DESIGN CONCEPT OF RETROFITTED RC BEAM-COLUMN JOINTS BY PRESTRESSED JOINT ENLARGEMENT

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ABSTRACT

Post-earthquake inspections of damaged RC buildings have demonstrated that poorly detailed beam-column joints can suffer serious damage. A retrofit technique called “joint enlargement using prestressed steel angles” was experimentally investigated by authors and found to be an effective and practical technique for the seismic retrofit of non-seismically detailed reinforced concrete beam-column joints. In this method, the beam-column joint is enlarged by locatingstiffened steel angles at the re-entrants corners of the beam-columnjoint, both above and below the beam, with the steel angles mountedand held in place using high tensile strength bars.The main objective of designing the proposed retrofit method isto avoid joint shear failure and to encourage beam flexural hinging.The size of the joint enlargement should be designed such that toincrease the joint shear capacity through increasing effective jointarea and to improve the anchorage bond of the beam longitudinalbar within the joint panel zone through increasing apparent columndepth. In this paper the design concept of retrofitted RC beam-column jointsby joint enlargement using prestressed steel angles is discussed with considering the load transfer mechanism in the beam-column joint sub-assemblages.

INTRODUCTION

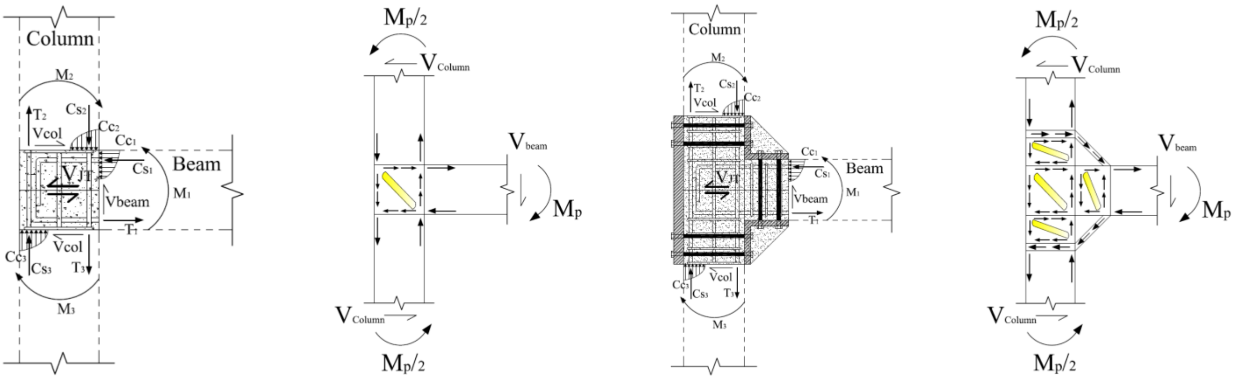
Recent earthquakes, particularly the 22 February 2011 Christchurch, NZ earthquake, caused numerous fatality and financial losses due to the failure of reinforced concrete (RC) structures (Dizhur *et al.* 2011, Elwood *et al.* 2012, Kam *et al.* 2011). Post-earthquake inspections of damaged RC Buildings (Elwood *et al.* 2012, Kam *et al.* 2011, Norton *et al.* 1994) as well as Laboratory tests of non-seismic RC beam-column joints (Park 2002) were demonstrated that beam-column joints suffered serious damage because of deficient reinforcement details in the joint region. Because of the poor detailing of the reinforcement and the absence of capacity design philosophy, undesirable brittle failure mechanisms are observed at either the local level (i.e. shear failures in joints, beam or column members) or globally in the structure (i.e. soft-story mechanisms). The joint region is of particular interest in such systems, as it is likely to be the critical and possibly the weakest link according to capacity design or hierarchy of strength considerations (Paulay 1998).

Several retrofit techniques have been studied in the past, Sugano (1996) presented an overview of seismic retrofitting techniques for existing pre-1970s buildings and also Engindeniz *et al.* (2005) studied efficiency of various rehabilitation techniques for non-seismically detailed beam-column joints. From this review it can be concluded that issues of cost, invasiveness, and practical implementation still remain the most challenging aspects of these solutions. A retrofit technique called “joint enlargement using prestressed steel angles” were experimentally and analytically investigated and found by (Shafaei *et al.* 2014a, Shafaei *et al.* 2014b, Shafaei *et al.* 2014c)that it is a simple, low-invasive, practical and cost-effective retrofit technique for seismic retrofit of non-seismically detailed reinforced concrete beam-column joints(Hosseini *et al.* 2012, Shafaei *et al.* 2013a, b, Shafaei *et al.* 2012). In the current study, design recommendations for the proposed technique called “joint enlargement using prestressed steel angles” which relies on two dimensional joint enlargement using prestressed steel angles installed at the re-entrants corners of the beam-column joint, both above and below the beam to protect the panel zone and to force a more desirable hierarchy of strength, is conceptually presented.

PRESTRESSEDJOINT ENLARGEMENT CONCEPT

Following the significant number of weld fractures at the beam-column connections of steel moment-resisting frames observed after the Northridge earthquake, a series of retrofit strategies was developed by FEMA-267 (1995), including haunch at bottom flange, top and bottom haunch, cover plate sections, upstanding ribs, top and bottom bolted bracket and side-plate connections. The welded haunch and bolted bracket connection and upstanding ribs at the top and bottom flange of beam section, protect the welded section by migrating the plastic hinge some distance away from the column face 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 (Martinez-Rueda 2000). The full design procedure of these retrofit strategies including selecting of geometry and stiffness of haunch for steel connections was proposed by Yu et al. (2000).

In the current study, a similar concept with some changes was adopted and applied for retrofitting of non-seismically detailed beam-column joints in reinforces concrete structures. In the proposed method called “joint enlargement using prestressed steel angles” the role of the prestressed steel angles are as same as haunch or bolted bracket or ribs in the steel structures. The prestressing force on the steel angles that applied by high tensile strength bars confined the joint region and provide the compression stress in the joint and delay tensile cracking of the joint. The proposed method as shown in Figure 1 mobilized part of the beam and column member directly adjacent to the joint panel zone to assist in resisting the shear input to the joint and thus, shear failure of the joint was delayed. In fact, the demand placed on the original as-built joint to transmit the force acting at the member-joint interfaces for the elements framing into the joint was diminished by re-directing the force-flow around the original joint region and by relocating beam plastic hinges to instead develop at the edge of the joint enlargement as shown in Figure 1. This figure indicates that when using a joint enlargement and post-tensioning force the depth of the joint diagonal strut can be expected to increase (Ingham et al. 1998) which will generally improve reinforcement embedment conditions and joint mechanism responsible for force transfer in the expanded joint can mobilize relatively more reinforcement. A special feature of the joint enlargement mechanism is that it benefits from broadening of joint main strut. This reduces the demand in the strut, increases the strut capacity, and alleviates possible compression failure.

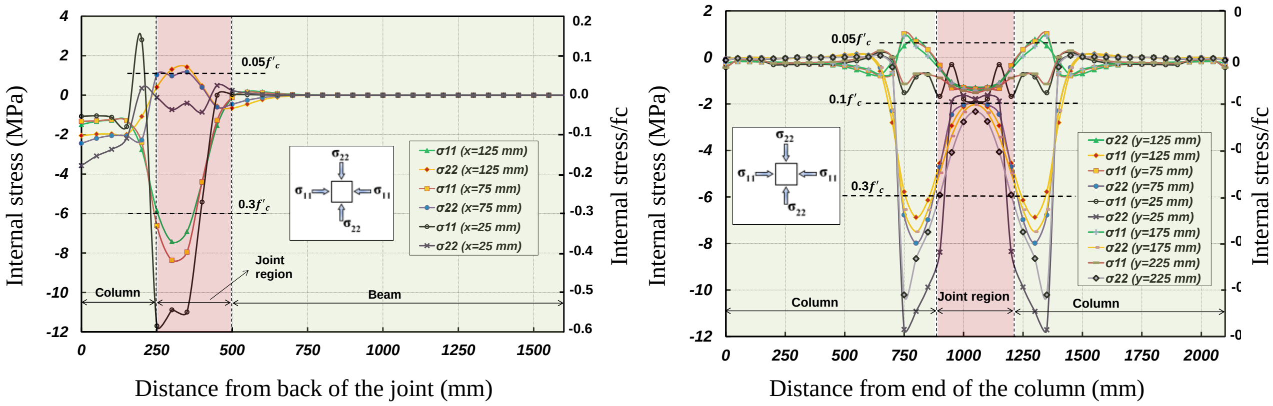


(a) Before retrofitting (b) After retrofitting

Figure 1. Load transfer in the joint before and after retrofitting.

DISTRIBUTION OF PRESTRESSING FORCE IN THE JOINT REGION

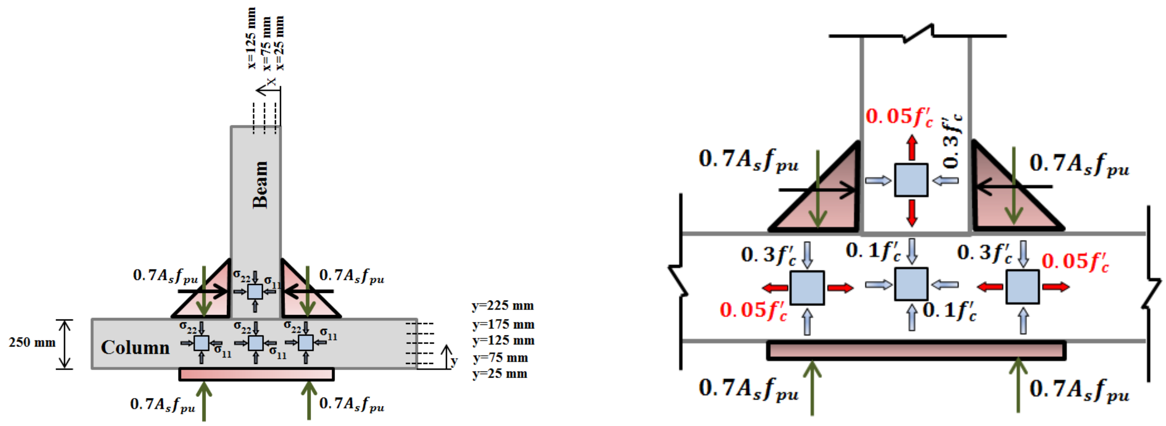
To study stress distribution due to post-tensioning force on the steel angles at the re-entrant corner of the beam-column joint in the joint region and part of the beam and column in the joint enlargement zones, the finite element model of beam-column joint was constructed in the ABAQUS software. Figure2illustrates horizontal (σ_{11}) and vertical (σ_{22}) stress distribution along and in various depth of beam and column. As shown in Figure2and Figure 3the effective horizontal (σ_{11}) and vertical (σ_{22}) compression stress in the joint is 10% percent of the concrete compressive strength. This compressive stress is very effective to improve joint confinement in the non-seismically detailed beam-column joints.



(a) $\sigma_{11}$and σ_{22} along beam (a) $\sigma_{11}$and σ_{22} along column

Figure2. Stress distribution along beam and column due to post-tensioning force.

The final average stress condition in the joint region and part of the beam and column in the joint enlargement zones are illustrated inFigure 3.



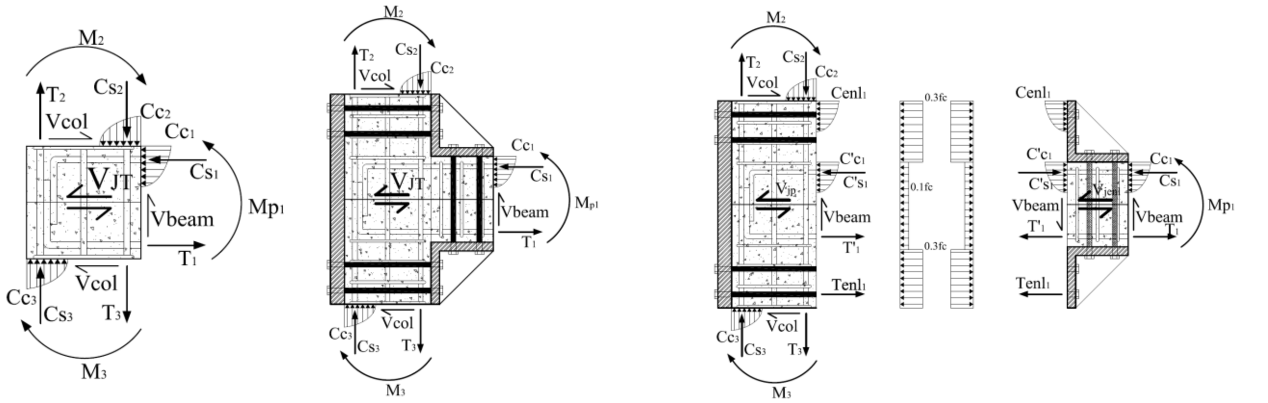
(a) definition of depth in the beam and column

(b) final stress condition

Figure 3. final stress condition in the beam-column joint after retrofitting.

DISTRIBUTION OF SHEAR STRESS BETWEEN JOINT PANEL AND JOINT ENLARGEMENT

The model of force equilibrium for the enlarged joint is displayed inFigure 4. This model is constructed to analyze the horizontal shear force carried by the beam sections within the enlarged joint and by the joint panel. As shown in Figure 4b, based on the equilibrium of the entire model the total horizontal shear force (V_{jT}) at the mid-plane can be calculated in the same manner as the horizontal shear force in the joint without enlargement (see Figure 4a).



$v_{JT}=T_1-V_{col}$

(a) Before retrofitting

$v_{JT}=T_1-V_{col}$

(b) After retrofitting (entire model)

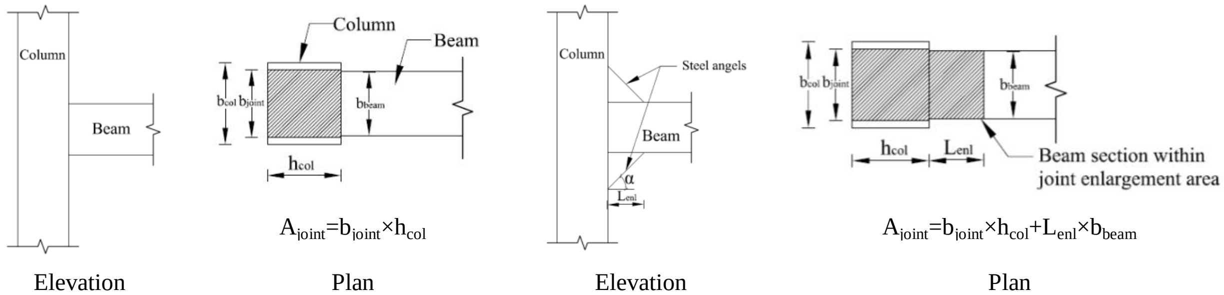
$V_{jT}=V_{jp}+V_{jenl1}, V_{jp}=T_1^{'}+T_{enl1}-V_c, V_{jenl}=T_1-T_1^{'}-T_{enl1}$

(c) After retrofitting (sub-models)

Figure 4. Force equilibrium model for retrofitted joint.

It should be noted that the total shear force is the sum of shear force in the joint panel (V_{jp}) and shear force carried by the beam sections within the enlargements (V_{jenl}). To understand how the total horizontal shear force is shared between the joint panel and beam sections within the joint enlargement, the entire equilibrium model is dissembled into two sub-models that isolate the enlargements and the joint panel (see Figure 4c).

The equilibrium equations shown in Figure 4c can be written to derive the horizontal shear force in each part of the retrofitted joint, where T_1 is force in tensile reinforcements at the edge of joint enlargement; T'_1 is force in tensile reinforcements at column faces; T_{enl} is tension forces in post-tensioning bars. All tension forces in these equilibrium equations can be experimentally obtained from measured strains through Ramberg-Osgood 's cyclic stress-strain law (Kent and Park 1971). As it is obvious and shown in Figure 6 the joint enlargement can increase the effective joint area, but based on the equilibrium of the upper half of the joint (Figure 4c) and the results presented byPimanmas and Chaimahawan (2010) the horizontal shear stress may not be distributed uniformly along the horizontal plane passing the beam sections and joint panel. Thus, the conventional ACI Committee 318M-11 (2011) design concept using effective joint area cannot be used for designing the size of joint enlargement because the joint shear strength does not proportionally increase with the joint area. The distribution of shear stresses depends on the mechanisms by which the joint enlargement and the joint panel resist the force. In this paper, horizontal shear stress is analyzed based on the equilibrium of force transfer in the joint. Finite element analysis is conducted to identify the flow of forces in the joint panel and enlarged areas and the load resistant mechanism of retrofitted joints. Strut and tie model is also developed to serve as a mechanical model to simulate the shear resistant behavior of retrofitted joints.



(a) Before retrofitting (a) After retrofitting

Figure 5. Effective joint area before and after retrofitting

EFFECT OF JOINT ENLARGEMENT ON THE BEAM, COLUMN AND JOINT INTERNAL FORCES

When stiffened steel angles are installed at the re-entrants corners of the beam-column joint, both above and below the beam (see Figure 2), the internal forces of the beam-column sub-assemblage are significantly changed. Assuming the points of contraflexure in the columns at mid-story height and at mid-beam length under applied lateral load the comparison of moment and shear diagrams before and after retrofitting of beam-column joint is shown in Figure 6.

If the dimension of enlargement designed adequately and designed as much as rigid and stiff, the presence of the two haunches can significantly reduce the beam and column moments at the joint panel zone interface. The maximum moment in the beam and in the column is relocated away from the original critical sections to the edge of the joint enlargement and can be exploited to force a plastic hinge in the beam.

It should be noted and as pointed by Yu et al. (2000) and Pampanin and Christopoulos (2003) that if the steel angles being rigid the efficiency of the proposed method in modifying the internal shears and moments in the beams and columns is significantly increased.

As showed in Figure 6 and Figure 7, it can be concluded that the effect of the joint enlargement in the shear and moment diagrams of beam and column is: (1) the migration of the maximum moment in the beam to the edge of the joint enlargement from the face of the column, (2) the reduction (relative to the maximum moment in the beam) of the moment at the face of the column, (3) the reduction of the column moment at the level of the beam-column joint, (4) the relocation of the maximum moment in the column to the edge of the steel angle.

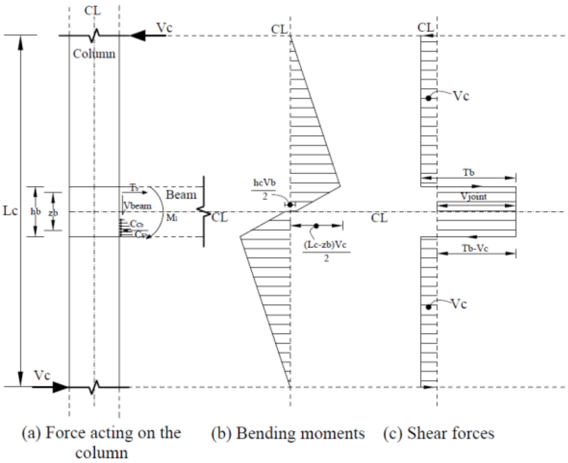


Figure 6. Moment and shear diagram before retrofitting.

The concentrated moment reduction at the edge of joint enlargement, βV_{bret} (db/2)/tanα, where d_b is the depth of the beam, is due to the offset of the beam centerline from the point where the steel angles are connected to the beam. As illustrated in Figure 6 and Figure 7, once the shear in the beam between the point of connection of the haunch (to the beam) and the face of the column βV_{bret} is determined, the moment and shear diagrams from the point of inflexion to the face of the column are known. The value of β is determined by writing deformation compatibility equations between the axial deformation of the steel angles as a haunch and the local deformations of beam and column where the haunch is connected. The full procedure for calculating the value of β is discussed in detail and proposed by Yu et al. (2000) and Pampanin and Christopoulos (2003).

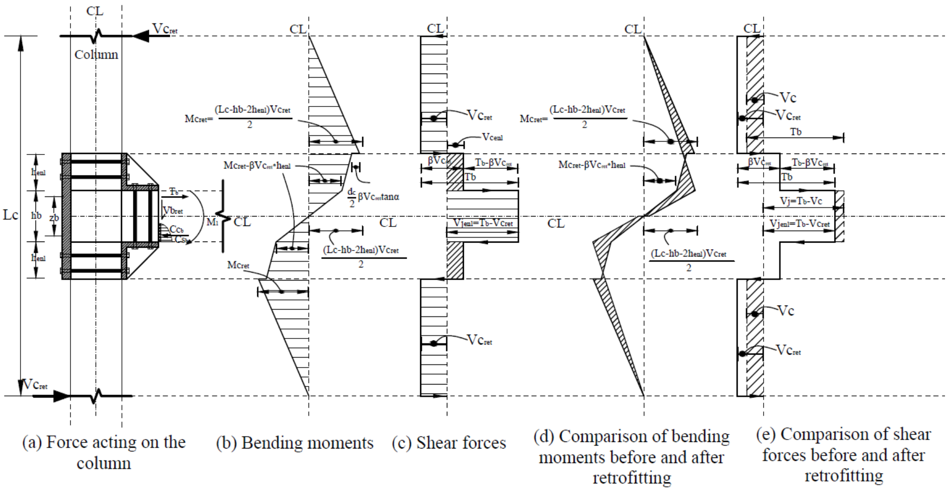


Figure 7. Comparison of moment and shear diagram before and after retrofitting.

A first derivation of the β factor has been proposed by Yu et al. (2000) in the formulation of a (single) haunch retrofit solution for steel frame buildings, which for simplicity accounts only for the beam flexural deformations. Adapting this equation to the configuration proposed in this paper (shown inFigure 1 and Figure 4), the Equation 1 can be derived:

$$\beta = \left(\frac{b}{a}\right)\frac{6Ld_b + 3ad_b + 6bL + 4ab}{3d_b^2 + 6bd_b + 4b^2 + \left(12EI_b / K_d a\cos^2\alpha\right)} \tag{1}$$

Where I_b is the effective moment of inertia of the beam, K_d is the axial stiffness of one steel angles, a and b are horizontal and vertical dimension of joint enlargement respectively. Considering the moment diagram presented in Figure 6, it can be seen that values of β bigger than 1 are desirable for a more efficient protection of the beam-to-column joint.

HIERARCHY OF STRENGTH AND CAPACITY DESIGN IN BEAM-COLUMN JOINTS

Professor Thomas Paulay at The University of Canterbury, New Zealand for the first time proposed hierarchy of strength and capacity design concept in reinforced concrete structures. The chain of Paulay is very well known among engineers that involved with rehabilitation and retrofitting projects. If a hierarchy in the chain of resistance is to be established, then the designer must rationally choose weak links and strong links. Thus strengths or capacity may be compared. It is for this reason that the term “capacity design” was coined. In the capacity design of earthquake resisting structures, elements of primary load resisting systems are chosen and suitably designed and detailed for energy dissipation when subjected to sever inelastic deformation (Paulay and Priestley 1992). All other structural elements are provided with sufficient strength so that the chosen means of energy dissipation can be maintained. The dimension and geometry of the joint enlargement is designed according to the capacity design concept to guarantee that a proper hierarchy of strength is developed. The beam-column joint sub-assemblage can be assumed as a chain that has five links (see Figure 8). Designing the proposed retrofit technique must be such that the equivalent beam shear corresponding to the development of a relocated plastic hinge in the beam at the face of the joint enlargement (weakest link) is lower than those corresponding to undesirable critical mechanisms (strong links). These mechanisms, from the least severe to the most severe on the overall integrity of the structure are (strong links): (i) column hinging, (ii) joint shear failure, (iii) beam shear failure, and (iv) column shear failure. The global target hierarchy of strength of the whole design can be thus summarized as:

$$V_{b,beam-plastic\ hinge} \leq V_{b,column-plastichinge} \leq V_{b,joint} \leq V_{b,column-shear} \leq V_{b,beam-shear} \tag{2}$$

In this conditions the designer is sure that plastic hinge in the beam will be occurred.

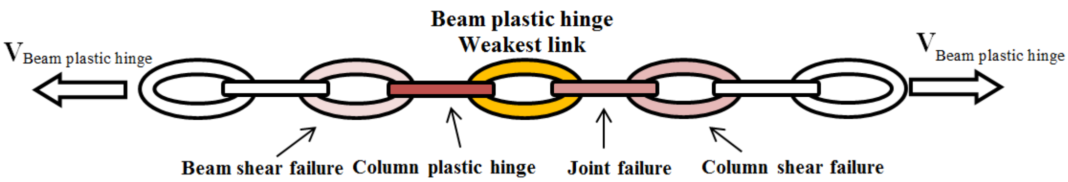


Figure 8. Hierarchy of strength in beam-column joint sub-assemblages.

TRIPLE PANEL ZONE OF ENLARGED BEAM-COLUMN JOINT

The presence of steel angles at both top and bottom of the beam creates an enlarged (or ‘‘triple’’) panel zone so that some parts of the column in the enlarged area contribute to transfer shear force form beam to column. Existing RC buildings usually do not meet current seismic detailing especially in the joint regions and so the joint shear capacity is very lower than joint shear demand and caused to joint shear failure. With the proposed retrofit method joint panel zone was extended to some parts of the beam and column in the joint enlargement area and caused to reducing joint shear demand in the existing joint panel zone. Moreover the presence of post-tensioning forces in the joint region significantly increase confinement of the non-seismically detailed joint and caused to increasing joint shear strength (see Figure 9). It is worth mentioning that Lee and Uang (1997) proposed same concept in the steel connections that retrofitted with haunch method and also proposed the procedure to compute the shear strength of the dual panel zone.

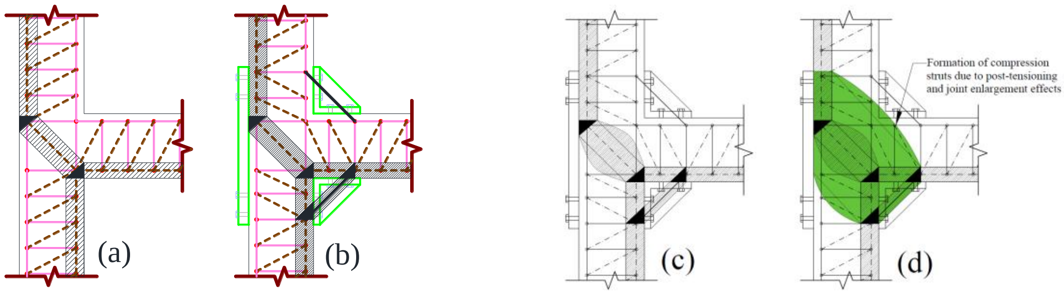


Figure 9. Effects of post-tensioning forces on the effectiveness of proposed method.

CONCLUSIONS

The proposed retrofit method can significantly delayed brittle jointshear failure and prevented beam bottom reinforcement slippagethrough increasing joint area, and improving bondbetween deformed bars and concrete and the proposed retrofit method can result in relocation of the plastichinge away from the column face to outside of the joint panel.

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