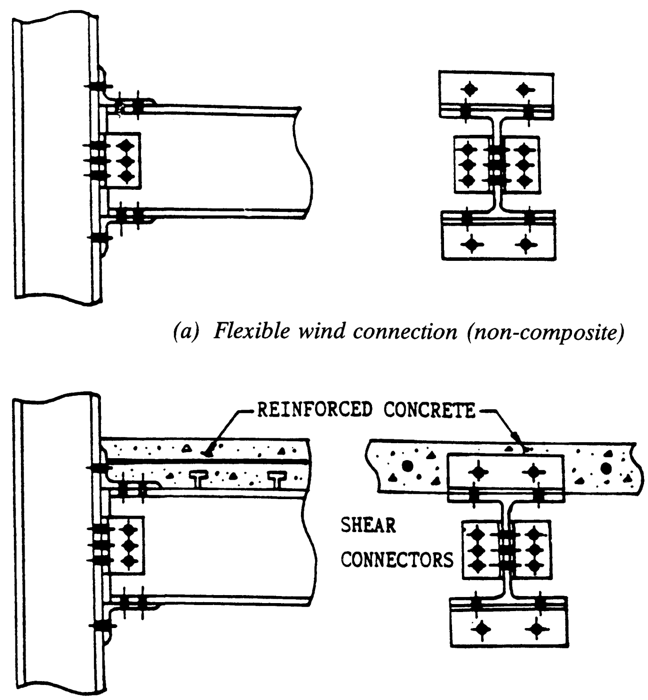
Semi-rigid Composite Steel Frames

ROBERTO T. LEON, DOUGLAS J. AMMERMAN, JIHSHYA LIN and ROBERT D. McCAULEY

Semi-rigid, composite steel frames are a new type of structural system being developed to better utilize the composite floor slabs and flexible connections present in common AISC Type 2 frames. This new structural system extends the beneficial aspects of composite action to the negative moment region of continuous beams by providing slab reinforcement across column lines. The resultant system offers significant gains in stiffness and strength not only in the members themselves but also at the connections. Because the connections benefiting most from this action are the relatively flexible ones (not rigid), the name "semi-rigid composite frames" (SRCF) has been coined.

The economies in materials and erection cost associated with composite and mixed construction are widely recognized.[1] Composite construction generally results in (1) reductions of steel area needed to support a given load, (2) an increase of overload capacity over non-composite sections, (3) reductions of construction depths and (4) an increase on the safety of the system by providing redundant load paths. Typically, composite action is used to increase the capacity of beams designed as simple spans, but its effect on the overall frame strength and stability is ignored.

Most of the connections used in simple framing have a small but finite stiffness and moment capacity. This is the rationale for the so-called wind connections currently under Type 2 in the AISC Specifications.[2,3] Since a large number of different connections fall in the semi-rigid category and it would be prohibitively expensive to investigate them all, the top and seat angle connection was selected for this study. A flexible connection (Type 2) and its corresponding semi-rigid composite connection (Type 3) are shown in Fig. 1. The stiffness and strength of the Type 2 connection can be increased greatly by replacing the top angle with Gr. 60 slab steel, so their behavior can be categorized as rigid rather than flexible. The increase requires only a moderate amount of steel across column lines and represents no new construction costs.



(a) Flexible wind connection (non-composite)

(b) Semi-rigid composite connection

Fig. 1. Composite vs. non-composite connection

This paper gives a brief summary of the experimental and analytical work carried out under the sponsorship of AISC on SRCF. It will serve as the basis for a design methodology currently under development and based on LRFD principles. The moment-rotation curves developed for the semi-rigid composite connections can also be used to analyze and design Type 2 frames as proposed by Ackroyd.[4,5] The main emphasis will be on the results of a one-story two-bay, full-scale frame tested under a combination of gravity and lateral loads. The tests demonstrated the inherent redundancy of the system and the large improvements in

Roberto T. Leon is an Assistant Professor, University of Minnesota.
Douglas J. Ammerman is a Research Assistant, University of Minnesota.
Jihshya Lin is a Research Assistant, University of Minnesota.
Robert D. McCauley is a Research Assistant, University of Minnesota.

strength, ductility and stiffness characteristics of semi-rigid composite frames. These characteristics make it a competitive system for buildings in the five- to ten-story range, both in braced and unbraced construction.

Semi-Rigid Connections and Composite Action

The potential savings from the use of semi-rigid connections are well known.[6,7] The rationale for extending semi-rigid action to composite construction becomes clear once some limitations of current analysis and design are understood. These limitations are associated with the nonlinear moment-rotation curves characteristic of Type 3 connections, with the joint panel flexibility due to shear, and with the issue of partial vs. fully composite action.

Semi-rigid connections (Type 3) generally are not considered a viable alternative in design of frames because of the lack of both a complete design methodology and data on their dynamic response characteristics. The use of Type 3 connections requires the designer to know their moment-rotation characteristics accurately.[3] Two alternatives are currently possible. The first is based on the use of empirical curves derived from statistical analysis of the few available tests.[8,9,10] The second alternative is to actually test some of the connections and use the data obtained in the laboratory in the design process. Both of these alternatives are expensive and outside the capabilities of most design firms. The testing approach has been used in cases where a more accurate estimate of the strength and stiffness were required for evaluation of a structure, but seldom as a design tool.[11]

Even if the moment-rotation curve is known, the ultimate strength analysis of a structure incorporating nonlinear springs is not simple. The nonlinear character of the moment-rotation curves for semi-rigid connections can be improved by the addition of a floor slab. From Fig. 1a, it is clear the cyclic behavior of these connections will depend on the behavior of the angles or other elements in tension. The problem is that angles bolted together to make up connections do not yield in tension, but in bending near the corner, often at very low loads (Fig. 19). Moreover, as the displacement increases, the bolts tend to slip unless the proper torque is applied and surface preparation done. The alternative is to field-weld the connections, but this is difficult and expensive.

Many of the disadvantages can be circumvented by adding a composite floor slab, with the slab reinforced with bars continuous over the column lines. Under gravity loading most of the tension force will be carried by the slab reinforcement. Since the moment arm is slightly increased and the rebar yields in tension at a stress higher than typical structural steel angles, the connection gains significant initial stiffness and strength. Under lateral or cyclic loads the connection is still stronger and stiffer than one without a slab. And its behavior can be easily improved by using a much heavier bottom angle.

Another concern raised by recent studies is the joint flexibility problem. If large moments are going to be transferred to the columns, the shear stresses in the column web (joint panel zone) must be investigated carefully. Most analytical work so far has assumed the column flanges are infinitely stiff and no deformation of the joint panel zone can occur. In fact, many tests have been carried out on semi-rigid connections to stiffened column stubs. However, the column panel zone can undergo significant deformations, as shown by recent research,[12,13] and for efficient use of semi-rigid composite construction a large range of beam-to-column stiffnesses would need to be studied.

The issue of partial versus fully composite action has not been investigated for this type of connection. All the tests described in this paper had enough shear studs to insure fully composite action. It is conceivable partially composite action can lead to further optimization of the system.

Experimental Studies

The idea of using the composite action of a slab along with a flexible connection was originally proposed by Barnard.[14] The effect of a composite slab on a semi-rigid connection has been studied experimentally by Van Dalen and Godoy in Canada[15] and by Echeta and Owens in England.[16] Both series indicated the large strength and stiffness that can be gained by adding the composite floor slab. Unfortunately, the Canadian tests were carried out on small specimens without web angles, while the English tests had only a small web clip, primarily for erection purposes. Consequently, it is difficult to extrapolate the results to details common in U.S. construction. Both of these test series, however, clearly showed the advantage of a composite system.

Over the past three years, a pilot study sponsored by AISC at the University of Minnesota resulted in testing four full-scale SRCF specimens. The first two were based on modifying non-composite connections that had been extensively studied by Radziminski and Azizinimini.[17] The first test was conducted under a cyclic lateral loading (SRCC1C). The first test had one connection incorporating both the top and bottom angle plus the slab steel and web angles (SRCC1MR), and the other connection without the top angle (SRCC1ML). The results of these tests are in Fig. 2 and more detailed descriptions have been reported previously.[18] Figure 2 shows a comparison of the moment-rotation curves for the bare steel connection (RADZIMI), the composite connection under monotonic loads (SRCC1MR and SSRC1ML) and the envelopes for the cyclic load test (SRCC1CR and SRCC1CL), where the last letter refers to the right (R) or left (L) beam. The appreciable gains in stiffness and strength from the bare to the composite connection are obvious, as well as the ability of the connections to withstand severe cyclic loads. The third test was a full-scale, two-bay one-story frame incorporating the same connections as the first two tests (SRCF2C). And the fourth test incorporated a much thicker and wider angle for the seat connection (SRCC3C). This paper describes the last two tests.

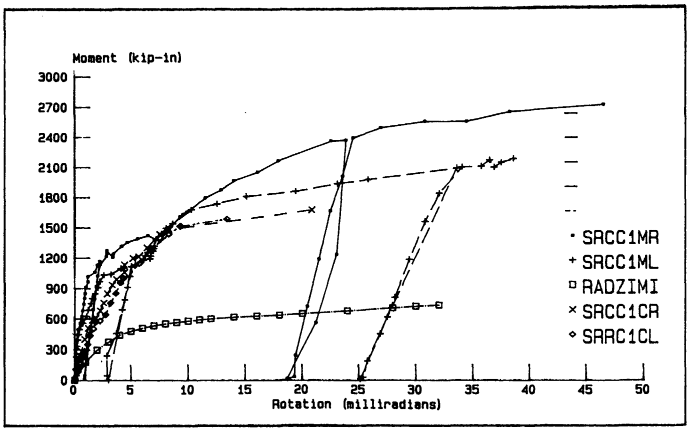


Fig. 2. Comparison of negative moment capacities

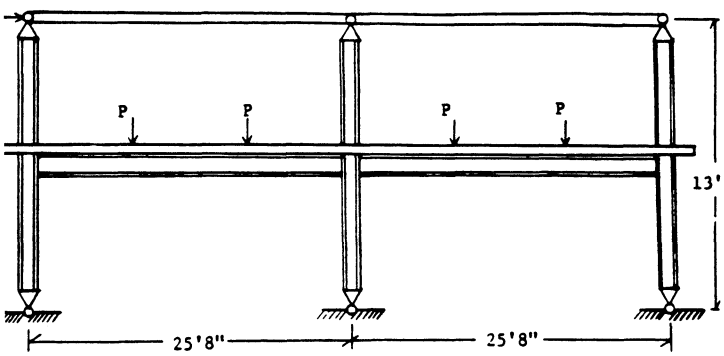


Fig. 3. Overall dimensions and locations of loads for SRCF2C

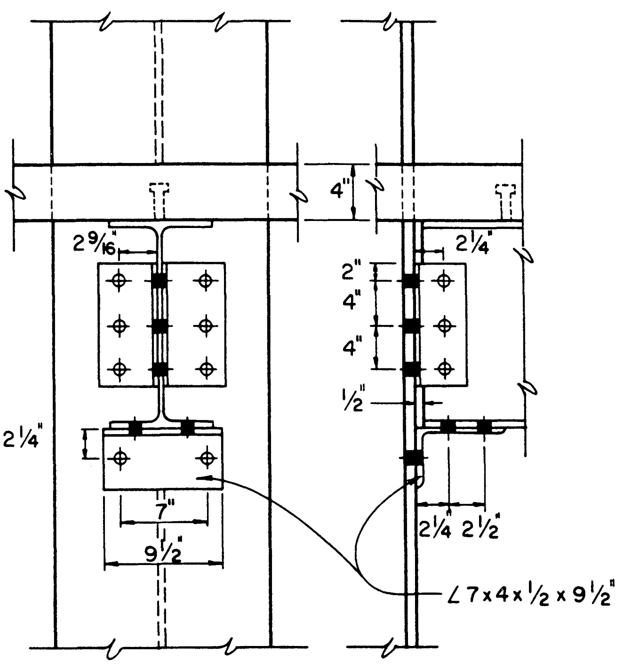


Fig. 4. Details of connection for SRCC3C. Details for SRCF2C are similar, except seat angle is 7 × 4 × 3/8 × 8"

Test SRCF2C—Frame

The specimen for the full-scale test was a two-bay frame with bay lengths of 25 ft-8 in. and story height of 13 ft (Fig. 3). The columns were W14 × 120 pinned at top and bottom to model inflection points at mid-story height and the beams were W14 × 38 with a 3-in. lightweight concrete composite slab on 2-in. formed metal deck. The entire setup is shown in Fig. 18. Complete composite action was provided by a pair of headed-stud shear connectors placed in each rib of the metal deck (12-in. spacing along the beam).

The beam-column connection consisted of L7 × 4 × 3/8 seat angle 8-in. wide, and 2L4 × 4 × 1/4 double-web angles 11-in. long. These angles were bolted with 1-in. A325 bolts tightened to AISC specifications by turn-of-the-nut method (Fig. 4). There were eight No. 4 reinforcing bars with a yield point of 63 ksi continuous across the column. For the external column connections the composite slab extended 2 ft beyond the centerline of the column in order to provide anchorage for the slab reinforcement. This external portion of the slab was reinforced with one No. 4 transverse bar for one external connection and with three No. 4 bars for the other external connection.

The gravity load on the specimen was applied as two point loads symmetric about the center of the span in each bay, to simulate the action of floor beams framing into the girder at these points. This load was applied slowly up to a level of 16 kips at each load point and then held constant for the lateral load portion of the test. The lateral load was applied, with stroke control, through a rigid strut attached to the top of all three columns, so each column would have the same amount of story drift.

Reactions of the specimen were measured using shear-beam-type load cells for the pin connections at the bottom of each column. This, coupled with the load cells at each actuator, allowed for the independent checks of moments at each connection. The rotation at each connection was measured using a pair of LVDTs rigidly attached to the beam 12 in. away from the face of the column flange, one below the bottom flange of the beam and one above the slab. For all connections, the other end of these LVDTs was attached to the flange of the column, and an additional pair of LVDTs was used at one of the interior connections measuring rotations relative to the centerline of the column web. In addition, LVDTs were used to monitor the slip of the angles with respect to the beam flange (Fig. 5). Strain gages also were used to locate the neutral axis of the composite beam and to determine the stress distribution in the slab reinforcing.

The specimen was subjected to a combination of gravity and lateral loads, as shown in Fig. 4. The gravity loads were chosen to simulate a total floor load of about 70 psf if 25 ft-8 in. × 25 ft-8 in. bays are assumed. It was felt this represented a reasonable service load level. The lateral loads were applied to the top of the structure and were deflection controlled. The structure was cycled at interstory drifts of

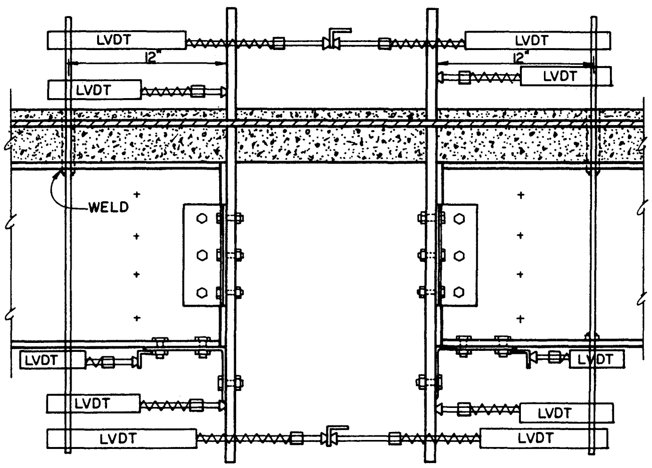


Fig. 5. Instrumentation to measure rotation to face and center of column, and slip of angles.

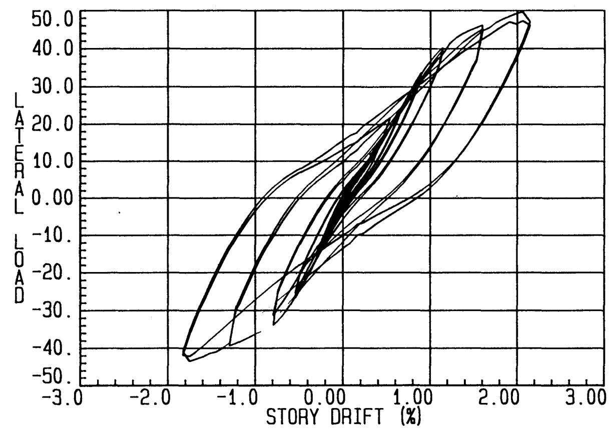


Fig. 6. Lateral load vs. interstory drift for SRCF2C

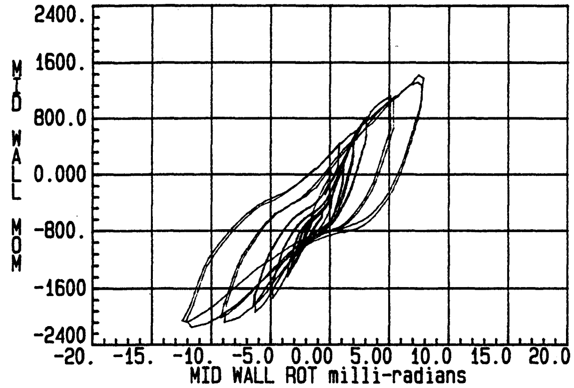


Fig. 7. Moment vs. rotation curve for interior connection for SRCF2C

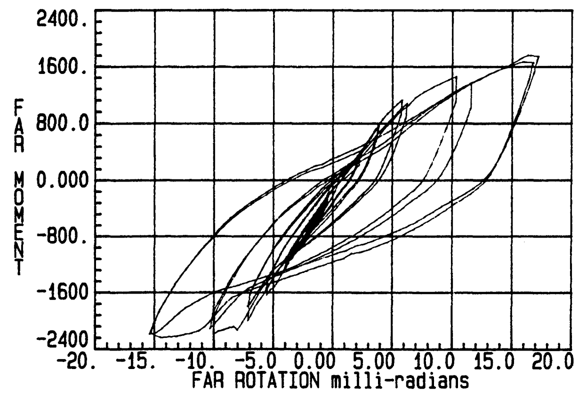


Fig. 8. Moment vs. rotation curve for exterior connection for SRCF2C

0.1, 0.25, 0.5, 0.75, 1.00, 1.50 and 2.00 percent. This was not intended to model any particular wind or seismic load; the intent was to extract information on the system's behavior at convenient, or design drifts.

During the gravity loading of the specimen the semi-rigid composite connections behaved as linear springs, with stiffness nearly equal to that of rigid connection. At the end of this loading there were small cracks in the slab at the column flange, and about 12 in. away from the face of the column. At this point, the connection was carrying the full gravity service load and was still in the initial, high stiffness region of the moment rotation curve. The connection stiffness was about 2.07×10^6 k-in./rad at a rotation of about 0.3 mRad.

For levels of load corresponding to service conditions (0.1% and 0.25% drifts), the connections behaved essentially linearly, with little or no hysteresis. Figure 6 shows the total lateral load applied to top of the specimen vs. the interstory drift. Cracking of the slab continued with the formation of small transverse cracks in the negative moment regions at the supports. Starting with cycling at 0.75% drift the hysteresis loops began to grow, mostly due to cracking of the slab and yielding of the seat angles. The toe of the angles in tension separated about $\frac{1}{32}$ in. from the column flange at this stage. At this level, the first nonlinearities in the moment-rotation curves were noticed. Figure 7 shows the moment-rotation curve for one of the interior connections and Fig. 8 that for an exterior one.

Beginning with cycles at 1% drift, the hysteresis loops began to grow, but no appreciable deterioration in strength or stiffness was noted up to 2% drift. This behavior was achieved even though the exterior connection with one transverse bar in the overhang failed at a drift of 1.5%. The failure occurred by propagation of very large shear cracks from the tip of the far column flanges to the end of the slab. While the size of the cracks exceeded $\frac{1}{8}$ in. and the connection unloaded slightly, with increasing drifts the connection regained and surpassed its previous maximum load. New cracks indicated most of the load was transferred to the column by bearing on the near flange. Thus, while there was visual evidence of failure, the structure was able to readjust its load-carrying mechanism.

Increasing the drifts to 3% and 3.5% resulted in more slab cracking and yielding of the reinforcing bars and seat angles. No slippage of the angles was noted throughout the test. Even at this stage no significant loss of strength was noted with cycling. And the test was stopped at 3.5% drift because of the failure of the tension angles in low-cycle fatigue. At this level the seat angles had very large cracks at the bolt lines on the column. It is estimated the angles had been subjected to at least 12 cycles of yielding and at least six of alternating plasticity. A cumulative ductility of about 38 was achieved, proving the excellent ductility and energy absorption capacity for this system.

While the structure had a permanent deformation close to 2% when unloaded, it was decided to attempt repairing it by replacing the cracked seat angles. New angles with slotted holes were fitted and the testing resumed. The load-deformation curves were very similar to those of the original structure; better performance was not to be expected because no effort was made at repairing the very large cracks in the slab.

All these test runs were done in a quasi-static fashion, with rate of loading varying between 0.01 in./sec. at the beginning of the test to about 0.05 in./sec. at the end. The repaired structure was tested at the 1% drift level at a rate of approximately 0.3 in./sec. to see if any behavior difference could be detected in the low part of the dynamic range. No such differences were found.

The most important finding in this test, from a design standpoint, was the yielding of the column panel zone. The shear-panel deformations were measured by using a crossing frame attached to three corners of the panel zone and measuring the change in angle between these three points with an LVDT. An idea of the shear strain in this region could also be obtained by a strain gage rosette placed at the center of the panel zone. Figure 9 shows the instrumentation used to measure the shear strain and Fig. 10 the horizontal shear vs. the shear strain measured for the panel zone of the interior connection. Yielding began at drifts of 1% and occurred primarily in the interior connection. One exterior connection did not show any yielding, while the other had two small yield lines. Figure 20 shows the yielding in the panel zone of the interior column at a drift of 2.5%. The columns met or exceeded all pertinent AISC recommendations for rigid connections, with a width-to-thickness ratio of 24.5 for the web. Since no data was available for semi-rigid composite connections, the recommendations for steel fully welded moment connections were assumed to be a conservative design approach.

The discrepancy can be explained by studying Fig. 11. The mechanism envisioned for the welded moment connection is based on the force distribution shown in Fig. 11a, while the force distribution for the semi-rigid composite connection after yielding is shown in Fig. 11b. The main differences stem from the absence of the top tension force and the much larger area over which the top compression force is applied. Further research in this area is in progress,

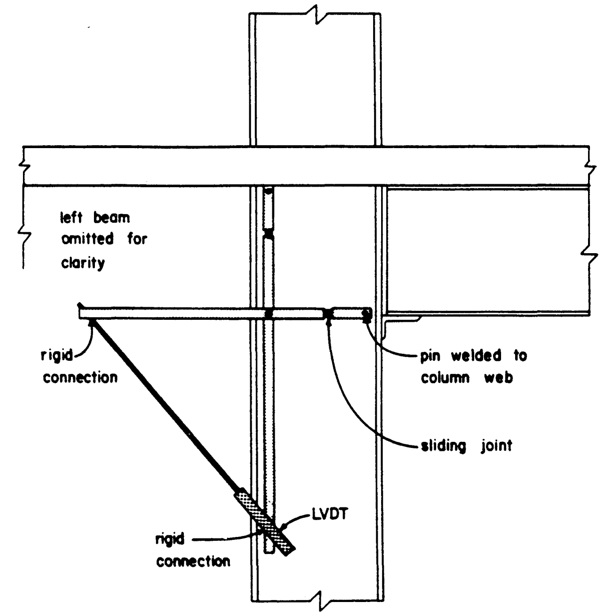


Fig. 9. Frame used to measure joint shear strains

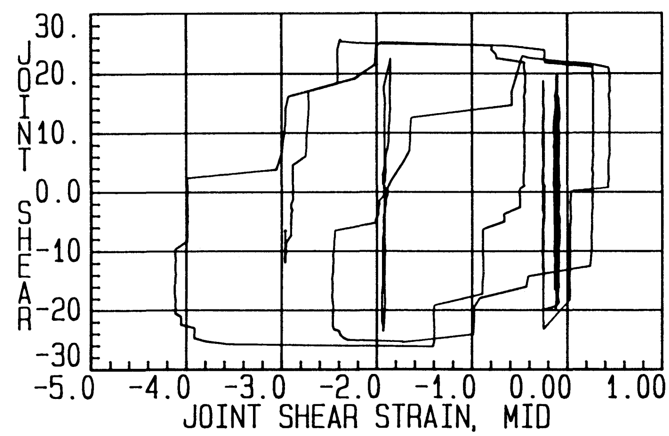


Fig. 10. Measured joint shear (kips) vs. joint shear strain, SRCF2C

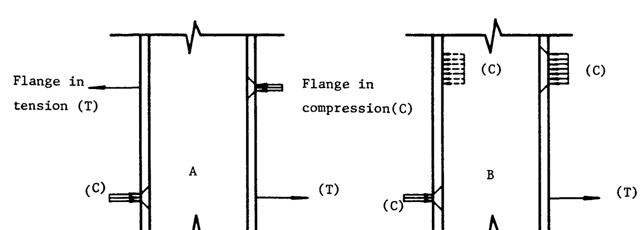


Fig. 11. Transfer of forces in a moment connection and in semi-rigid composite connection after yield of reinforcing bars

as similar results had been observed in the tests on isolated connections conducted previously.[18]

Test SRCC3C—Interior Connection

The specimen for the fourth test was similar to the interior connections tested before (Fig. 4), except the seat angle was a $L7 \times 4 \times ½$, 9.5-in. wide, the bolts were A325 ¾ in. and gage lengths were adjusted accordingly. The instrumentation and loading were very similar to those used in the second test and the slab was a solid rectangular section 60-in. wide and 4-in. thick. This test is comparable directly to SRCC1C, except for the size of the seat angle and slight difference in the gages to the bolts.

The set-up for test SRCC3C is shown in Fig. 12. The load-deflection diagram is shown in Fig. 13 and a typical moment-rotation curve in Fig. 14. Compared to previous tests with thinner angles (½ in. for SRCC3C and ⅜ in. for SRCC1C), SRCC3C shows an increase of positive moment capacity at ultimate of about 23% in the west beam and about 24% in the east beam; the negative moment capacities were almost equal since the slab reinforcement was the same. The stiffnesses were comparable, with initial uncracked values of 2.40×10^6 kip-in./rad and 3.72×10^5 kip-in./rad at 4 milliradians.

The behavior was elastic up to a 0.75% drift, with cracking of the slab beginning the column face during the 0.25% drift cycles. This cracking was minor and did not lead to any measurable hysteresis. At 0.75% drift, the negative moment was 1,277 kip-in. at 3.72 milliradians. The positive moments at this drift were 1,034 kip-in. with a rotation of 3.60 milliradians for SRCC3C compared with 923 kip-in. and 4.02 milliradians for SRCC1C.

At the 1.0% story drift, cracks in the slab began to open, signalling the first yield of the slab steel. Because the steel closest to the column will carry the most load, the steel strains are distributed parabolically with tensile strains close to yield at the middle bars, but only half of that value on the outside bars. This strain distribution results in a gradual rather than abrupt softening of the load-deflection curves. Yielding of all the reinforcing bars did not occur until 2.0% drifts were reached, signalling the attainment of the ultimate strength of the connection. At ultimate, the west connection carried 1,878 kip-in. in the negative direction and 1,718 kip-in. in the positive direction; the east beam carried 1,702 kip-in. and 1,577 kip-in. respectively.

There was no observable slip of the tension angles up to a drift of 1%; some minor slip occurred at that level for the east connection, while both east and west connections slipped significantly at the 2% level. The maximum slip measured was 0.09 in. at a drift of 3.5%. Once again, extensive shear yielding of the web of the column was observed beginning at drifts of 1.0% and increased steadily throughout the test. Figure 15 shows the joint shear stress vs. shear strain. A comparison with Fig. 10 indicates that much more severe shear problems were present in SRCC3C. Clearly there is a limit to the effectiveness of

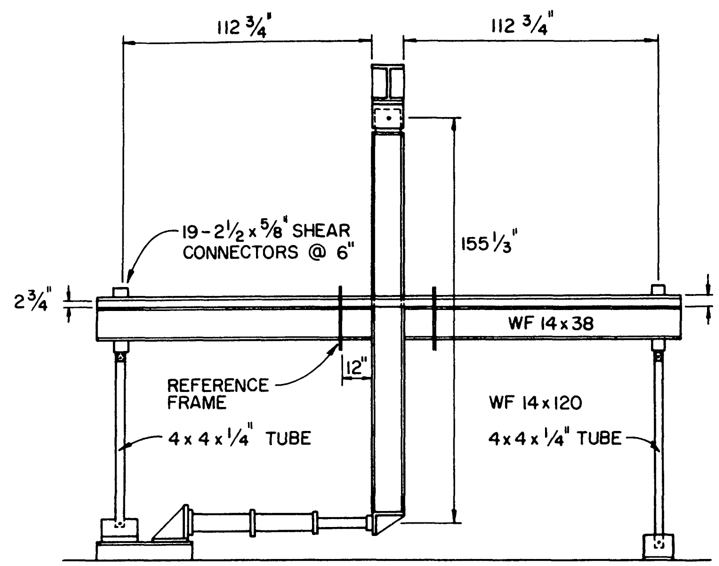


Fig. 12. Setup and dimensions for SRCC3C

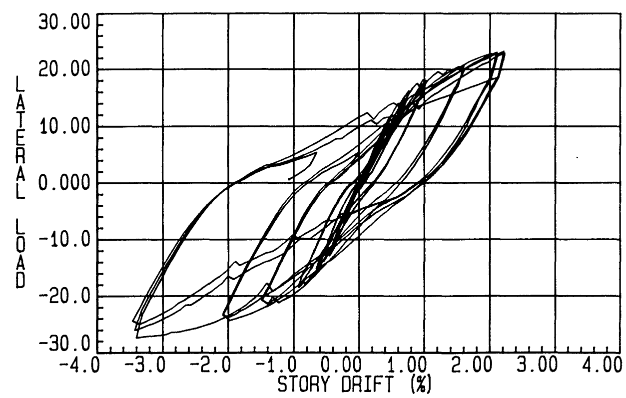


Fig. 13. Lateral load vs. interstory drift for SRCC3C

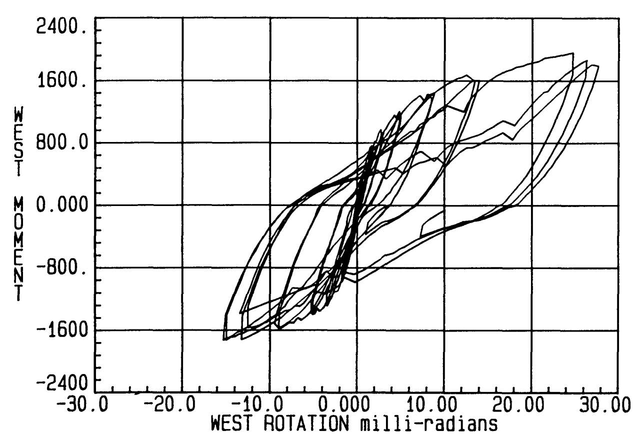


Fig. 14. Moment vs. rotation curve for west beam of SRCC3C

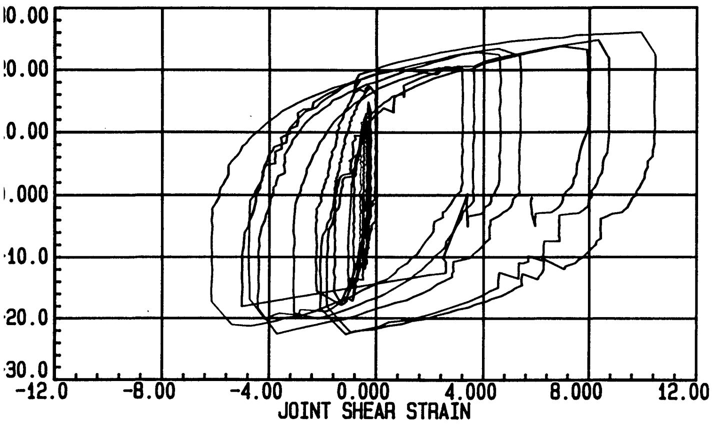
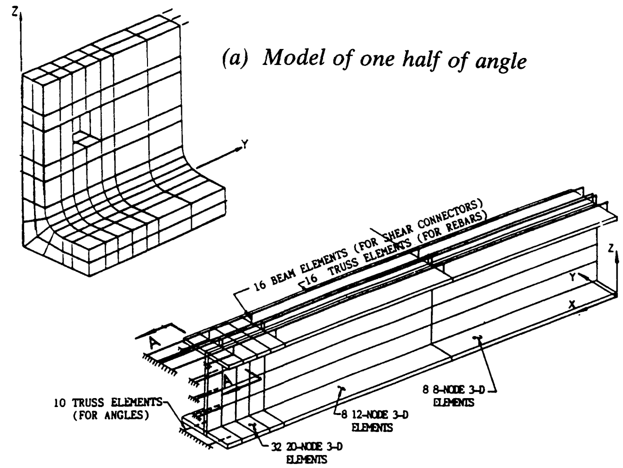


Fig. 15. Measured joint shear (kips) vs. joint shear strain, SRCC3C



(a) Model of one half of angle

(b) Model of connection

Fig. 16. Finite element models used in analysis

increasing the angle thickness, since the larger the force that the angle transmits to the joint panel, the larger the shear strains in the web and the more damage to the column.

Analytical Studies

In conjunction with the experimental work, an analytical effort is underway to develop a comprehensive model for semi-rigid composite connections. The model developed so far, which will be described next, can only work for monotonically increasing loads. Extension of the model to reverse cyclic loading, including stiffness and strength deterioration is underway.

The development of a model for semi-rigid composite connections requires:

1. The distribution of stresses in both the beam and the slab be properly modelled. As pointed out before, the distribution of stresses is neither uniform across the slab nor linear in the beam near the connecting angles.
2. Effect of bolt tension on the slip of the connections. The tensile load in the bolts is uncertain and difficult to measure, and has a significant impact in the slip of the connection.
3. Effect of slip in angles and beams due to the tolerances of bolt holes. The coefficient of static friction between steel varies due to the different smoothness of steels and the geometric shapes of connective elements. This friction force determines the initial amount of slip and the type of slip (a sudden or gradual slip) that will occur.
4. Effectiveness of shear connections. The composite action of the beam and the slab depends on the performance of shear connectors. The local behavior of shear connectors is a function of shape, size, arrangement and location of connectors and the type and history of loadings.
5. Problems associated with the yielding of the reinforcing bars, growth of cracks in the concrete slab and possibility of local bond failures can be accounted for.
6. The contribution of the web angles, included to carry the shear force from the beam to the column, must be included in the calculations for ultimate strength capacity and rotational ductility of the connection.

The analytical study was divided in two stages. The first stage was intended to provide a simple model to compare the test results of a semi-rigid connection with and without composite slab. In the second stage, the model was calibrated using existing experimental data and was used to conduct parametric studies. The model development was carried out using the finite element code ADINA, employing three-dimensional elements and a refined mesh. The model is essentially a two-step procedure, with an analysis of the connection angle under study carried out first, and its load-deformation curve then used to generate a stress-strain diagram for a truss element (Fig. 16). The second step involves replacing all the angles in the connection with equivalent truss elements, and then calculating the moment-rotation characteristics of the entire connection. This procedure has yielded a very good agreement with test data, as shown in Fig. 17.

Some Notes on Design

The original design for these connections was based on replacing the maximum force permissible in the top angle with an equivalent amount of reinforcing bars. Thus, the area of the top angle (L7 × 4 × 3/8) which was 8-in. wide and

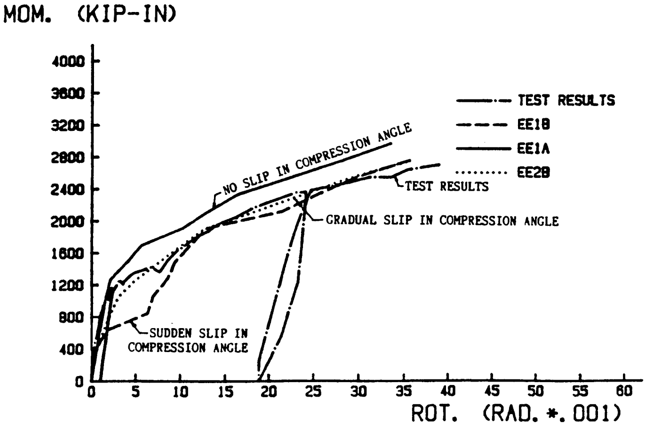


Fig. 17. Results of model analysis with and without slip of compression angle

made of A36 steel (total force = 8 in. × 3/8 in. × 36 ksi = 108 kips) should be replaced by an equivalent area of steel in the slab. Using Gr. 60 rebars, the required area is 1.60 in.^{2} or eight No. 4 bars (0.20 in.^{2} each). This slab steel was distributed over a column strip 5-ft wide since, in the service range, the effectiveness of the slab steel decreases as its distance from the column increases. At ultimate, of course, all the slab steel would contribute, but excessive cracking and deformations are required to reach that stage. Therefore, the slab steel was kept close to the column, with a criteria of effective width on each side of the beam equal to eight times the slab thickness being arbitrarily selected.

A limitation of the test data generated that must be recalled is the depth of the beam. A W14 section was used, while a W16 to W24 may be considered more typical. It is expected that, if current allowable stress design guidelines for compactness are followed, the extension of this work to deeper members should be quite straightforward. Note the increases in strength should be linear with increasing depth and the stiffness should be proportional to the square of the increase in depth.

An evaluation of the experimental data indicates a LRFD approach would be most suited to simplify the design of semi-rigid composite frames. The initial analysis for loads could be done by assuming rigid connections, or more accurately by modifying existing computer programs to account for the flexibility of the joint. The latter is a relatively simple task requiring the coefficients of the stiffness matrix be modified, for example by changing the coefficients from $4EI/L$ to $3.8EI/L$ for a connection with 90% of full rigidity. The moments and shears would then be modified by the usual resistance factors, and the beams and column proportioned accordingly. The connections would then be designed for the required moment capacity as per the assumptions discussed above. Alternatively, the moment-rotation curves developed in this research project can be incorporated into the ASD Type 2[4] or LRFD Type PR[5] design recently proposed by Ackroyd.

Conclusions

This experimental investigation, coupled with the ones previously conducted by the authors,[18] indicates composite semi-rigid frames offer very large gains in strength and stiffness over "bare" steel connections. For the service load range, these connections offer rigidities similar to those of rigid frames; while for the ultimate state they provide excellent ductility and energy-dissipation capacity. For the stability limit state, the continuous composite action over the column lines provides significant additional stiffness resulting in decreased drifts and associated P-δ effects. Moreover, semi-rigid composite frames provide a large degree of redundancy and have excellent force redistribution characteristics leading to increased safety. The construction of SRCF requires only small changes from current practice.

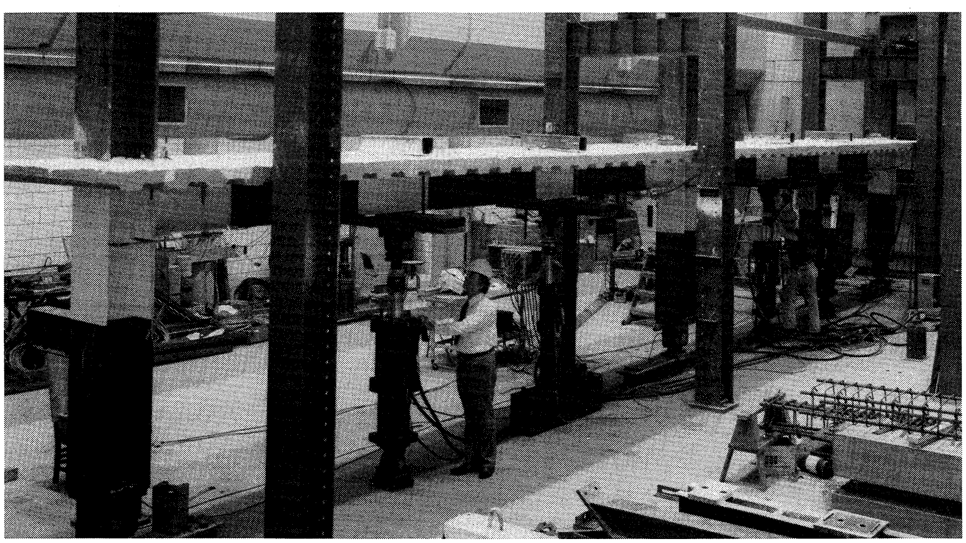


Fig. 18. View of test set-up for SRCF2C

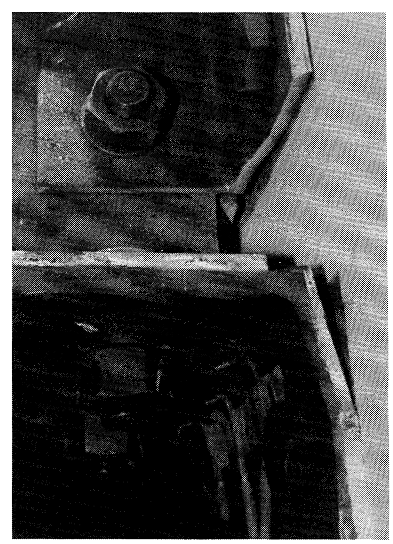


Fig. 19. Pullout of angles in tension

The cost and labor for the additional reinforcing bars result in a very economical method of increasing strength and stiffness. Overall, semi-rigid composite frames represent a very economical and structurally efficient solution to the design of low-rise frames.

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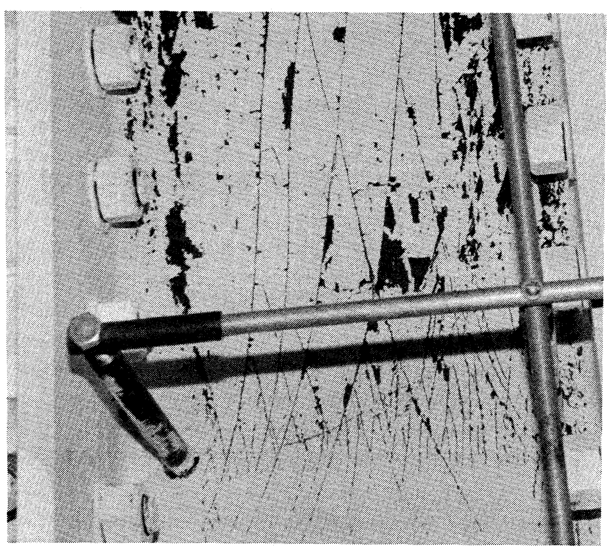

Fig. 20. Shear yielding of interior column web for SRCF2C

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