Fig. 5: Failure of the bar in tension, two cycles after buckling. The picture shows stirrups firmly connected to the longitudinal bars, and a brittle failure, i.e., without necking (from [Saisi and Toniolo, 1998]).

REPAIRING AND STRENGTHENING OF THE FRAME

The main cracks in the slab have been restored by means of fluid epoxy resin. As to strengthening, the structure in its original configuration did not fulfil three main design criteria for constructions in seismic zones: 1) the amount of confinement near the joints, 2) the requisite of capacity design to enforce a strong column/weak beam mechanism, and, 3) the minimum anchoring length of the beam bars. These requirements are included in Eurocode 8 for structures in the ductility classes medium or high.

Strengthening in columns consisted in two different interventions, Fig. 2: 1) increasing the confinement of the critical regions of the columns, by the application of horizontally oriented FRP sheets, and, 2) increasing the tension strength in the central span of the columns, by the application of longitudinally oriented lamina of FRP.

As to beams, confinement of the beams critical regions, by CFRP sheets (or jackets), was attempted. For both columns and beams, to ensure satisfactory adhesion, curvature around the edges has been smoothed as much as possible, by a suitable levelling of the epoxy adhesive as Fig. 6 suggests.

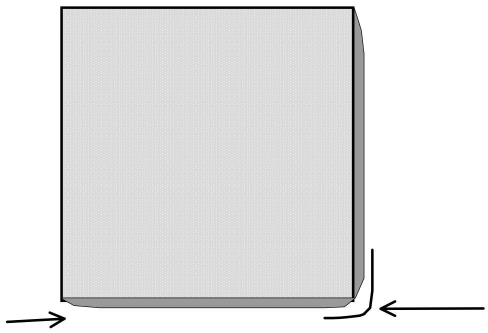


Fig. 6: Horizontal cross section of columns. Smoothing the curvature at edges, with epoxy grout.

The mechanical characteristics of the carbon fibres reinforced membranes which have been used are:

	Modulus of elasticity *N/mm^2*	*Tensile strength* *N/mm^2*	*Thickness* *mm*
Carbon fiber reinforced polymers (Sika Carbodur [1]), 50 mm wide	165000	2800	1.2
Carbon fiber fabric (SikaWrap Hex-230C [2])	230000	3500	0.13

[1] used for columns strengthening
[2] used for concrete confinement

The carbon fibre reinforced polymer has a Young modulus of elasticity only marginally smaller than that of the steel. With respect to deformed bars, it may offer a largely better adherence under cyclic loads.

With reference to a strong motion earthquake, the target of the intervention was to provide a ductile behaviour of the structure, by the concrete confinement of the potential critical regions of columns and beams.

TEST RESULTS

The peak of the time history of acceleration in the reference earthquake is 0.45 g. This is equivalent to 0.3 g in the prototype, when the acceleration scale is taken into account. This excitation has been applied two times: 1) to the bare structure; 2) to the strengthened structure.

During and after the second earthquake simulation, no further cracks have been detected. Besides, some cracks, which opened before, did not reopen during the second earthquake excitation. Detailed results can be found in the paper by [Castellani et al., 1999]. In the present paper results obtained under imposed displacements of increasing amplitude are discussed.

Cycles of harmonic displacement have then been applied in order to reach the collapse of the frame. The amplitude of cycles has been increased up to ± 245 mm. The main observations are as follows:

Under cycles of small to medium amplitude, all membranes and the lamina did not show any initial detachment from concrete. At the end of the tests, when the amount of applied displacement was around 245 mm, the central portion of some laminas began to detach, Fig. 7. On the other hand, both ends of lamina had no problem of loosing adhesion to concrete, due to the over application of wrap.

The confinement of the column ends was satisfactory, as no sign of damage appeared in the wrapped regions.

The beam to column rotation, measured by means of inclinometers, emphasised the column inflection near the base. At the end one longitudinal bar broke, without previous buckling. The break occurred with a loud clack, and a sudden loss of lateral stiffness.

The sheets appeared quite effective in delaying the longitudinal bar failure. The strain in the vertical bars at the column base has been measured on the basis of the inclinometers and displacement transducers. It has been found that the axial strain has been within the bounds:

$$10.5\ \varepsilon_y < \varepsilon_{max} < 21.6\ \varepsilon_y.$$

Under these deformations, in normal conditions the concrete cover would have spalled from the column, and the bar would have buckled. Thus, it has been concluded that the applied sheets have provided a suitable confinement to the column. Notice moreover that in these conditions the bar has entered the hardening branch of the stress-strain diagram. One or two load reversals in this range of deformation lead invariably the bar to a brittle fracture (Brittle = without necking). The fact that a one bar failure did occur is thus not surprising. It would had occurred well before if the bar had buckled.

The final failure was of the strong-beam, weak-column type. Measurements from inclinometers as well as from visual inspection (Fig. 8) demonstrated that plastic hinges took places at columns ends, with the only exceptions of the external joints of the first storey, where plastic hinges formed in the beams, as a result of poor detailing of the anchorage of longitudinal bars.

Fig. 7: Resulting damage.

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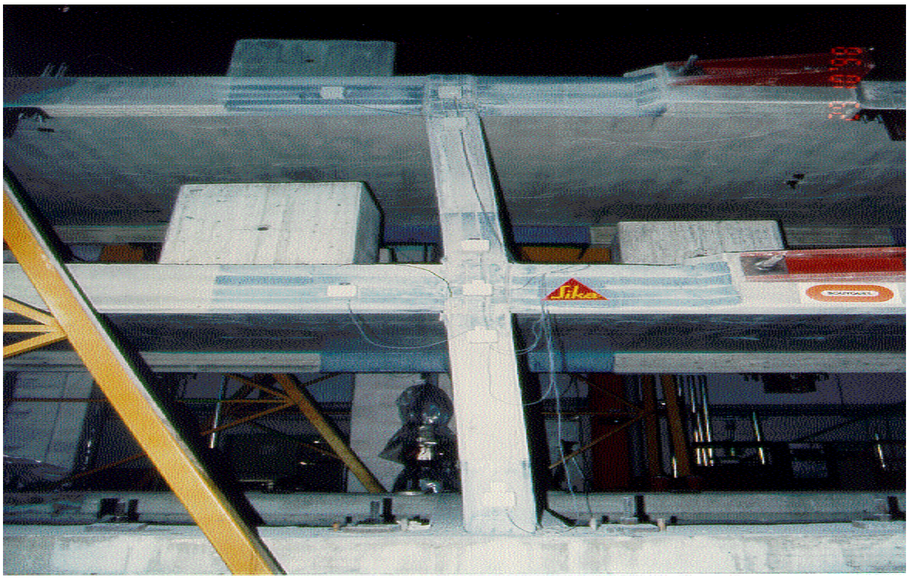

Fig. 8: Deformed frame at maximum top displacement.

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