Retrofitting RC Members with External Unbonded Rebars

A. Tinini, F. Minelli(✉), and G. A. Plizzari

Department of Civil, Environmental, Architectural Engineering and Mathematics, University of Brescia, Brescia, Italy
fausto.minelli@unibs.it

Abstract. External unbonded rebars represents a suitable strengthening technique for the retrofitting of existing Reinforced Concrete (RC) members. Advantages regard the ease of installation, a minimum invasiveness and possibility of future inspections. Structurally, increment of flexural stiffness and bearing capacity and enhancement of shear-flexure behavior can be achieved. The presence of both bonded and unbonded bars introduces a change in the way the shear actions are resisted. Unbonded rebars develop an arch action component, with no bond present and constant force in the rebars, in addition to the beam action component, normally developing in presence of bonded bars. The present paper reports the results of four point loading tests on full-scale beam, with the aim of studying the influence of different bond condition. Moreover, the Double Harping Point technique, using external rebars and vertical deviators, is presented, with attention to the definition of the vertical equivalent stiffness of the deviators.

Keywords: Shear strength · Unbonded rebars · Beam and arch action
Retrofitting

1 Introduction

In recent years, an increasing attention has been devoted to the assessment and the structural rehabilitation of existing reinforced concrete (RC). This is mainly due to problem connected with structural deterioration, with more onerous requirements in Standards and Building Codes or with the change in use of the building, resulting in an increase of the loading.

The retrofitting of existing RC elements by means of external steel rebars was already studied in previous works (Cairns et al. 2005; Minelli et al. 2008). The technique is illustrated in Fig. 1. High yield threaded bars are applied to both sides of (or, in some application, under) a RC beam. The bars pass through yokes at the ends of the beam, where they are anchored by locknuts. On all but short spans there are benefits from use of deflectors to avoid a reduction in effective depth as the beam deflects. External bars can thus easily be installed by hand: no significant prestress is required and only sufficient force to firmly secure the bars or to obtain low post-tension action is applied.

M. di Prisco and M. Menegotto (Eds.): ICD 2016, LNCE 10, pp. 184–199, 2018.
https://doi.org/10.1007/978-3-319-78936-1_14

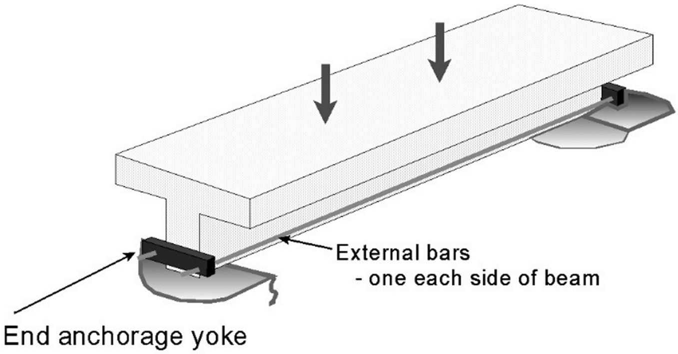

Fig. 1. External unbonded bars reinforcement

The Authors highlighted some of the advantages of that reinforcement technique, resulting a more cost effective and less disruptive solution to the problem of strengthening simply supported RC elements. Moreover, the use of external unbonded reinforcement presents many of the merits of post-tensioning with unbonded tendons, dispensing at the same time with the need of specialist stressing operation. Less clearance is needed for the end anchorages, since there is no need to provide access for jacks.

The corrosion protection can be based on systems used for external prestressing. The technique is also compatible with principles of conservation, which require that a structure be returned to its original condition after any interventions.

Some disadvantages are also present: the system is limited to be used for the strengthening of simply supported beams, the end anchorages position represents a risk of anticipated failure (only in case it is not possible to install them over the support points) and space limitation may be present for systems applied under the element to be reinforced.

The concept of retrofitting by means of unbonded external reinforcement developed from observations of reinforced concrete beams when concrete around bars is broken out during repair actions. The study showed that the debonding of the longitudinal rebars might cause little reduction in strength (Cairns and Zhao 1993) or even an enhancement in ultimate strength of beams deficient in shear (Cairns 1995), thanks to development of an arch action component, which represent a more stable shear resistant mechanism. Without bond, the force in reinforcement must be constant along the length of the beam. The lever arm between reinforcement and concrete, and not bar force, must vary with bending moment. The neutral axis depth therefore also varies, in a manner consistent with arch rather than shear-flexure action. Concrete is, of course, better at resisting compressive stresses than shear stresses. It might thus be expected that this change from a purely flexural mode of failure towards a tied arch/flexure hybrid would be accompanied by an increase in shear capacity.

The potential of retrofitting using external bars anchored only at the ends of simply supported flexural elements (Fig. 1) has been demonstrated for flexural modes of failure (Cairns and Rafeeqi 2003), reaching a strength enhancements of the order of 100%.

In this paper, results concerning the behavior of member with embedded plain bars are also presented. Many existing structures, built after the second world war (up to the 70 s), present plain bars as longitudinal reinforcement. That type of rebars, not having lugs or any other surface deformation, are not able to transfer the bond stress by means of mechanical interlock. For this reason, the bond transfer mechanism is less efficient than in the case of members reinforced with deformed bars.

In the early Twentieth Century, an intensive experimental campaign on members with plain bars was performed (Abrams 1913). The Author highlighted two main resistant mechanisms: adhesive resistance and sliding resistance, the latter arising from inequalities of the bar surface and irregularity of its section and alignment together with the corresponding conformation in concrete.

Other researches further demonstrated that adhesive resistance can be attributed to the chemical adhesion mechanism and the micro-interlocking of concrete keys generated by the penetration of cement paste into the indentation of the bar surface (Stoker and Sozen 1970).

Tassios (1979) pointed out that a small slip is needed for activating micro-interlocking and developing the maximum adhesive resistance.

In recent years, an experimental research was carried out to investigate the effect of flexural cracking and bond loss on flexural behavior (Feldman and Bartlett 2008). The Authors tested two beams, critical in flexure, presenting the same contact area but different reinforcement ratios. The beam with the higher reinforcement ratio (0.98%) exhibited arch action because of the bond failure at about 60% of the maximum load. The other member, with a ratio of 0.33%, presented a predominant beam action up to the failure.

The present study was therefore undertaken with the aim of studying the influence of different bond condition (deformed, plain and unbonded rebars) on the shear-flexure behavior of RC members. Plain rebars, behaving similarly to deformed rebars for very low slip and to unbonded rebars for higher slip values (Tinini 2016), represent a type of reinforcement of great interest from the structural rehabilitation point of view. Therefore, they are included in the study in order to consider a more comprehensive case study.

2 Experimental Investigation

2.1 Specimen Geometry

The experimental program concerned seven full-scale beams tested under a four point loading system with a shear span-to-depth ratio *a/d* of 2.5, the most critical for shear resistance of RC members with no transverse reinfocement.

A total depth of 500 mm was chosen, with a gross cover of 40 mm. According to the *a/d* ratio considered, the shear span length was 1150 mm, while the total span length was of 4300 mm and the overall length 4500 mm. Four of the experimental specimens were made considering different type of embedded rebars (deformed (*B*), plain (*S*) and 2 unbonded (*U*), with deformed rebars placed through a corrugated plastic pipe that covered them), and three combining in pairs the different levels of bond condition considered (*BS*, *BU* and *SU*). Rebars having a diameter of 20 mm were used in case of bonded and unbonded rebars, while 22 mm diameter bars were used for plain bars. This resulted in a reinforcement ratio varying from 1.09% to 1.32%.

Figure 2 reports the main geometrical properties of the specimens.

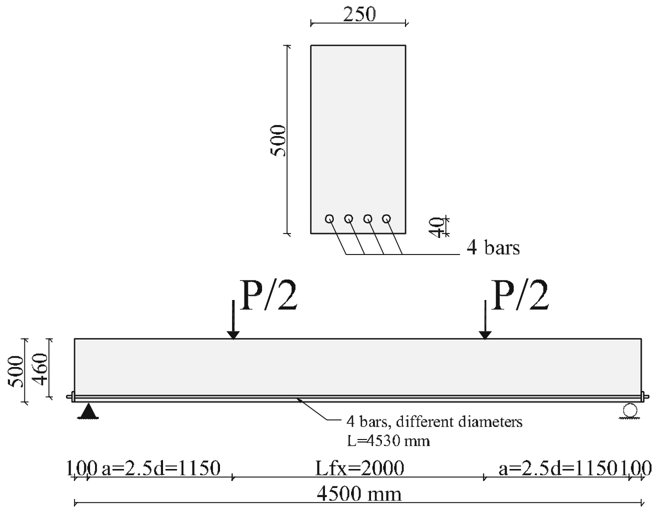

Fig. 2. Specimen geometry

2.2 Material Properties

All the beams were made with the same concrete mixture. The mix design consisted in 430 kg/m^3 of Cement Portland II/A-LL 42.5R and 168 kg/m^3 of water, resulting in a water/cement ratio (w/c) of 0.39. The maximum aggregate size was 14 mm. An amount of 3.85 kg/m^3 of superplasticizer was added to the concrete in order to enhance the workability. According to European Standard EN 12350-2, the concrete showed a S4 consistency class.

In accordance with EN 12390-13 and EN 12390-3 five cylinders 80(Ø) × 210 mm and ten 150 mm cubes were used for the determination of the Modulus of Elasticity and the cubic compressive Strength of concrete, respectively. After 28 days of curing, the secant Young's modulus, E_{cm}, was 29.7 GPa, while the cubic compressive strength, R_{cm}, was 54.04 MPa. The cylinder compressive strength of concrete was analytically derived as $f_{cm} = 0.83R_{cm}$ and resulted 44.85 MPa.

Rebars properties were evaluated according to EN 15630-1; the yielding and ultimate tensile strength resulted respectively 522.0 MPa and 639.1 MPa for deformed rebars and 388.1 MPa and 567.6 MPa for plain bars.

2.3 Test Set-up and Instrumentation

A displacement-controlled test was adopted to allow a suitable test control during critical steps, such as in the case of abrupt cracking phenomena or load drops. This was obtained by adopting a worm electro-mechanical screw-jack having a loading capacity of 1000 kN and a stroke of 350 mm.

In the test loading frame (Fig. 3), the actuator was hanged at the laboratory strong floor (with a thickness of 1050 mm) and the load transferred to the top through transverse steel beams (two 2-UPN400) and 32 mm dywidag rebars, passing through holes in the strong floor. The applied load was measured by two load cells placed between the bearing steel plates of the dywidag bars and the upper 2-UPN400 beam. The top cross beam loaded the longitudinal steel beam at its midspan. The longitudinal beam, in turn, loaded the specimen in the two desired points through steel cylinders. These cylinders, welded to a 300 × 100 × 5 mm steel plate, were placed above a layer of neoprene with a thickness of 25 mm to better transfer and distribute the load between the frame and the beam.

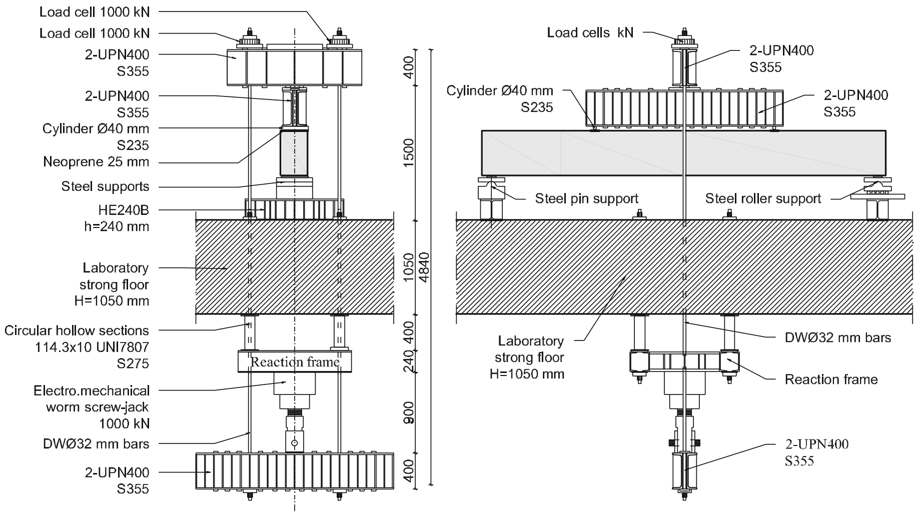

Fig. 3. Scheme of loading system adopted

Being the flexural span of specimens equal to 2000 mm, 2-UPN400 beam, identical to the lower transverse beam, were used as longitudinal beam.

A consistent number of instruments were used for monitoring the most important displacements and deformations of specimens, besides the applied loads. Linear Variable Differential Transformers (LVDTs) allowed the measurement of the vertical displacements of the beam and the supports. Potentiometric transducers were placed on both sides (back and front) of the specimens in the area of shear stresses (shear spans), to measure crack openings and strut deformations and, in the flexural span, to measure top compressive chord shortening and the elongation of the bottom chord at the bar level. The transducers for crack openings were placed with an inclination of 135° with respect to the horizontal.

The ones for the strut deformation were placed along the ideal connection line between the center of the support and the center of the loading point.

3 Experimental Results

3.1 Failure Mode

Figure 4 reports the comparison of the load-deflection (at midspan) curves for every specimen. Beam IDs report the number of bars with the related bond type, the shear slenderness and, in case of duplicate, an identification number. Firstly, it can be noticed that the influence of bond is very significant, determining a rather different structural behavior. For load levels significant for service conditions (around 100 kN), all specimens with bonded rebars exhibited a quite stable behavior with a crack onset and propagation highly controlled, and with overall deformations much smaller than those shown by the fully unbonded specimens.

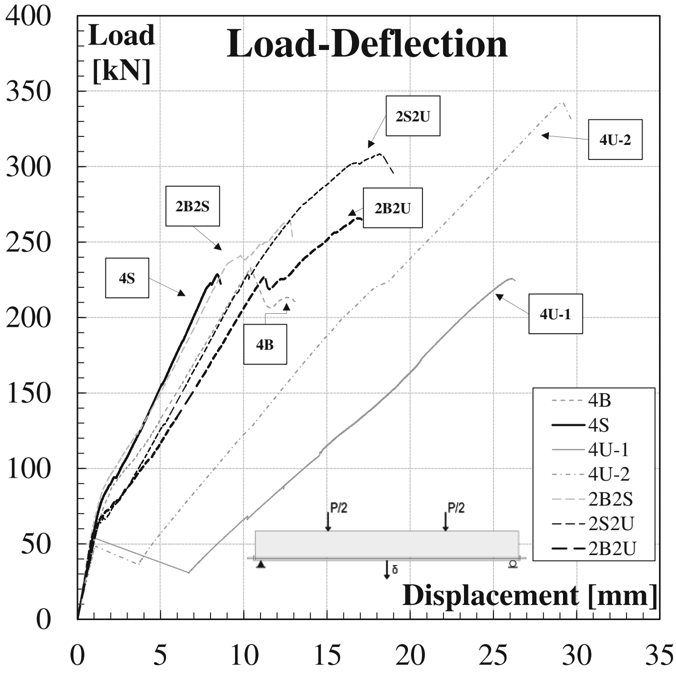

Fig. 4. Load-displacement chart

A noticeable tension stiffening effect was also reported. This behavior resulted more evident for the 4B specimen (4 deformed rebars), but became less effective with decreasing of bond level, resulting in a reduction of the specimen cracked stiffness.

In beams 4B and 4S, however, at a very low load level, a classical, sudden and brittle shear failure appeared through a wide crack running from the point load to the bottom reinforcement, toward the support, in the shear span. This brittle collapse occurred for crack widths, measured in the shear span, of about 0.3 mm, which represents a quite small value (i.e., one would think to be in a safe situation under such a structure), even though in accordance with classical literature on shear.

The failure load was almost the same, therefore no significant influence of the rebar type (deformed or plain) was shown for this specific cross-section. The reason is probably related to absence of debonding at the failure load level.

The beam with a mix of the two types of bars (2B2S) presented a higher bearing capacity than the other two members with only plain or only deformed rebars. Approaching the failure load of 4S and 4B specimens, a change in stiffness and a small load drop for the mixed beam can be observed, but no brittle failure occurred; indeed, the load further increased, reaching a value 16% higher than the one for bonded and plain specimens.

Also the other mixed specimen (2B2U and 2S2U) presented a greater bearing capacity, due to the presence of unbonded rebars and to the development, from the very beginning of the experimental test, of an arch action contribution.

The behavior exhibited by the two specimens with unbonded rebars was really different: after the first cracking load, a single, very wide flexure crack appeared in the flexural span between the loading points, in combination with considerable energy release and displacement. After that, only a few other flexure cracks, which were very wide, developed. For the specimens a flexure failure was attained through concrete crushing in the top chord, but well before the yielding of the reinforcing bars.

The need to perform an additional test on a copy of 4U-1 beam was related to a problem with the anchorage steel plates, due to concrete shrinkage, unrestrained by unbonded rebars. The shortening of the specimen resulted in a gap between the plate and the edge of the beam of about 1.5 mm, which did not allow the mutual interaction between concrete and longitudinal reinforcement (for unbonded elements, with no steel-to-concrete bond, the load can only be transferred by means of the anchorages). When the cracking load was reached for 4U-1 specimen, the gap was still open and the beam behaved like an unreinforced concrete member up to a deflection of 7 mm.

This problem was solved in 4U-AD2.5-2 specimen by inserting, a few days before the test, a layer of bedding mortar between the anchorage steel plates and the edges of the beam and welding the plates to the longitudinal bars. At the formation of the first flexural crack, the energy release and the displacement of the new member were significantly lower than in 4U-1 (Fig. 4). The vertical propagation of the crack was reduced and a top compressive strut thickness of 55 mm was measured. Because of that, the bearing capacity of the element was 50% greater but a ductile failure was not reached anyway.

The maximum deflection of unbonded members was 2–2.5 times greater than the one exhibited by the bonded specimens, while the bearing capacity of 4U-AD2.5-2 specimen turned out to be almost 1.5 times greater. No crack in the shear span, or inclined crack elsewhere, were reported.

The unbonded specimens, however, exhibited a cracking onset unacceptable under a design point of view. The first crack that arose in specimen 4U-1 was about 2.7 mm wide (1.5 mm for 4U-2 beam) and the midspan deflection suddenly dropped from 1.1 to 6.7 mm for 4U-1 (from 1.2 to 3.7 mm in the case of the 4U-2 beam). Moreover, the first vertical crack ran for almost 90% of the depth of the beam. Such a situation would have considered much more dangerous that the one mentioned for the bonded specimens. It is observed, on the contrary, that after this stage, flexural cracks develop in a fairly stable fashion.

Figure 5 illustrates the main shear crack width versus the load, for five of the specimens (the two unboned beam were obviously excluded, not presenting any crack in the shear span). The different bond condition influenced the cracking process; in fact, in bonded specimens (deformed and plain rebars), the collapse arose for very small crack width (~0.3 mm), as already explained. Decreasing the bond level, a more stable and wider crack propagation can be observed, reaching a crack width of 3 mm in 2S2U specimen. In specimen 2B2U, a sudden crack opening occurred for a width of 0.5 mm; afterwards, the beam was able to further increase the load and to reach a final crack width of 5.2 mm (with a more distributed crack pattern anyway). From that consideration, it can be concluded that the presence of both bonded and unbonded bars, introducing a mixed arch/beam action behavior, is able to increase the shear bearing capacity of reinforced concrete members and to allow stable shear cracks up to widths ten times greater than deformed rebars only.

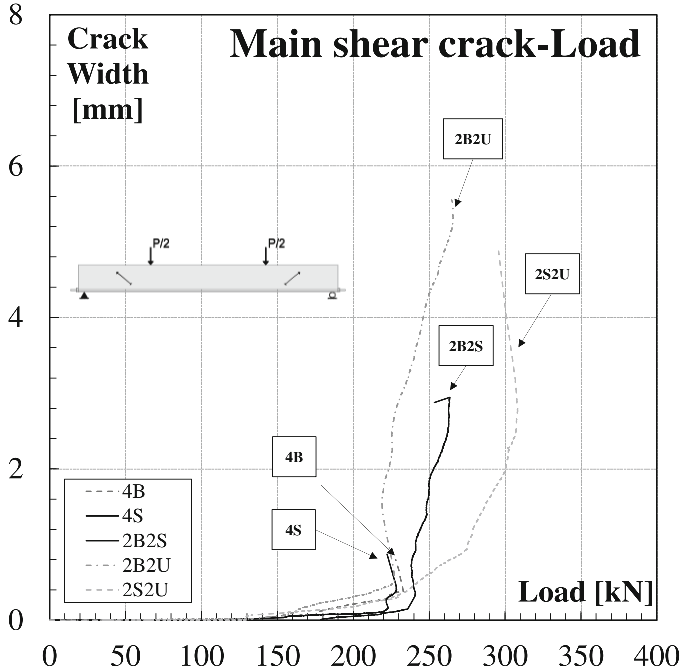

Fig. 5. Main shear crack-load chart

3.2 Crack Pattern

Figure 6 presents the crack patterns at failure for six of the specimen.

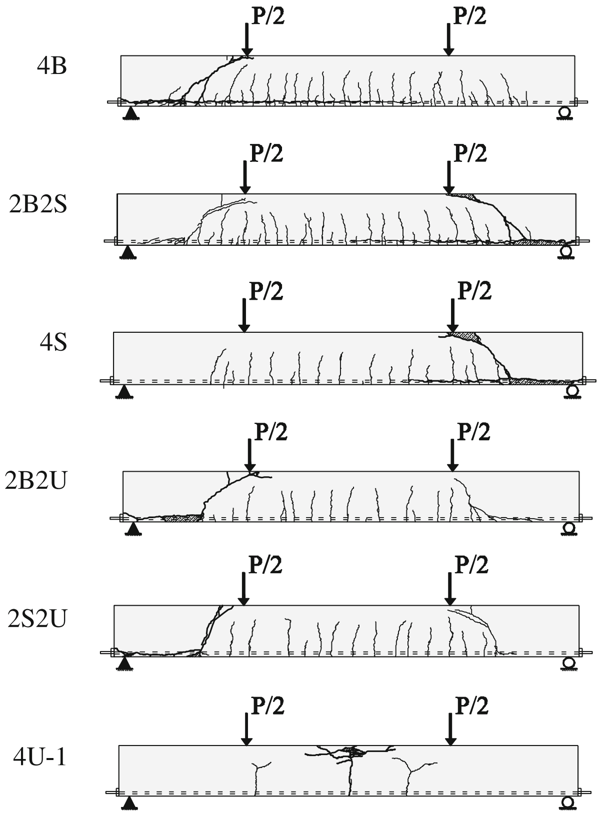

Fig. 6. Crack patterns at failure

Concerning 4B, flexure-shear crack appeared at a load intensity of 180 kN at a distance from the load point of almost 500 mm (similar to the beam depth), developing in an inclined crack towards the same loading point. This crack resulted the critical one along which the typical shear failure (block mechanism) took place. A considerable splitting along the longitudinal reinforcement could also be observed.

A similar behavior can be observed in most of the other members (2B2S, 4S, 2B2U and 2S2U), with the main difference that the critical shear crack resulted closer to the loading point in rebars having a lower bond resistance. Accordingly, the critical crack (failure crack) became steeper.

Looking at the flexural span it can be noted that, with rebars having a lower bond resistance, the flexural crack pattern resulted more spaced, with fewer and wider vertical cracks. In the evaluation process for assessment and rehabilitation of existing buildings, this could result in possible durability issues.

Concerning specimen 4U-1, a 2.7 mm wide crack (vertical) arose at a load level of 54 kN (first cracking point) with a propagation in 90% of the beam depth; a second shorter flexural crack developed for a load intensity of around 67 kN; another macro crack formed later on. All of the cracks tended to bifurcate and to become horizontal, clearly showing the arch shape. The final collapse occurred at the top chord of the member with concrete crushing and without cracks in the shear span.

4 Double Harping Point System

4.1 Introduction

This strengthening system presents many similarities with the unbonded tendons post-tensioning technique: however the use for retrofitting of RC elements is derived by a very similar technique quite common for the retrofitting of timber roof beam (Giuriani 2012).

The posts *m* (Fig. 7), due to the presence of the steel bar system, can be considered as two new vertical supports elastically unrestrained, with an equivalent stiffness k_V.

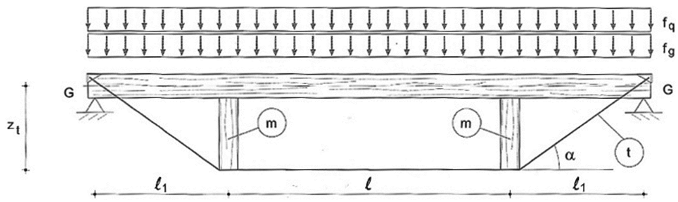

Fig. 7. Double harping point reinforcement for a timber roof beam (Giuriani 2011)

Giuriani suggested to consider these support as fixed ($k_V = \infty$) for normal dimension timber beams and for the area of reinforcement usually adopted ($A_s \geq 5$ cm^2). In this case, the ratio between the displacement of the reinforced member, w_r, and the displacement of the unreinforced, w, is less than 1/10.

These considerations are generally questionable in case of retrofitting of reinforced concrete elements. The material and geometrical properties in that case do not allow this simplification. Because of that, a proper evaluation of the equivalent stiffness offered by the reinforcement system is very important for considering the real redistribution of the load and avoiding underestimation of the internal forces on the member.

4.2 Evaluation of Actual Equivalent Stiffness

In a double harping point system for a reinforced concrete beam (Fig. 8), the distance between the anchorage point in the proximity of the supports and the centroid *G* of the transformed concrete section (e_1), the distance between the horizontal bars and the centroid *G* (e_2), the position of the posts (l_1) are all very important design aspects, influencing the efficiency of the reinforcement.

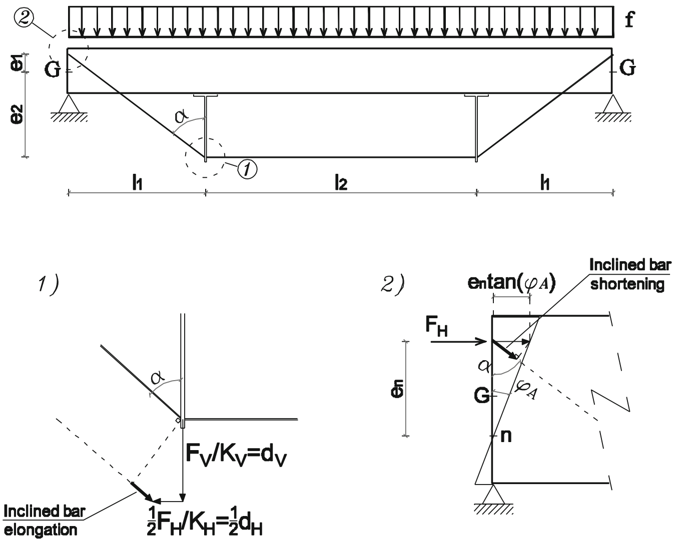

Fig. 8. Main geometric parameters and inclined bar compatibility (particulars 1 and 2)

The inclination of the bars in the external spans is a function of those parameters:

$$\alpha = \tan^{-1}\left(\frac{l_1}{e_1 + e_2}\right) \tag{1}$$

It is possible to observe that increasing the distance of the horizontal bars from the edge of the beam (increasing e_2), the value of α decreases, resulting in a higher vertical force in the posts. Assuming the effect of the post on the beam similar to that of a vertical axial spring of stiffness k_V, that results in an increase of equivalent stiffness.

However, due to maximum distance requirements for the reinforcement, the value e_2 cannot be increased freely.

If $e_1 \neq 0$, the anchorage point at the support level is not coincident with the centroid of the cross section of the beam. Because of that eccentricity, the element is not only subjected to a horizontal axial force but also to a concentrate bending moment at support.

The vertical equivalent stiffness of the reinforcement, k_V, does not depend only on the geometrical and material properties of the reinforcement, but also on the characteristic of the RC beam (flexural stiffness in the first time). The properties of the beam (neutral axis position, moment of inertia, moment of areas) for an element subjected to both bending moment and axial force are a function of the values of the internal forces. On their side, the internal force distribution depends on the equivalent stiffness of the support. That result in a circular reference, in which it is possible to consider three main parameters (Fig. 9):

- The equivalent vertical stiffness of the posts [k_V];
- The neutral axis position [c];
- The eccentricity of load to centroid [$e = u + y_{cen}$].

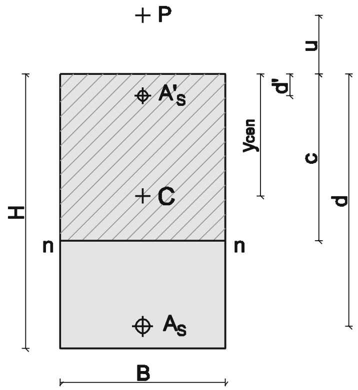

Fig. 9. Geometrical properties of the RC element (rectangular cross-section)

Only two of these unknowns are independent from the others and so, to solve the system, two equations are needed:

- Rotational equilibrium with respect to the point of eccentricity of load (P);
- A compatibility equation in term of elongation of the inclined reinforcement bars. The compatibility equation can be expressed as reported in Eq. 2:

$$\frac{F}{K} = \frac{F_V}{K_V} \cos\alpha - \frac{1}{2}\frac{F_H}{K_H} \sin\alpha - \left(e_n \tan\varphi_A\right) \sin\alpha \tag{2}$$

$$K = \frac{E_s A_{r,1}}{l_1} \sin\alpha \tag{3}$$

$$K_H = \frac{E_s A_{r,2}}{l_2} \tag{4}$$

Where K, K_H (Eqs. 3–4) are the axial stiffness of the inclined and horizontal bars and $A_{r,1}$, $A_{r,2}$ their cross sectional area respectively.

The first member of the equation represents the total elongation of the inclined bars (F/K). The second reports the elongation due to the vertical displacement of the system at the deviation point (F_V/K_V), the shortening due to the elongation of the horizontal rebars (F_H/K_H) and the shortening related to the rotation of the beam at the support (φ_A). The vertical displacement and the rotation can be evaluated considering the effect of three different load condition and using the Elastic Line Method for the concrete beam. The presence of the external reinforcement is taken into account including in the analysis a vertical axial spring with stiffness K_v, an Axial Force (N) and a Bending Moment (M_A) at the support location (Fig. 10a).

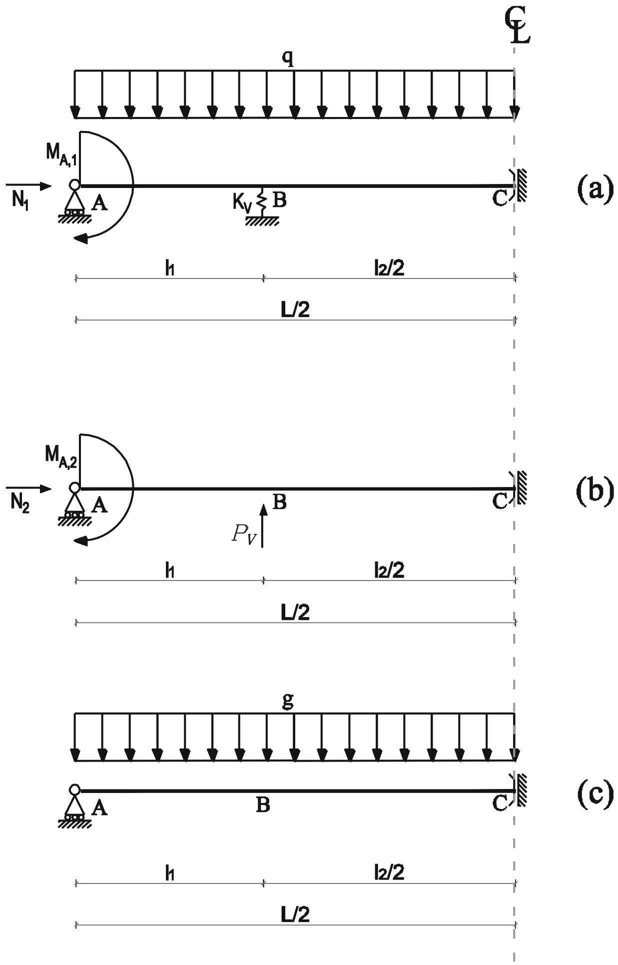

Fig. 10. Elastic line method: load cases

The first load (Fig. 10a) condition consider the effect of the load applied on the structure after the installation of the reinforcement system (q). The second condition (Fig. 10b) consider only the effect of the prestress action (P_V) that can be apply to the external reinforcement. Finally, the last load condition (Fig. 10c) considers the effect of the part of load already applied before the installation of the reinforcement (g).

The rotation in A and the internal force along the length of the member can be evaluated from the combination of the three load cases. It is worth noting that only rotation $\varphi_{A,1}$ (from load case in Fig. 10a) must be considered in Eq. 2, being the only one contributing to the deformation of the inclined bars after the application of external loads.

Figures 11 and 12 report the shear and bending moment diagram in case of a retrofitted one-way slab. The load applied comes from a typical SLS combination that can be considered in the rehabilitation of an existing building. A pretension of about 89 MPa is applied to the external bars.

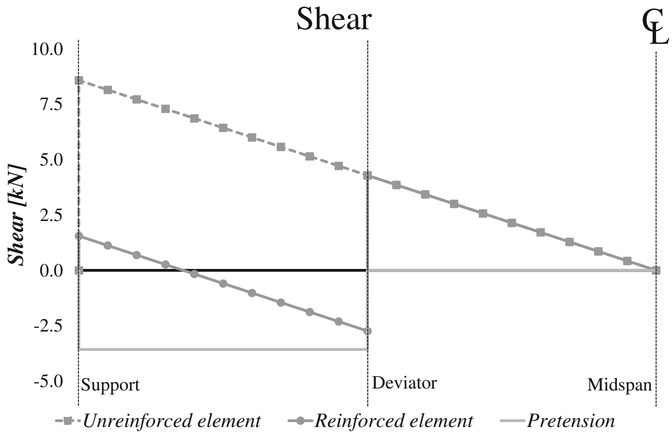

Fig. 11. Shear force over half of the span for reinforced and unreinforced one-way slab

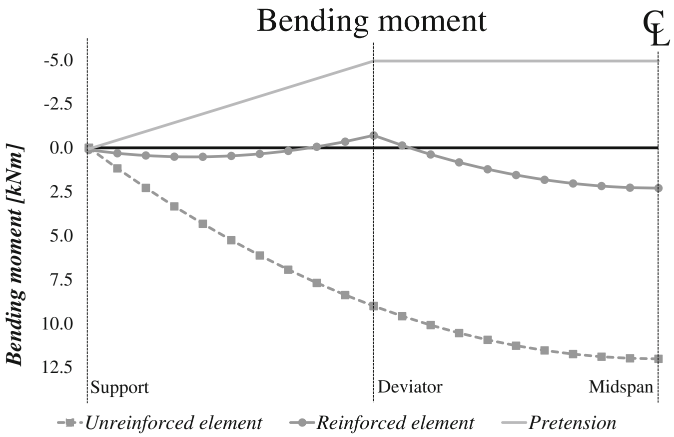

Fig. 12. Bending moment over half of the span for reinforced and unreinforced one-way slab

The external reinforcement system leads to a change of the static configuration of member. The effect is more evident in the external part of the span, between the support and the deviator; in the internal span, instead, the shape of the diagrams is similar to that for the unreinforced structure. Due to the presence of the reaction from the post, a discontinuity is present in the shear and a peak in the bending moment diagram.

The shear (Fig. 11) in the central span is exactly the same for the reinforced and the unreinforced element. The concentrated force at the deviation point leads to a drop in the diagram and to an important reduction in term of shear action in the external part of the smember. Therefore, the improvement in the behavior of the element is mainly related to a reduction of the shear demand. Looking at the bending moment diagram (Fig. 12), at the deviation point the maximum negative moment is reached; in the central span, the shape of the diagram is similar to the one for the unreinforced beam but the reaction coming from the post translates that portion of diagram, reducing the maximum positive moment at the midspan. Also in flexure, the reinforcement effect leads to a reduction in term of forces acting on the element

5 Concluding Remarks

In this paper, the results of seven tests on flexure-shear critical beams and a model for the evaluation of the equivalent stiffness of double harping point system have been presented. Based on the results, the following conclusions can be drawn:

1. Good steel-to-concrete bond (deformed rebars) tends to anticipate the collapse by determining a critical shear failure at very low load and deflection levels; this is mainly due to the cracking process governed by the compatibility between steel and concrete. The lack of bond, on the other hand, determines a flexural behavior, even though under considerable deflections and localizations of cracks.
2. Intermediate levels, in which both beams and arch action can develop together, demonstrate the possibility to increase the shear bearing capacity and to reach wider shear critical cracks.
3. The double harping point system results an interesting technique for the retrofitting of existing RC elements experiencing shear-flexure deficiency. The presence of the deviators as elastically restrained supports lead to a reduction of the shear and bending moment action. An improvement in term of stiffness, due to the presence of the external unbonded longitudinal reinforcement can also be achieved.

Acknowledgements. The Authors gratefully acknowledge Engineers Esti Federica, Caldera Silvia and Migliarini Mauro for assistance in carrying out laboratory testing and data processing.

The fundamental contribution of the rebar provider (Priuli s.a.s. and Alfa Acciai S.P.A.) and the concrete provider (Italcementi Group) is also gratefully acknowledged.

References

Abrams DA (1913) Test of Bond Between Concrete and Steel, University of Illinois Bulletin No. 71, University of Illinois, Urbana, IL, p 240

Cairns J (1995) Strength in shear of concrete beams with exposed reinforcement. In: Proceedings of ICE structures & buildings, vol 110, May 1995, pp 176–185

Cairns J, Minelli F, Plizzari GA (2005) Strengthening RC beams by external reinforcement. In: Proceedings of the international conference on concrete repair, rehabilitation and retrofitting, Cape Town, South Africa, 21–23 November 2005

Cairns J, Rafeeqi SFA (2003) Strengthening of reinforced concrete beams by external unbonded bars: experimental investigation: theoretical investigation. Proc Instit Civil Eng 156(1):27–48

Cairns J, Zhao Z (1993) Structural behaviour of concrete beams with reinforcement exposed. Proc Instit Civil Eng Struct Build 99:141–154

EN 12350-2 (2009) Testing fresh concrete – Part 2: Slump Test

EN 12390-13 (2013) Testing hardened concrete. Determination of secant modulus of elasticity in compression

EN 12390-3 (2009) Testing hardened concrete. Compressive strength of test specimens

EN15630-1 (2004) Steel for the reinforcement and prestressing concrete – Part 1: Test methods

Feldman LR, Bartlett FM (2008) Bond in flexural members with plain steel reinforcement. ACI Struct J 105(5):552–560

Giuriani E (2012) Consolidamento degli Edifici Storici. Utet Scienze Tecniche, Torino (in Italian)

Minelli F, Plizzari GA, Cairns J (2008) Flexure and Shear behavior of RC beams strengthened by external reinforcement. In: Proceedings of the international conference on concrete repair, rehabilitation and retrofitting II, Cape Town, South Africa, 24–26 November 2008, pp 377–378

Stoker MF, Sozen MA (1970) Investigation of Prestressed Reinforced Concrete for Highway Bridges. Part V: Bon Characteristic of Prestressing Strends, University of Illinois Bullettin No. 503, University of Illinois, Urbana, IL, pp 116–119

Tassios TP (1979) Properties of Bond between Concrete and Steel under Load Cycles Idealizing Seismic Actions, Comité Euro-International du Béton, Bulletin No. 131, Paris, France, pp 67–121

Tinini A (2016) Investigating Existing Beams and Slabs Experiencing Flexure and Shear deficiency, PhD thesis, University of Brescia, Brescia