

Shear Strengthening of Reinforced Concrete Beams with External Steel Bars and with CRFP

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SUMMARY

The seismic retrofitting of existing buildings presents significant problems all around the world. Their study and assessment indicate that many different problems can be detected. Structural engineers need to apply several strategies to fulfil retrofitting process objectives. Achieving appropriate solutions is difficult task, particularly in developing countries: economical reasons or lack of technology reduce the available alternatives. This paper describes main results and conclusions obtained from an experimental research program conducted to evaluate response of two different shear strengthening schemes for applying to a high rise building. Objectives of proposed solutions are to (i) increase the shear strength of beam allowing the development of ductile plastic hinge at their ends and (ii) provide lateral support to longitudinal beam bars, to control their buckling. The paper also intends to verify soundness of some simplified models recently proposed to assess yield and ultimate curvature, rotation and displacement, comparing with experimental results here reported.

Keywords: Shear Strengthening, Reinforced Concrete beam, Threaded Bars, CFRP.

1. SEISMIC RISK AND BUILDING CODE REGULATIONS

The Poder Judicial Building, PJB, to be retrofitted is located in Mendoza, 1100 Km west of Buenos Aires, Argentina. The city was destroyed on March 1861 by a very strong shaking. It is within the highest seismic region of the country. Since that devastating event, just minor tremors have occurred nearby and therefore all building stock constructed in the last 150 years have not been tested by the current ultimate state design earthquake.

The PJB was designed and constructed during the 70's, just before the first official seismic code was to be enforced (CCA, Mza, 1970). As in any part of the world, at that time, the capacity design principles were not known and, even for relatively high buildings, the shear reinforcement resulted from the fictitious code forces rather than from the flexural capacity. Furthermore, in those days the 4.2mm diameter steel bars were allowed to be used as transverse reinforcement in Argentina. Lateral seismic coefficient adopted was 10 %, and reinforced concrete members were proportioned and detailed according to German DIN 1045 regulations. The proposed retrofitting was based upon the new Argentine seismic code (IC-103, part I, 2005) which follows the New Zealand earthquake resistant design philosophy (Paulay and Priestley, 1992).

2. DESCRIPTION OF THE BUILDING STRUCTURE

Strengthening of PJB girders are main objective of this paper, is a ten story public office construction, with one basement, 9500 m² covered area and almost 40 m total height. Figure 1 shows the PJB structural plans and the front façade. It is a complete in situ cast reinforced concrete structure. The original design attempted to consider spatial frames as the only main lateral resistance system for both directions. However, as can be appreciated in Figure 1, some squat reinforced concrete walls are present in the lower stories in the longitudinal direction, and from the foundation to the top two channel-shaped walls enclose the area of elevators and stairs. As was the practice of those days in Mendoza, all

reinforced concrete walls were ignored in the design and the whole seismic demand was allocated to the frames. Under such circumstances, should an intense quake hit the building, brittle shear failures can be anticipated since, once again, transverse reinforcement was assigned just to cover minimum steel requirements, typically 6mm bars spacing 150mm , many times in a single mesh at the middle of the wall thickness. Strengthening of the walls is outside the scope of this paper.

At a glance, the structure was reasonably symmetric, particularly in the longitudinal direction and, for both directions, in the emerging eight stories high portion tower. The vertical circulation core is almost centered. In lower stories, the centre of mass migrates some distance to the front facade but the mass induced eccentricity is not dramatic and it is not an issue.

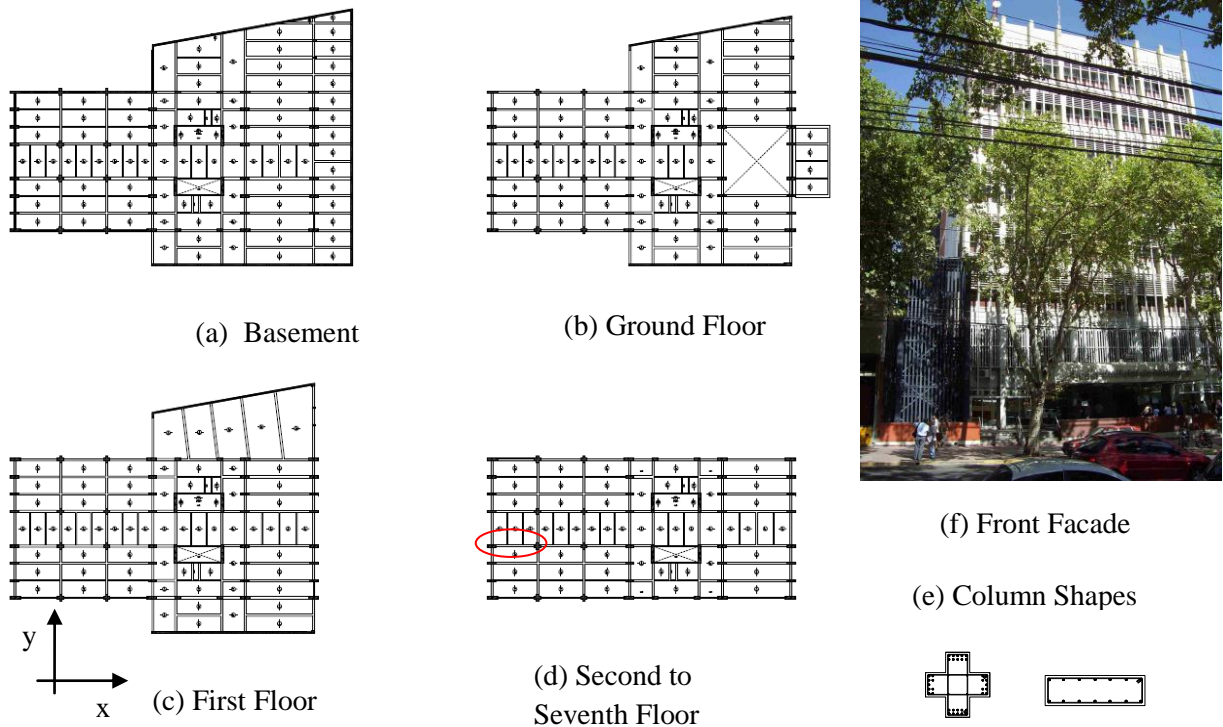


Figure 1. Structural Plans of the building and front facade. In read ellipse (d) tested girders.

Most of columns were either rectangular 400mm or 360mm wide by 1000mm or 1200mm high, or cross-shaped section with similar dimensions. It was fortunate that the column design to flexure was very conservative and therefore, in almost all cases columns are stronger than girders. Furthermore, for the transverse steel in columns, 8mm bar diameter were used.

Slabs are supported in one direction with 100mm or 120mm thickness. Once again, 4.2mm bars are used in the weak direction. No damage for acting gravity loads were observed to this time and the same applies to foundations and the supporting soil.

Structural walls were ignored for the seismic lateral resistance. However, exterior squat walls have enough longitudinal steel for supporting lateral soil pressure. Even though squat shear mechanisms (Paulay and Priestley, 1992) were not enough to withstand elastic demands, during the retrofitting design process, the assigned force were tuned according to what is needed for the rest of the structure. The U-shaped interior walls, due to high reinforcement in the flange borders, lack of transverse reinforcement and will need also to be shear strengthened. Typical interior girders are 300mm - 600mm , width-total height. In the outside frames proportions change to 250mm - 1100mm respectively. For all girders, transverse reinforcement consist of two legs of 4.2mm diameter spacing from 100mm to 250mm . Figure 2 shows typical reinforcement details for interior girders at roof level. Regarding flexural reinforcement, it was a positive fact that for gravity loads the original criteria was to consider girders almost simple supported on columns and therefore when superimposed to seismic actions the demands for positive and negative moments were almost identical. As a consequence, and can be seen

in the figure, girders resulted with almost the same reinforcement for both positive and negative support moments (6-20mm bars on the left, 5-20mm on the right). However immediately, and just by inspection, several simultaneous problems become apparent: (i) diameter of the stirrups, 4.2mm, are very small and too spaced to preclude premature buckling of the longitudinal bars; (ii) for the same reason, there is not enough stirrups to resist shear when flexural resistance along with large ductility demands; (iii) large discontinuity in flexural and shear strength since only 2-10mm bars as basic top beam reinforcement and all bend-up (normally 3-20mm bars) located at the same position. The overcoming of these drawbacks regarding the potential poor response of the building against seismic attack is presented next.

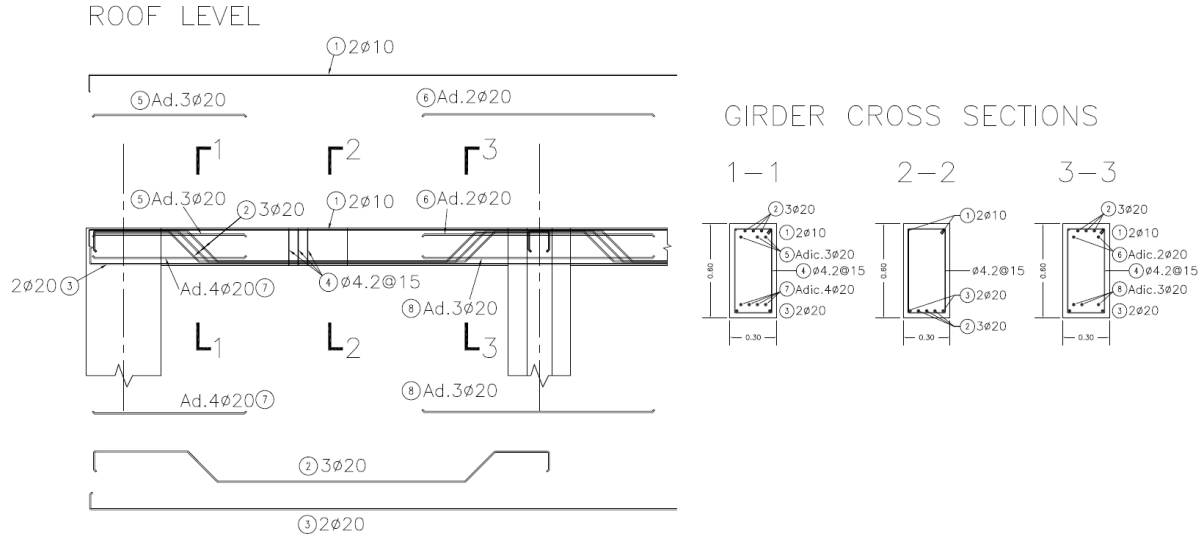


Figure 2. Typical reinforcement for an interior girder at roof level.

PJB did not presented at time of inspection any visible damage. However, regarding seismic events, it was never compromised since its construction. There exists minor cracking as observed in any normal reinforced concrete construction. Quality of concrete, evaluated through cylindrical 3 inches (75mm) diameter samples, was good, average 21MPa for slabs and girders and 17MPa for walls and columns. The steel available at the time of construction was cold twisted, $f_y=420\text{MPa}$. After tested in the lab, 4.2mm and 6mm diameters resulted with average $f_y=600\text{MPa}$ and $f_{max}=650\text{MPa}$, while 12mm and 16mm diameters resulted $f_y=500\text{MPa}$ and $f_{max}=550\text{MPa}$. No defined yield region, low strain hardening and low ultimate strain were observed. Beam-column joints, in general, satisfied the code condition (IC-103-2005) that the maximum nominal shear stress should not exceed $0.16f'_c$.

3. CODE REGULATIONS FOR DUCTILE BEHAVIOUR

A summary of current code regulations (IC-105-2005) are revised and contrasted against the design of the girders under study, in particular with respect to transverse reinforcement.

- (i) Minimum diameter bar 6mm: not satisfied (used 4.2mm)
- (ii) Maximum spacing in plastic hinge regions $6d_b=120\text{mm}$ (for 20mm longitudinal bar) and $d/4=137\text{mm}$, normally not satisfied (150 mm was the usual spacing)
- (iii) Minimum area of stirrup with respect to area of supported longitudinal bar against buckling: $A_{te}=(\sum A_b/16)(s/6d_b)$, not satisfied, since for one 4.2mm leg spacing 150mm, the maximum area allowed to support is 177mm^2 , i.e., not even one 20mm (area 314mm^2) bar is code protected.
- (iv) Minimum steel reinforcement for shear in plastic hinge, where $v_c=0$. For the case of Figure 2, seismic action from the left, positive nominal moment of the left is about 420KNm and negative nominal on the right is close to 360KNm. Overstrength moments are 588KNm and 504KNm respectively, and when shear due to gravity, about 100KN, is incorporated, the maximum shear demand results $V_u=[(588+504)/5.60+100]\text{KN}=295\text{KN}$. The resulting shear stress is $v_u=1.80\text{MPa}$. The required transverse reinforcement would be then $A_v=(1.80 \times 300 \times 120/420)\text{mm}^2$, and therefore, three 8mm bar legs,

spacing 120mm is required. Only two 4.2mm spacing 150mm are supplied, i.e., almost seven times less than code requirements. Minimum shear steel reinforcement given by $A_{min}=(0.333/f_y)b_ws=(0.333/420)\times30\times120\text{mm}^2=29\text{mm}^2$ would be almost satisfied if the spacing for the 4.2mm stirrups would have been 120mm , not 150mm as it was.

4. PRELIMINARY 1:2 SCALE TESTS

Final objective of this research was to investigate 1:1 scale models to implement strengthening of girders at column supports. Dimensions of the column-beam-slab subassemblages are considerable as it is the associated cost for construction, handling and testing, among other aspects. Therefore, it was decided to build the minimum necessary amount of 1:1 scale models but to perform previously a series of tests on 1:2 scale models, Figure 3, which served to tune up the final proposals at real scale.

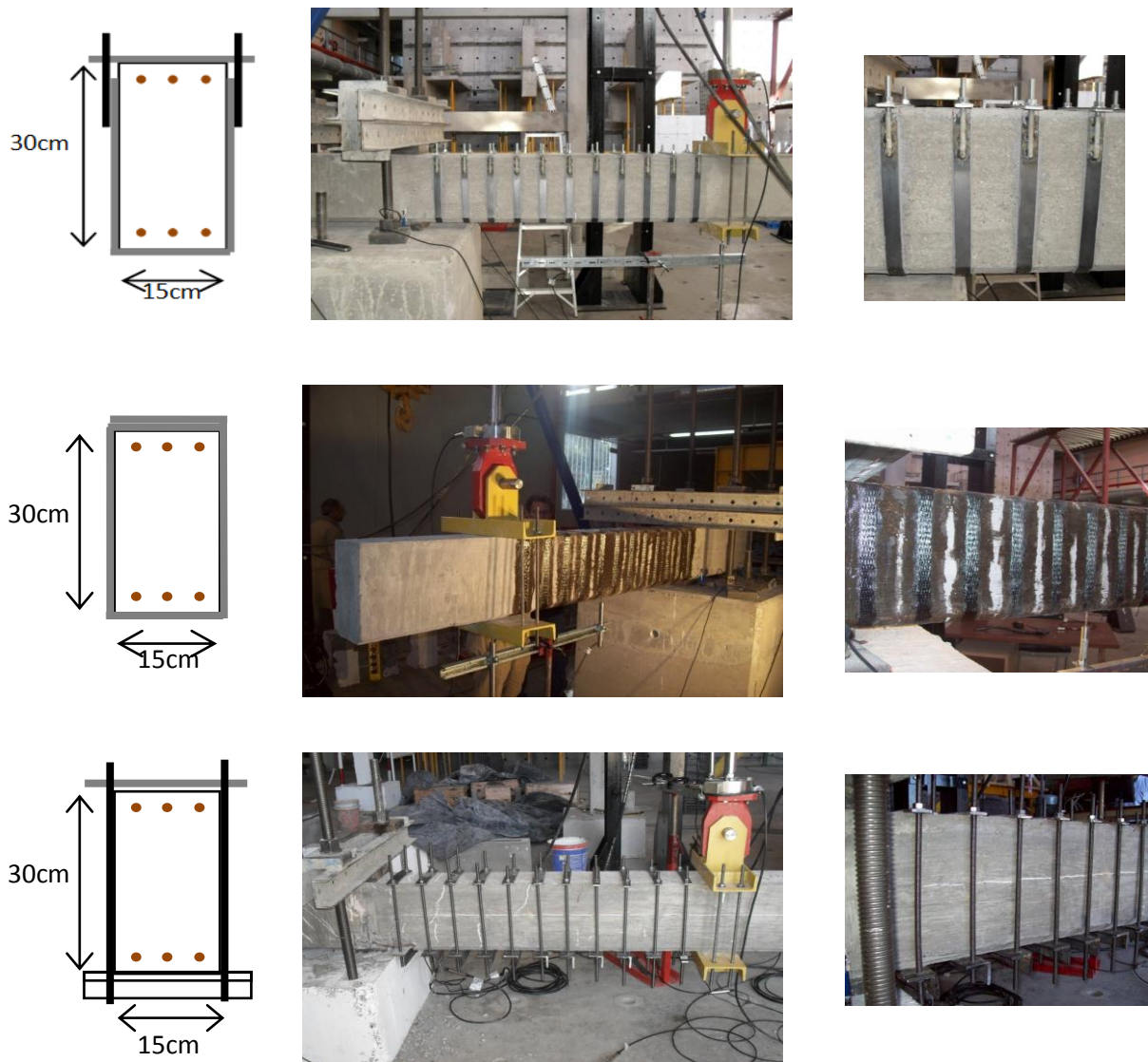


Figure 3. Half scale models strengthened for shear and to delay buckling of longitudinal bars:
(a) Steel plates with welded threaded bolts and top plate;(b) CFRP full wrap and (c) Threaded bars with top plate and bottom angle

Dimensions of scaled models are 150mm width and 300mm total height. Three strengthening strategies, as shown in Figure 3, were implemented: (a) see Figure 3(a), U-shaped steel plates whose flanges are

welded to crossing-slab threaded bars (to simulate the real construction) which in turn are adjusted through a crossing transverse plate; (b) Figure 3(b), strips of Carbon Fiber Reinforced Polymers) CFRP, making a full wrapped member, and (c) Figure 3(c) threaded bars each side of the web, adjusted by bolting to plates at top and angles at bottom of girder faces. Two additional half scale models were tested, one practically without stirrups (just one in the middle of the length to keep longitudinal bars in position), and the other with transverse reinforcement designed according to capacity design. Both served as reference since the first showed the behavior controlled by shear failure and the other gave the response to be imitated to. Main conclusions from these tests are:

- (i) The element without stirrups had a sudden and very brittle failure.
- (ii) The girder designed for capacity against shear reached the maximum flexural strength as predicted, and was able to develop a displacement ductility exceeding 5 (five) without loss of strength. The test was interrupted before total collapse occurred.
- (iii) The 3 (three) girders strengthened to shear behaved with similar response to the previous well designed. However, the element with CFRP did not showed good response when only one FRP wrapped the section. It is recommended, (ACI 440-F, 2010, and ACI- 440.2R-08, 2008), that a minimum of two plies be used for proper confinement response in plastic hinge regions. The response with FRP improved substantially when this recommendation was implemented in the tested girder.
- (iv) The authors of tests reported here felt more confidence with use of the U steel shape, case (a), but its implementation is more expensive and difficult to build in the field than the simplest solution which resulted to be case (c). This is also the cheapest among all.
- (v) The use of FRP requires good knowledge of the fiber to apply as well as experienced workmanship to follow all necessary previous preparation of the surface.

5. FULL SCALE MODELS

Typical interior frame girders are T shaped, 6600mm span at column axes, 300 mm web width, 600mm total height supporting slabs of 100mm thickness. Therefore, the full scale models are subassemblages of same dimensions, with flange of the 1650mm as effective width since it responds to the ratio $l/4$ and close to the (b_w+16h_s) permitted for the code. The girder longitudinal steel was almost symmetric for positive and negative moments at supports. For negative moments there exists some small additional contribution coming from the basic steel adopted as top bars, $2\text{-}10\text{mm}$ diameter bars, and from the secondary steel of the slab, namely 4.2mm spacing 150mm .

Three models with identical steel arrangement were constructed, as shown in Figure 4, except for the first subassemblage with transverse girder reinforcement designed applying principles of capacity design. In all cases, free end of the girders were heavily reinforced with stirrups to avoid any failure at point loading. In the PJB girders, longitudinal steel changed from 4 to 8 bars- 20mm diameter. Web steel ratio then changes from 0.77% to 1.55% , top and bottom. Figure 4(d) indicates 1256mm^2 for bottom reinforcement and 1417mm^2 for the top negative, corresponding almost 0.8% longitudinal steel ratio. Figure 5 shows test scheme, both top and bottom of columns hinged to simulate zero moment at middle column. Static reversible load applied to girder end, with length simulating half span of the girders. On the right appears the hydraulic jack 50ton capacity, with 400mm maximum stroke.

The end girder vertical displacements were measured by two LVDTs, and vertical and horizontal movements of the supporting blocks were also monitored for corrections. Rotations and curvatures are measured at three girder sections (three for each section, one on top and two at the bottom), close to the column face. Relative displacements are detected through linear potentiometers spanning between pieces of steel bars 4.2mm diameter which are fixed to girder longitudinal bars. A total of fourteen sensors were installed. At the centre, Figure 5(b) scheme of instrumentation installed.

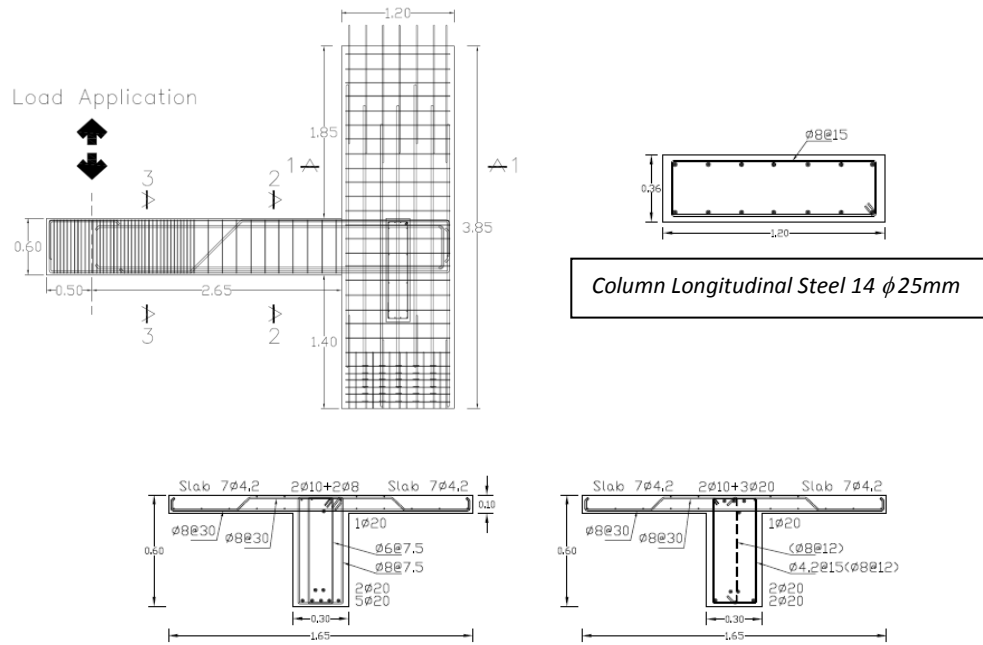


Figure 4. Full scale model: (a) Above left, longitudinal view section of the tested girder, showing also the transverse beam framing also to joint; (b) Above right, cross section of column; (c) Left below, girder cross section 3-3 and (d) Right below, girder cross section 2-2. The transverse reinforcement under ($\phi 8\text{mm}$ diameter) applies to the girder designed for capacity.

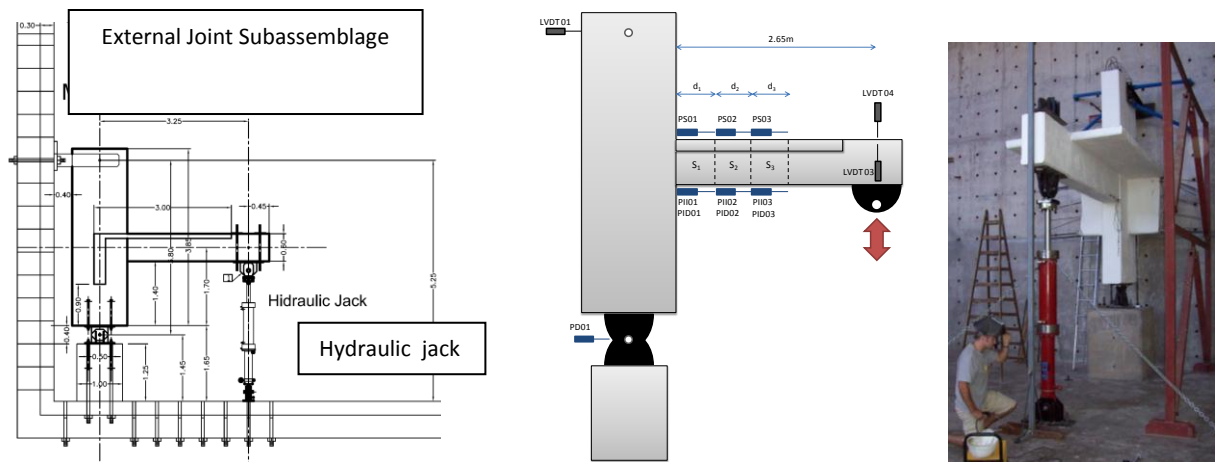


Figure 5. Full Scale Model: (a) testing scheme; (b) instrumentation and (c) full scale model fixed to 800mm thickness L-Slab-Wall reacting system at the IMERIS structural laboratory.

5.1 Shear Capacity Design Model

5.1.1 Local and Global Response.

The first model tested, shown in Figure 6, intended to satisfy all current argentine requirements (IC-103-2005) regarding transverse reinforcement. Figure 7 shows the complete local response in terms of Moment vs. Curvature. The result for the testing the ADN-420, designation for the type of steel, $f_y=420\text{MPa}$, used as longitudinal reinforcement in the girders, 20mm diameter bar is included.

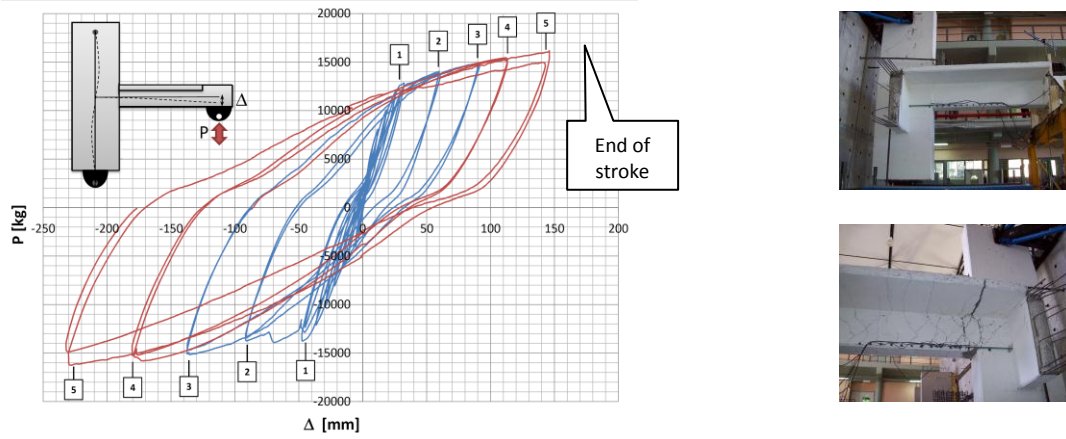


Figure 6. Global response of the model with shear reinforcement designed after flexural capacity.

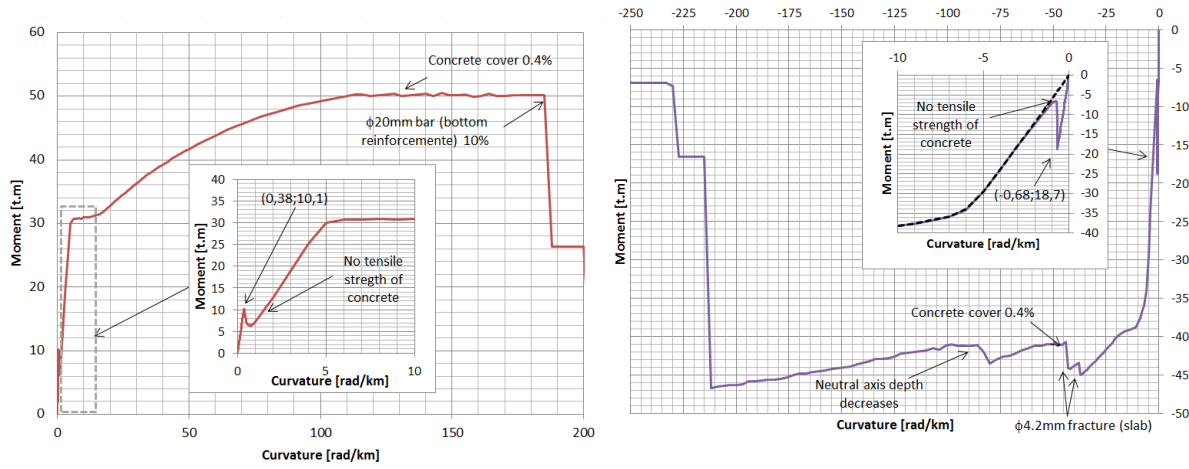


Figure 7. Girder $M-\phi$ complete response: (a) positive and (b) negative moments. $f-\epsilon$ for steel 20mm bar.

Concrete was $f'_c=25MPa$. It can be seen that: (i) for the steel, $f_y=450MPa$ and $f_{max}=720MPa$; (ii) from $Mvs\phi$, $M_y^+=310KNm$, $M_{max}^+=500KNm$ and $M_y^-=390KNm$, $M_{max}^-=460KNm$; and (iii) $P_y=125KN$ and $P_{max}=160KN$. Quick verifications can be done applying some simplifications. For example: $M_y^+=4\pi 100mm^2 450Nmm^{-2} 0.55m=311 \times 10^3 N=311KNm$. A good approximation is done (Priestley, 2003) for the yielding curvature, which depends on geometry but not in strength. For T-Section beams $\phi_y=1.70\epsilon_y/h_b=1.7 \times 0.00214/600mm=6.10 \times 10^{-6} rad/mm$. From Figure 7(a) $\phi_y=(500/310)4 \times 10^{-6} rad/mm$, just 5% difference w.r.t. the more sophisticated analytical prediction.

Regarding global response, the yield positive load is well approximated since $(310KNm/2.65m=117KN)$, close to the value given in Figure 7(a). Furthermore, yield displacements Δ_y can also be approximated (Priestley et al, 2007), applying the expression for yield curvature, and taking into account the rotation of the joint and the yield penetration or L_{sp} , length of strain penetration following notation of the reference. Other components of the beam displacement are the flexural deformation of columns and the shear deformation of both, beam and joint. In this paper L will designate the distance from column axis to loading point of the beam, i.e., $L=3250mm$; L_b is the clear length of beam, i.e., $L_b=2650mm$. The length of strain penetration can be approximated (Priestley et al, 2007) by the equation:

$$L_{sp} = 0.022 f_y d_b = 0.022 \times 450MPa \times 20mm = 198mm. \text{ Next, each component of } \Delta_y \text{ is evaluated.}$$

Elastic Flexural deformation of the beam (taking into account yield penetration):

$$\Delta_{by}^b = \theta_y^b (L_b + L_{sp}) 2/3 = \phi_y^b (L_b + L_{sp})^2 / 3 = 6.5 \times 10^{-6} mm^{-1} (2650 + 198)^2 mm^2 / 3 = 17.60mm$$

Flexural deformation of column:

It is necessary evaluate the curvature distribution in the columns above and below the joint. Then a rotation of column is calculated which multiplied by L gives the contribution to yield displacement. The moment at column axis when beam yields is $M_{by} = 125KN \times 3.25m = 406KNm$. It is assumed that half moment is taken for each portion of column above and below, i.e., $203KNm$. From the same reference, the yield curvature of a rectangular reinforced concrete section can be approximated to:

$$\phi_y^c = 2.10\varepsilon_y / h_c = 2.10 \times 0.00214 / 1200mm = 3.75 \times 10^{-6} mm^{-1}.$$

An estimation of the curvature developed by the columns when beam yields must be done and this will depend upon the column stiffness which in turn is a function of its strength. The nominal flexural strength of the column can be approximated (no axial is present) by:

$$M_n^c \cong A_{st,t} f_y [(h_c / 2) - d'] = 12 \times 491mm^2 \times 0.7 \times 700MPa (600 - 50)mm = 1580KNm$$

It was assumed that maximum strength would be reached at 70% of $f_{max} = 700MPa$, and that 12 out of 14 bars are yielding in tension. It is clear that the columns will remain elastic, and by a linear relationship, the curvature in the columns when beam is yielding results:

$$\phi_{by}^c = 3.75 \times 10^{-6} mm^{-1} (203 / 1580) = 0.48 \times 10^{-6} mm^{-1} \text{ (column curvature when beam yields). Then:}$$

$$\Delta_{by}^c = \theta_{by}^c L = \phi_{by}^c 0.5H_c L = 0.48 \times 10^{-6} mm^{-1} \times 0.5 \times 3800mm \times 3250mm \cong 3.0mm$$

Shear deformation of beam and joint:

For brevity reasons they are not presented here but it was found, (Llopiz et al, 2011), to be $2.80mm$ and $4.40mm$ respectively. Finally, the prediction would be:

$$\Delta_y^b \cong (17.6 + 3.0 + 2.8 + 4.4)mm \cong 28mm, \text{ close enough to } 30mm, \text{ Figure 7(a) for positive moment.}$$

Proportions of each source ($0.63 + 0.11 + 0.10 + 0.16 = 1.0$) also agree well with the reference.

Next, an estimation of the maximum or total displacement reached in the test is attempted. The total displacement is $\Delta_{max} = \Delta_y + \Delta_p$. The last term can be evaluated from the simplified approach based on the concept of plastic hinge length, L_p , (Priestley et al, 2007). The plastic hinge length is given by:

$$L_p = kL_c + L_{sp} \geq 2L_{sp}, \quad k = 0.2[(f_u / f_y) - 1] \leq 0.08. \text{ Then, } k = 0.2[(7 / 4.5) - 1] = 0.11, \quad k = 0.08.$$

L_c is the length from the critical section to point of contraflexure in the member, $L_c = L_b = 2560mm$ in this case. Then, $L_p = 0.08 \times 2650mm + 198mm = 212mm + 198mm = 410mm$ (almost $0.70h_b$).

Next, each component of Δ_p is evaluated.

Plastic Flexural deformation of the beam (takes into account yield penetration):

$$\Delta_{bp}^b = \theta_p^b L_p L_b = (\phi_u^b - \phi_y^b) L_p L_b = (136 - 6.5)10^{-6} mm^{-1} 410mm 2650mm = 141mm,$$

where ultimate curvature ϕ_u was selected for the stage where concrete reaches at top $\varepsilon_c = 0.4\%$, bottom steel $\varepsilon_s = 6\%$, as seen in Figure 8(a).

Flexural deformation of column:

As before, the moment at column axis for ultimate load is $M_{bu} = 160KN \times 3.25m = 520KNm$, then $260KNm$ for each column, and the moment increase from the yielding state is $57KNm$.

$$\phi_{bp}^c = 3.75 \times 10^{-6} mm^{-1} (57 / 1580) = 0.135 \times 10^{-6} mm^{-1}. \text{ Then:}$$

$$\Delta_{bp}^c = \theta_{bp}^c L = \phi_{bp}^c 0.5H_c L = 0.135 \times 10^{-6} mm^{-1} \times 0.5 \times 3800mm \times 3250mm \cong 0.85mm$$

Shear deformation of beam and joint:

Adjusting for the increment of 28% from $P_y = 125KN$ to $P_u = 160KN$, results $0.84mm$ and $1.26mm$ respectively for beam and joint shear deformations. Finally, the prediction would be:

$$\Delta_p^b \cong (141 + 0.85 + 0.84 + 1.26)mm \cong 144mm,$$

Total positive deformation is then $\Delta_{max} = 28mm + 144mm = 172mm$, close enough to the value of $150mm$ given in Figure 7(a), with the tendency to increase. The end of stroke makes the test to stop. Assuming that $170mm$ could be reached, $\mu_\Delta = \Delta_u / \Delta_y = 170 / 28 = 6$, is the displacement ductility factor corresponding to a curvature ductility factor $\mu_\phi = \phi_u / \phi_y = 136 / 6.5 \cong 21$.

By inspecting Figure 6(a), it can be seen that the maximum negative displacement was close to 230mm, and since yield displacement is almost 40mm, the ductility on reverse load was quite similar.

5.1.2 Design of Plastic Hinge for Ductile Behaviour.

From analytical predictions, $M_{max}=500KNm$ for positive and $M_{max}=460KNm$ for negative case, considering real $f_s-\epsilon_s$ relationship for the steel. Code nominal moment with specified $f_y=420MPa$ was $M_n=300KNm$. Applying the code material overstrength factor $\lambda_o=1.40$, code overstrength moment is $M_o=420KNm$. The tested beam will be subjected, should an earthquake occur, to an additional 37KN/m due to gravity loads and therefore, the ultimate shear demand is:

$$V_u = [(500 + 460)KNm / 5.30m] + 37KN / m \times 2.65m = 180KN + 100KN = 280KN$$

Maximum stirrup spacing is $s=6 \times 20mm=120mm$, or $s=d/4=560mm/4=140mm$.

It was adopted three 8mm legs spaced 120mm, as shown in Figure 5(c). The nominal shear strength is:

$$V_n = 3 \times 50mm^2 \times (560mm / 120mm) \times 0.420KN / mm^2 = 294KN > 280KN$$

This shear resistance was taken as reference for designing the strengthening of the other two models.

5.1.3 Analysis of observed experimental response.

Stable elastic response resulted after many reversal cycles. Very ductile behaviour was achieved up to displacement ductility $\mu_\Delta=6$. Typical flexure-shear cracks appeared along the test, and at large ductility demands longitudinal steel starts buckling. Minor cracks observed in the joints and columns.

5.2 Model Strengthened by Vertical Threaded Bars

Two threaded bars 11.11mm diameter, net area 58mm², spacing 100mm, steel SAE 1010, $f_y=400MPa$, Considering the existing stirrups, 2-4.2mm spacing 150mm, $f_y=420MPa$, result in a shear resistance:

$$V_n = (560/100) \times 2 \times 58mm^2 \times 0.4KN / mm^2 + (560/150) \times 2 \times 14mm^2 \times 0.42KN / mm^2 = 260 + 44 = 304KN$$

As can be seen from Figure 8, the global response was very similar to the standard code capacity design model.

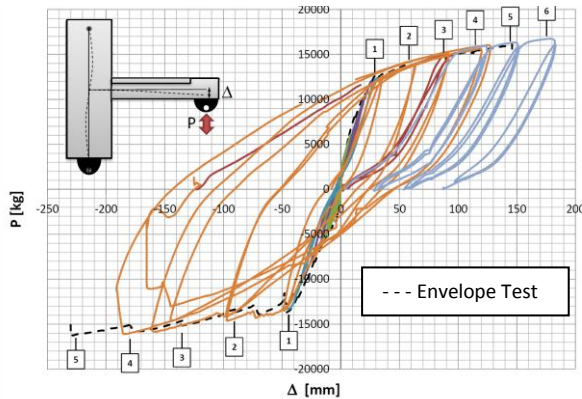


Figure 8. Global Response of Model with Vertical Threaded Bars.

5.3 Model Strengthened by CFRP

Code requirement, effective strain is $\epsilon_{fe}=0.04 \leq 0.75 \epsilon_{fu}=0.75 \times 0.015=0.01125$

Double strip must be used in plastic hinge regions. CRFP supplier, Elasticity modulus $E_f=234.5KN/mm^2$. Effective tension Stress $f_{fe}=E \epsilon_{fe}=234.5KNmm^{-2} \times 0.004=0.938KNmm^{-2}$. Effective strip thickness is 0.34mm. Strip width adopted 85mm. Spacing is 200mm (should not exceed $d/4 + \omega_f=140mm+85mm$), ω_f width of the strip. Then:

$$V_{nf}=(560/200) \times 4 \times 85mm \times 0.34mm \times 0.938KNmm^{-2}=304KN$$

From Figure 9, the global response was also very satisfactory. Wrapping the girder with the same web strip will not (and cannot) be used in the real building. Fibre anchors 5/8 inches=15.90mm diameter will

be used, not available at the time of testing. This will avoid the strong weakening of the slab, as observed in the figure.

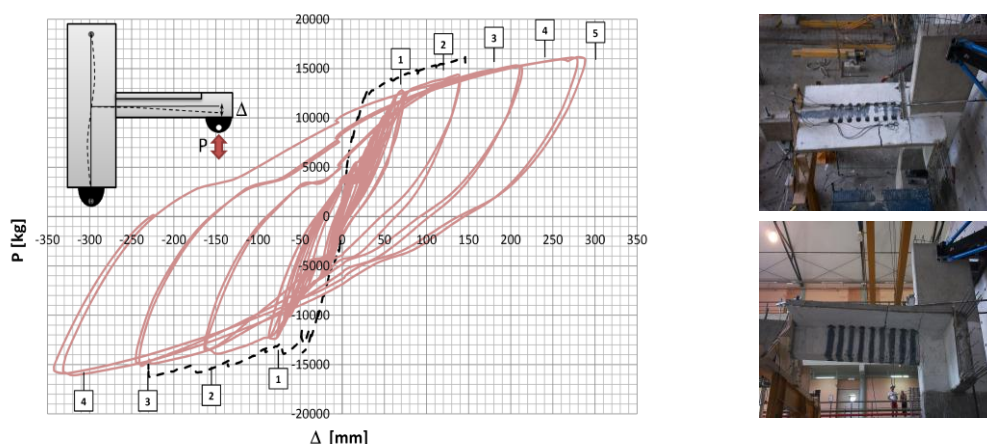


Figure 9. CFRP Strengthening Global Response.

6. CONCLUSIONS

Experimental tests on three full scale beam-column joints were conducted to evaluate response of two different shear strengthening schemes for the retrofitting of beams in an existing RC building, namely, (i) the use of vertical threaded bars with top plate and bottom angle and (ii) full wrapping of the beam section with CFRP bands. Test results showed that both techniques are adequate to increase the shear strength and to prevent buckling of the longitudinal steel. Both retrofitted models were able to develop a structural response similar to that obtained from a specimen designed according to modern seismic requirements. However, the solution with vertical threaded bars appears very attractive: it is simple to build, economical and does not require specialized workmanship.

Experimental data obtained from the tests were compared to analytical results. This comparison indicates that the simplified models used to predict yield and ultimate curvatures, rotations and displacements are very effective.

ACKNOWLEDGEMENT

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