NON-INVASIVE RETROFIT OF EXISTING RC FRAMES DESIGNED FOR GRAVITY LOADS ONLY

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1 INTRODUCTION

Recent experimental and analytical investigations on the seismic performance of existing reinforced concrete frame buildings, designed for gravity loads only, as typically found in Italy before the introduction of seismic-oriented design codes in the 1970s, confirmed the expected inherent weaknesses of these systems [1,2]. Because of the poor detailing of reinforcement, the absence of capacity design philosophy and the use of plain round reinforcing bars, peculiar brittle failure mechanisms were observed. At the local level, most of the damage occurred in the beam-column joint panel zones, through particularly brittle failure mechanisms. At the global level, the formation of undesirable soft-stories, resulting from a combination of column hinging and exterior joint damage, greatly impaired the overall structural performance of these RC frames.

The need for retrofit strategies providing adequate protection to the joint region while modifying the strength hierarchy between the different components of the beam-column connections, according to a capacity design philosophy, was recognized. Several retrofitting solutions have been studied in the past, and adopted in practical applications. An overview of seismic rehabilitation techniques was presented by Sugano [3]. Conventional techniques which utilize braces, jacketing or infills as well as more recent approaches including base isolation and supplemental damping devices have been considered. Another main thrust of these studies has considered retrofits involving structural strengthening with advanced materials (i.e. Shape Memory Alloys [4] or Fiber Reinforced Polymers [5]. Most of these retrofit techniques have evolved in viable upgrades to these structures. However, issues of cost, invasiveness, and practical implementation still remain the most challenging aspects of these solutions.

Following the Northridge earthquake, where weld fractures were observed at the beam-column connections of steel moment-resisting frames, a series of retrofit strategies was developed. One of these retrofit schemes, the welded haunch connection [6], protects the welded section by migrating the plastic hinge some distance away from the face of the column and by redirecting the beam shear forces to the column through axial straining of the haunch. This concept was further extended to an energy dissipating haunch which increases the performance level through supplemental damping [7].

In this paper, a similar retrofit strategy is proposed for reinforced concrete structures and is investigated numerically. Stiffening haunch type element are introduced locally near beam-column joints to significantly reduce the nominal shear stresses in the panel zone region. The geometry and stiffness are chosen to reverse the strength hierarchy with respect to capacity design principles, protecting the panel zone from high level of damage or brittle failure mechanisms and preventing the development of soft-story mechanisms. A discussion is also extended to cases where the stiffening elements are replaced by axial elastoplastic energy dissipating devices to provide supplemental damping.

After a brief overview of the main issues concerning the behaviour of frames designed for gravity loads only and of the modeling techniques to adequately capture the non-linear behaviour of the panel zone, a set of principles for implementing the haunch retrofit technique are proposed. The efficiency of the proposed solution is investigated through cyclic push-pull analyses on a beam-column exterior joint subassembly. A case study of a 6-story moment-resisting frame is also considered and the seismic performance of the as-built frame is compared to that of the retrofitted frame through non-linear time-history analyses.

2 SEISMIC RESPONSE OF RC FRAMES DESIGNED FOR GRAVITY LOADS ONLY

2.1 Typical structural deficiencies

As it has been widely reported in the literature [1,8-10], typical structural deficiencies of existing reinforced concrete frame systems can be related to:

a) inadequate confining effects in the potential plastic hinge regions;
b) insufficient amount, if any, of transverse reinforcement in the joint regions;
c) low amount (nominal) of longitudinal reinforcement;
d) inadequate anchorage detailing (including end-hook solutions), for both longitudinal and transverse reinforcement;
e) lapped splices of column reinforcement just above the floor level;
f) lower quality of materials (concrete and steel) when compared to current practice:
  - plain round (smooth) bars for both longitudinal and transverse reinforcement
  - low-strength concrete.

The main variations between construction practices in major seismic-prone countries are related to the percentage of column longitudinal reinforcement, which strongly affects the beam-to-column moment capacity ratio (increasing the tendency of developing soft-storey mechanisms) and the alternative solutions in anchorage details in lap splice regions or within joint regions.

2.2 Vulnerability of the panel zone region

In a recent experimental and analytical research program on the seismic vulnerability of existing reinforced concrete frame buildings designed for gravity loads only, as typical in Italy before the introduction of seismic-oriented codes in the mid-1970s [1,2], particular attention was given to the vulnerability of the panel zone region. Peculiar brittle damage mechanisms at both the beam-to-column subassembly level and at the global frame level were observed, due to the combination of plain round bars and hook end anchorages (see Fig. 1).

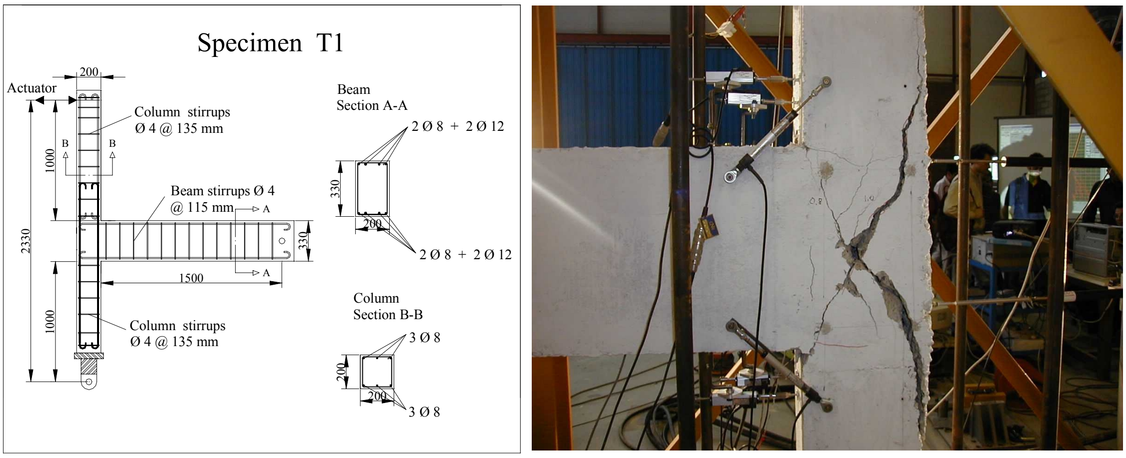

Fig. 1. Exterior tee-joint specimen T1 and joint damage ([1])

Different damage or failure modes are expected to occur in beam-column joints [11,12] depending on the typology (exterior or interior joint) and of the adopted structural details (i.e. presence of transverse reinforcement in the joint; use of plain round or deformed bars; alternative bar anchorage solutions). In absence of transverse reinforcement in the joint region, alternative post-cracking behaviour depends solely on the efficiency of the compression strut mechanism to transfer the shear within the joint. Thus, while rapid joint strength degradation after joint diagonal cracking is expected in exterior joints (Fig.2a), a hardening behaviour after first diagonal cracking can be provided by an interior joint. Furthermore, when hinging in the columns occurs, significant displacement ductility can be developed at a subassembly level. At the global level, however, the response of the system can be seriously impaired if a soft-storey mechanism is caused by the hinging in the columns.

This is illustrated in Fig. 2 where the experimental force-deflection response of an exterior joint (joint shear damage and beam hinging) and of an interior joint (column flexural damage) are shown.

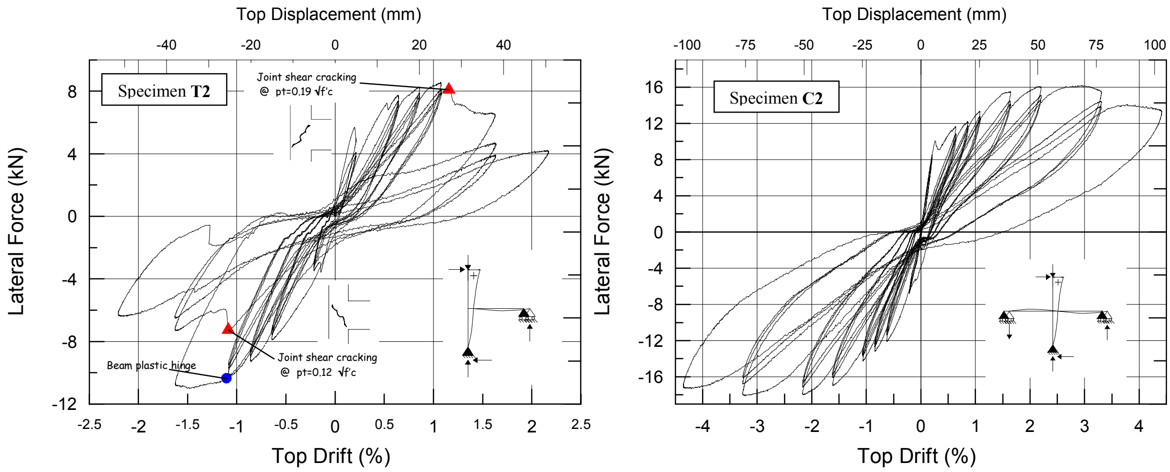

Fig. 2. Experimental hysteretic response of exterior tee-joint (T2) and interior joint (C2) ([1,12])

The joint shear stress is generally expressed in terms of either the nominal shear stress (v_{jn}) or the principal compression/tensile stresses (p_c , p_t). Although current codes tend to limit the nominal shear stress v_{jn} expressed as a function of the concrete tensile strength, $k_1\sqrt{f'_c}$, or the concrete compressive strength, $k_2 f'_c$, where k_1 and k_2 are empirical constants, it is commonly recognised that principal stresses, by taking into account the contribution of the actual axial compression stress (f_a) acting in the column, are better indicators of the stress state and consequently of the damage level in the joint region. Strength degradation curves for different joint typologies (exterior knee or interior tee-joint) and different structural detailing (i.e. plain round or deformed bars, anchorage solutions) based on principal tensile stresses-shear strain deformations have been suggested in the literature [11, 12].

2.3 Shear hinge mechanism and effect on global response

A critical discussion on the effects of damage and failure of beam-column joints in the seismic assessment of frame systems has been given in [13].

Based on experimental evidence and numerical investigations, the concept of a shear hinge mechanism has been proposed as an alternative to flexural plastic hinging in the beams. The concentration of shear deformation in the joint region, through the activation of a shear hinge, can reduce the deformation demand on adjacent structural members, postponing the occurrence of undesirable soft-storey mechanisms which can lead to a collapse of the whole structure.

The drawback of this apparent favourable effect on the global response is the increase in shear deformations in the joint region that can lead to possible strength degradation (depending on the detailing) and loss of vertical load-bearing capacity. Limit states based on joint shear deformations have recently been defined and are reported in [14]. Based on this detailed assessment of the local damage and corresponding global mechanisms, a more realistic seismic rehabilitation strategy can be defined.

2.4 Modeling issues

A simplified analytical model for the joint non-linear behaviour (shear hinge mechanism) has been proposed and described in [14]. According to a concentrated plasticity approach, the model consists of a rotational spring able to describe the variation of principle tensile stresses at mid-depth of the joint panel zone. The monotonic characteristics of the springs are derived from equilibrium considerations on the relative principal tensile stress-shear deformation curves. For any given level of principal tensile (or compression) stress in the joint (first cracking or higher damage level), the corresponding joint moment M_j which is either the sum of the beam moments or the sum of the column moments can be evaluated. It is therefore important to include the axial load variation (due to applied lateral forces) when evaluating the strength hierarchy in beam-column subassemblies for both columns and joint panel zones.

The cyclic behaviour of the joint is represented using hysteresis rules with pinching behaviour. Satisfactory analytical-experimental comparisons were obtained using the proposed model and adopted to define limit states based on joint shear deformation. More details on the modeling of shear critical beam-to-column panel zones can be found in a companion paper [14].

3 HAUNCH RETROFIT OF BEAM-TO-COLUMN RC JOINTS

The primary aim of the proposed retrofit strategy is to eliminate the damage in the beam-to-column panel zones while enhancing the global response of non-seismically designed RC frames. Since the panel-zone nominal shear or principal stresses (typically assumed as indicators of joint damage) are directly related to the maximum moment developed in the beam at the beam-to-column interface and the axial load in the columns, the solution aims to reduce the moment at the face of the columns by introducing local haunch type elements. Figure 3 shows a haunch upgrade of an existing RC building. The haunch type elements can be designed as stiffening elements with sufficient strength to remain elastic under the applied loads, or as passive elastoplastic devices which rely either on hysteretic yielding or on friction type elements to provide supplemental damping to the system.

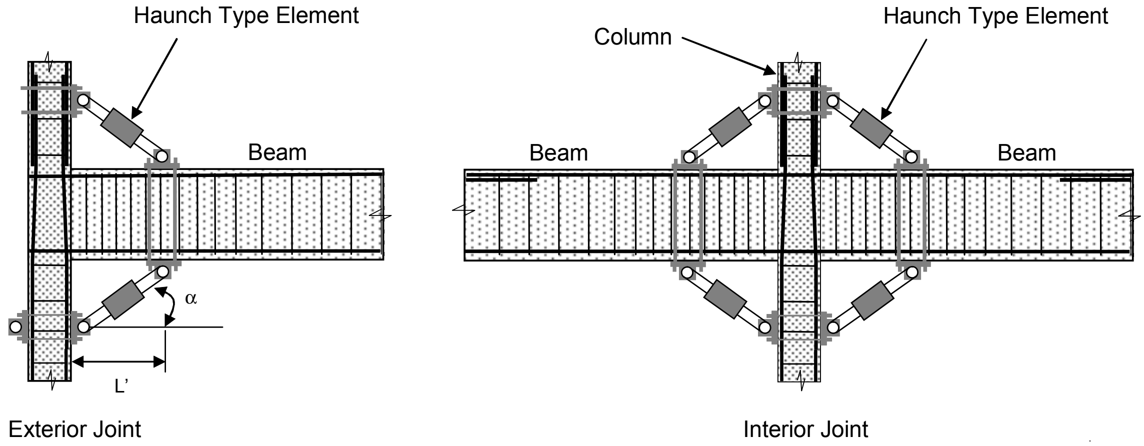

Fig. 3. Proposed haunch retrofit configuration for exterior and interior joints

3.1 Retrofit strategies

The ideal retrofit strategy would not only protect the beam-to-column joints that were identified as the major deficiency in these frames, but would further upgrade the structure to exhibit the desired weak-beam strong-column behaviour which is at the basis of the design of new seismic resistant RC frames. Considering that the beams in such structures are usually under-reinforced, with flexural reinforcement ratios as low as 1%, they inherently exhibit good curvature ductility capacity [1,8,15]. However, due to the disproportionate flexural capacity of the beams when compared to the columns this is difficult to achieve in all cases and for all beam-to-column connections without major interventions. This is especially true for interior beam-to-column connections where the moment imposed on interior columns from the two framing beams is significantly larger than for exterior columns. As indicated in the previous paragraph, interior joints are less vulnerable than exterior joints and exhibit a much more stable hysteretic behaviour with hardening after first cracking. It is thus conceivable, in a bid to protect the interior columns from hinging, to tolerate joint damage. Two levels of retrofits can therefore be considered, depending on whether or not interior joints can be fully upgraded.

A *complete retrofit* would consist of a full upgrade by protecting all joint panel zones and developing plastic hinges in beams while columns are protected according to capacity design principles. A *partial retrofit* would consist of protecting exterior joints, forming plastic hinges in beams framing into exterior columns, while permitting hinging in interior columns or limited damage to interior joints, where a full reversal of the strength hierarchy is not possible. The viability of the partial retrofit strategy must be investigated on a case by case basis to assure that the localized damage to interior joints does not severely degrade the overall response of the structure or jeopardize the ability of the interior columns to safely carry gravity loads.

3.2 Effect of haunch elements on beam, column and joint behaviour

When a haunch type element is introduced at the beam to column interface, the shear forces and moment diagrams of the beam-column assembly are significantly altered. Assuming inflexion points in the columns at mid-story height and at mid-beam length under applied lateral load, the free body diagram of an exterior joint is shown in Fig. 4 (as-built solution). In this figure, the maximum moment in the beam M_{bc} occurs at the face of the column, while moments M_c represent moments along the centerline of the columns located a distance $d_c/2$ from the face of the column, with d_c being the depth of the column. This offset is taken into account in the following equations but is not shown on the figure for simplicity reasons.

When the moment in the beam at the face of the column, M_{bc} reaches a critical value M_j^*, progressive cracking and failure under cyclic loading of the joint will take place if no other mechanism such as hinging of the beam occurs first. As discussed earlier the value of M_j^* depends on the principal stresses in the joint and is therefore dependent on the axial force and shear in the column. The value of interstorey shear causing the joint failure is given by:

$$V_c = M_j^* \frac{(1 + d_c / L_b)}{H_c} \tag{1}$$

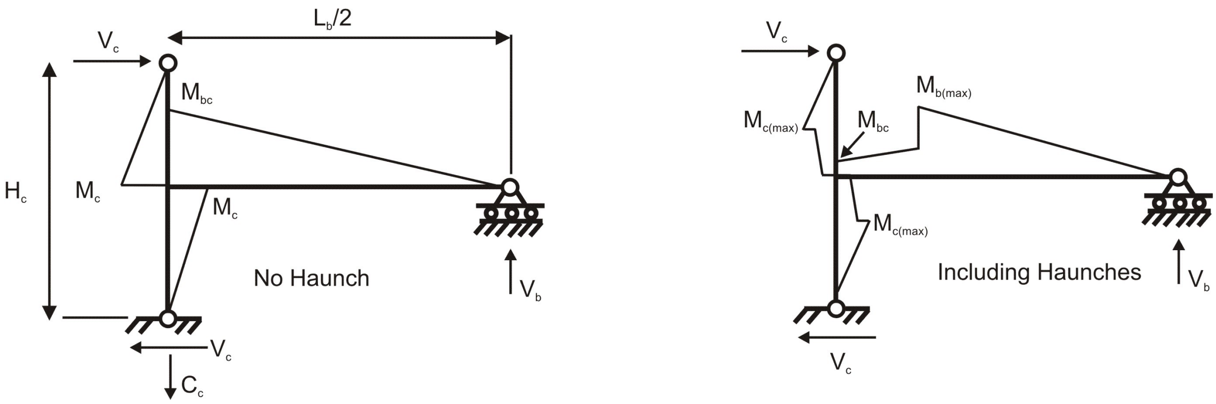

Fig. 4 Free-body and moment diagram of exterior joint

When haunch type elements are connected above and below the beam at a distance L' and at an angle α, (see Fig. 3.) the moment diagram in the exterior joint sub-assembly is modified and follows the diagram shown in Fig. 4 (including haunches case). It can be seen that the effect of the added haunches is:

- the migration of the maximum moment in the beam to a distance L' from the face of the column
- the reduction (relative to the maximum moment in the beam) of the moment at the face of the column
- the reduction of the column moment at the level of the beam-to-column connection
- the relocation of the maximum moment in the column to the point where the haunches are connected.

Note that because of the symmetry of the top and bottom haunches, no axial load is introduced in the beam and the shear force, applied to the beam at a distance L' from the face of the column, is contributed to, equally, by both haunches. It is of interest to also note that a similar behaviour can be achieved by using a single haunch element, introduced only below the beam. However, since in this case axial forces are induced in the beam and in the joint panel zone, it would be discouraged *a priori*.

If the total shear force introduced in the beam by the two haunches is expressed as a function of the beam shear as βV_b, the moment and shear diagrams in the beam are given in Fig. 5. The concentrated moment reduction at a distance L' from the face of the column, $\beta V_b(d/2)/\tan\alpha$, where d is the depth of the beam, is due to the offset of the beam centerline from the point where the haunches are connected to the beam.

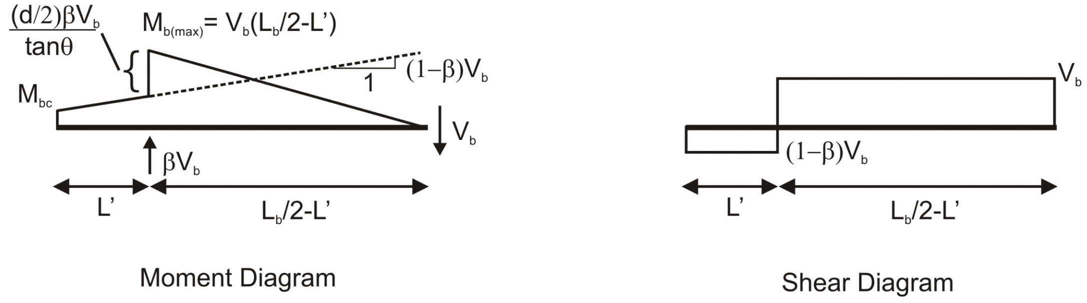

Fig. 5 Moment and shear diagrams in beam

The moment in the beam at the face of the column, is now given by:

$$M_{bc} = M_{b(\max)}\left[1 - \frac{\beta d}{2L\tan\alpha} - \frac{(1-\beta)L'}{L}\right] \tag{2}$$

where L = (L_b/2)-L'.

The shear in the column for a given value of $M_{b(max)}$ is given by:

$$V_c = M_{b(\max)}\frac{(1 + d_c / L_b)}{H_c} \tag{3}$$

and the corresponding maximum moment in the column is:

$$M_{c(\max)} = M_{b(\max)}\frac{(H_c/2 - d_b/2 - L'\tan\alpha)(1 + d_c/L_b)}{H_c} \tag{4}$$

The value of β is determined by writing deformation compatibility equations between the axial deformation of the haunch and the local deformations of beams and columns where the haunch is connected. The complete formulation of such an equation would involve axial, flexural and shear deformations in both beams and columns as well as panel zone elastic shear deformations. However, depending on the relative stiffness of elements and the relative contribution of these deformations to the total local deformation, simpler equations neglecting some of these contributions can be derived. For example, considering the beam flexural deformations only, the value of β can be shown to be:

$$\beta = L'\frac{-6Ld\sin\alpha\cos\alpha - 3L'd\sin\alpha\cos\alpha - 6L'L + 6L'L\cos^2\alpha - 4L'^2 + 4L'^2\cos^2\alpha}{-3\cos\alpha^2 d^2 L' - 6dL'^2\sin\alpha\cos\alpha - 4L'^3 + 4L'^3\cos^2\alpha - 12EI_b/(2K_d)} \tag{5}$$

where I_b is the moment of inertia of the beam and K_d is the axial stiffness of one haunch element.

Considering the moment diagram presented in Fig. 5, it can be seen that values of β bigger than 1 are desirable for a more efficient protection of the beam-to-column joint. For given properties of beam and column sections, a number of combinations of L', α and K_d are possible and must be chosen to limit the invasiveness of the added haunch elements and to provide the necessary upgrade to the system. As a general rule, larger values of α reduce the effect of the haunches on the maximum moments in the columns and are therefore preferred for cases where weak-column behaviour is expected.

3.3 Energy dissipating elastoplastic haunch element

When the haunch elements are designed as yielding or slipping devices exhibiting an elastoplastic hysteresis, the system behaves exactly as previously described until the forces in the passive haunches reach the strength of the devices F_s. Up to that point, the moment and shear distribution in the beam follows the distribution presented in Fig. 4 (including haunches case) and Fig. 5. When the haunches reach their maximum load, assuming they do not exhibit significant post-yielding stiffness, any additional lateral loads applied to the system will cause internal forces following the distribution presented in Fig. 4 (no haunch case). After the device reaches F_s, the moment in the beam, at the face of the column will increase at a much higher rate than it does before the haunch slips. If the beam does not form a plastic hinge at the location where the haunches are attached after the devices have slipped or yielded, the joint will suffer damage. Considering this, F_s must be chosen such that when the devices yield, the moment at the location where the haunches are attached to the beam is sufficiently close to the plastic moment of the beam, to assure that the damage to the joint does not occur before yielding of the beam.

3.4 Design scheme

The starting point of the design scheme is to first define the value of M_j^* from principal tensile stress considerations, including axial loads and shear forces in the columns. The beam plastic moment M_{pb} corresponding to the formation of a plastic hinge is also evaluated.

Once this is determined, the geometric characteristics and the stiffness of the haunch elements are determined by successive iteration. This can be implemented in a numerical procedure according to the following flow chart:

- Choose the properties of the haunches α, L' and K_d
- Using Equation (5) or any other applicable compatibility equation, compute β
- Using Equation (2) compute M_{bc} by setting $M_{b(max)}$ to M_{pb}
- If M_{bc} is lower than M_j^*, then compute the maximum moment $M_{c(max)}$ using Equation (4) with $M_{b(max)}$ equal to M_{pb}, otherwise return to the first step and change the properties of the haunches
- If $M_{c(max)}$ is lower than the maximum permissible moment in the column, then the shear in the beam and in the columns is checked, otherwise return to the first step and change the properties of the haunches

From a practical point of view, especially for frames with weak column problems, starting with higher values of α will lead to a more effective retrofit and a quicker convergence to feasible solutions. Furthermore, to reduce the invasiveness of the retrofit strategy, the lowest possible values of L' are preferred. The value of K_d is also limited by the choice of the haunch element sections and materials. More than one combination of the haunch properties may satisfy these requirements, and it may be useful to investigate a number of possible combinations

For the elastoplastic haunches, the design is first carried out assuming elastic elements following the procedure described above, and then the lowest value of F_s, to assure yielding of the beam before the joint is damaged can be determined.

4 APPLICATION OF PROPOSED RETROFIT STRATEGY

4.1 Validation of the retrofit strategy on a beam-column subassembly

The efficiency of the proposed retrofit solution and of the design approach is numerically investigated with an exterior beam-column subassembly (tee-joint specimen T1), that was experimentally tested as part of the aforementioned research program on seismic vulnerability on existing buildings typical of the Italian construction practice before the 1970s [1,2]. The as-built specimen suffered severe joint shear damage mechanism and a marked pinching effect on the hysteresis loop due to opening and closing of shear cracking (see Fig. 2).

A complete retrofit strategy following the procedure described above was adopted, with the intention of protecting the panel zone region and forcing the development of a flexural hinge in the beam at the level of the beam-haunch connection. For simplicity, the haunch connections were initially assumed connected to the beam and column centrelines. Within a conservative design, the evaluation of the haunch stiffness K_d sufficient to maintain the beam moment at the interface M_{bc} under the critical value M^*j that corresponds to the first cracking in the joint, was carried out neglecting the shear deformation of the panel zone as well as considering gross section properties for both beam and column elements.

The analytical model, presented in a companion paper [14] and briefly described above, was adopted within the finite element code Ruaumoko [16] to predict the response of the retrofitted solution and compare it to the as-built tested specimen. According to a concentrated plasticity approach, a moment-rotational spring with pinching hysteresis loops was used to capture the non linear joint panel zone response (shear hinge mechanism [14]). The plastic hinge in the beams relocated at the level of the haunch connection was modelled with a Takeda hysteresis loop [17].

Push-pull analyses consisting of two cycles at 1% and 2% of column drift were carried out. The selected haunch solution (α=60°, L'=0.4 m, K_d= 150000 kN/m) resulted in a significant improvement of the assembly response, confirming the efficiency of the proposed solution and retrofit design strategy. A marked increase in the lateral force capacity of the retrofitted system when compared to the initial one can be seen in Fig. 6. This increase is not due to the increase of the individual strength of any of the components, but rather to a modification of the internal forces and the hierarchy of strength. Given the geometric properties of the adopted haunch solution, simple equilibrium considerations (from internal moment distributions such as in Fig. 4) can be used to predict the value of increased lateral force at equivalent yielding of the subassembly.

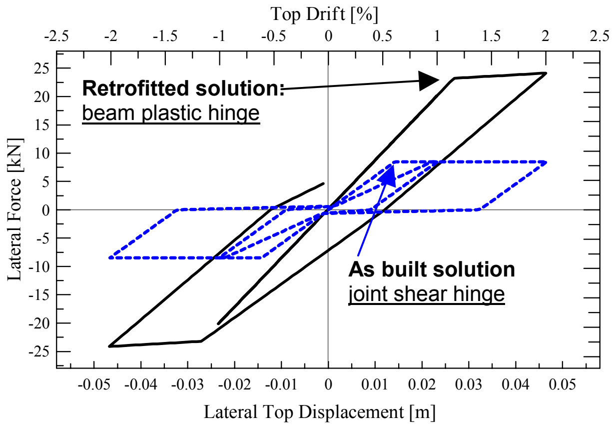

Fig. 6 Comparison of hysteresis response of as-built and retrofit solutions

More importantly, in terms of actual performance of the subassembly, no damage occurs in the panel zone region (which remain elastic Fig. 7a) while a plastic hinge develops in the beam (Fig. 7b) at the level of the haunch connection, exhibiting good energy dissipation and a controlled curvature demand. Furthermore, the relocation of the plastic hinge reduces the global pinching behaviour due to joint shear cracking as well as bond deterioration and slipping of beam bars within the joint.

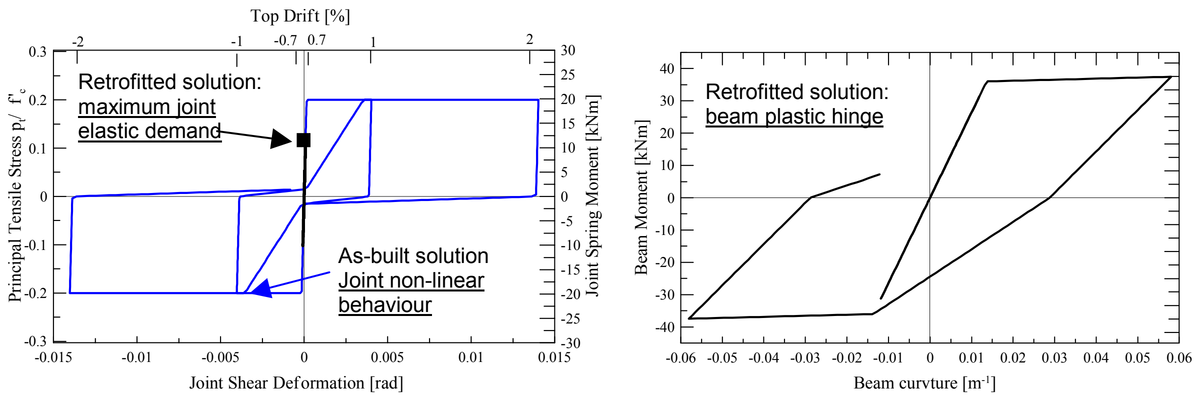

Fig. 7 Comparison of sources of damage and non-linear behaviour: a) shear hinge in the as-built solution; b) beam hinge relocated at the haunch connection level in the retrofitted solution

5 CASE STUDY: 6-STORY FRAME

5.1 Description of analyzed structure

The effectiveness of the above described retrofit technique is investigated numerically through time-history analyses of a 6-story frame building designed for gravity-loads-only according to the Italian code provisions available in the 1950s and 1960s. The prototype system is comprised of 3-bays, with a shorter central bay used as a corridor. The storey height is assumed to be 3 m (see Fig. 8). This 6-storey frame has been extensively studied through analytical investigations to assess the behaviour of the as-built structure [13,18].

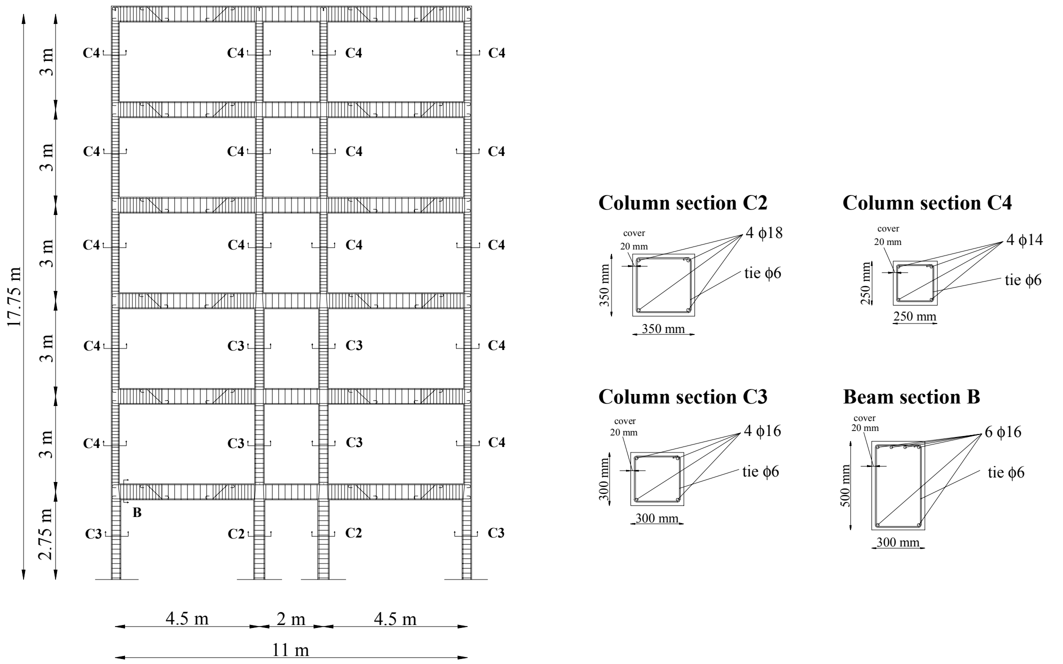

Fig.8 Six storey frame system designed for gravity loads only [13]

5.2 Retrofit Strategy

Considering the geometry of the frame, only a partial retrofit could be achieved because of the short middle spans. In fact, forming plastic hinges in the middle span beams is very difficult, especially considering that with the inclusion of haunches, the plastic hinges forming in each side of the beam would be too close, thus greatly increasing the curvature ductility and shear demands on the beam. Therefore, the partial upgrade consisted of retrofitting all joints, but interior joints were equipped with haunch elements only on the beams in the exterior bays (see Fig.10). In this retrofit, the goal is to protect all exterior joints, introduce hinges in beams framing into exterior columns to provide a source of stable energy dissipation to the system and avoid hinging in all exterior columns. In interior joints and columns, controlled damage is tolerated as long as the global behaviour of the frame is not significantly affected (soft-story mechanism).

A prototype subassembly consisting of column section type C2 and beam section type B, (see Fig. 8) was first designed. It was found that other than for the first floor, the haunch design which satisfied this configuration could be used for all joints. Further optimization of other connection retrofits is therefore possible. It is worth underlining that inflexion points in the columns are not always located at mid column height since they are affected by gravity loads and, more significantly by, higher modes under dynamic loading [19]. As a consequence of this, while the protection of the joint panel zone is guaranteed if Eq. (2) is respected, further attention should be paid to avoid column hinging (particularly undesired in exterior joints), by taking into account the variability related to the column inflexion points when defining the geometric characteristics of the haunch retrofit.

The adopted haunch solutions for both exterior and interior joints (only on the side of the exterior bay), for all floors other than the first one were defined for α=60°, L'=0.8 m and K_d= 150000 kN/m.

For the first floor, in an attempt to reduce the stiffness irregularity between the ground floor and the floors above, while still maintaining a relatively non-invasive solution, the first storey beams were connected to the base of the columns (see Fig. 10). The characteristics of these haunches were also chosen to meet the partial retrofit strategy with L' = 0.8 m and K_d =150000 kN/m.

6 TIME-HISTORY ANALYSES

6.1 Choice of accelerograms

The seismic response of the 6-storey building with and without the retrofit were compared under three different ground motions, chosen within a set of records whose average spectrum is compatible with the design spectrum defined by the International Building Code (ICC 2000, [20]) for a soil class C. Details on these records can be found in [21].

Table 1: Earthquake Records Used in Analyses

Name	Earthquake	Year	M_w	Station	Scaling Factor	Scaled PGA (g)
EQ1	Superstition Hills	1987	6.7	Plaster City	2.2	0.409
EQ2	Northridge	1994	6.7	Canoga Park - Topanga Can	1.2	0.427
EQ3	Loma Prieta	1989	6.9	Gilroy Array # 4	1.3	0.542

6.2 Results of analyses

Results in terms of maximum interstorey drifts, envelopes of floor displacements and maximum floor accelerations under the EQ2 record are shown in Fig. 9, while maximum values for the three records are given in Table 2. It can be noted that the selected haunch retrofit solution, although not optimised, provides a considerable improvement in the global system response. A significant reduction in maximum interstorey drifts (as well as maximum floor displacements) can be observed and the predicted collapse due to soft-storey mechanism at the fourth floor is avoided. Slight increases in floor accelerations especially at the top floors are also observed. This is a direct consequence of the increased overall elastic stiffness and strength of the system. However, it is reasonable to expect that an optimised solution with different properties of the haunch connection along the elevation (i.e. reduced stiffness) as well as the introduction of an energy dissipating haunch element would reduce these effects.

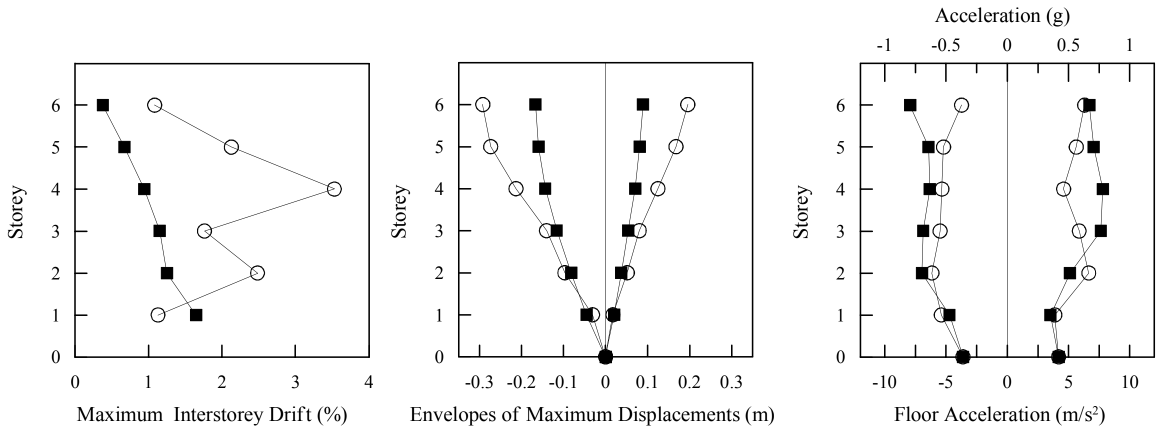

Fig.9 Comparison seismic response of as-built and retrofitted 6-storey frame under the EQ2 record

Table 2: Earthquake Records Used in Analyses

		EQ1	EQ2	EQ3
Maximum Drift (%)	As-Built	2.45	3.53	1.97
	Retrofitted	0.947	1.64	1.47
Residual Drift (%)	As-Built	0.4	0.61	0.33
	Retrofitted	0.17	0.47	0.64
Maximum Acceleration (g)	As-Built	0.63	0.67	0.53
	Retrofitted	0.85	0.81	0.90

The distributions of damage in the as-built and retrofitted frames are shown in Fig. 10. It can be noted that unlike the as-built frame, the retrofitted frame sustains no joint damage in exterior joints, while almost all the beams framing into the exterior joints develop flexural hinges. The hinging in a few interior columns and the damage to two interior joints can also be observed. This was expected, since only a partial retrofit strategy was applied to the frame. However, the overall response of the retrofitted frame confirmed that even with these interior columns hinging the system behavior is still very satisfactory since no soft stories are formed and floor displacements are reduced to acceptable values. The shear forces in columns and beams were also increased as a result of the retrofit strategy and since shear failure is not modeled in the beam elements, this was checked subsequently to verify that the estimated shear capacity was sufficient to resist these loads.

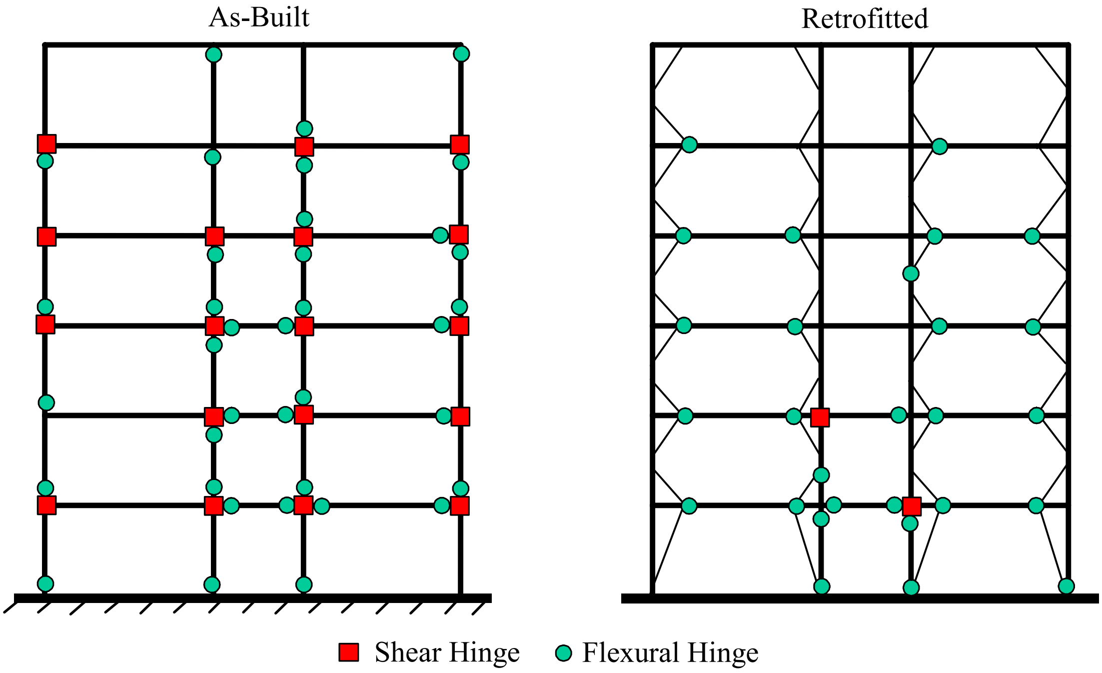

Fig.10 Comparison seismic response of as-built and retrofitted 6-storey frame under the EQ2 record

7 CONCLUSIONS

An overview of the major deficiencies of RC frames designed in the 1950s and 1960s without any seismic provisions was first presented. The concept of shear hinge mechanisms in beam-column joints, its critical effects on the global response of a frame system, as well as appropriate models to capture the non-linear behaviour of the panel zone regions, were also briefly reviewed.

A retrofit strategy consisting of introducing haunch type elements locally, close to the beam-to-column connections, has been investigated numerically as a means to significantly enhance the seismic performance of these buildings. A simplified design approach, to control the hierarchy of strength within beam-column subassemblies, reducing the damage in exterior joints as well as avoiding soft storey mechanisms, was presented.

The efficiency of the proposed retrofit strategy was first numerically confirmed by the enhanced cyclic behaviour of an as-built beam-column exterior subassembly.

The seismic response of a 6-storey frame building, designed for gravity only, was then compared through time-history analyses to that of a retrofitted frame. The retrofit strategy for this specific building consisted of fully protecting exterior beam-to-column joints while allowing limited damage to interior columns and joints, but eliminating global soft-storey mechanisms. Results indicate that the proposed retrofit strategy would lead to a significant enhancement of seismic performance, by i) protecting the all exterior joint regions ii) changing the hierarchy of strength in exterior joints to induce hinging of beams which provide a good energy dissipation to the system iii) reducing the maximum interstorey drifts as well as maximum floor displacements iv) controlling the overall damage level.

Further studies on the application of this retrofit technique to other types of non-seismically designed RC frames and on the practical definition of the elastic and elastoplastic haunch elements are needed. Experimental validations of the proposed retrofit technique both at the local sub-assembly level and at the global frame level are required to confirm results from this numerical study.

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