Connections Between Steel Frames and Concrete Walls

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Mixed structures are frequently used in modern construction.[1,2] These structures typically combine a stiff reinforced concrete shear wall or central core with a flexible steel frame, as shown in Fig. 1. This structural system can be very economical, because the concrete element can be quickly constructed by modern slip-forming techniques. Further, the steel frame is typically very light, because the lateral deflections are controlled by the reinforced concrete.

This type of structure has many potential applications, including seismic resistant design. However, these applications require that the behavior of the mixed structure be well understood. Considerable study has been devoted to the behavior of the steel frame and the reinforced concrete components, but the behavior of the connections between those very different components is not well understood.

This paper describes an analytical and experimental study into the behavior of one such connection. This connection combines a steel plate, which is embedded into the concrete with headed metal studs, with a typical steel frame connection between the plate and the beam, as shown in Fig. 2. Variations of this connection are used in modern construction, but there is only minimal knowledge of the strength and ductility exhibited by this connection. This study investigates several variations in the subject connections. A number of prototype structures are designed and then analyzed for different loading conditions. The computed behavior is compared to findings from previous research, and an appropriate design procedure is developed. Finally, the results of a series of experiments are described. These results verify the effectiveness of the design procedure, and they provide valuable evidence on the strength and ductility of these connections.

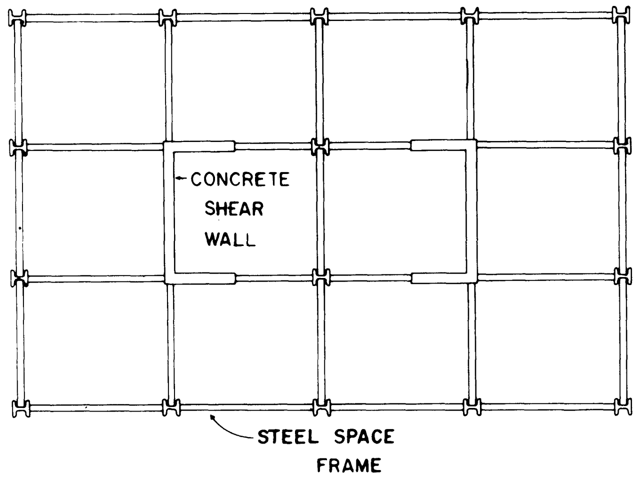

Fig. 1. Plan view of a typical steel frame-shear wall structure

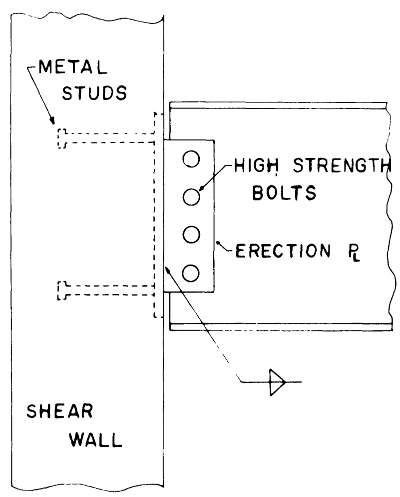

Fig. 2. Typical frame-wall connection detail

DESIGN AND ANALYSIS OF PROTOTYPE STRUCTURE

Several alternate prototype structures were designed and analyzed. These buildings were generally of intermediate height (5–15 stories), and were mixed steel frame-reinforced concrete shear wall structures of the type shown in Fig. 1. The structures were designed to resist gravity, wind, and Uniform Building Code (UBC-1976)[3] seismic design loads for Seattle. They were then analyzed under the different loadings by a linear elastic finite element program, SAP IV.[4] The analyses were performed for a number of

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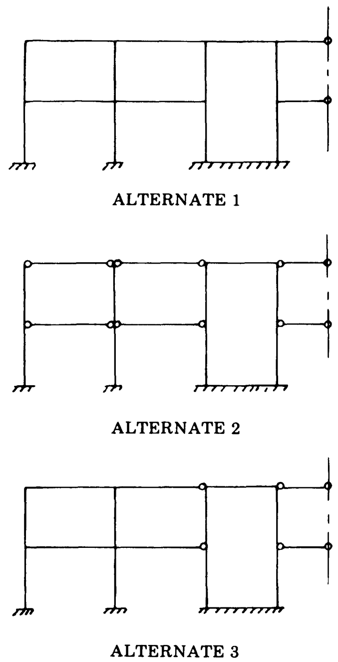

Fig. 3. Definitions of alternate connection conditions

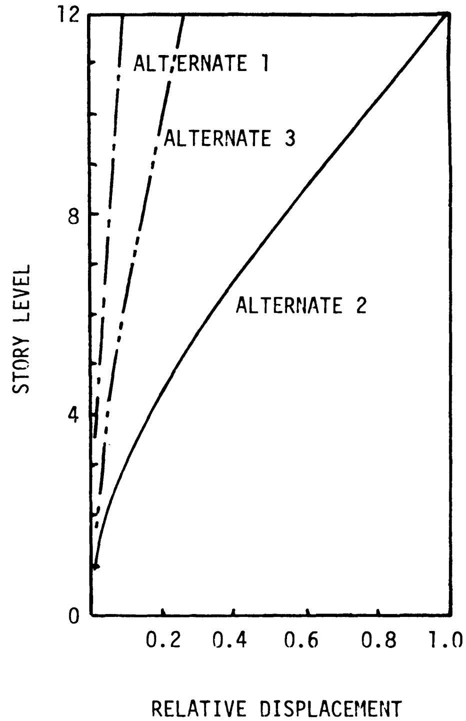

Fig. 4. Relative deflections of a 12-story structure, similar in plan to Fig. 1, under alternate connection conditions

different geometries and three different connection conditions. The connection conditions were varied between fully rigid beam-column and beam-wall connections, Alternate 1, and fully pinned connections, Alternate 2. Alternate 3 was an intermediate condition with rigid beam-column connections and flexible shear wall connections. The three alternate conditions are shown schematically in Fig. 3.

These analyses provided some simple but useful results. They showed that the use of rigid moment resisting connections significantly increased the lateral stiffness of the structure. The magnitude of the differences in stiffness and deflection varied with the geometry of the structure, but the lateral deflections were always largest with Alternate 2 connections and smallest with Alternate 1. These observations can also be verified by noting the results of other similar analytical studies.[5,6] Typical results are shown in Fig. 4 for a 12-story structure with coupled shear walls, similar in layout to Fig. 1. This figure shows the lateral deflections for each floor level of the three Alternates with all member sizes and the UBC-1976 seismic loads held constant.

This increased stiffness provides a strong incentive for using rigid connections, but analysis also indicates that rigid connections place greater strength demands on the connections. Rigid frame-wall connections must be designed for both large shear forces and moments. The moments are typically of the same order of magnitude as the full bending capacity of the connecting beam. Pinned connections need only to resist a smaller shear force with no moment, but they must be free to rotate. Rigid connections are subjected to larger shear forces, because the beams are in double curvature when the structure is loaded laterally.

Linear elastic analysis is useful in evaluating the stiffness of a structure. However, it is well known that seismic resistant construction also requires an understanding of the inelastic behavior and ductility of the structure. An examination of the inelastic behavior of the mixed structural system clearly indicates that this ductility requirement will place constraints upon the design of the frame-shear wall connection. Most seismic resistant building structures are designed with the expectation that the structure will not fail during a severe earthquake before the structure has experienced lateral deflections which are 6 to 8 times the maximum elastic deflection. If this requirement is applied to the mixed structural system, rigid shear wall connections will be required to sustain unusually large plastic rotations without losing their shear or moment capacity. Pinned connections must sustain similar rotations without losing their shear strength.

These analyses define the design requirements of connections between a stiff shear wall and a flexible steel frame. They show that rigid connections are desirable, since they produce a stiff structure. However, these connections must develop large moments and shear forces, and they must maintain this capacity while experiencing significant inelastic rotation, if the structure is to survive a severe earthquake. Pinned connections produce more flexible structures, but they can be designed for smaller shear forces and only minimal moments.

COMPARISON WITH PREVIOUS RESEARCH

The literature was then reviewed to determine if there is any evidence that mixed connections can actually satisfy these required strength, stiffness and ductility conditions. No record of previous analytical or experimental studies into the behavior of such steel frame-reinforced concrete shear wall connections was found. However, several related studies were identified.

A recent report by Hawkins *et al.*[7] investigated the shear and moment resisting behavior of metal stud connections. This study indicated that stud connections can resist large shear forces, but their moment resistance is severely limited by the tensile capacity of the studs. The study also showed that the failure was ductile when the shear forces were high and the bending moments were low, but the failure mechanism was likely to become more brittle as the beam bending moment increased. This study also developed a design procedure for predicting the shear and moment capacity of such stud connections. Application of that design procedure clearly indicated that stud connections cannot develop the moment capacity which is required for rigid moment-resisting (Alternate 1) connections. Further, that study showed that the required connection ductility cannot be developed in the studs.

Other studies[8,9,10] have examined the behavior of steel framing connections. Those connections have been designed as pinned and rigid connections, and both types of connections have been shown to be very ductile, if properly designed and constructed. Connections with bolted webs and unconnected flanges, such as that shown in Fig. 2, are commonly assumed to be pin connections (Alternate 2 or 3) during the design process. However, research[8,9] has shown that these bolted joints also develop significant moment due to friction and bearing on the bolts, and the shear and moment capacity may be much larger than predicted by accepted working stress design methods.[11] This moment capacity is not a serious problem in steel structures, because structural steels are ductile, but it is a potentially serious problem in composite connections. Since stud connections become brittle under high bending moments[7] and ductility within the connection is required for seismic resistant design, the stud connection must be conservatively designed and be capable of resisting the full moment capacity of the bolted connection.

Crawford and Kulak[9] provide equations for predicting the plastic strength of bolt groups. They assume that the moment capacity of the connections, M_u, is defined as the shear capacity, V_u, multiplied by the eccentricity, e_2, as shown in Fig. 5. Then,

$$V_u = KA_s \tag{1}$$

where A_s is the total shear area of a single bolt and K is a constant defined by the moment of inertia of the bolt group, I, and the eccentricity. For a single line of bolts,

$$K = \alpha I^{\beta} \tag{2}$$

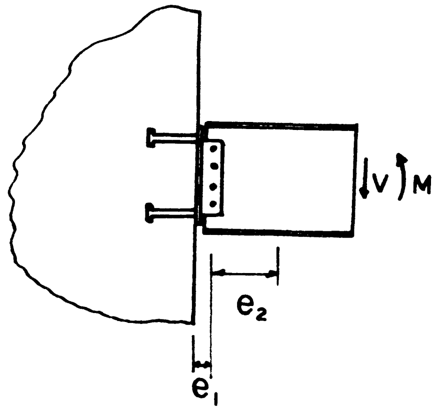

Fig. 5. Eccentricity in the connection

where

$$\alpha = \frac{2.36e - 123.0}{1 - 1.2e} \tag{3}$$

and

$$\beta = 0.296 + 0.0589e - 0.003475e^2 - 0.0000718e^3 \tag{4}$$

PROPOSED DESIGN PROCEDURE

The concepts, which were introduced by this previous research, were used to develop a design procedure for simple connections between a steel beam and a concrete wall. For this procedure, the shear wall and the members of the steel frame are first designed by the usual design methods. The bolts, erection plate, and welds between the erection plate and the embedded plate are designed as typical shear connections by the usual methods for Type 2 steel construction.[11] After the bolted connection is designed, the plastic moment and shear capacity of the bolts are determined. This is accomplished by determining the design plastic shear force, V_{DP}, for the bolt group and employing the method of Crawford and Kulak[9] to find the plastic moment capacity of the group. V_{DP} is found by multiplying the service load shear force by the appropriate load factors and selecting the largest magnitude of plastic shear which is produced by these factors. The shear area of the bolt is then inserted into Eq. (1) to find the required K-value, for the bolt group. K and the moment of inertia of the bolt group, I, are inserted into Eqs. (2), (3), and (4) to solve for the maximum eccentricity of the bolt group, e_2. Figure 6 is a graphical solution of these three equations, which can be used to simplify the solution for e_2. The eccentricity, e_2, is a fictitious eccentricity, which is used only to estimate the maximum moment capacity of the bolt group. It has no physical meaning. However, there is a real eccentricity, e_1, between the bolts and the studs as shown in Fig. 5. Thus, the design plastic moment for the stud connection, M_{DP}, becomes

$$M_{DP} = V_{DP} (e_1 + e_2) \tag{5}$$

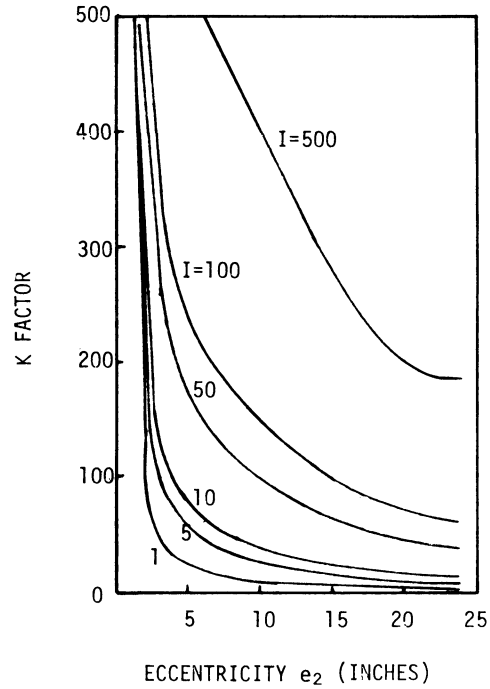

Fig. 6. Graphical solution of the equations of Crawford and Kulak

The moment, M_{DP} , is the minimum design moment for the stud connection. However, previous research[7] has shown that a brittle, cone pull-out failure of the tension studs is possible when the stud connection is loaded with combined moments and shear forces. Further, stud connections which were loaded under inelastic cyclic loading had a reduced capacity. To ensure that the stud connection does not fail and that the ductility is developed through inelastic actions in the bolted connections, it was concluded that the stud connection should be designed for a moment and shear of 1.5 times M_{DP} and V_{DP}. That increase reduces the possibility of a brittle stud connection failure, due to the yield stress of the steel or ultimate strength of the

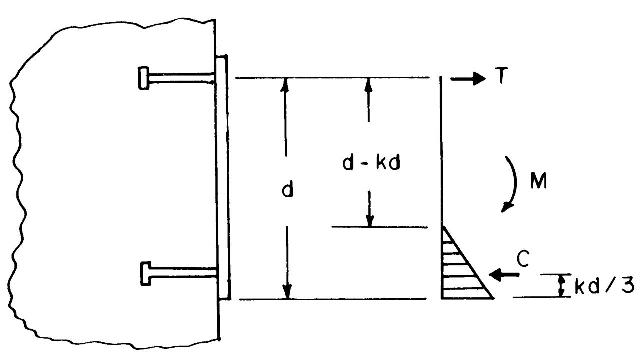

Fig. 7. Assumed stress distribution at the interface between steel plate and column

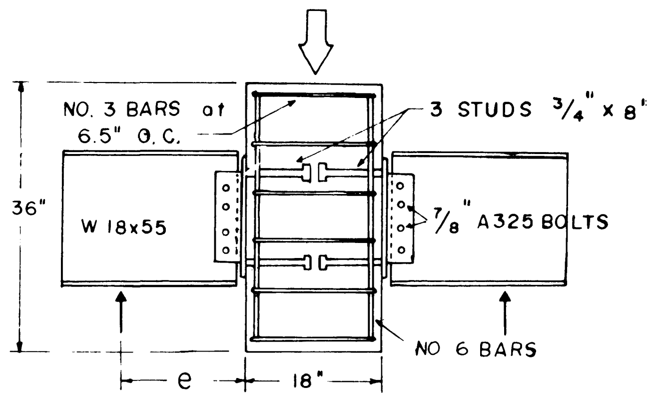

Fig. 8. Design of the test specimen

concrete being higher than the design strength, strain hardening of the steel, prying action, reversals of loading or other phenomenon that can increase the loading on the studs.

The studs are then designed for the increased moment by the methods proposed in Ref. 7. The tensile forces on the studs are computed from the applied moment using the model shown in Fig. 7. Initially, the shear is equally distributed among the studs in the compression zone of the connection. If the tensile studs reach their full tensile capacity before the compression studs reach their full shear capacity, then failure is assumed to occur when the tensile studs reach their full capacity. However, if the compression studs reach their full shear capacity first, then a plastic redistribution of the excess shear force is assumed. The excess shear force is distributed equally among the tensile studs until all tensile studs reach their full combined load capacity. The shear and tensile strength of the individual studs are computed by normal procedures.[7]

It should be noted that while Eqs. (2), (3), and (4) were developed for bolts in double shear, they are applied to bolts in single shear in this design procedure. That action is believed to be conservative. Bolts in single shear have additional stresses due to non-symmetric loading of the bolt and prying action, and thus this method is conservative, because it overestimates the design moment for the stud connection.

EXPERIMENTAL PROGRAM

The design procedure was verified by an experimental study. Six specimens of the type shown in Fig. 8 were tested. Each specimen was a symmetric, full-scale model of a typical frame-shear wall connection. The beam was a W18 x 55 of A36 steel, bolted to an erection plate with four 7/8-in. A325 bolts. The erection plate was welded to a 3/4 x 12 x 16-in. steel plate with a 3/8-in. fillet weld on both sides. This steel plate was anchored into the concrete with six 3/4 x 8-in. studs. The studs were designed by the proposed design procedure, except that the capacity of the studs was only 44% greater than the computed capacity of the bolts, rather than 50% recommended in the design procedure. The studs were provided and installed by the Seattle Office of the Nelson Division, TRW Corporation. All specimens were identical except Specimen 5. Specimen 5 had an additional weld between the beam web and the erection plate. The weld was installed to prevent any slippage in the bolted connection and to identify the effect of such a movement. Thus, that specimen in part simulated the behavior of a rigid (Alternate 1) shear wall-steel frame connection. In order that the bearing strength of the concrete would not affect the capacity of the stud connection, the stud plate was positioned outside the concrete, rather than recessed in the manner shown in Fig. 2.

The concrete column was designed to simulate a shear wall. The reinforcement had size and spacing typical of those likely in a seismic resistant wall. The concrete was designed to have a 7-day strength of 4000 psi. Concrete strengths at the time of test, taken as the average strength determined from tests on three 6 x 12-in. cylinders are noted in Table 1.

The load was applied with the 2400 kip Baldwin Testing Machine. The specimens were supported so that there was an eccentricity, e, between the support and the face of the concrete column as shown in Fig. 8. Eccentricities of 8.25, 12.75, 17.75, and 22.75 in., respectively, were used for Specimens 1 through 4. The variation produced a wide range of shear forces and moments, and thus provides a reasonable basis for a general check of the design procedure. Specimen 5 was also tested at the intermediate eccentricity of 12.75 in. but, as previously explained, that specimen had an additional weld between the beam and the erection plate. Comparison of the results for Specimens 2 and 5 provides a measure of the importance of bolt displacement to the connection behavior. Specimens 1 through 5 were loaded monotonically to failure.

A 12.75-in. eccentricity was used for Specimen 6 and that specimen was subjected to severe cyclic loading. The specimen was first loaded monotomically to 75% of the capacity of Specimen 2. Then it was unloaded, inverted on its supports and cyclic effects simulated by loading in the opposite direction to, again, 75% of the capacity of Specimen 2. Two complete cycles were applied in that manner and then, in the third cycle, the specimen was monotonically

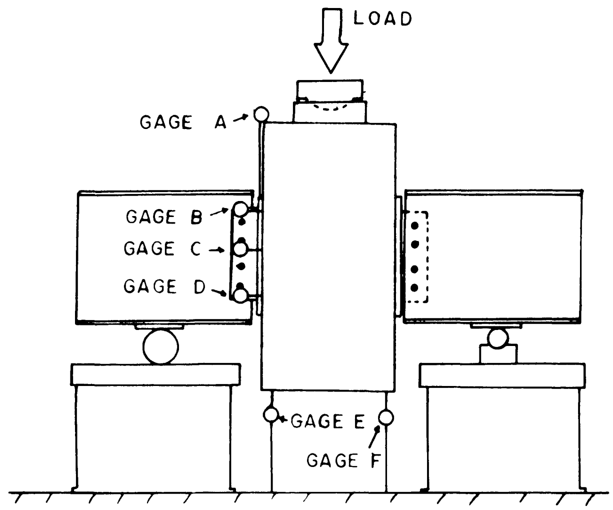

Fig. 9. Instrumentation and test set-up

loaded to failure. Comparison of the results for Specimens 2 and 6 provides an indication of the effects of seismic loading on the behavior of the connection. An actual earthquake would probably produce a larger number of cycles, but with smaller changes in rotation.

Dial gages were used in the test set-up as shown in Fig. 9. Gage A was used to measure slip between the steel plate and the concrete. Gages B and D measured the rotation in the joint. Gage C recorded any separation between the plate and the concrete. Gages E and F recorded the vertical displacement of the concrete column.

EXPERIMENTAL RESULTS

The more important experimental results are summarized in Table 1. Specimens 1 through 4 were all proportioned in accordance with the design procedures described previously. The behavior of all four specimens was similar. Shown in Fig. 10 is the typical shear-rotation curve for Specimen 2. The bolts initially were tightened to develop frictional resistance, and this connection was very stiff for shear forces up to approximately 13 kips. The frictional resistance was overcome at that shear, and the bolted joint rotated freely until the bolts began to bear firmly on the bolt holes at 1.7 degrees rotation. The connection then again

Table 1. Test Results

Spec. No.	Eccentricity e (in.)	Concrete strength f'_c (ksi)	Measured ult. capacity V_{um} (kips)	Predicted capacity of bolts V_{up} (kips)	ϕ	Failure mode
1	8.25	3.8	96.0	102	4.18°	Bolt
2	12.75	3.7	66.0	58	6.46°	Bolt
3	17.75	3.9	51.0	39	6.86°	Bolt
4	22.75	3.7	32.0	25	6.50°	Bolt
5	12.75	3.8	66.0	N/A	1.98°	Conc.
6	12.75	3.0	51.0	58	7.16°	Conc.

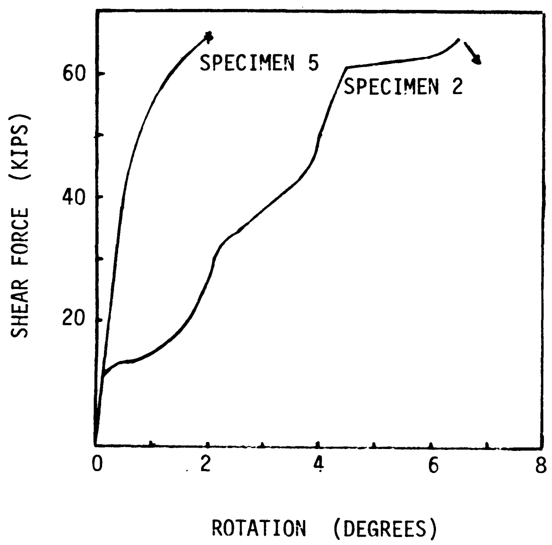

Fig. 10. Force-rotation curves for specimens 2 and 5

became very stiff, and the shear increased rapidly with only small increases in rotation up to a value of approximately 33 kips. At the 33 kip shear force, local yielding began in the metal surrounding the bolt holes. This yielding caused elongation of the bolt holes and another significant reduction in stiffness. With continued yielding, the joint rotation increased rapidly up to a value of approximately 4 degrees. At that rotation, the beam web came in contact with the top of the embedded plate as shown in Fig. 11. This contact

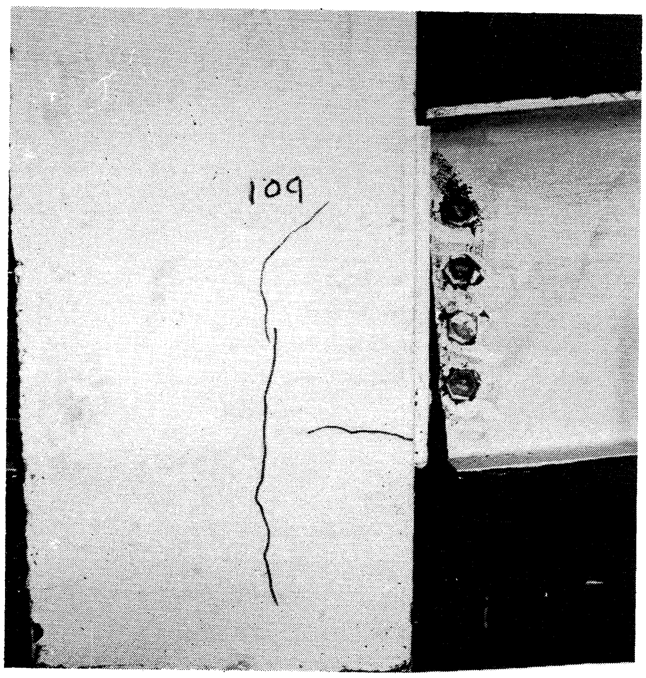

Fig. 11. Specimen 2 at ultimate load

caused a prying action, which internally redistributed the bolt forces and stiffened the connection. Then, the shear force increased sharply to a value of 60 kips. At the 60 kip shear, the connection was approaching its ultimate capacity, and it rotated freely with only small increases in shear. The bolts on the tension side tore through the web, as shown in Fig. 12. The shear capacity then decreased for increasing rotations and the test was terminated. The failure was not sudden or brittle. Additional rotational capacity at a reduced shear force was available. However, rotation and deflection measurements were terminated to prevent damage to the instrumentation.

Specimens 1, 3 and 4 exhibited behavior similar to Specimen 2. Each had 3 stiff zones and 3 flexible zones limited by bolt slippage, local yielding in the web and erection plate, and attainment of the ultimate capacity of the connection, respectively. The shear forces that defined the zones increased with decreasing eccentricity. All four specimens exhibited considerable rotational capacity. The minimum capacity was well in excess of 4.2 degrees which implies that the mixed structure could sustain at least a 7% interstory drift without loss of shear capacity in the connection. All four specimens displayed the same ductile failure mode.

The bolts in Specimens 2, 3, and 4 transmitted shear and moments which were well in excess of the capacity pre-

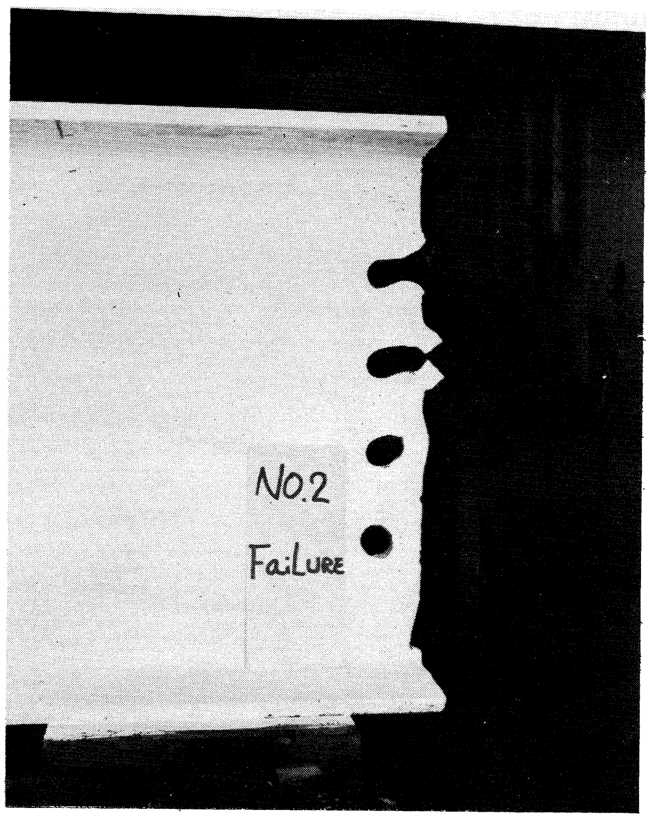

Fig. 12. Failure mode for specimen 2

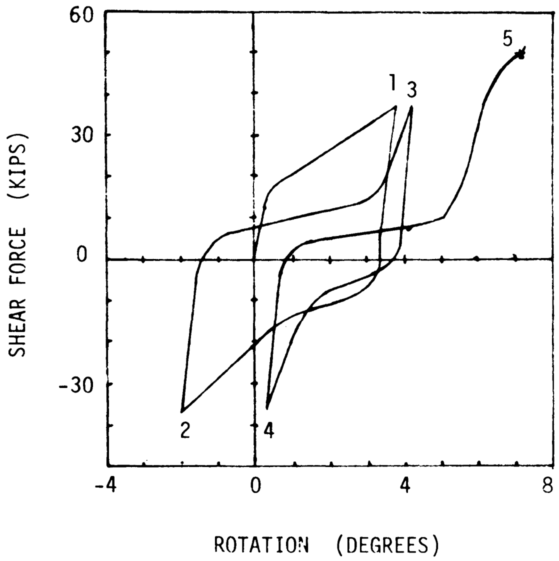

Fig. 13. Force-rotation hysteretic curves for specimen 6

dicted by Crawford.[9] This excess capacity was apparently caused by an internal redistribution of bolt forces that occurred when the web of the beam came in contact with the embedded plate as shown in Fig. 11. This excess capacity clearly indicates the need for the 50% increase that was applied to the theoretical stud forces in the design procedure.

Specimen 5 was identical to Specimen 2 except that it had an additional weld between the web and the erection plate. The weld prevented slippage of the bolted connection and caused a relatively brittle connection behavior. Shown in Fig. 10 is the shear force-joint rotation plot for Specimen 5. That curve does not show the three zones of behavior noted for Specimens 1 through 4, because the connection was restrained against bolt slippage. The connection retained its initial stiffness up to a shear of approximately 50 kips, where it experienced a minor loss in stiffness due to local yielding. The specimen failed at a shear of 66 kips and a rotation of 1.98 degrees. The failure was a brittle, concrete cone pull-out, failure of the tension studs. While the general ductility of Specimen 5 was much smaller than that of Specimen 2, the ultimate shear capacities were the same for both specimens. This result indicates that the 50% increase in the theoretical forces on the studs in the design procedure is not overly conservative. Comparison of the results for Specimens 2 and 5 also indicate the relatively brittle behavior that can be expected with a rigid (Alternate 1) frame-wall connection. That brittle behavior developed in spite of the embedded length for the studs being considerably greater than the length customarily recommended by stud manufacturers.

Specimen 6 was identical to Specimen 2, but it was tested under cyclic loading. The shear force-rotation curves for Specimen 6 are shown in Fig. 13. These hysteretic curves are severely pinched in the later cycles, because of the slippage of the bolts and the elongation of the bolt holes due to local yielding. The ultimate shear capacity of the connection was 51 kips, a value equal to only 77% of the capacity of Specimen 2. This result appears to indicate that the cyclic loading reduces the strength of the connection. However, it should be noted that Specimen 6 had a slightly lower 7-day concrete strength than Specimen 2, and that reduction may have contributed to the loss in strength. Additional tests are needed to properly define the effect of cyclic loading. The maximum rotation of 7.16 degrees, which was obtained with Specimen 6, was larger than the rotation of 6.46 degrees, which was recorded for Specimen 2. However, the failure of Specimen 6 was a brittle concrete cone pull-out failure, rather than the ductile failure observed for Specimens 1 through 4.

These experiments clearly indicate the effectiveness of the proposed design procedure. Specimens 1 through 4 were designed by the proposed procedures. Those specimens developed the required connection strength and also exhibited sufficient rotational capacity to assure ductile behavior during an earthquake. Specimen 5 was designed to inhibit rotation in the bolted joint, and it had a brittle failure with no increase in total strength. Thus, the design procedure has been shown to be an effective technique for seismic resistant construction.

SUMMARY AND CONCLUSIONS

1. Theoretically, the most desirable connection between a moment resistant steel frame and concrete shear wall is a rigid connection. With such connections the frame-shear wall system can be made stiffer and the members lighter than with flexible frame to shear wall connections. However, in practice those benefits are likely to be accompanied by a decrease in the rotational capacity of the connection and the increase in the likelihood of a brittle failure of the connection. Those disadvantages are an important consideration for structures located in high risk seismic zones.
2. Flexible frame-wall connections result in a more flexible structure and require larger member sizes than rigid connections. However, flexible connections can be designed so as to have reliable strengh and ductility characteristics. Reliable design characteristics can be achieved with the procedures developed and experimentally verified in this paper.
3. Even with flexible connections, inelastic cyclic loading can cause a significant reduction in the strength of the connection, pinched hysteresis curves that deteriorate under cyclic loading, and a change in the mode of failure of the connection at high rotations from ductile to brittle. Additional study is needed to fully assess cyclic loading effects.
4. With flexible connections, welding of the web connections prevents slippage of the bolts and can create a stiff,

brittle connection which might not survive a severe earthquake.

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