BEHAVIOR AND STRENGTH OF RC COLUMN-TO-STEEL BEAM CONNECTIONS SUBJECTED TO SEISMIC LOADING

Gustavo J PARRA-MONTESINOS[1] And James K WIGHT[2]

SUMMARY

The seismic behavior of reinforced concrete column-to-steel beam (RCS) connections is studied in this paper. Experimental results from the tests of nine exterior RCS joints subjected to large load reversals are presented. Also, a simple model to predict the shear strength of RCS connections is proposed. The influence of different joint details on the seismic response of the connections is discussed. These joint details are two-part U-shaped stirrups passing through drilled holes in the steel beam web, steel cover plates or band plates surrounding the joint region, dowel bars attached to the steel beam flanges, and steel fiber concrete or engineered cementitious composite (ECC) material in the connection. Experimental results indicate that RCS frames are suitable for use in high seismic risk zones. In addition, good agreement was found between experimental results and the shear strengths predicted by the proposed model.

INTRODUCTION

The use of hybrid structures has gained popularity in the last twenty years. One of the most efficient hybrid systems is represented by RCS frames, which consist of reinforced concrete (RC) columns and steel (S) beams. In RCS frames, the advantages of reinforced concrete and steel structures are combined to form a cost- and time-effective type of construction. RC columns are more cost effective in terms of axial strength and stiffness than steel columns [Sheikh et al. 1987]. Also, they offer superior damping properties to the structure, especially in tall buildings. On the other hand, steel floor systems are lighter and require little or no formwork, reducing the weight of the building and increasing the speed of the construction.

Despite the advantages offered by RCS structures, their use has been restrained primarily to low and moderate seismic risk regions because of the lack of design provisions that consider the behavior of these hybrid systems under large load reversals. In addition, the study of RCS joint behavior has been limited primarily to interior connections. Therefore, an experimental and analytical program was undertaken at the University of Michigan to develop information on the inelastic cyclic response of RCS joints, especially in exterior RCS connections. In this paper, the behavior of nine exterior RCS joints is discussed. Also, a shear strength model is proposed for both interior and exterior RCS joints.

EXPERIMENTAL PROGRAM

The experimental program included the testing of nine 3/4-scale exterior RCS connections subjected to lateral cyclic loading. The specimens consisted of an RC column and a steel beam passing through the column. A steel section was embedded in the RC column to simulate the steel columns commonly used for erection purposes in RCS structures. Each specimen was subjected to twenty cycles of increasing lateral displacements from 0.5% to

[1] *Ph.D. Candidate, Dept. of Civil and Envir. Eng., Univ. of Michigan, Ann Arbor, MI, USA 48109-2125. Email:gjpm@umich.edu*
[2] *Professor, Dept. of Civil and Envir. Eng., Univ. of Michigan, Ann Arbor, MI, USA 48109-2125. Email:jwight@umich.edu*

5.0% story drift. Each cycle to a higher story drift level was performed twice to evaluate the stiffness retention capacity of the specimens during repeated cycles.

The test specimens were divided into two groups depending on the beam depth to column depth ratio. This ratio was 0.60 for group 1 (specimens 1 thru 5), and 1.0 for group 2 (specimens 6 thru 9). Specimens 1 and 6 were used as the control specimens for groups 1 and 2, respectively. These two specimens had normal connection details that included overlapping U-shaped stirrups passing through holes drilled in the web of the steel beam, and face bearing plates between the beam flanges at the front and back column faces. The joint transverse reinforcement volumetric ratio was 0.9% for these two specimens. Different joint details and new material concepts were used in the other seven specimens to evaluate their influence on the joint strength and inelastic cyclic behavior of RCS connections. They included steel cover plates surrounding the joint (Fig. 1a), steel band plates wrapping the columns in the regions just above and below the steel beam (Fig. 1b), dowel bars attached to the beam flanges, steel fiber reinforced concrete, and engineered cementitious composite (ECC) in the connection. The specimens were designed such that most of the inelastic activity would concentrate in the joint region to better evaluate the shear strength and inelastic cyclic response of the connections. Table 1 summarizes the main features of the test specimens.

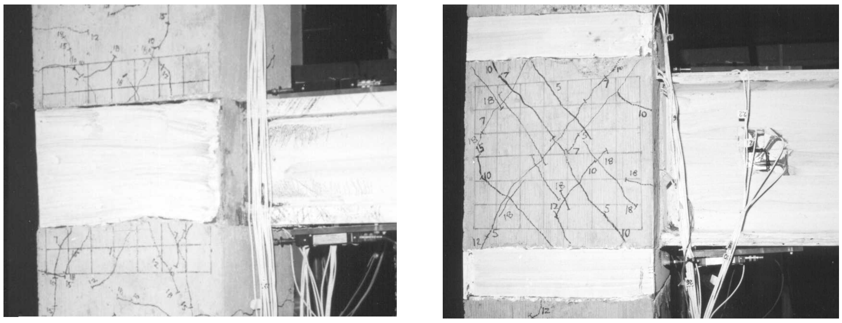
Table 1 - Description of Test Specimens

Specimen	Features
1	Built-up beam, face bearing plates (FBP), 2 #13 (ϕ = 13mm) stirrups in joint.
2	W8x58 beam, FBP, 2 #13 (ϕ = 13mm) stirrups in joint.
3	Built-up beam, steel fiber concrete, FBP, 2 #10 (ϕ = 10mm) stirrups in joint.
4	Built-up beam, dowel bars, FBP, 2 #13 (ϕ = 13mm) stirrups in joint.
5	Built-up beam, steel cover plates around joint, no stirrups in joint.
6	W14x38 beam w/ cover plates (CP), FBP, 3 #13 (ϕ = 13mm) stirrups in joint.
7	W14x38 beam w/ CP, FBP, steel band plates, 3 #13(ϕ = 13mm) stirrups in joint.
8	W14x38 beam w/ CP, FBP, ECC, no stirrups in joint.
9	W14x38 beam w/ CP, FBP, transverse beams, band plates, no stirrups in joint.

EXPERIMENTAL RESULTS

All nine specimens showed good overall response and were able to maintain their strength up to lateral displacements of 5.0% story drift. In addition they exhibited a good stiffness retention capacity throughout the test and during repeated cycles at the same drift level.

Control specimens 1 and 6 suffered the greatest damage in the joint. Diagonal cracks started to form at 1.0% drift and continued growing as the lateral displacement was increased in the specimens, leading to spalling of cover concrete at the end of the test. Even though severe damage was observed in these two specimens, they showed a stable hysteresis response with no decay of shear strength (Figs. 2a and 2c). Low to moderate damage in these specimens corresponded to joint shear deformations of up to 0.0075 radians. When the joint shear distortion exceeded 0.01 radians, the specimens exhibited severe damage, characterized by large diagonal cracks in the joint and spalling of the concrete. Another important phenomenon observed in these specimens was the presence of gaps between the steel beam flanges and the surrounding concrete caused by bearing forces and bar slip. This led to a rigid body rotation of the steel beam inside the joint and the formation of large bearing cracks that originated from the corners of the beam flanges. The rigid body rotation of the steel beam led to a softening of the subassembly and significantly contributed to the overall specimen drift.

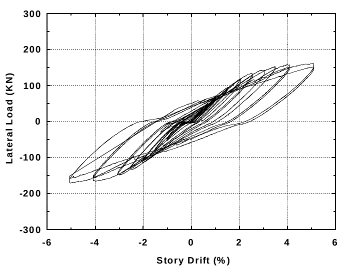


a) Steel Cover Plates in Specimen 5 **b) Steel Band Plates in Specimen 7**

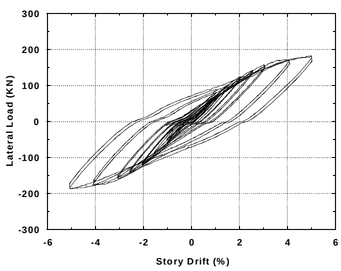
Fig. 1 - Steel Cover Plates and Steel Band Plates in Test Specimens

The addition of special details and new materials to the joint region, such as dowel bars, steel cover plates and band plates, steel fiber reinforced concrete, and ECC material, led to significant improvements in the inelastic cyclic response of the joints compared to the control specimens. Dowel bars attached to the steel beam flanges effectively distributed the stresses caused by bearing of the steel beam flanges on the surrounding concrete, as evidenced by a decrease in the damage observed in these regions of the columns. This led to a reduction in the rigid body rotation of the steel beam inside the joint, and thus to an increase in the stiffness of the subassembly.

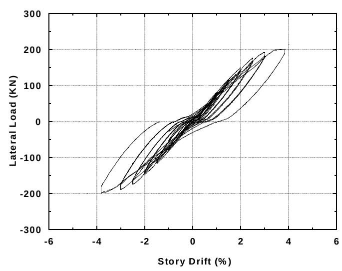
The influence of steel cover plates and band plates was studied in specimens 5, 7, and 9. Specimen 5 had steel cover plates surrounding the joint region over the beam depth. Because of the confinement provided by the cover plates, this specimen showed a 12% increase in shear strength at 5.0% drift compared to specimen 1 and had the best stiffness retention capacity of all the specimens in group 1. Specimen 7, with steel band plates wrapping the columns in the regions just above and below the steel beam, exhibited a 50% increase in shear strength compared to the control specimen 6 (Fig. 2d). Despite the increase in joint shear, the damage in the joint region was moderate (Fig. 1b) because the steel band plates effectively confined the concrete in the regions of the joint outside the width of the beam flanges. Specimen 7 also showed the best stiffness retention capacity among the specimens in group 2. Specimen 9 also had steel band plates. However, in this specimen no steel transverse reinforcement was used in the joint because of the presence of transverse beams. This specimen showed only moderate damage up to 3.0% drift. As the lateral displacement was increased, diagonal cracks became larger, leading to a spalling of the concrete in the joint at the end of the test. Even though severe damage was observed in the joint at 5.0% story drift, specimen 9 showed a 25% increase in shear strength compared to specimen 6 (Fig. 2f) and had a good stiffness retention capacity. This suggests that stirrups can be eliminated from the connection if steel band plates are used in the joint.



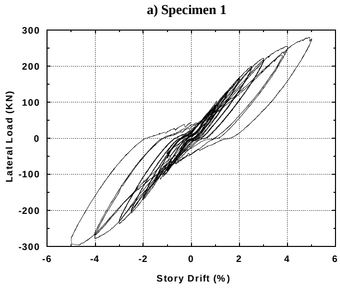
a) Specimen 1



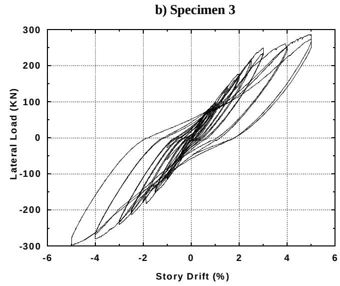
b) Specimen 3



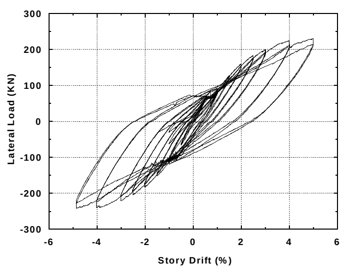
c) Specimen 6



d) Specimen 7



e) Specimen 8



f) Specimen 9

Fig. 2 - Load vs. Displacement Behavior

The influence of new material concepts on joint response was studied in specimens 3 and 8. In specimen 3, with steel fiber concrete, the amount of joint transverse reinforcement was approximately 50% of that used in specimen 1. This specimen showed a 7% increase in strength at 5.0% drift and a better energy dissipation capacity, compared to specimen 1, as shown by the wider load vs. displacement hysteresis loops (Fig. 2b). Specimen 8, with ECC, showed slight damage in the joint and a 50% increase in joint shear strength compared to specimen 6 (Fig. 2e), even though no steel transverse reinforcement was used in the connection. It should be mentioned that this specimen sustained joint distortions greater than 0.02 radians without significant damage because of the tension ductility of the ECC.

SHEAR STRENGTH OF RCS JOINTS

The shear strength of RCS joints is given by contributions from the steel web panel and concrete compression struts. Numerous detail configurations can be used in RCS joints, thus it would be impractical to present strength equations and factors for all possible cases. In this paper, design equations are proposed for RCS connections having the following joint details: stirrups passing through holes in the steel beam web, steel cover plates, steel band plates, extended face bearing plates, and steel columns embedded in the RC columns. The influence of fiber reinforced concrete and engineered cementitious composite on shear strength is also considered in the proposed equations.

The design equations proposed in this paper are expressed in terms of the horizontal shear strength. Therefore, a relationship must be established between the external forces applied to the joint and the shear resisting mechanisms. From equilibrium of horizontal forces in an RCS joint and assuming a lever arm between the joint horizontal shear resistant forces equal to the beam depth, d_{beam}, minus the flange thickness, t_f, the following equation can be derived,

$$\frac{\sum M_{ub}}{d_{beam} - t_f} - V_{ucol} = V_{uh} \leq \phi V_{njh} \qquad [1]$$

where $\sum M_{ub}$ is the summation of factored moments in the beams framing into the joint, V_{ucol} is the average factored shear force in the columns, ϕ is a strength reduction factor, and V_{njh} is the nominal horizontal shear strength of the joint.

Steel web panel

The behavior of the steel web panel in an RCS joint is similar to that observed in steel structures. Yielding starts to occur in the middle of the web panel, extending towards the beam-column interface as the load is increased. The distribution of stresses in an interior RCS joint follows a symmetrical pattern while in exterior connections higher stresses are concentrated in the front half of the joint. It was found that in exterior RCS connections more than 80% of the web panel width effectively contributes to the shear strength of the joint at high drift levels. In design of exterior RCS connections, the contribution of the steel web panel to horizontal joint shear strength is taken to be approximately $0.80\tau_y\, h_{col}\, t_w$, where τ_y is the yield shear strength of the steel web panel, h_{col} is the column depth, and t_w is the thickness of the web panel. In interior connections, a higher effective web panel width contributes to joint shear strength due to the symmetrical distribution of stresses in the steel web. Using an effective web panel width equal to $0.90h_{col}$ and $0.80h_{col}$ for interior and exterior joints, respectively, the horizontal shear strength of the steel web panel can be expressed as follows (Fig.3a),

$$V_{wh} = \tau_y\,(0.90 \text{ or } 0.80)\,h_{col}t_w = \frac{f_y}{\sqrt{3}}(0.90 \text{ or } 0.80)\,h_{col}t_w \qquad [2]$$

where f_y is the yield strength of the steel web panel.

Concrete compression struts

The contribution of the concrete to joint shear strength can be represented by the action of compression struts. The location and strength of the compression struts will depend on the joint details. In this paper, it is assumed that all RCS connections have face bearing plates between the steel beam flanges. Other joint details such as steel columns, extended face bearing plates, and steel band plates, are optional. Two different compression struts can be simultaneously contributing to joint shear strength: an *inner compression strut* and an *outer compression strut*.

Inner concrete compression strut

The inner compression strut is activated by direct bearing of the concrete on the face bearing plates and flanges of the steel beam (Fig.3b). The contribution of this strut is estimated by assuming that it has an angle of inclination, θ_i, equal to $arctan(d_{beam} / h_{col})$, and an effective depth, d_i, equal to 30 percent the length of the diagonal compression strut, L_i. This inner compression strut depth was determined based on the crack patterns observed in the test specimens after the removal of the concrete outside the beam flanges. In order to fully activate the inner compression strut, the face bearing plates should extend over the width of the steel beam flanges. Hence, the inner compression strut width would correspond to the width of the flanges, b_f, minus the thickness of the steel web, t_w.

In order to determine the contribution of the inner compression strut to joint shear strength, an effective concrete strength in the strut must be established. Different values have been proposed for concrete strength in strut and tie models [MacGregor 1997; Schlaich et al.1987]. The effective concrete strength has been usually expressed as $fce = v\ f'_c$, where v is a factor that accounts for cracking of the strut, reinforcement, and confinement. The strength of the compression struts in RCS joints has also been found to be affected by the type of connection (i.e. interior, exterior). This effect is accounted for by a factor k_1, which is 1.0 and 0.8 for interior and exterior joints, respectively. The decrease in the value of the k_1 factor for exterior connections is due to the lack of direct bearing on the concrete compression struts at both sides of the joint caused by compression flange forces. Therefore, the concrete strength of the inner compression strut can be expressed as $fce = k_1\ v\ f'_c$. Based on the experimental results obtained from the testing of nine exterior RCS connections, a factor v equal to 1.0 is proposed. This factor has been increased from those proposed for similar struts in RC members to account for the excellent confinement provided by the steel beam flanges, face bearing plates, web panel, and surrounding concrete. If transverse beams are framing into the joint, v can be increased by 15%. The horizontal strength of the inner compression strut, V_{ih}, can then be expressed as,

$$V_{ih} = k_1 v\ f'_c\, d_i b_i \cos\theta_i = 0.30\, k_1 v\ f'_c\, h_{col}(b_f - t_w) \qquad [3]$$

where f'_c ` *55 MPa (8,000 psi).*

Outer concrete compression strut

The outer compression strut acts in the regions outside the width of the steel beam flanges. In order to activate a diagonal compression strut in these regions special details, referred to as "shear keys", must be used to achieve a proper transfer of shear forces from the flanges of the steel beam to the regions outside the width of the flanges. The shear keys considered in this paper are: steel columns embedded in the RC columns, extended face bearing plates, and steel band plates wrapping around the columns just above and below the steel beam. The embedded steel columns are generally used for erection purposes. A steel frame is first built and then RC columns are cast around the steel columns. The extended face bearing plates consist of the extension of the face bearing plates to the regions of the columns just above and below the steel beam. In this paper, the face bearing plates are considered to have the same width as the flanges of the steel beam. The third shear key considered is steel band plates surrounding the column regions just above and below the joint. These band plates are connected to the steel beam by direct welding over the width of the beam flanges and by stiffeners to increase the stiffness of the band plates.

The mechanisms of transfer of forces from the steel beam to the regions outside the beam flanges are illustrated in Fig. 4. In all three cases the shear forces are transferred by direct bearing of steel plates on the surrounding concrete. If the transfer of shear forces is achieved by means of a steel column or extended face bearing plates, stirrups in the column regions above and below the steel beam are required to equilibrate the outward thrust produced by the bearing on the concrete of these two shear keys [Deierlein et al. 1989]. This translates into tensile forces in the stirrups that then activate the outer diagonal compression strut. A key factor in this case is to determine the width of the outer compression strut, b_o. Figs. 4a and 4b show the recommended outer compression strut widths when steel columns or extended face bearing plates are used in the connection. This recommended strut widths were determined based on a 3:1 horizontal projection from the outer edges of these two shear keys, and have been validated by experimental results. If steel columns and extended face bearing plates are combined in the same joint, an average width can be used. When steel band plates are used in the joint, it has been found from the testing of two RCS connections that the outer compression strut is primarily activated by direct bearing of the concrete on the band plates. This was indicated by the high stresses measured in the bearing zones of the plates and negligible stresses in the plates parallel to the direction of loading. The recommended width of the outer compression strut for this case is shown in Fig. 4c. When steel band plates are used in combination with steel columns, it is recommended to use the strut width corresponding to the presence of steel band plates.

A similar procedure to that used for determining the strength of the inner compression strut is used for the outer compression strut. In the outer strut, confinement can be provided by stirrups passing through holes in the web of the steel beam, steel cover plates wrapping the joint, steel band plates just above and below the steel beam, or by fibers in the concrete or cementitious matrix. The shear keys are assumed to be effective over a depth equal to $0.25d_{beam}$, as suggested by Deierlein et al. [1989], given that the shear keys extend to at least this depth. Hence, the outer compression strut is assumed to act with an angle of inclination, θ_o, equal to $arctan(1.25d_{beam} / h_{col})$ (Fig. 3c). The effective depth, d_o, is also assumed to be equal to 30 percent of the length of the outer diagonal compression strut, L_o, as assumed for the inner compression strut. The effective concrete strength of the outer compression strut is expressed as $fce = k_1\, v\, k_2\, f'_c$, where k_1 accounts for the type of connection as described before, and the factors v and k_2 are defined below. Therefore, the horizontal shear strength of the outer compression strut is given by,

$$V_{oh} = k_1 v\, k_2 f'_c\, d_o b_o \cos\theta_o = 0.30\, k_1 v\, k_2 f'_c\, h_{col} b_o \qquad [4]$$

Based on experimental results, the following values of the factors v and k_2 are proposed:

v = 1.0 if cover plates wrapping the joint over the beam depth, or ECC are used in the joint.

v = 0.85 if stirrups passing through the steel web, or fiber concrete are provided in the joint.

v = 0.65 if no confinement is provided between the beam flanges (only if steel band plates are used).

k_2 = 1.0 if steel band plates wrapping the column regions just above and below the steel beam are used.

k_2 = 0.75 if no steel band plates above and below the steel beam are used.

As was assumed for the inner compression strut, v can be increased by 15% if transverse beams frame into the joint. The contribution of the outer compression strut is also limited by the bearing strength of the concrete mobilized by the shear keys. ASCE [1994] proposed a concrete bearing strength of $2f'_c$. Therefore, the contribution of the outer compression strut is limited to $2f'_c A_{bearing}$. The bearing area, $A_{bearing}$, is determined based on the geometry of the shear keys and assuming that the forces are transferred within a depth of $0.25d_{beam}$. For the cases when the shear forces are transferred through shear keys and stirrups above and below the steel beam, it must be assured that enough ties are provided to carry the horizontal shear forces being transferred to the outer compression strut. The area of stirrups effectively transferring shear forces to the outer strut, A_{veff}, is given by the area of ties within the outer compression strut width and a distance equal to $3d_{bs}$ from this outer strut (Fig. 4a), where d_{bs} is the diameter of the ties. Hence, the minimum area of ties over a depth of $0.25d_{beam}$ above and below the beam shall be $A_{veff} \geq V_{oh} / f_{yv}$, where f_{yv} is the yield strength of the ties. Finally, the distance between the ties parallel to the direction of loading should not be greater than $12d_{bs}$ (Fig. 4a). This limit is given to assure that the ties perpendicular to the direction of loading have sufficient flexural stiffness to anchor the outer compression strut.

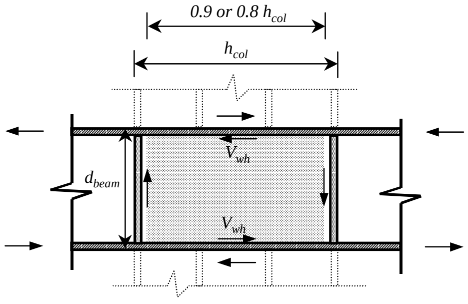
After the contributions of the steel web panel and concrete compression struts have been found, the horizontal joint shear strength is determined as follows,

$$V_{njh} = V_{wh} + V_{ih} + V_{oh} \qquad [5]$$

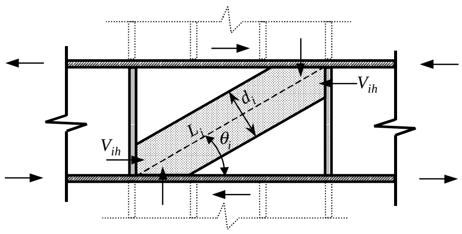
and, combining Eqs. [1] and [5], the required joint horizontal shear strength is expressed as follows:

$$\phi(V_{wh} + V_{ih} + V_{oh}) \geq \frac{\sum M_{ub}}{d_{beam} - t_f} - V_{ucol} \qquad [6]$$

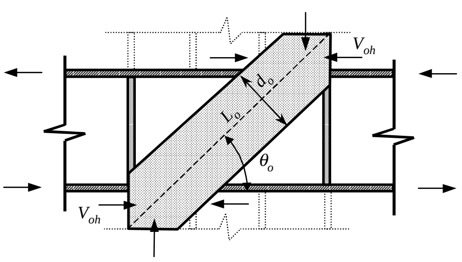
where the strength reduction factor, ϕ, is taken equal to 0.80 for shear strength in RCS joints.



a) Steel Web Panel

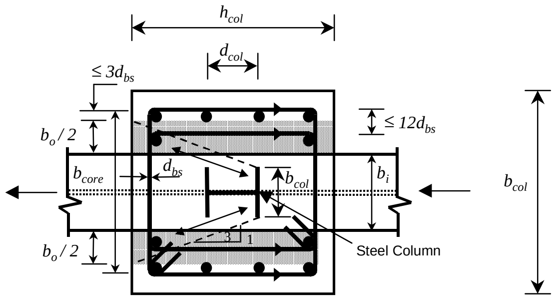


b) Inner Compression Strut



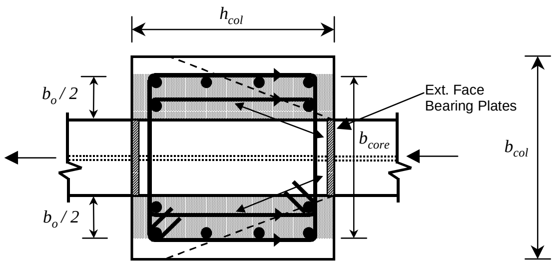
c) Outer Compression Strut

Fig. 3 - Shear Strength Mechanisms



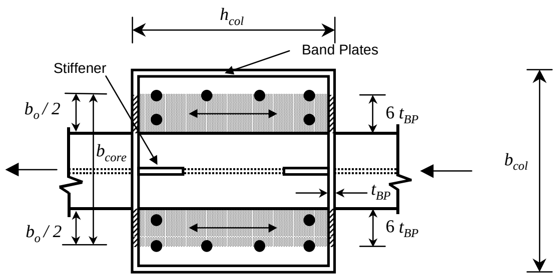
$b_o = 1/3\ (h_{col} + d_{col}) + b_{col} - b_i \leq\ b_{core} - b_i$

a) Steel Column



$b_o = 2/3\ (h_{col}) \leq\ b_{core} - b_i$

b) Extended Face Bearing Plates



$b_o = 12\ t_{BP} \leq\ b_{core} - b_i$

c) Steel Band Plates

Fig. 4 - Outer Compression Strut Widths

The proposed model was applied to different interior and exterior RCS connections tested by Kim et al. [1997], Kanno [1993], and the writers. The specimens were chosen such that they included most of the joint details covered in the proposed model. These joint details were stirrups passing through the web of the steel beam, steel band plates, extended face bearing plates, and transverse beams. In addition, the model was applied to connections that did not have shear keys, and one specimen with high strength concrete (f'_c > 55 MPa). Fig. 5 shows the ratio between the predicted and the experimental shear strengths, V_{calc} / V_{exp}, for twelve interior and exterior RCS joints. It can be seen that the proposed model accurately predicted the shear strength of these connections, with a mean for V_{calc} / V_{exp} of 0.93 and a standard deviation of 0.06.

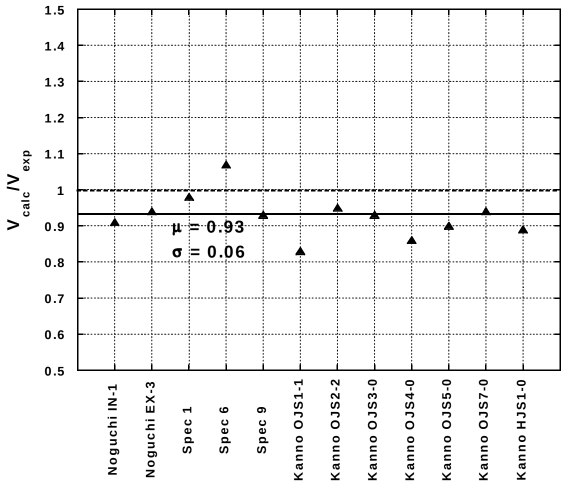


Fig. 5 - Calibration of Proposed Model

CONCLUSIONS

Results from the testing of nine exterior RCS joints show that hybrid structures consisting of RC columns and steel beams are suitable for use in high seismic risk zones. All nine specimens showed good overall response to cyclic load reversals. They were able to maintain their strength at levels of story drift higher than those expected during a major earthquake, with little or no deterioration of stiffness during repeated cycles at the same drift level. Moreover, all the specimens showed good stiffness retention when the lateral displacement was increased beyond 2.0% story drift. The addition of steel cover plates or band plates significantly increased the strength and enhanced the stiffness retention capacity of RCS joints. In addition, the use of steel band plates, ECC, and fiber concrete allowed the reduction or even the elimination of steel transverse reinforcement in the connection. This simplifies the joint detail, especially when beams are framing into the same column from different directions. Finally, a model has been proposed to predict the shear strength of RCS joints. Good agreement was found between experimental results from tests of interior and exterior RCS connections and the shear strengths predicted by the proposed model.

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