Behavior of Semi-rigid Composite Connections

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The 1978 AISC Specification[7] allows three types of framing. Type 1 is for fully rigid moment connections, Type 2 is for pinned connections (although the moment capacity inherent in these connections is recognized and assumed to act in resistance of wind loads), and Type 3 is for semi-rigid connections. Semi-rigid frames offer significant savings in materials for a combination of the following reasons:

a. By varying the beam end-fixity a more balanced distribution of positive and negative moments can be achieved in continuous structures, resulting in a more efficient use of the material.
b. The effective length of columns is decreased over typical Type 2 framing due to the end restraint provided by the connection.
c. The additional stiffness and strength provided by semi-rigid connections in combination with composite action will result in decreased drifts and reduced non-structural damage under lateral loads when compared to Type 2 construction.

Many of the common connection details in use today could be considered semi-rigid, and the economies outlined above could be achieved with little change from current design and construction practices. However, use of Type 3 connections under present specifications requires the designer to know their moment-rotation characteristics accurately. Two alternatives are currently possible. The first is based on the use of empirical curves derived from statistical analysis of the few available tests. The second alternative is to actually test some of the connections, and utilize the data obtained in the laboratory in the design process. The first alternative provides an approximation at best, and offers very limited reliability since the statistical database is small and the number of variables involved is large. The experimental alternative is expensive and outside the capabilities of most design firms.

The new Load and Resistance Factor Design Specification[8] recognizes two types of framing: Type FR (fully restrained) is the same as Type 1 in the 1978 Specification, and Type PR (partially restrained) combines Types 2 and 3. For the limit state of strength rigid-plastic analysis can be used, requiring only the ultimate moment capacity of the connection and sufficient ductility to sustain the rotations associated with plastic hinge formation at this location. For the serviceability limit state the drifts must be calculated, requiring an accurate knowledge of the moment rotation curve up to service load levels. If the ultimate moment capacity and its moment-rotation characteristics in the service load range can be easily determined, the design of semi-rigid frames could be greatly simplified under the LRFD Specification.

The results of an experimental program reported here indicate this behavior can be achieved by semi-rigid composite connections. Composite semi-rigid connections offer moment-rotation curves which are very stiff and fairly linear in the service range, and have an easily calculated ultimate capacity. Moreover, the system exhibits large ductilities, good energy dissipation capacity, and ease of construction.

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PAST RESEARCH

It has been recognized for a long time that connections were neither "rigid" nor "pinned," but in reality fell somewhere in between these two extremes. The first research in an attempt to quantify the amount of rigidity in an actual connection was conducted by Batho and Rowan[22] in the 1930s for riveted connections composed of top and seat angles and double-web angles. Later tests on riveted connections were carried out by Rathbun;[21] Young and Jackson;[24] and Hechtman and Johnston.[16] Excellent compilations of past results have been made by Frye and Morris;[14] Jones, Kirby, and Nethercot;[19] Ang and Morris;[10] and Goverdhan.[15] An excellent computer model for the analysis of frames with semi-rigid connections has been developed by Ackroyd et al.[1,2,3,4,5]

The use of continuous reinforcing over column lines to provide for composite action at the connections was first proposed by Barnard[12] in 1970. Since then, several investigators have conducted tests to determine the characteristics of composite semi-rigid connections. In 1972, Johnson and Hope-Gill[18] tested several short double cantilever composite beams at the University of Cambridge in England. In 1980, Echeta and Owens[13] tested two semi-rigid composite connections fastened to a concrete filled rectangular tube section at Imperial College. Van Dalen and Godoy[23] tested another series of composite and bare steel connections at Queen's University in Canada in 1981. These tests showed composite action greatly increased the strength of semi-rigid connections and that semi-rigid composite connections could sustain large rotations.

EXPERIMENTAL DESIGN

The first phase of the authors' project was intended to provide some baseline data to compare the behavior of a semi-rigid connection with and without a composite slab. Given the time and economic constraints, it was decided to use a connection similar to those tested by Radziminski et al.[6,11] for which the moment-rotation characteristics of the non-composite connection were well known. It was recognized that by adding a slab to Radziminski's test specimen without any other modifications some AISC design criteria might not be met. The possibility of making meaningful comparisons, however, overrode these objections.

The first specimen tested is shown in Figs. 1 and 2 and will be labelled SRCC1Mx (semi-rigid composite connection, Number 1, monotonic loading, $x = L$ or R for left or right side). It consisted of a W14 × 99 column, about 12.5 ft high with two W14 × 38 about 10.0-ft long beams framing into it along the strong axis. A composite slab 60 in. wide and 4 in. thick with 8-#4 reinforcing bars was cast on shored forms above the beam to simulate a continuous floor slab across an interior column.

It was anticipated the column would behave essentially as a fixed end for the gravity load case. Given this assumption, different connection details were used on each side of the column. On the right side, a connection very similar to that labelled 14S1 by Radziminski was used. As shown in Fig. 3, this connection consisted of a L6 × 4 × 3/8 at the top and bottom and two L4 × 3-1/2 × 1/4 in the web. The top and bottom angles had an 8-in. length across the beam flange, and were connected by two pairs of 3/4 in. ASTM A325 heavy hex bolts. All holes were 1/16 in. oversized to minimize construction problems, thus introducing the possibility of connection slippage. The gage length in the column was 2-1/2 in. with a single pair of bolts. The web angles had an 8-1/2 in. length on the beam web and were bolted so the beam would be 1/2 in. away from the column flange, which would enable slip in the connection. The connection on the left side was similar, except it lacked the upper angle.

The test setup for the cyclic test (SRCC1C) was similar to that used for the monotonic test except a lateral load was applied at the bottom of the column instead of a vertical load at the beam ends. The actuators at the beam ends were

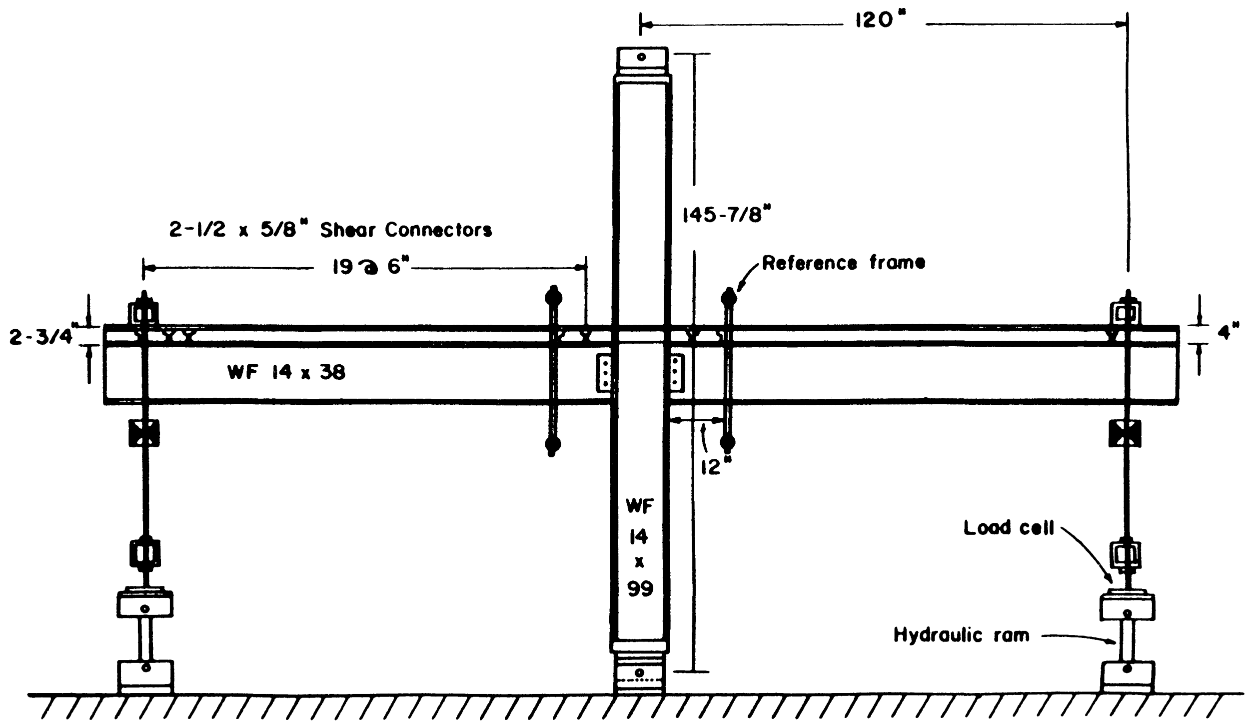

Fig. 1. Overall view of test specimen for SRCC1M

replaced by rigid links which were instrumented to form load cells. For reasons of symmetry under lateral load both sides had the same connection. The connection tested was the same as the left connection for the monotonic test (without the top flange angle).

The load histories are shown in Fig. 4 for the gravity load test and Fig. 5 for the lateral load test. Rotations were measured using a pair of LVDTs on each connection. These LVDTs were connected between the column flange and the beam 12 in. from the column face. Moments were determined by load cells at the end of each beam. In the cyclic load test story drift was also measured with an LVDT and lateral force with a load cell.

EXPERIMENTAL RESULTS

During load history one of the gravity test (GL1) the behavior of the specimen was entirely elastic, with no slab cracking or yield observed. Perfectly elastic behavior was observed up to a moment of 340 kip-in. on each connection.

The initial loading during GL2 closely followed the curves for GL1 to a moment of about 500 kip-in. when first cracking was observed on the left beam. The initial rotational stiffnesses measured were 2.26×10^6 kip-in./radian for the right connection and 2.00×10^6 for the left connection. By comparison specimen 14S1, without a composite slab, had only 1.95×10^5 kip-in./radian (see Table 1 and Fig. 6). The cracks observed on the left connection began at the column flange tips and extended outwards to the slab edge. The crack penetrated through the slab and corresponded with a significant jump on the strain readings from the reinforcing bar strain gages. Similar cracking occurred on the right beam at about 560 kip-in.

When moments reached about 700 kip-in., loud noises were heard, signifying the beginning of slippage of the left bottom angle. At this point the bolts were carrying about 10.4 kips of shear each, above the allowable load of 7.7 kips given by the 1978 AISC Specification. Increasing the load resulted in more loud noises and the beginning of a pronounced loss of stiffness, particularly for the left beam. At a

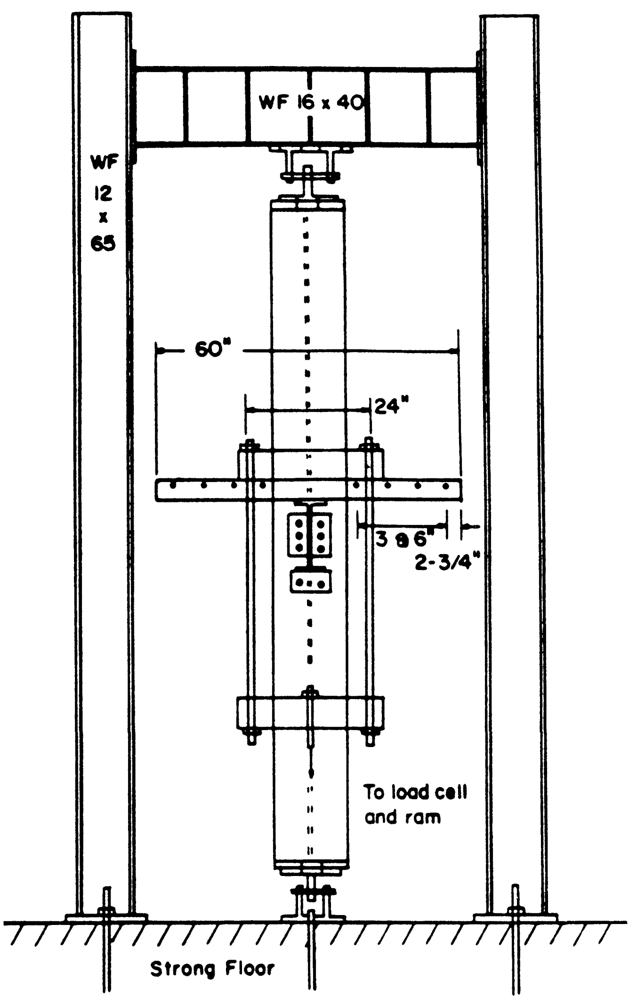

Fig. 2. Specimen in loading frame

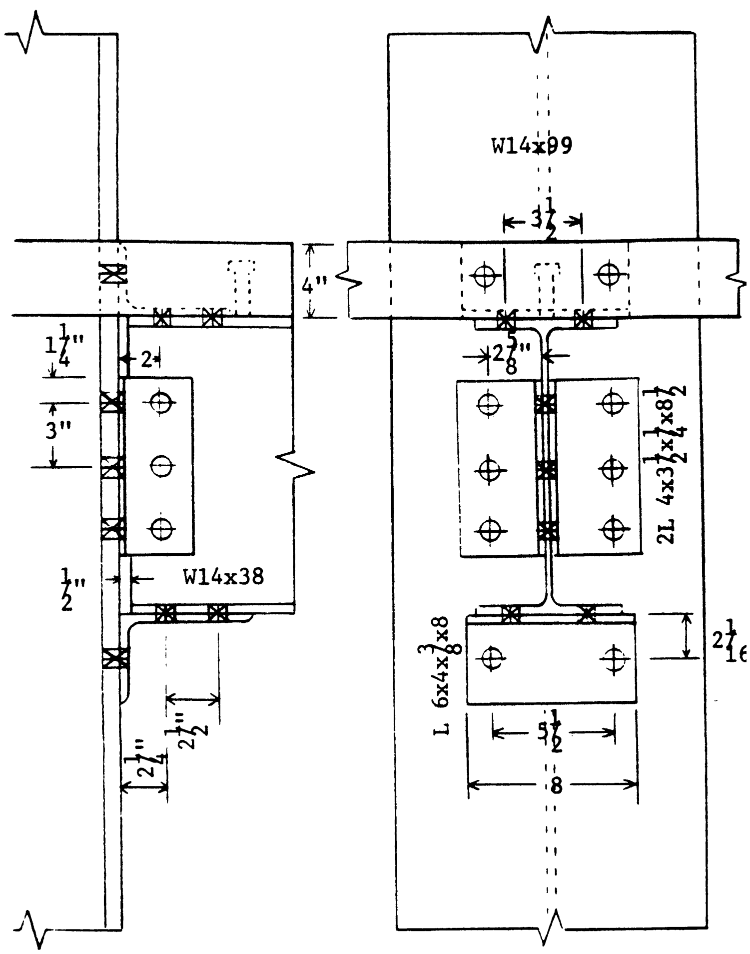

Fig. 3. Detail of connection SRCC1MR. Connections SRCC1ML and SRCC1C were similar except the top flange angle was omitted

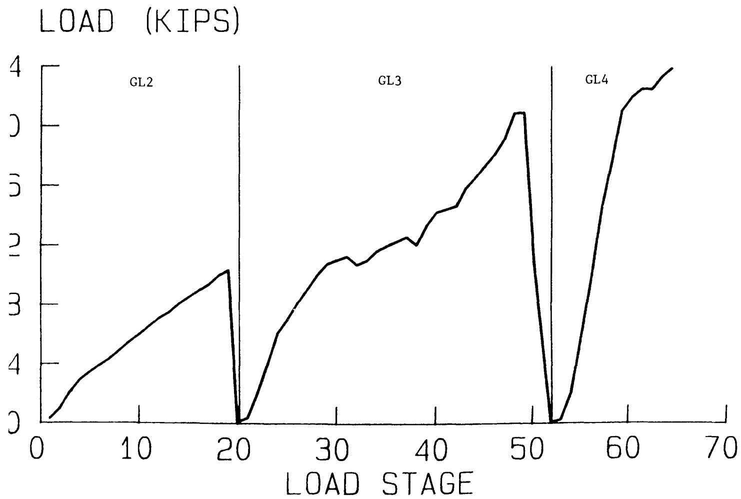

Fig. 4. Load history for gravity load test SRCC1M

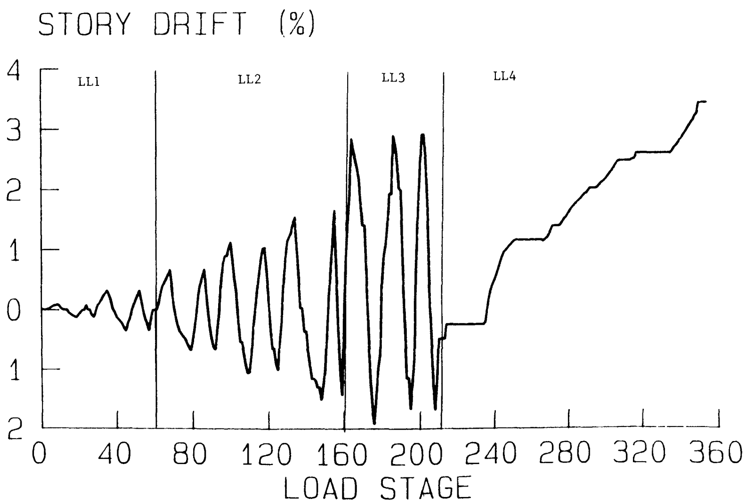

Fig. 5. Load history for cyclic test SRCC1C

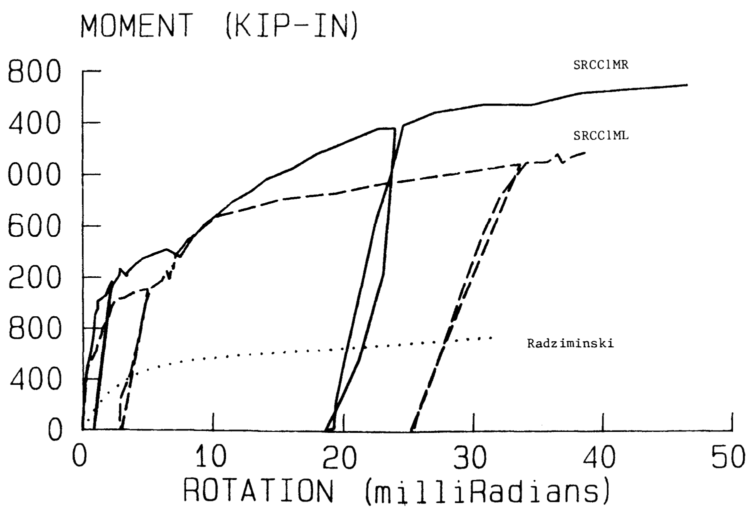

Fig. 6. Complete moment-rotation curves for SRCC1M

Table 1. Stiffness at Selected Points—SRCC1M

Condition	SRCC1MR	SRCC1ML	Radziminski
Initial stiffness	2,260	2,000	195
Slope of secant at 4.0 mR	327	270	109
Slope of tangent at 4.0 mR	95.2	63.3	53.5
Moment at 4.0 mR	1,306	1,078	435
Slope of tangent at 24 mR	40.0	14.0	5.8
Moment at 24 mR	2,404	1,939	668
Slope of tangent at 38 mR	9.1	7.5	—
Moment at 38 mR	2,640	2,173	—

All stiffnesses and slopes are given in kip-in. per milliradian; all moments are given in kip-in.

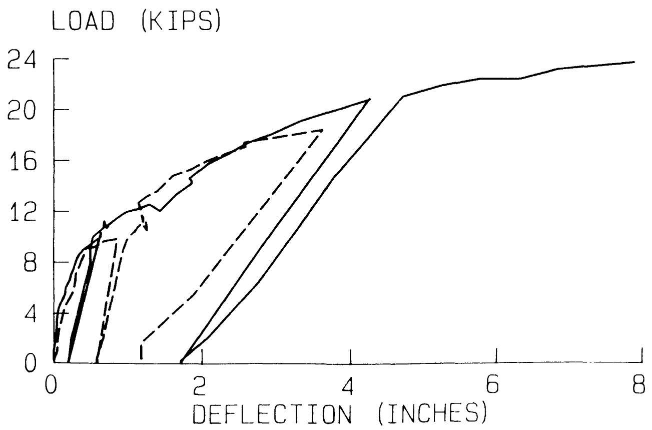

Fig. 7. Complete load-deflection curves for SRCC1M

moment of about 970 kip-in., the nut on one of the bolts connecting the left beam flange and the connection angle showed a large crack. The deformations in this nut indicated it had ceased to work in friction, transferred its load to the remaining three bolts and caused substantial slip of the connection by exceeding the frictional capacity of the remaining bolts. No local yielding or damage was observed as a result of this nut cracking. It was decided to unload the specimen and replace the nut to avoid an undesirable local failure. The unloading branch of the curves for this test indicated essentially elastic unloading for the left beam, but a much lower stiffness (about 23 kip/in.) for the right beam. The unloading resulted in residual deformations due to the bolt slippage. These were particularly large for the left beam, about 0.57 in., and about a third of that, 0.19 in., for the right one. When the damaged nut was removed it was clear the angle had slipped considerably and the bolts were acting in bearing rather than friction.

After the nut was replaced and retightened to 350 ft-lbs of torque, the specimen was reloaded. Since the left connection had slipped into bearing during the prior loading run, it followed a reloading curve without a plateau until the slab rebars began to yield at a load of about 1,800 kip-in. After this the stiffness decreased significantly, and its ultimate strength was achieved at a total rotation of about 0.030 radians, corresponding to a moment of 2,080 kip-in. and a beam end deflection of 2.5 in. The right connection on the other hand had not slipped as much and therefore exhibited a pronounced softening beginning at a load of 1,000 kip-in. At a moment of 1,400 kip-in., the right connection had slipped into bearing and the load began to increase again as the rebar progressively yielded. The ultimate capacity of the right connection was reached at a moment of 2,700 kip-in., a rotation of about 0.039 radians and a total deflection of about 7.5 in. Failure occurred when two of the bolts in the bottom right angle connection to the beam flange fractured in shear, and the whole beam slipped. The beam ended bearing on the column flange and sustaining about two-thirds of the ultimate load. The horizontal shear load at fracture in the bolts was calculated at about 45 kips, or about three times the amount allowed in bearing by the 1978 AISC Specification.

Of interest is the fact some yielding of the column web began to occur at a moment of 1,250 kip-in., with the formation of yield bands inclined at about 50°, beginning near the points where the compression angles were connected to the beam. Assuming the force in the flange was about 80 kips, the column web still complied with the 1978 AISC Specification Eqs. 1.15-2 and 1.15-3 with a d_c of 11.25 in. (less than 22.9 required if 5/3 factor is used) and a t_f of 0.78 in. (where 0.71 in. is required). Equation 1.15-1 would have begun to require a stiffener when the flange load was about 94 kips (still using the 5/3 factor) or 156 kips (without factor). The latter, which is the most reasonable case since the flange load was known, would give a moment of about 2,500 kip-in. or close to the yield capacity of the beams. After a moment of 1,300 kip-in. some localized yielding of the bottom left beam flange was noted near the connection, as well as some yielding in the left web angles. With increase in load, most of the deflection was produced by yielding of the steel reinforcing bars. No visible signs of yielding could be found in the angles except as noted.

In summary, the behavior of the specimen can be divided roughly into three phases. The first phase consisted of essentially linear behavior and lasted until the friction capacity of the bolts was exceeded in the bottom angles (moments below 600 kip-in.). The second phase comprised the slippage of the bolts until they began to work in bearing (moments between 600 and 1,500 kip-in.). The last phase consisted of a long, almost plastic, curve with no strength deterioration and excellent ductility characteristics.

The large ductilities and strength evidenced by the gravity load test indicated it should be possible to use this type of connection to carry some of the lateral loads imposed on structures. Clearly, the lack of full connection rigidity prevents its use other than as a secondary system in zones of high seismic risk (UBC Zones 3 and 4), but enough strength and stiffness may be present for satisfactory performance under wind loads and small earthquakes (UBC Zones 1 and 2). For this cyclic test the left connection from the gravity load test was chosen. The Load-story Drift curve and Moment-rotation curves obtained from this test are shown in Figs. 8–10.

The initial loading of the specimen consisted of four cycles of load, two each at deflections of 0.2 in. and 0.5 in. at the bottom of the column (LL1). A deflection of 0.5 in. at the bottom of the column corresponds to about 0.34% interstory drift. These cycles resulted in beam end loads of 3 and 9 kips, and no evidence of non-linear behavior. The

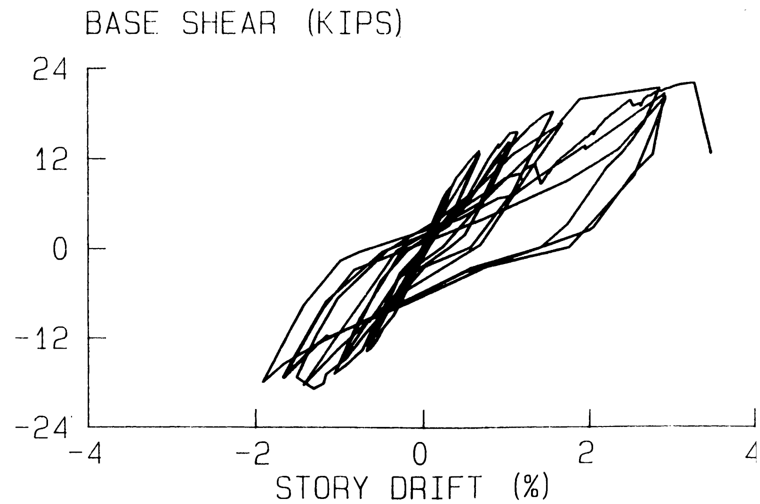

Fig. 8. Complete load-story drift curve for test SRCC1C

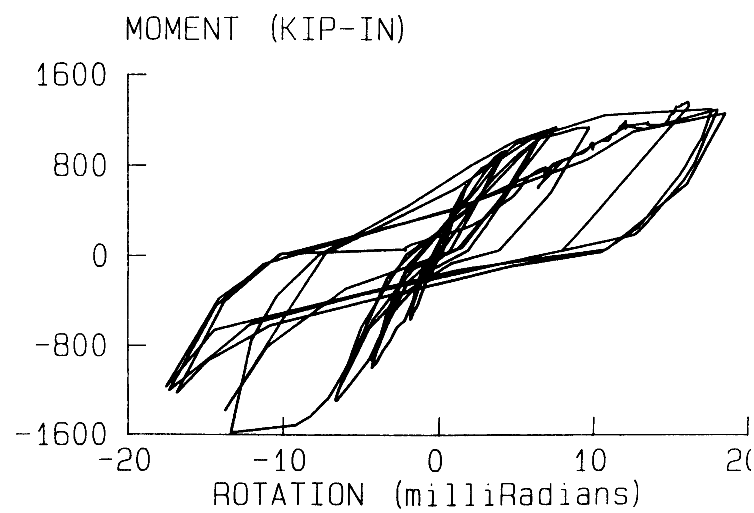

Fig. 9. Complete moment-rotation curve for test SRCC1CL

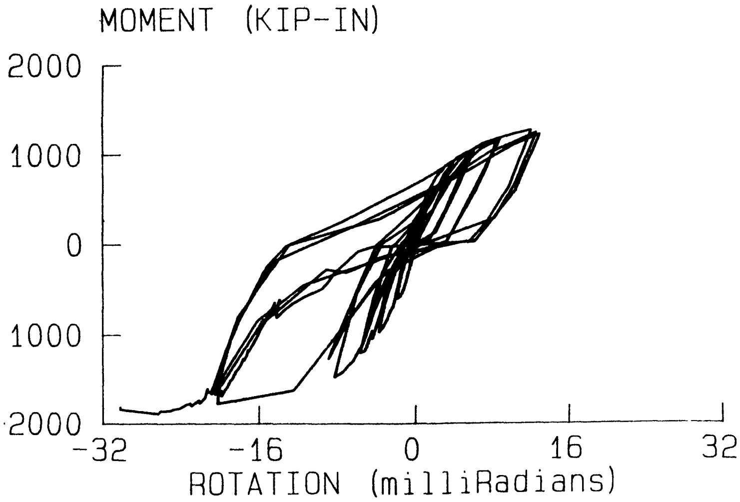

Fig. 10. Complete moment-rotation curve for test SRCC1CR

only damage which occurred during these cycles was the cracking of the slab at the column face. In this load sequence the maximum moments and rotations achieved were about 600 kip-in. and 1.50 milliradians, respectively. By comparison the rotation in the monotonic test at this level of moment was about 0.70 milliradians. This less stiff behavior is most likely due to the earlier cracking of the concrete slab in the cyclic tests and loss of its contribution to stiffness.

The first two cycles at an interstory drift of 1.00 in. (0.68%) did not produce any observable distress. However, beginning with the next two cycles at 1.50 in. (1.02%), the first visible signs of damage were evident. This damage was of two types: for the connection loaded with negative moment there was an increasing number of cracks and the existing cracks opened more; for the connection with positive moment the flange angle began to separate from the column face. In this region, slip between the flange angles and beam flange began to have an effect on the behavior of the connection and the moment rotation curves deviated from linear. The large non-linearities in this region are due primarily to yielding of the slab reinforcement. This yielding began in the first loop with a deflection of 1.50 in. for the right connection and in the first loop with a deflection of 2.00 in. for the left connection. This indicates a possible influence of the initial direction of loading.

As the loading proceeded into LL3 the loading became unsymmetrical due to loading frame limitations, with maximum positive displacements of 4.25 in. and negative ones of 2.91 in. At this level significant hysteresis losses began to occur. The opening up of cracks and the pulling away from the column face of the flange angle were the causes of pinching of the hysteresis loops in this region. Even with this detrimental behavior of the connections, as load was increased to the point where the cracks were closed and the flange angle was bearing, the stiffness increased to the same values obtained in the monotonic test for similar levels of rotation (Table 2). As the level of load for each cycle

Table 2. Stiffness of Envelope—SRCC1C
Left Connection

Load Stage	Moment (kip-in.)	Rotation (mR)	Stiffness (kip-in./mR)
0-9	240	0.4	557.0
24-35	625	1.7	296.9
52-68	923	4.0	126.1
86-100	1052	6.4	52.74
117-134	1145	9.6	36.51
155-164	1313	17.7	16.38
0-18	−217	−0.7	326.7
28-45	−569	−1.9	304.0
57-79	−997	−4.3	197.8
92-109	−1293	−6.6	157.8
125-146	−1513	−9.2	124.5

Right Connection

Load Stage	Moment (kip-in.)	Rotation (mR)	Stiffness (kip-in./mR)
0-9	−238	−0.5	487.7
24-35	−610	−1.9	287.8
52-68	−987	−3.7	229.1
86-100	−1209	−5.6	159.5
117-134	−1480	−8.3	137.9
155-164	−1779	−20.3	44.02
201-350	−1894	−26.4	40.35
0-19	162	0.6	264.3
28-45	536	1.4	485.8
57-79	926	3.8	162.9
92-109	1048	6.1	59.22
125-148	1188	8.5	60.19
159-176	1283	12.0	31.79

increased there was greater residual deformation at the end of the cycle. At the zero load position in the final set of cycles there was a residual deformation of about 2 in. in the positive direction and 1 in. in the negative direction.

At low levels of drift the instrumentation of the panel zone for the column web showed very little shear strain (less than 0.03 milliradians during LL1). At larger levels of drift significant shear strains were shown, and by the first cycle of LL3 they had reached 2.65 milliradians, which contributes about 10% to the connection rotation. These shear strains were accompanied by a small amount of yielding in the panel zone, but there was no indication of loss of web stability.

The final monotonic test (LL4) did not produce any new results, except to prove very large rotations can be accommodated by the system and ductile behavior can easily be achieved.

The results of the cyclic test indicate good lateral load behavior can be obtained with composite semi-rigid connections. The first large non-linearities in the hysteresis loops were noted at an interstory drift of 1.5%. Normal design procedures would limit this drift to 0.5% under most conditions for the type of construction envisioned. The large ductilities and good energy-absorption capacity evidenced by the system, as well as its inherent redundancy will probably make it a very attractive structural system in large areas of the USA.

CONCLUSIONS

The results of the gravity load test leads to the following conclusions:

1. The behavior of a composite connection is similar to that of a non-composite connection, with the slab steel replacing the top angle. The higher strength of the rebar steel, the increased moment arm and the presence of a slab result in a stronger, stiffer and more ductile system. The substitution of the angle with the rebar also results in a much more linear initial behavior.
2. The moment-rotation curves are fairly linear within the range that should be used for service loads, and thus serviceability checks might not be as difficult as previously thought. For design purposes, the connection can probably be considered linear with a stiffness similar to those obtained at the beginning of GL3, if shakedown can be assumed to have taken place during GL2. Thus the use of a complex approach, requiring B-splines or polynomials to approximate the moment-rotation characteristics, might not be required for everyday office use.
3. The behavior of the specimen was governed primarily by the yielding of the slab steel. This specimen had only 8-#4 bars (A = 1.60 in.) for a reinforcement ratio of 0.67 in the slab. Substantial gains in the linearity of the moment rotation curves can be expected if the reinforcement ratio is increased. The strain gauge data available from the slab rebars indicates the stresses were not distributed uniformly across the slab, but more on a parabolic fashion, with the outermost bars carrying about half as much load up to yield as the innermost ones.
4. Although the forces in the column web were very large, and at the end of the test exceeded AISC allowable values, only very limited yielding was observed. Thus web crippling might not be as severe a problem for semi-rigid connections as with rigid ones.
5. If the slippage of the bolts can be limited, either by increasing their size or number, the linear behavior can probably be extended far beyond what was shown in this test.
6. The use of a top angle provided reserve strength capacity and stiffness at ultimate. The deformations observed in the angles were very small until the end of test GL3.

The results of the cyclic load test lead to the following conclusions:

1. The behavior of a composite connection under cyclic loads is considerably different than similar non-composite connections. The increased strength of the connection makes the yielding of the bottom flange angle occur at a moment that is a lower proportion of the ultimate moment of the connection than is the case for a non-composite connection. This characteristic leads to increased pinching of the hysteresis loops, but the area they enclose is still about twice as large as enclosed by loops for the same range of rotations in non-composite connections.
2. The envelope of the moment rotation curves from the cyclic test follows the moment rotation curve of the monotonic test very closely.
3. The shear strains observed were large enough to cause some yielding in the column web, but not nearly as large as those obtained in tests of rigid connections.[20] Two reasons for this are the increased bearing area of this connection type spreads the tensile and compressive forces over a larger area and the concrete between the column flanges acts as a stiffener. This suggests the use of web stiffeners may not be required as often in semi-rigid construction as in rigid construction.
4. The behavior of the connection was governed by the plastic bending of the flange angle and the yielding of the rebar. The more important of these at lower levels of load (in the service range) is the bending of the flange angle, as it occurs first for a configuration like the one tested. This problem can be eliminated by using a thicker and/or longer angle, by using a plate welded to the column in place of the angle, or—especially in the case of retrofits—a weld could be added along the top of the leg adjacent to the column. The addition of more reinforcement in the slab along with these modifications would produce a longer linear portion in the moment-rotation curves, and enable the designer to detail a connection capable of developing the full moment capacity of the beam.
5. The lack of a top angle did not have an affect in the cyclic test any more than it did in the monotonic case. The removal of this connection element simplifies erection and decreases cost without having an adverse affect on the redundancy and safety of the system.
6. The hysteresis loops for the third cycle at the maximum deflection did not change appreciably from the loops from the second cycle at this deflection. This indicates there is no increase in damage as the number of cycles at a given loading increases, suggesting incremental collapse should not be a problem.
7. The direction of first damage did not appear to have any affect on the behavior of the connection. The moment rotation curves obtained for the right connection are the same as those for the left connection.

The results of this project indicate that the particular connection configuration tested could be used to provide lateral stability if design drifts are kept below 0.5%, which is the allowable drift specified by UBC and ANSI.[9,17] It should be noted these results are only for one connection, and to generalize them to all of the many possible connection configurations without further testing should be strongly discouraged. These results suggest the use of semi-rigid composite connections may be an economical way to resist lateral load in low-rise structures or to help in the resistance of these forces in larger structures.

FUTURE WORK

The work described in this paper encompasses the first half of a project funded by AISC. The second part of the experimental effort consists of testing a full-scale, two-bay subassemblage as well as testing another isolated interior connection. The preliminary results from these tests confirm the results and expand the database described in this article.

In addition to the experimental work, a companion analytical and design-oriented effort are underway to translate these results into practical applications. A design procedure based on the new LRFD provisions for the use of these connections for both gravity and lateral loads is the ultimate goal of this project.

ACKNOWLEDGMENTS

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REFERENCES

1. *Ackroyd, M. H.* Automatic Design of Steel Frames with Flexible Connections *Report of American Iron and Steel Institute, July 1977, University of Colorado, Boulder.*
2. *Ackroyd, M. H. and K. H. Gerstle* Strength and Stiffness of Type 2 Frames *Report of American Iron and Steel Institute, July 1977, University of Colorado, Boulder.*
3. *Ackroyd, M. H. and K. H. Gerstle* Behavior of Type 2 Steel Frames *ASCE Journal of the Structural Division, Vol. 108, No. ST7 (pp. 1,541–1,556).*
4. *Ackroyd, M. H. and K. H. Gerstle* Elastic Stability of Flexibility Connected Frames *ASCE Journal of the Structural Division, Vol. 109, No. ST1 (pp. 241–255).*
5. *Ackroyd, M. H.* Design of Flexibly-Connected Steel Building Frames *Final Report to American Iron and Steel Institute, November 1985.*
6. *Altman, W. G., A. Azizinamini, J. H. Bradburn and J. B. Radziminski* Moment Rotation Characteristics of Semi-Rigid Steel Beam-Column Connections *Technical Report, Department of Civil Engineering, June 1982, University of South Carolina.*
7. *American Institute of Steel Construction, Inc.* Manual of Steel Construction *8th Ed., 1980, Chicago, Ill.*
8. *American Institute of Steel Construction, Inc.* Proposed Load and Resistance Factor Design Specification for Structural Steel Buildings *January 1986, Chicago, Ill.*
9. *American National Standards Institute* ANSI A58.1-1982 *1982, New York, N.Y.*
10. *Ang, K. M. and G. A. Morris* Analysis of Three Dimensional Frames with Flexible Beam-Column Connections *Canadian Journal of Civil Engineering, Vol. 11, No. 2 (pp. 245–254).*
11. *Azizinamini, A., J. H. Bradburn and J. B. Radziminski* Static and Cyclic Behavior of Semi-Rigid Steel Beam-Column Connections *Technical Report, Department of Civil Engineering, March 1985, University of South Carolina.*
12. *Barnard, P. R.* Innovations in Composite Floor Systems *Canadian Structural Engineering Conference Proceedings, Canadian Steel Industries Construction Council, 1970 (pp. 13).*
13. *Echeta, C. B. and G. W. Owens* A Semi Rigid Connection for Composite Frames—Initial Test Results *Joints in Structural Steelwork, John Wiley & Sons, 1981, New York, N.Y. (pp. 6.93–6.121).*
14. *Frye, M. J. and G. A. Morris* Analysis of Flexibly Connected Steel Frames *Canadian Journal of Civil Engineering, Vol. 2, No. 3 (pp. 280–291).*
15. *Goverdhan, A. V.* A Collection of Experimental Moment-Rotation Curves and Evaluation of Prediction Equations for Semi-Rigid Connections *Master's Thesis, Vanderbilt University, December 1983, Nashville, Tenn.*
16. *Hechtman, R. A. and B. G. Johnston* Riveted Semi-Rigid Beam-to-Column Building Connections *Progress Report No. 1, AISC Committee of Steel Structures Research, November 1947.*
17. *International Conference of Building Officials* Uniform Building Code *1985.*
18. *Johnson, R. P. and M. Hope-Gill* Semi-Rigid Joints in Composite Frames *Preliminary Report of the Ninth Congress of IABSE, 1972 (pp. 133–144).*
19. *Jones, S. W., P. A. Kirby and D. A. Nethercot* Columns with Semirigid Joints *ASCE Journal of the Structural Division, Vol. 108, No. ST2 (pp. 361–372).*
20. *Popov, E. P., N. R. Amin, J. J. C. Louie and R. M. Stephen* Cyclic Behavior of Large Beam-Column Assemblies *Earthquake Spectra, Vol. 1, No. 2 (pp. 203–238).*

21. *Rathbun, J. C.* Elastic Properties of Riveted Connections *First Report, Transactions of Steel Structures Research Committee, Department of Scientific and Industrial Research, HMSO, 1931, London.*

22. *HMSO* First Report, Steel Structures Research Committee *Department of Scientific and Industrial Research, 1931, London.*

23. *Van Dalen, K. and H. Godoy* Strength and Rotational Behaviour of Composite Beam-Column Connections *Canadian Journal of Civil Engineering, Vol. 9, No. 2 (pp. 313–322).*

24. *Young, C. R. and K. B. Jackson* The Relative Rigidity of Welded and Riveted Connections *Canadian Journal of Research, Vol. 2, Nos. 1 & 2 (pp. 62–100, 101–134).*