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Effect of Varying Top Beam Reinforcement Anchorage Details on Ductility of HSC Beam-Column Joints

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Abstract

The ductility of external beam-column joints effects seismic behavior of a structure and can be a reason for structural collapse. High strength concrete and large amount of steel present in the joint region reduces the ductility even further. This study presents a radical way of increasing high strength concrete external beam-column joint's ductility by varying the beam top reinforcement detailing. Two types of joints (three specimens each) were tested, type-1 specimens were prepared as per regular standards while type-2 specimens were prepared by reducing the beam's top reinforcement anchoring in the column. Load-deflection behavior was studied to observe the stiffness degradation and energy dissipation by joints. Experimental results demonstrated that type-2 specimens were considerably more ductile as compared to type-1 specimens. The energy dissipated by type-2 joints (with 40% less reinforcing bar area in the beam) was 41.5% more than type-1 joints without substantial stiffness degradation. Furthermore, it was observed that by reducing the steel reinforcement in beam, the cracks location changed from inside the joint to beam-column joint's face which may be attributed for increased ductility in type-2 joints. This study shows that by changing the steel reinforcement detailing near external beam-column joints, fatigue behavior and energy absorption capacity can be enhanced hence providing better performance against seismic activities.

Keywords: *beam-column joint, seismic performance, fatigue life, ductility, stiffness degradation, shear capacity of joints, high strength concrete*

1. Introduction

The local construction industry in south Asian region is facing difficulties due to recent wave of earthquakes hitting the region. These have exposed poor performance of reinforced concrete frame buildings. This poor performance can be attributed to the brittle behavior of structures against lateral loads. While considering the seismic performance of structures the ductility of beam-column joints act as a key factor. Several researchers have emphasized on studying the inelastic response of beam-column joints as it effects the overall structural performance to a great degree, as expressed by Durrani and Wight (1985); Park and Ruitong (1988); Leon (1990); Lowes and Moehle (1999); Clyde *et al.* (2000); Mazzoni and Moehle (2001); and Walker (2001).

Earlier studies have successfully evaluated the impact of joint flexibility on structural frame response as explained by. These studies emphasize that beam-column joints are a major concern as far as the seismic performance of buildings is concerned. Several researchers have further investigated the shear distortion in beam-column joints due to anchorage failure which is again a problem associated with reinforcement detailing in the joints as expressed by El-Metwally and Chen (1988) and Alath and Kunnath (1995).

Following the facts stated earlier authors feel that there is a need to improve the ductility of beam-column joints especially for the joints constructed using high strength concrete. The performance of joints depends upon many parameters that effect the behavior of joints. These parameters include concrete compressive strength, stress-strain behavior of concrete, dimensions/geometry of the joint, confinement by reinforcement, column axial compression and bond demand level of the longitudinal reinforcement as studied by Kim and LaFave (2007); Yoshitake (2010) and Baig and Nagai (2013).

Considering the seismic performance of a beam-column joint, one important factor is the amount of steel reinforcement present in the joint region. Although steel reinforcement adds considerable amount of tensile strength to concrete, its amount may change the behavior of reinforced concrete sections from ductile to brittle. By changing the amount of steel, the ductility of beam-column joint changes and it can affect overall seismic performance

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of the buildings. By varying reinforcement in longitudinal and transverse directions, the location of plastic hinges may be changed by controlling the strain in the reinforcement, explained by Lee *et al.* (2009). Similarly Fattah (1987) stated that by controlling the amount of reinforcement and its curtailment the location of plastic hinges may be shifted within a beam member. Several researchers have studied seismic performance of beam-column joints and proposed improved models for the investigation of joint's behavior as expressed by Mitra and Lowes (2007); Sharma *et al.* (2011); and Metelli *et al.* (2015).

A lot of research has been carried out on improvement of structural behavior against earthquakes and performance of beam-column joins. However, there is a need to study the effect of reinforcement detailing on the cyclic/fatigue performance of beam-column joints in terms of stiffness degradation and energy dissipation. Current research study focuses on this approach to investigate the effect of beam's longitudinal reinforcement on the ductility of high strength external beam-column joints. As per current construction practice a lot of steel is provided in joint which is extension of beam's longitudinal steel anchored with column's steel. By changing the reinforcement detailing the proportioning of beam and joint strength can be changed and the plastic hinge can be moved out of joint zone. It is expected that the change in reinforcement detailing will affect the cracking pattern and cracks location in beam-column joint region which will further affect the energy dissipation capacity. Furthermore, stiffness degradation of beam-column joints needs to be studied using cyclic behavior of joints. In this study, the prevailing detailing methods were investigated to study the formation of hinges and overall seismic performance of beam-column joints. The experimental results were compared with FEM (finite element method) models and a three well known empirical models for shear capacity calculation in beam-column joints.

2. Experimental Investigation

To investigate the behavior of joints subjected to low cycle fatigue loading, it was decided to cast two types of joint specimens with two different reinforcement detailing patterns.

All the samples were casted with same column reinforcement and same beam bottom reinforcement. The columns were casted with 4–#13 (4–#4) bars (Figs. 1 and 2) while the beams in both types of joints were having 2–#13 (2–#4) bars at the bottom. Top reinforcement of beams was varied in two types of beam-column joints. Half of the samples were casted with 2– #13 (2–#4) & 1–#16 (1–#5) steel rebar as top reinforcement dowelled into the column while in other samples 16 mm rebar was removed from beam top reinforcement assuming its discontinuity at the joint face. The analogous shape of reinforcement has been shown in Fig. 3. The middle bar in case of type-2 joint is assumed to be curtailed at the joint face. To eliminate the effect of curtailed reinforcement, it was not placed in the beam top portion. Beam-column joints with two 13 mm (0.5 in) and one 16 mm (0.625 in) rebar on top

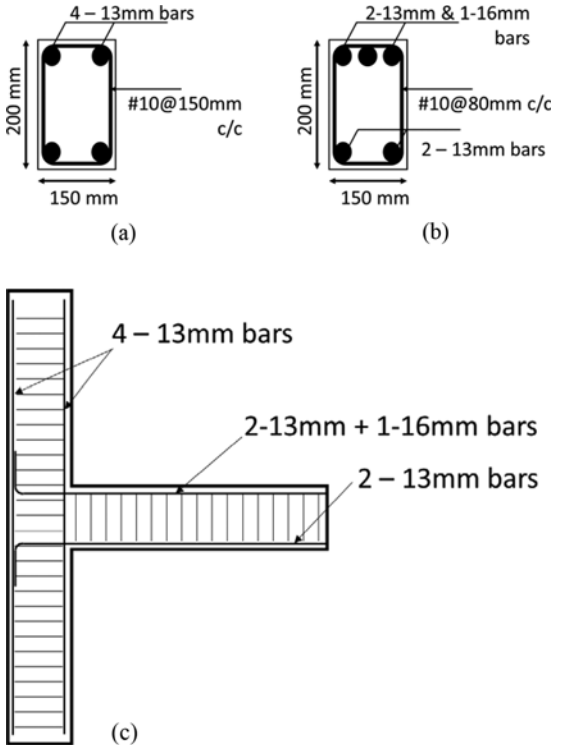


Fig. 1. Type-1 Beam-column Joint Reinforcement Details [1 mm = 0.0394in]: (a) Column Cross-section, (b) Beam Cross-section, (c) Cross Section of Beam-column Joint

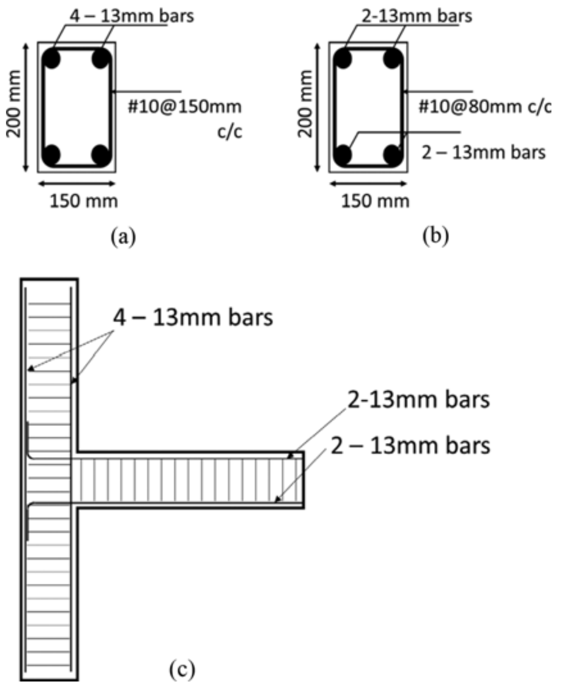
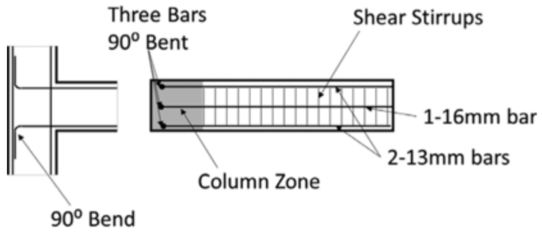
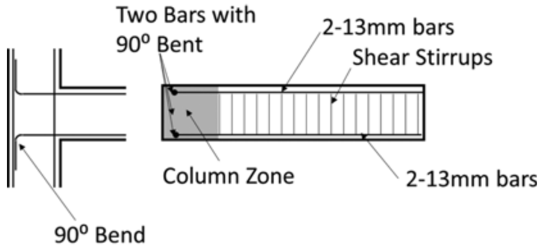


Fig. 2. Type-2 Beam-column Joint Reinforcement Details [1 mm = 0.0394 in]: (a) Column Cross-section, (b) Beam Cross-section, (c) Cross Section of Beam-column Joint

were designated as type-1 joints while others were designated as type-2 joints. Hence, in this experimental study type-2 joints have around 40% lesser top longitudinal steel in the beam



(a)



(b)

Fig. 3. Type-2 Beam-column Joint Reinforcement Details [1 mm = 0.0394in]: (a) Beam Top Reinforcement Anchorage Details for Type-1 Joints, (b) Beam Top ReinforcemenT Anchorage Details for Type-2 Joints

Fig. 4. Specimen Casting and Testing

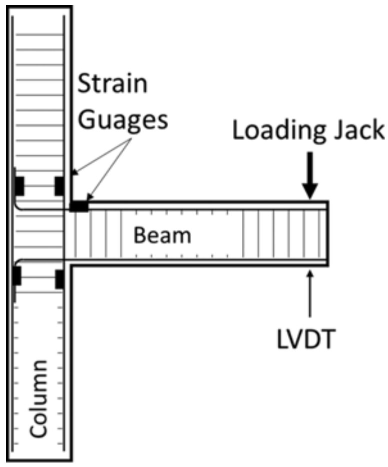


Fig. 5. Test/Loading Setup

projecting from the joint.

The cross-sectional dimensions of beam and column were kept same (150 mm [6 in] wide and 200 mm [7.87 in] deep). The ratio of beam width to its height has been kept ¾ to create similarity with a few previously published experimental details such as Lee *et al*. (2009). Length of the column in joint was 1.5 meters (4.92 ft) while the length of beam projecting from the joint was 0.7 meters (2.3 ft).

In order to keep the joint stable during loading cycles, a special rectangular frame was prepared for bottom end of the column whereas two channel sections were used to hold the upper end of column in place. The bottom rectangular frame would hold the bottom portion of column from all sides. The two channel sections were placed on the sides of column at its upper end to arrest its movement in the direction of applied load. Two channel sections were used so that during negative loading cycle column portion should not backwards.

For determining the theoretical shear capacity of the joint shear strength, the equation given by ACI Committee Report 352 (2002):

$$V_n = 0.083\,\gamma\,\sqrt{f_c'}\,b_j\,h_c \tag{1}$$

where, b_j is the effective joint area, h_c is the depth of column, and f_c' is the concrete compressive strength on 28 days.

The beam joint shear strength calculation was carried out using the ACI Committee Report 318 (2008):

$$V_{jby} = \left(\frac{l_c}{z} - 1\right)V_{by}\,\frac{l_b}{l_c} - V_{by}\,\frac{h_c}{z} \tag{2}$$

where, l_c is length of column face, l_b is the length of beam face, h_c is the depth of joint/column, z is the distance between the centers of upper and lower beam bars and V_{by} is the beam shear strength.

2.1 Loading Details

A low cycle fatigue load was applied on beam's free end to study the load-deflection behavior of beam-column joint. To study pure effect of moment transferred by the beam to joint, no axial load was applied on the column. By avoiding the axial load on column any possibility of secondary moments was also circumvented. To avoid lateral movements special rectangular frame was prepared for bottom end of the column whereas two channel sections were used to hold the upper end of column in place as shown in Fig. 4. The column's upper and lower end conditions should be assumed fixed. LVDTs (linear variable differential transformers) were used to determine the deflection in the beam on application of load. Strain gauges were also used

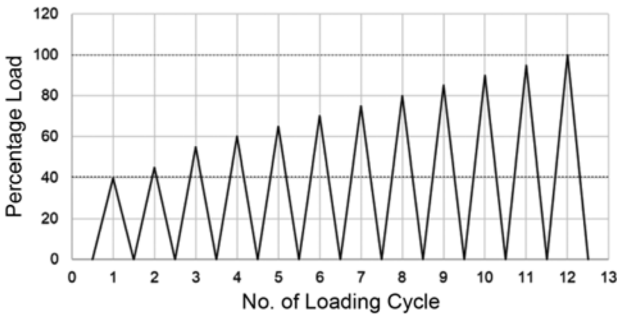


Fig. 6. Loading Cycles

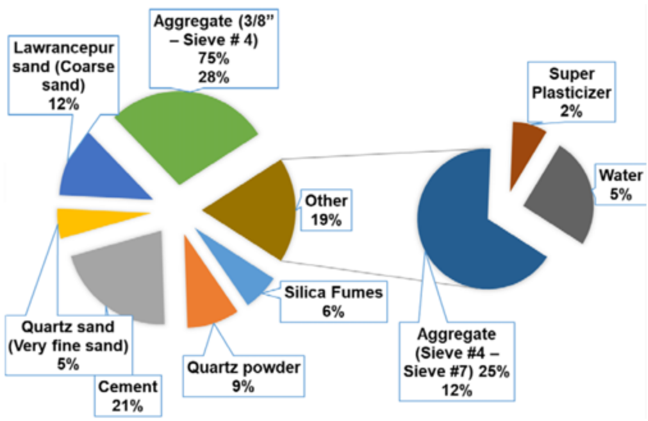


Fig. 7. High Strength Concrete Mix Proportions

Table 1. Properties of Cement used

Properties	Cement
Specific gravity	3.65
Specific surface area, m^2/Kg (sq. ft/lb)	340 (1,660)
Normal consistency (%)	29.3
Initial setting time (min)	103
Final setting time (min)	220

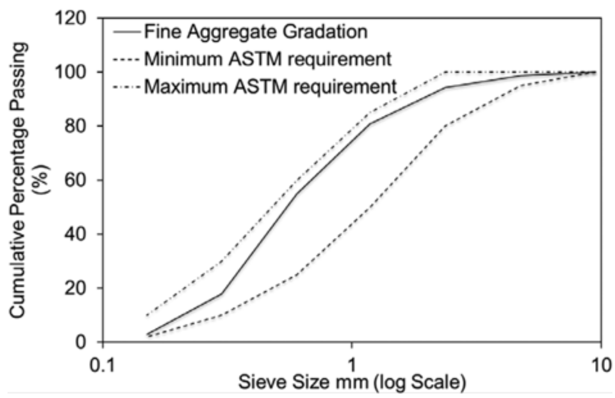


Fig. 8. Gradation of Fine Aggregate (Lawrancepur Sand)

to monitor the state of steel reinforcement across the joint as shown in Fig. 5. Cyclic load ranging from 40% to 100% of the joint's ultimate capacity was applied at the cantilever end of beam (determined by Eqs. (1) and (2). The load was varied gradually from 40% to final failure as shown in Fig. 6.

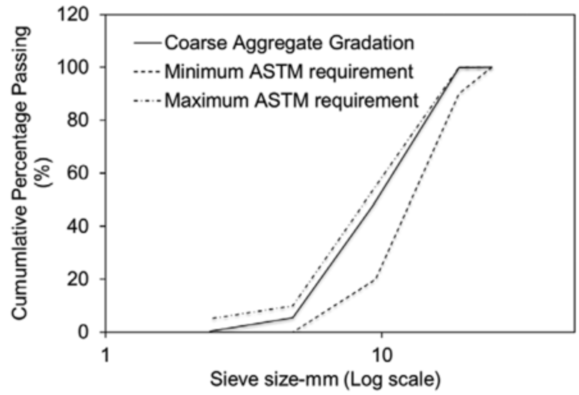


Fig. 9. Gradation of Coarse Aggregate (Margalla Crush)

2.2 Material Details

As the construction industry is moving towards high strength concrete a 60 MPa (8,702.26 psi) concrete was used for casting the specimens. To achieve high strength, silica fumes were added as additional pozzolans and very fine quartz powder was used as additional filler. Mixing of concrete was done by initially making powder slurry and later adding sand and coarse aggregates to it. The proportion of different constituents have been shown in Fig. 7. Table 1 carries the physical properties of ordinary Portland cement used for mixing concrete. Well graded fine and coarse aggregates were used for making concrete (Figs. 8 and 9).

3. Empirical Models Verification

The experimentally determined shear capacity is compared with the shear capacity calculated from three analytical shear capacity models (discussed in Section 3.2) to determine the most suitable empirical model to predict shear capacity of beam-column joints with different ductility values (due to difference in the reinforcement amount) so that it can be used for joint performance studies before structural designing.

3.1 Empirical Models Used

Three empirical relations were used to calculate the shear capacity of the beam-column joints. First one was presented by Tsonos (1992):

$$\left[\frac{\alpha\gamma}{2\sqrt{f_c}}\left(1+\sqrt{1+\frac{4}{\alpha^2}}\right)\right]^5+\frac{5\alpha\gamma}{\sqrt{f_c}}\left(\sqrt{1+\frac{4}{\alpha^2}}-1\right)=1 \tag{3}$$

where, α is the ratio of beam depth to column depth and γ is the joint shear strength in multiples of square root of concrete compressive strength.

Second relation one was suggested by Bakir (2002) and is given below:

$$V=\left(\frac{b_c+b_b}{2}\right)h_c\lambda\left(0.092f_c'+0.55\ln\left(\frac{h_c}{d_b}\right)+0.23\frac{A_{sh}f_{ys}}{\left(\frac{b_b+b_c}{2}\right)h_c}\right) \tag{4}$$

where, b_b is width of the beam, b_c is width of the column, h_c is total depth of column, d_b is diameter of the larger longitudinal bar and λ is the capacity reduction factor (= 0.78)

Third model was presented by Jiuru *et al.* (1992) and is reproduced below:

$$V = V_c + V_f + V_s \tag{5}$$

$$V_c = 0.1\left(1 + \frac{N}{b_c\, h_c\, f_{ac}}\right) b_j\, h_j\, f_{ac} \tag{6}$$

$$V_f = 2\frac{l_f}{d_f} V_f\, b_j\, h_j \tag{7}$$

$$V_s = f_{ys}\frac{A_{sh}}{S}\left(d - a_s'\right) \tag{8}$$

where f_{ac} is the compressive strength of concrete in column, N is the axial compressive load of column, h_j is effective depth of joint parallel to the direction of shear, b_j is effective width of joint transverse to the direction of shear, a_s' is the distance from extreme compressive fiber to the centroid of compressive reinforcement.

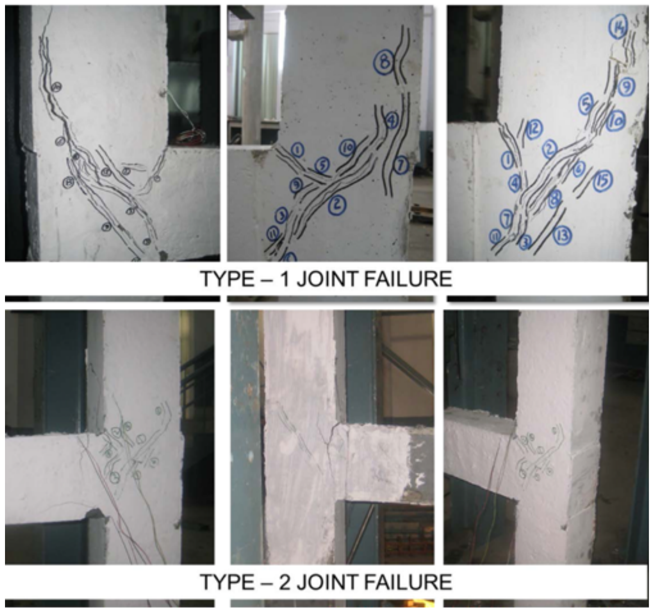


Fig. 10. Crack Pattern in Type-1 and Type-2 Specimens

4. Results and Discussions

As explained in Fig. 5 the load was applied on the edge of the beam (in direction of gravity). The load creates moment which is applied to the beam and eventually transferred to the joint. The efficiency of stress transfer in the joints is enhanced by the anchorage of beam top and bottom reinforcing steel bars in the joint on opposite side. The applied load (gravity load) is transferred through the beam in form of counter moment created by compressive and tensile forces generated in the top and bottom portion of beam. These transferred stresses generate shear stresses in the joint region. The failure might be either due to flexural failure in the beam or shear failure in the joint region. The final failure in flexure in beam portion may occur due to tensile failure at the top or compression failure at the bottom portion of beam at the face of beam-column joint. The two types of failure patterns effect the overall ductility of the system.

4.1 General Behavior and Crack Pattern

The purpose of this study was to investigate the effect of reinforcement detailing on the performance of beam-column joints. The development of cracks during loading cycles was carefully monitored and marked on the samples. Fig. 10 show the cracks pattern for type-1 and type-2 joints. From the figure it can be clearly seen that type-1 joints developed major cracks in the joint region while the other type developed major cracks on the joint-beam face.

In type-1 joint specimen all the cracks were developed in the joint region. Main cause of failure was one major crack developed inside the joint region that propagated in the column. The depth of this crack (at maximum load level) was measured to be more than 15 mm in each specimen of this type. The strain in steel during loading cycles show that the steel in the beam did not yield up to the time of failure. It shows that the steel present at the top of beam region may be enough to resist the moments but at the same time this steel may increase the strength of beam as compared to the joint region which will cause failure due to joint region concrete failure in shear. Although large number of smaller cracks help in dissipating more energy, but one wide/deep crack may cause sudden failure. Hence, this type of failure may not be desirable as it reduces the ductility of joints and repair is also very difficult.

On the other hand, type-2 joints exhibited crack formation on the face of the beam-column joint as well as a few minor cracks inside the joint region. The number of cracks were not large, yet the cracks were not wide open. Further, on releasing the load after each cycle the crack width reduced considerably which is a sign of improved ductile performance of this type of joint. The study of strain in steel shows that in case of type-2 joints the beam top reinforcement reached the yield point, effectively using the steel reinforcement and contributing towards better ductility. At this point the cracks started appearing at the joint-beam face. By this time, for the given reinforcement condition, some shear cracks were observed in the joint region, but the final failure was caused due to the cracks that appeared on joint face. As the cracks appeared outside the joint, the ductility of the beam-column joint system was not compromised.

4.2 Stiffness Degradation

The recorded values of load applied at the end of the beam and deflection at the same point was used to calculate secant stiffness at each loading cycle. Approximate stiffness was calculated as slope of the line drawn between minimum displacement and zero load of each cycle and maximum positive displacement at the peak load of each cycle. These values provided a qualitative measure for the comparison of stiffness degradation in beam-column joint specimens. The load deflection behavior of one specimen of each type of joint has been shown in Figs. 11 and 12

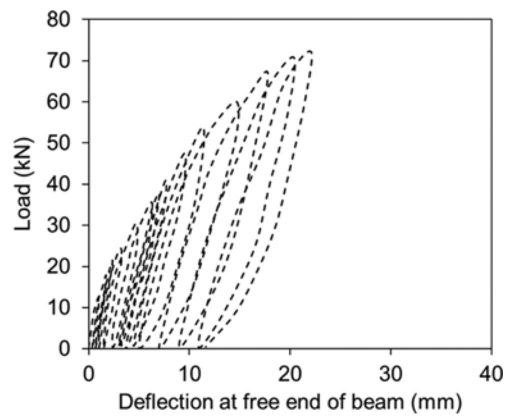


Fig. 11. Load-Deflection Behavior of Type-1 Beam-column Joints [1 mm = 0.0394in; 1 kN = 224.81lbf]

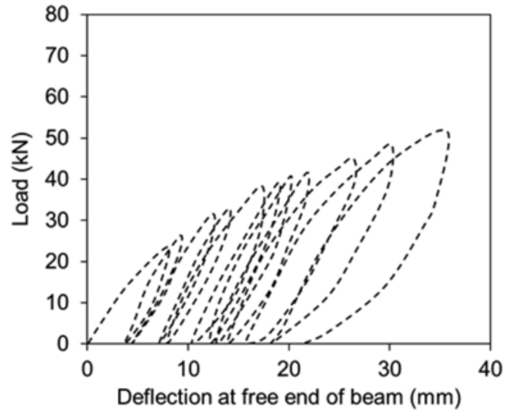


Fig. 12. Load-Deflection Behavior of Type-2 Beam-column Joints [1 mm = 0.0394in; 1 kN = 224.81lbf]

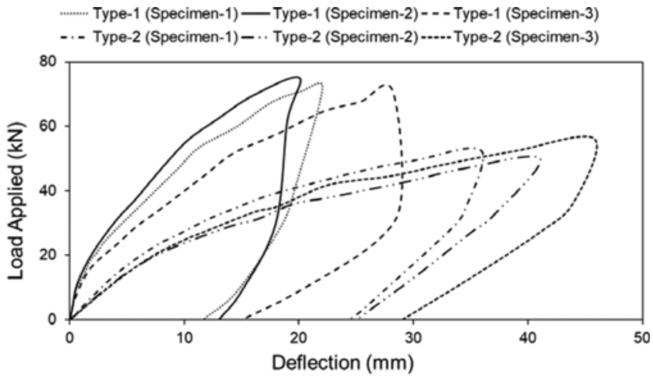


Fig. 13. Load-Deflection Envelop Curve Comparison for All Specimens

while Fig. 13 shows the load-deflection envelop curves comparison of all six specimens tested. Stiffness degradation curves have been shown in Figs. 15 and 16. The stiffness value for each specimen at each cycle of same type was averaged and that value was plotted in the figure.

From the figure it can be observed that type-1 joint has more stiffness during initial cycles due to more amount of steel in the beam's top portion which was carried in the column for anchorage purpose. The amount of steel also affects the load bearing capacity of the joint system. As there is more steel in type-1 joint, the load carrying capacity of this type is greater than the other type. But at the same time stiffness degradation of type-2 joint is lesser than type-1 joint. The stiffness degradation of type-2 joint was observed to be around 1.4% per 1 mm deflection while the stiffness degradation for type-1 joint was

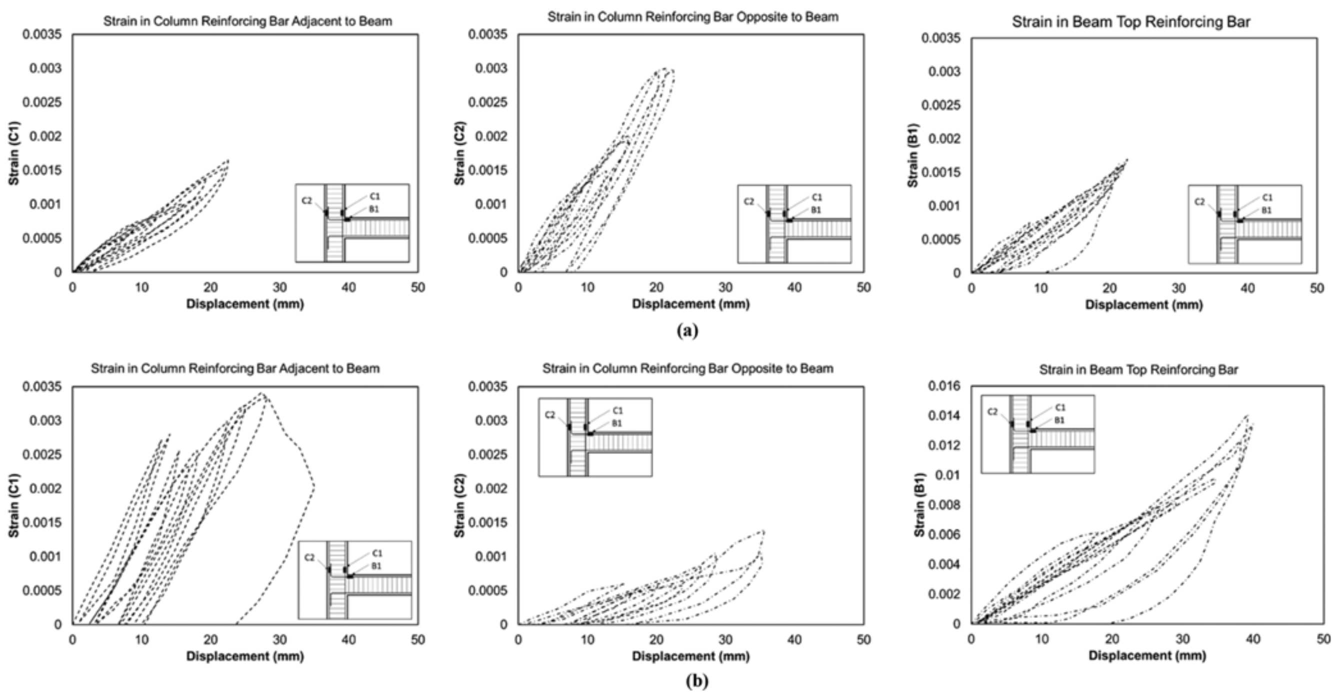


Fig. 14. Strain-Beam Displacement Relation: (a) Type-1 Beam-column Joint Specimen, (b) Type-2 Beam-column Joint Specimen

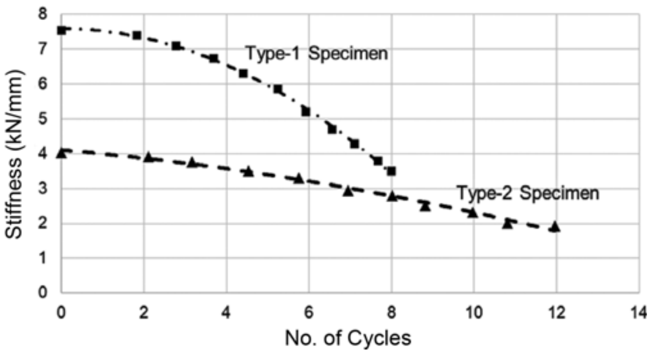


Fig. 15. Comparison of Stiffness Degradation of Type-1 and Type-2 Joints (average of three specimens)

around 7% per 1 mm deflection. As stiffness degradation represents the ductility of joint, it can be concluded that type-2 joints were more ductile as compared to type-1 joints. Lesser stiffness degradation represents better seismic performance in terms of ductility of the beam-column joint. This strengthens the primary concept of this study leading to a beam-column joint that can perform better against seismic activities.

4.3 Energy Dissipation Analysis

In continuation of the ductility comparison, the energy dissipation by the joints of type-1 and type-2 has also been compared. The total area under the load-deflection envelop for each loading

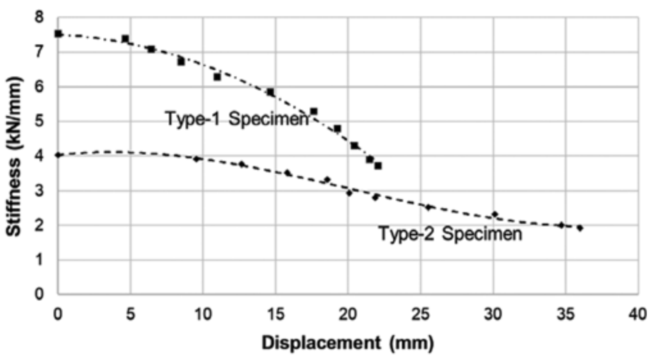


Fig. 16. Comparison of Stiffness Degradation of Type-1 and Type-2 Joints (average of three specimens)

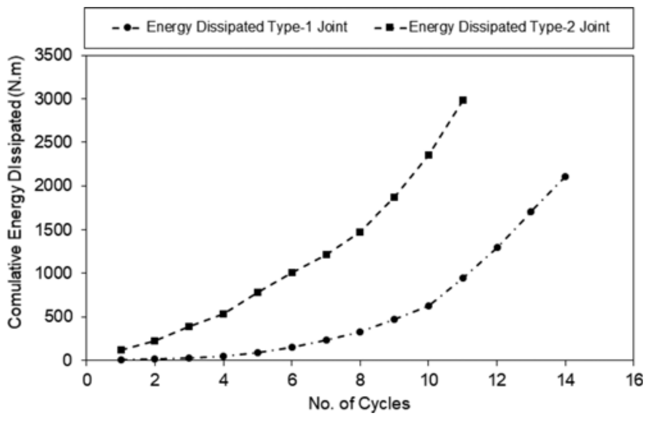


Fig. 17. Cumulative Energy Dissipation per Loading Cycle

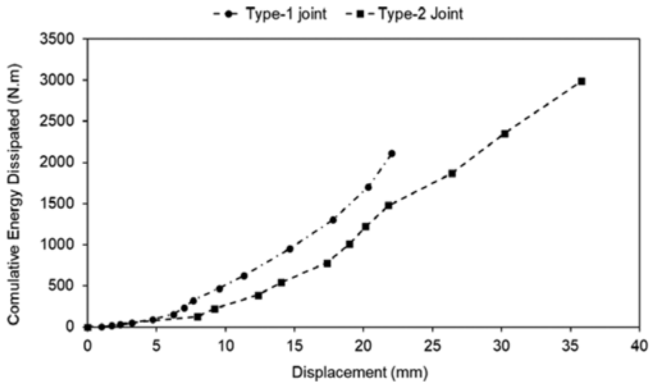


Fig. 18. Cumulative Energy Dissipation vs. Corresponding Deflection

cycle was calculated and considered as the energy dissipated by the joint. The cumulative energy dissipated was calculated and compared. The cumulative energy dissipated by each type of joint has been shown in Figs. 17 and 18 plotted against the loading cycles and deflection at the beam's end respectively.

It is important to mention that the total energy required to take each type of joint to failure was applied in cycles therefore the number of cycles also become an important consideration. The cumulative value of energy dissipation per cycle totals to around 2,109 N.m for type-1 joint and 2,984.5 N.m for type-2 joint and in this case type-2 joint has dissipated around 41.5% more energy. More energy dissipation verifies the improved ductility in type-2 joint specimens. Larger energy dissipation represents a resilient structure against seismic or similar type of loading.

These results show that type-2 joints can perform better against seismic activities as compared to type-1 joints. This improvement in performance is related to the absorption of amount of energy generated by earthquakes. Such structures may be damaged by earthquakes, but the chances of structural demolition are reduced considerably.

The experimental results confirmed our initial perception about the joint behavior. The type-2 model displayed better ductility at lesser maximum load carrying capacity, yet the total energy absorbed was far more as compared to type-1 beam-column joint.

4.4 Empirical Models Comparison

Furthermore, to compare results given by selected mathematical models given by Tsonos (1992), Bakir (2002) and Jiuru (1992) the ratio of values computed using the empirical models and experimental values have been compared (Fig. 19).

For the type-1 joint, Jiuru model gave precise results with estimated/experimental ratio nearly equal to 1.0. Tsonos predicted conservative results with the ratio nearly equal to 0.8. Bakir's model over-estimated the strength giving a ratio of around 1.4. In case of type 2 joints, all the methods over-estimated the strength (estimated / experimental ratio of Tsonos = 1.2, Bakir = 2.05 and Jiuru = 1.4). Bakir's relations in general gives very large values for under designed and other type of beam-column joint. Tsonos

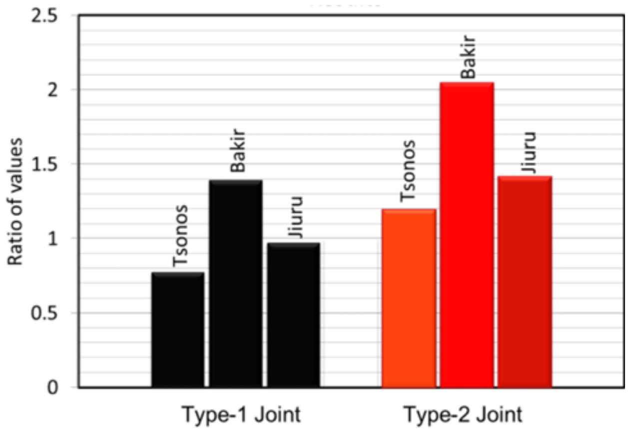


Fig. 19. Ratio of Shear Values Computed by Models of Tsonos, 1992, Bakir, 2002, and Jiuru, 1992 to the Experimental Values

relation is more applicable for type-2 joints while Jiuru's relation is well suited for type-1 joints.

5. Conclusions

Experimental investigation was held to study the improvement in seismic performance of beam-column joints by changing the steel reinforcement detailing. It was observed that by reducing the amount of beam top reinforcement at beam-column joint, seismic performance of these joints may be improved. Although it will reduce the capacity of joint but at the same time the increase in joint's ductility is considerable. If capacity of joint is a concern, the dimensions of columns and beams may be increased to higher values while reducing top reinforcement in beams. At the same time it should be kept in consideration that the most commonly used empirical models may over-estimate strength of joint with reduced top reinforcement. The energy dissipation by external beam-column joint can be enhanced to as much as 41.5% by reducing the beam top reinforcement by 40% at joint face. This practice may produce structures with better ductility and energy absorption capacity. Prime impact of this practice will be on structural collapse against seismic loads which will become very ductile as compared to regular ongoing practice.

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