Partially Restrained Composite Beam-To-Girder Connections

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INTRODUCTION

Advancements in design technology and construction materials have enabled engineers to design longer spanning and shallower composite beam floor systems. The beams in these floor systems are typically designed as simply supported members which, in many instances, leads to the design being controlled by construction or live load deflection criteria rather than strength. If the beams were designed as continuous beams (having rigid connections) or partially continuous beams (having partially restrained connections) then deflection problems would be minimized and the strength of the composite beam could be more fully utilized.

In recent years several research programs have investigated the strength and rotational stiffness of beam-to-column connections in buildings with composite slabs that are continuous over the connection region. These connections have been termed "Semi-Rigid Composite Connections" (SRCC) or more recently "Partially Restrained Composite Connections." This research has shown that SRCC are capable of providing a range of rotational stiffness from what could be classified as a pinned connection to what could be classified as a rigid connection. The key to SRCC behavior is the continuous composite slab which is reinforced with reinforcing steel and passes over the connection region. Because the composite slab is also continuous over beam-to-girder connections it seems probable that the idea of SRCC could be applied to these connections. This would lead to beam-to-girder connections with rotational resistance (partially restrained (PR) beam-to-girder connections) that could then be used to design composite beams as partially continuous rather than simply supported.

A research project aimed at developing design methods and criteria for PR beam-to-girder connections and partially continuous floor systems is currently in progress at Virginia Polytechnic Institute and State University (VPI). This paper has two main parts. The Background presents the reader with the reasoning for considering continuity in a steel framed floor system, a review and definition of the general connection classifications, the rationale for choosing a partially continuous floor system over a fully continuous floor system, and how continuous composite slabs can be utilized in developing a partially continuous floor system. The Experimental Investigation presents a summary of preliminary tests on PR composite beam-to-girder connections.

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BACKGROUND

There is a continuing demand for more open space in buildings. In particular, building owners want as much open space as possible to allow the flexibility of accommodating a variety of tenants. The amount of open space in a building is a direct result of the floor system used. Three changes in steel design over the last 30–40 years have allowed engineers to increase open space in buildings with steel framed floors. First, composite steel-concrete floor system technology has developed which allows designers to use the synergy of tying the two floor components (the beam and the slab) together to span longer distances. Second, the plastic section analysis and design procedures utilized in the Load and Resistance Factor Design (LRFD) methods[13] have allowed an additional increase in span length over designs which utilize Allowable Stress Design (ASD).[15] Third, high strength steel, particularly A572 Grade 50 steel, is becoming more readily available at a cost comparable with A36 steel. Longer and shallower floor systems, and thus more open space, have been the result of these changes. Along with these benefits have come an increased concern for serviceability limit states.

Serviceability issues, such as floor deflections and vibrations, are an increasing concern as floor systems become longer and shallower. In many cases serviceability criteria control the floor design.[16] It is the current belief that at least floor deflection problems will be minimized or solved by designing floor systems with a certain degree of continuity. The effect of continuity on floor vibrations is uncertain at this time.

Structural engineers are aware of the advantageous characteristics of a continuous beam over those of a simply supported beam. These include reduced design moments, deflections, and an increased structural redundancy. The moment-rotation behavior of the beam-to-girder connection is the key to determining the degree of continuity that a steel framed floor system exhibits.

Every connection has a distinct moment-rotation relationship that determines how the connection will behave as load is applied to the structural members that it is connecting. The American Institute of Steel Construction (AISC) classifies connections into three groups as shown in Table 1. Connections that have significant rotational restraint are classified as rigid and for analysis are treated as perfectly rigid (infinite moment capacity with no rotation). Connections with little rotational restraint are classified as simple and for analysis are treated as perfectly flexible (infinite rotation capacity with no moment). Despite the fact that neither perfect rigidity or perfect flexibility exists in real structures, these assumptions are justified as long as the behavior of the attached members based on the assumed connection behavior is not significantly different than would be predicted by accounting for the true connection characteristics. If a significant difference in member behavior occurs, then the connection should be classified as a semi-rigid (or PR) connection and appropriate analysis techniques must be used to account for the connection behavior.

The basis for the demarcation between these three general categories of connections is somewhat qualitative. A commonly used basis in the United States considers the rigid assumption valid if the connection can develop 90 percent of the fixed end moment of the structural member; and, if the connection develops less than 10 percent of the fixed end moment then the assumption of a simple connection is justified. This classification considers only the strength of the connection and ignores the stiffness of the connection which consequently ignores the possibility that the connection may not reach its full strength if beam end rotations are small. Bijlaard and Zoetemeijer[5] developed a non-dimensional basis for connection classification which is shown in Figure 1. This classification accounts for both strength and stiffness and was

Table 1.
AISC Connection Classification Groups

	ASD	LRFD
Rigid	Type 1	Fully Rigid (FR)
Simple	Type 2	Partially Rigid (PR)
Semi-Rigid	Type 3	Partially Rigid (PR)

modified and adopted for use in Eurocode 3[6] and Eurocode 4[7] with a distinction made between braced and unbraced frames as shown in Figure 2. The reader is referred to Reference 10 for a more detailed discussion on the background of the demarcation between the three connection classifications.

Because perfectly flexible connections assume no moment resistance, only rigid and PR beam-to-girder connections can be used in floor design to provide full or partial continuity. If rigid connections are used then a fully continuous floor system would be assumed and if PR connections were used then a partially continuous floor system would be assumed.

Some possible rigid beam-to-girder connections are shown schematically in Figure 3. These could be used to design a fully continuous floor system. Aside from the advantage of continuity, rigid connections have several disadvantages. First, rigid connections typically require substantial welding and thus cost more than simple connections in materials and labor. Second, rigid connections attract large moments to the connection. This is a particular disadvantage for composite beams because the largest moment capacity of a composite beam is in the positive moment region, while the negative moment capacity at the supports is typically reduced as a result of the cracked floor slab. These relationships are indicated schematically in Figure 4. Third, it is recognized that to develop a plastic collapse mechanism in a continuous beam, particularly a continuous composite beam, a large redistribution of moment from the supports to the midspan regions is necessary. This implies that the support region must have substantial rotational ductility. Rigid connections are suscep-

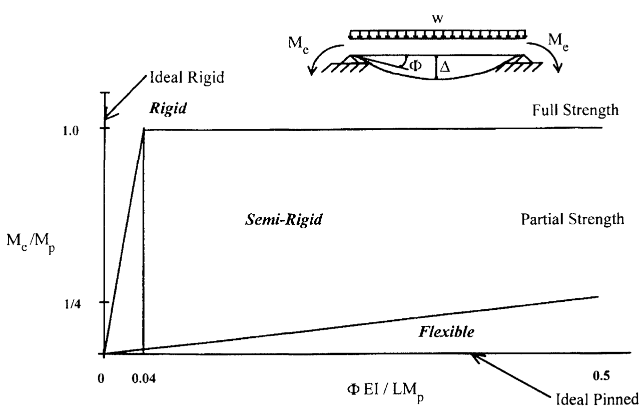

Fig. 1. Bijlaard & Zoetemeijer joint classification.[5]

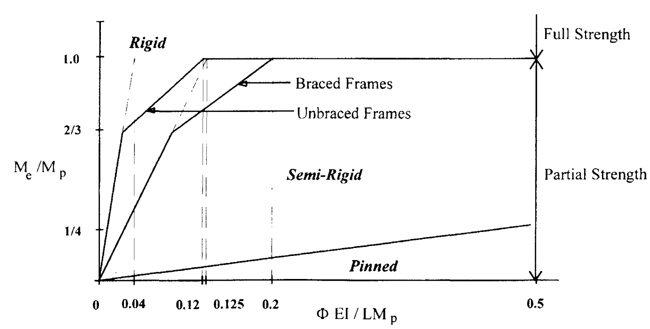

Fig. 2. Joint classification according to Eurocode 4.[12]

tible to local flange and or web buckling at relatively low rotations which would not allow adequate redistribution of moment in the member.

PR beam-to-girder connections could be used to design a partially continuous floor system. As illustrated in Figure 5, using PR connections allows the designer to control the distribution of positive and negative moments in the beam by varying the rotational restraint provided by the connection. Ideally, the designer would want to design the connection so that the distribution of moment is proportional to the beam strength. Thus, for a composite beam, the connection should be designed so that more moment is distributed to the midspan of the beam than to the ends.

Banard[2] introduced the idea of PR composite connections in 1970. He suggested that PR steel connections should be combined with composite construction to provide continuity over supports. He also proposed details for both PR composite beam-to-column and beam-to-girder connections. Since that time, PR composite beam-to-column connections have been the subject of several research programs. The most comprehensive research has been conducted by Leon and colleagues,[1,19,8-12] and Zandonini and colleagues.[3,4,16] Both of these research teams have studied numerous connections as well as developed design guidelines.

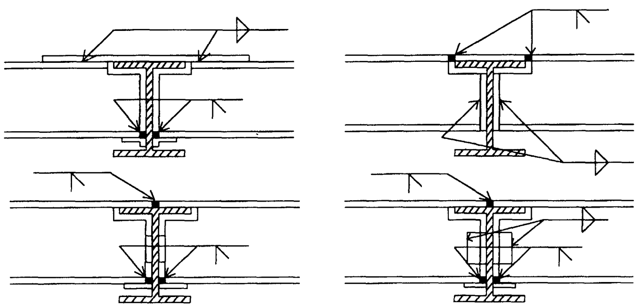

Fig. 3. Rigid beam-to-girder connections.

The research on PR composite beam-to-column connections has shown that slight modifications to typical beam-to-column connections can result in relatively stiff, PR composite connections. These modifications include adding reinforcing bars in the composite slab over the connection, using slightly larger steel elements in the connection, and increasing the number of shear connectors. The research has shown that by varying these details, the rotational restraint of composite connections can range from very flexible to rigid. Additional desirable characteristics that are evident from the research results include:

1. The moment-rotation behavior in the service load range has a high degree of linearity. This will permit the connections to be readily incorporated into linear analysis and design, with which designers are already familiar.
2. The use of composite PR connections can result in connections having the same capacity and rigidity as rigid bare steel connections. Although, unlike many rigid connections the composite connections have so far shown a tremendous rotation capacity prior to failure.[11]
3. The continuous steel reinforcing reduces problems associated with cracking across the support.
4. A reasonable estimate of the ultimate capacity of the connection is typically easy to determine.
5. The connection detailing and fabrication is basically the same as typical simple connections. This means that continuity can be developed at supports without significantly increasing the complexity of the structural detailing. This is a particularly important point as the cost of labor has increased more rapidly than the cost of materials in recent years.

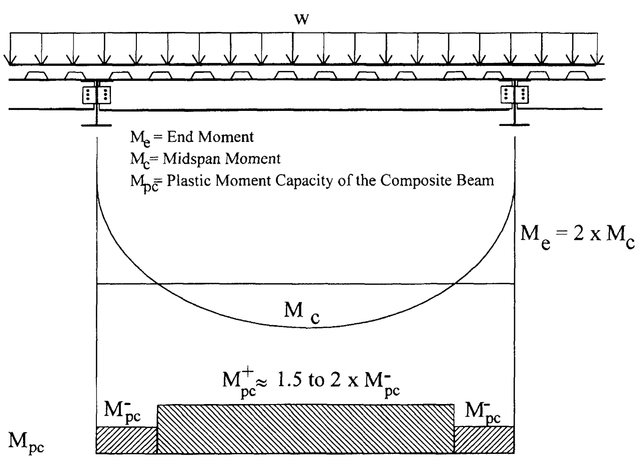

Fig. 4. Moment distribution for beam with rigid connections.

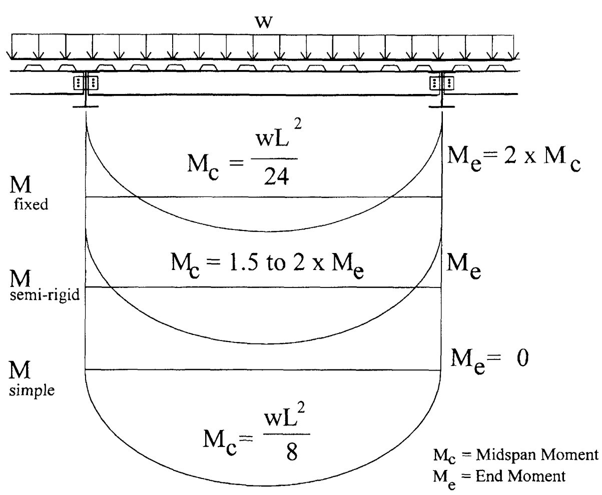

Fig. 5. Beam moments associated with rigid, semi-rigid, and simple connections.

All known experimental research with PR composite connections has been in refining or developing beam-to-column connections, while beam-to-girder connections have not been investigated. PR composite beam-to-girder connections appear to be a natural extension of the work done with composite beam-to-column connections and would appear to be a connection ideal for designing a partially continuous floor system. The results of the preliminary tests reported herein on composite beam-to-girder connections indicate that they exhibit many of the same advantages associated with the composite beam-to-column connections. In general, it is believed that much of the research in composite beam-to-column connections will be directly applicable to composite beam-to-girder connections.

EXPERIMENTAL INVESTIGATION

Test Specimens

Four PR composite beam-to-girder connections were tested in this portion of the research program. The four connection details are shown in Figures 6–9. For more detailed information than is presented in this paper regarding specimen configuration, instrumentation, etc., the reader is referred to Rex.[14]

Connection #1 was a single plate framing connection which is the second most commonly used beam-to-girder connection. Connection #2 was detailed the same as Connection #1 but with the addition of a seat angle. Attaching a seat angle to the bottom beam flange was shown in the research on PR composite beam-to-column connections to increase rotational stiffness and provide stability for the bottom flange. Connections #3 and #4 were two innovative connections developed in an attempt to increase the rotational stiffness of the beam-to-girder connections prior to placement of a composite slab (this initial connection behavior is believed to be particularly important because many composite beam designs are controlled by the construction loads). Both connections were combinations of a seat angle that attached the bottom flange of the beam to the girder and a tension plate that attached the top portion of the beam to the girder. The seat angle in Connections #3 and #4 was turned upside down from the typical seated beam connection to eliminate clearance

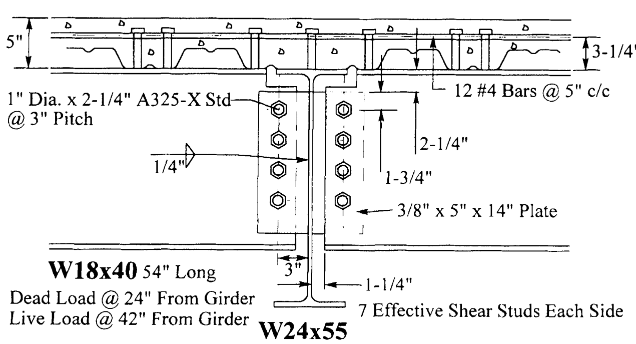

Fig. 6. Connection #1 details.

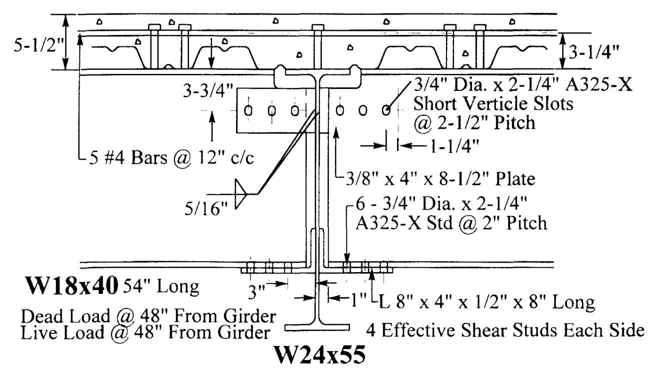

Fig. 8. Connection #3 details.

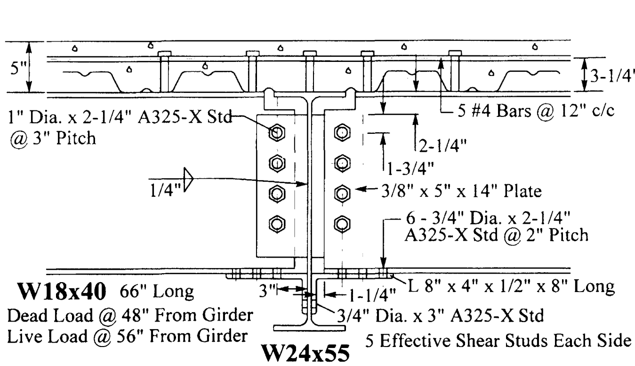

Fig. 7. Connection #2 details.

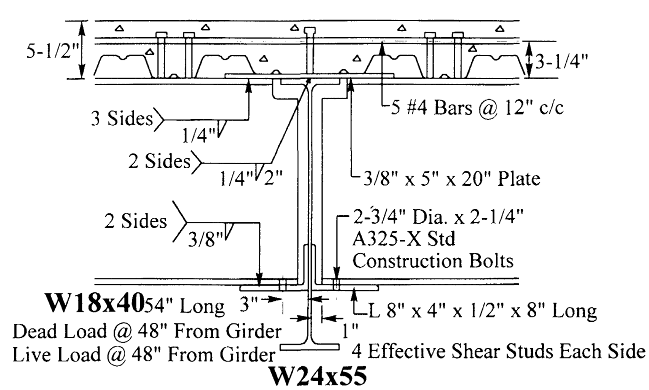

Fig. 9. Connection #4 details.

Table 2. Measured Material Properties			
Connection Component	Average Yield (ksi)	Average Ultimate Strength (ksi)	Percent Elongation
Seat Angle	42.6	65.4	30%
Shear & Tension Plates	48.9	71.4	29%
Beam	45.1	61.6	27%
Girder	47.5	64.0	31%
Reinforcing Steel	70.9	118.7	7%
Connection #1 Concrete	N/A	2.6	N/A
Connection #2 Concrete	N/A	5.3	N/A
Connection #3 Concrete	N/A	4.2	N/A
Connection #4 Concrete	N/A	3.5	N/A

problems between the bottom flange of the beam and the bottom flange of the girder.

A hypothetical design consisting of a 40-ft. long composite filler beam on 15-ft. beam spacing was used to determine the general details of the test connections. Un-reduced uniform live and dead loads of 100 psf and 60 psf respectively were used. It is recognized that the design utilizes relatively large beam spacing and high live loads. However, it is believed that spacing of this magnitude will not be uncommon with future improvements in composite steel floor decks. It is also believed that PR composite beam-to-girder connections will be particularly useful for designs that utilize relatively large beam spacing and high live loads.

Connection angles and plates were fabricated from A36 steel. All welds were shop welded using a shielded metal arc welding (SMAW) process with a continuous feed. The beams were W8×40s and the girder was a W24×55. Both were fabricated from A572 Grade 50 steel. Average measured material properties are presented in Table 2. The test specimen beam lengths were based on average projected inflection points for the beam in the hypothetical design. New beam sections were used for each connection test while the girder was reused because it sustained only minimal damage as a result of the symmetrical cruciform testing arrangement used. The beams, girder, and all connection parts were white-washed to identify yield patterns during testing. The connections utilized A325 bolts that were pre-tensioned by turn-of-the-nut method.

A 60-in. wide composite slab using 2-in. deep 20 gage composite steel deck and normal weight concrete was placed on top of the beam-to-girder connection. The 60-in. width was chosen because it was a common width used in the PR composite beam-to-column research. The steel deck was haunched over and puddle welded to the girder. The slabs were reinforced with WWF6×6–W1.4×W1.4 mesh and #4 grade 60 reinforcing bars. Connection #1 had 12 reinforcing bars (1.33 percent) while Connections #2 through #4 had five reinforcing bars (0.55 percent, 0.48 percent, and 0.48 percent respectively). Welded headed shear studs (¾-in.ϕ×4 in.) were used to attach the slab to the beams and girder. Five shear studs spaced at 12 inches on center were attached to the center line of the girder while the number of shear studs attached to the beams varied depending on the connection.

The shear studs attached to the beams were designed using the AISC Specification.[18] The nominal stud strength, Q_n, was determined to be 19.6 kips/stud by assuming nominal values for the concrete ($f_c' = 4$ ksi) and shear studs ($F_u = 60$ ksi) and by using a stud reduction factor of 0.75 based on recent recommendations.[17] In all four connections there was one stud placed in the deck rib adjacent to the ends of the specimen (near the point of loading). Because there was only three or four inches of concrete between the stud and the end of the slab, it was believed that this stud would not be very effective and was ignored for purposes of determining the required number of studs.

In general, the number of studs required was based on the nominal yield force of the reinforcing steel which was 144 kips for Connection #1 and 60 kips for Connections #2 through #4. Connection #1 required eight effective studs but because of minimum spacing requirements only seven studs could be used. The stud layout for Connection #1 is presented in Figure 10. Connections #2 through #4 required four effective studs which was the number used in Connections #3 and #4 but because of maximum spacing requirements (to prevent deck uplift) Connection #2 had one stud per deck rib which resulted in five studs instead of the four required.

For Connections #1 and #2 a shear connector was placed in the deck rib immediately adjacent to the girder. The writers were informed (by the advisory committee) after Connection #2 had been cast that a shear connector would not normally be placed adjacent to the girder because of special flashing that is used around girder haunches. Consequently Connections #3 and #4 did not have a shear connector immediately adjacent to the girder.

Instrumentation

The preliminary goal of the initial composite beam-to-girder connection tests was to determine the moment-rotation be-

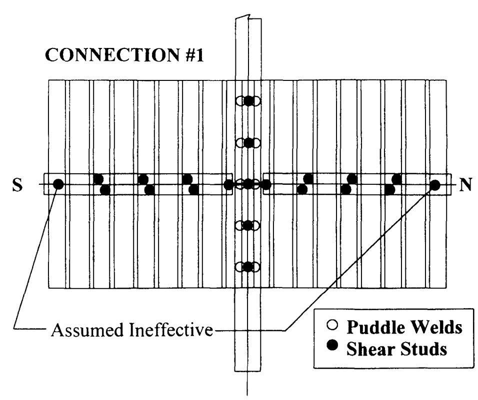

Fig. 10. Steel deck and stud layout.

havior for the connections. The moment was determined by multiplying the value of the load applied on the ends of the cruciform specimen by the distance from the load to the center line of the girder. The connection rotation was measured as the rotation of the beam end relative to the girder web. This was measured with spring loaded linear potentiometers attached near the top and bottom of the beam which then rested against the girder web. The distance between the top set of potentiometers and the bottom set of potentiometers was typically around 14 inches. In addition to measuring rotation and moment, a variety of other instrumentation was used to measure quantities such as bolt slip, slip between the composite slab and the beam, strain in the reinforcing steel, deflections of the beam and girder, and strains throughout the beam web and connection elements.

General Test Setup and Load Frames

A cruciform test configuration, illustrated in Figure 11, was used in which the specimens were subjected to two general phases of static loading; simulated dead load on the bare steel connection and simulated live load on the composite connection. The general test setups used during the dead load and live load simulations are presented in Figures 12 and 13 respectively. The test girder was supported by and bolted to three W10×49 columns which were spaced at four foot intervals. The two outside columns were bolted to a reaction floor while the middle column (which was directly under the connection) was bearing against a reaction floor.

The three column supports for the girder obviously provide constraints on the girder that would not be present in a real floor system, namely the elimination of vertical deflection of the girder as well as increased rotational resistance provided by the girder (since the bottom flange was bolted to the columns). A very complex test setup would be required to try to simulate the real behavior of the girder while testing the beam-to-girder connections. In the writers' opinion a more complicated setup is not necessary based on the following two assumptions: the vertical deflection of the girder will have little if any impact on the behavior of the beam-to-girder connection, the extra rotational resistance in the girder has no effect on the measured moment and rotation behavior of the connections on either side because they are measured relative to the girder web. The intent of the cruciform specimen was to load the two sides of the specimen equally which would ideally create a plane of symmetry through the center of the girder web in which case the girder should not rotate at all. In a real floor system, the writers believe that the girder will provide little if any rotational restraint that would cause un-balanced moments on each side of the girder.

The dead load simulation frame consisted of short cruciform columns that were bolted to reaction floor beams and a structural tube that spanned between the two cruciform columns. The specimen beam was connected to the dead load simulation frame by a series of U-bolts, threaded rod, and turnbuckles as shown in Figure 11. The turnbuckles were tightened to apply the simulated dead load. The magnitude of the load was measured by a portion of threaded rod that was instrumented as a load cell transducer.

The live load frame consisted of W21×62 columns, which were attached to the reaction floor. Two C15×50 sections spanned between the columns and supported a short reaction beam at their mid span. Two hydraulic rams powered by an electric motor were used to load the test specimen. The rams reacted against the composite slab through a block and roller arrangement with the load distributed across the end of the composite slab through a steel plate.

Testing Procedure

Because the majority of composite beams are currently built using unshored construction, it is apparent that the connections associated with these beams will have two distinct stages of behavior; before and after concrete hardens. Before the concrete hardens, the only rotational resistance of the beam-to-girder connection will be provided by the steel components of the connection. After the concrete hardens the composite slab contributes to the connection rotational restraint against all additionally applied load.

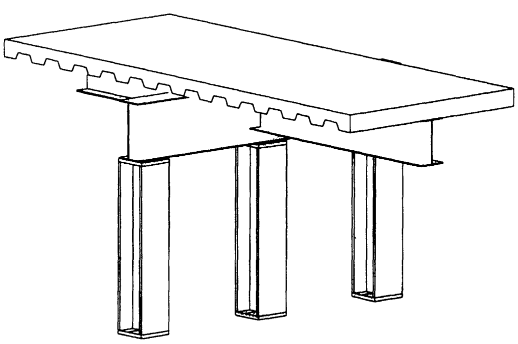

Fig. 11. Cruciform test specimen.

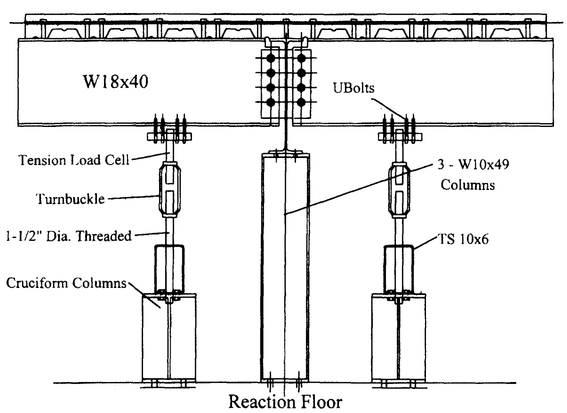

Fig. 12. Dead load simulation test setup.

The typical loading history for the test specimens was designed to simulate the loading history expected for unshored construction. Immediately after placement of concrete, the dead load frame was used to apply a simulated dead load which was left in place while the concrete cured. This load was based on the fresh concrete load for the beam in the hypothetical design (discussed previously) and was typically applied in one to two kip increments. The specimens were unloaded one or two days prior to the application of the simulated live load. After unloading, the dead load frames were removed and the larger live load frames were put into position. The positions of the dead load frames and live load frames with respect to the centerline of the girder varied from connection to connection and are indicated in Figures 6–9.

Once the live load frames were in position the connections were pre-loaded to a low load level and then unloaded to ensure all instrumentation and the test frames were operating properly. The specimens were then reloaded in one to four kip increments until the moment-rotation behavior of the connection started to become significantly non-linear or until it was necessary to unload for reasons associated with the test setup. Once the connection response became significantly non-linear, the specimen was unloaded and reloaded to determine the unloading and reloading stiffness characteristics of the connection. Subsequently, the load increments were based on both load and deformation with load application continuing until the connection failed.

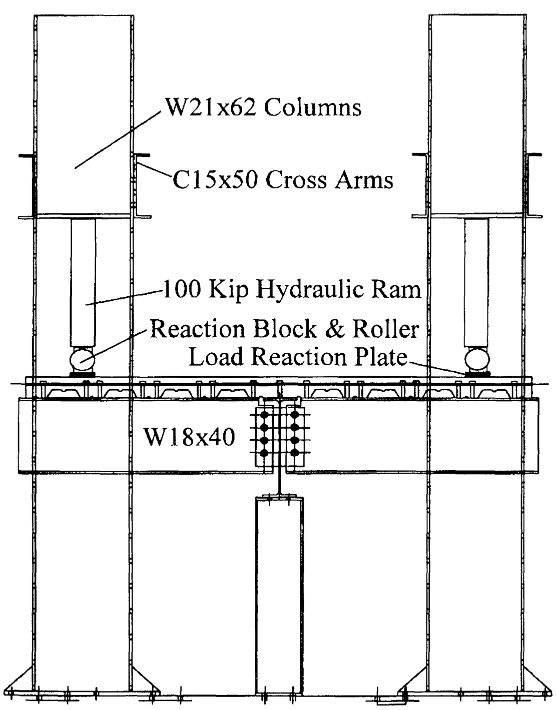

Fig. 13. Live load test setup.

Experimental Results

Joint Classification

Because of the two stages of connection behavior, associated with unshored construction, the problem of assigning stiffness classifications (rigid, semi-rigid, or pinned) to the connections becomes subjective. The stiffness classification depends on whether the designer chooses to look at the connection behavior from beginning to end or to look at the connection behavior independently in each stage of the construction. In addition, the stiffness classification will depend on what properties are chosen to normalize the data. The writers are unaware of any guidance on how to classify connections with two distinct stages of behavior.

The complete moment-rotation behavior for the north side of all the specimens from the beginning to the end of the testing is presented in Figure 14 along with the Eurocode 3[6] joint classification boundaries for braced frames. The behavior has been normalized using weighted averages of the nominal properties of the composite beam in the hypothetical design. The weighted moment of inertia (I_{cw}) is given by $I_{cw} = 0.4\, I_c^- + 0.6 I_c^+$ as recommended by Leon[19] where I_c^- and I_c^+ are the moments of inertia for the composite beam in negative and positive bending respectively. To be consistent, the weighted plastic moment capacity (ϕM_{pcw}) was also based on the same relative weighting between positive and negative moment capacity. Based on Figure 14 one would conclude that Connection #1 would be classified as a pinned partial strength connection while Connections #2 through #4 would be classified as semi-rigid partial strength connections.

The steel connection moment-rotation behavior for the north side of all the specimens is presented in Figure 15 along with the Eurocode 3[6] joint classification boundaries for braced frames. The steel connection behavior has been normalized using the nominal properties of the steel beam. Based on Figure 15 one would conclude that the steel Connections #1 through #3 would be classified as semi-rigid partial strength while steel Connection #4 may be classified as either

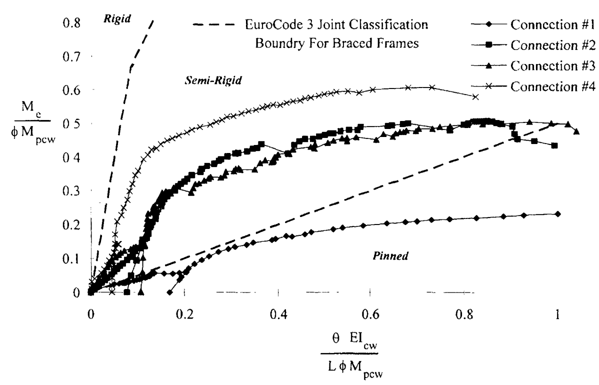

Fig. 14. Normalized complete M-θ behavior (north side of specimen).

Table 3. Measured Connection Behavior Properties				
	Con #1	**Con #2**	**Con #3**	**Con #4**
Steel Connection Behavior				
Initial Stiffness (k-in./mrad)	103	322	634	710
Point of Non-Linearity (mrad, k-in.)	3.6, 370	N/A	0.8, 490	N/A
Maximum Moment (k-in.)	432	730	810	827
θ After Removal of Dead Load (mrad)	6.9	1.6	2.3	0.95
Composite Connection Behavior				
Initial Stiffness (k-in./rad)	433	1,031	3,793	6,538
Point of Non-Linearity (mrad, k-in.)	8.1, 505	2.7, 1,060	2.6, 1,320	1.2, 1,430
Maximum Moment (k-in.)	1,367	2,884	2,861	3,435
θ at Maximum Moment (mrad)	30.7	18.7	18.5	12.8
Maximum Shear (kips)	32.5	51.5	59.6	71.6
Failure Moment (k-in.)	1,356	2,935	2,816	3,293
θ at Failure Moment (mrad)	26.5	24.9	23.5	17.0
Specimen Side that Failed	North	South	South	North
Mode of Failure	Distortional Buckling	Shear Stud Failure	Tension Rupture of Tension Plate	Beam Web Crippling

rigid partial strength or semi-rigid partial strength depending on how the data is interpreted.

The composite connection moment-rotation behavior for the north side of all the specimens is presented in Figure 16 along with the Eurocode 3[6] joint classification boundaries for braced frames. The composite connection behavior has been normalized using weighted averages of the nominal properties of the composite beam in the hypothetical design as previously described. Based on Figure 16 one would conclude that the composite Connections #1 and #2 would be classified as semi-rigid partial strength while composite Connections #3 and #4 would be classified as rigid partial strength.

General Characteristics of Connections

Various characteristic values of the connection behavior are presented in Table 3. The initial stiffness is based on a visual inspection of the moment-rotation data prior to a significant change in the initial slope. The point of non-linearity was chosen as the approximate point where a permanent deviation from the initial stiffness was noticed. The reader should note that in the writers' opinion the steel Connections #2 and #4 did not show significant non-linearity. The rotation of the connection after the dead load was removed and before the live load was applied is labeled "θ After Removal of Dead Load" in the table. These quantities as well as the point of maximum moment and shear are the average values measured for the two sides of the test specimen. The reader is reminded that the steel connections were not loaded to failure and that the maximum steel connection moment does not necessarily correspond with the ultimate moment capacity of the steel

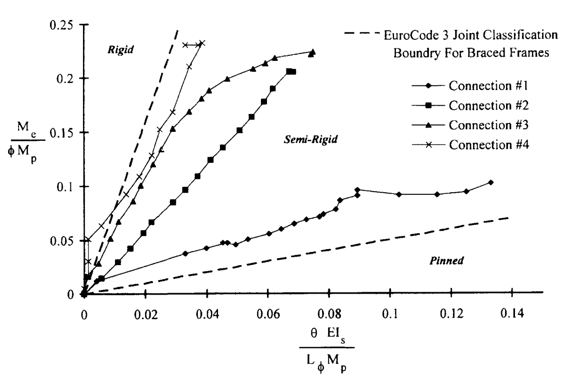

Fig. 15. Normalized steel M-θ behavior (north side of specimen).

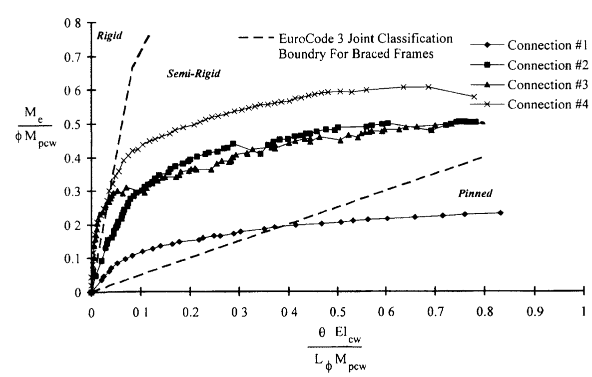

Fig. 16. Normalized composite M-θ behavior (north side of specimen).

connection. The values for the point of failure are values measured for the side of the specimen that failed. The point of failure was chosen as the point when rapidly decreasing moment associated with rapidly increasing rotations was first observed or when some part of the connection failed abruptly.

Based on a review of Table 3 the following observations can be made:

- The initial stiffness of both the steel and the composite connection behavior increased from Connection #1 to Connection #4.
- The moment capacity of the composite connections increased from Connection #1 to Connection #4 with the exception that Connection #2 and Connection #3 had approximately the same moment capacity.
- The rotation at failure decreased from Connection #1 to Connection #4.

In general connection stiffness and strength increased and connection ductility decreased as the connection details were modified from the simple details of Connection #1 to the more non-typical details of Connection #4.

Shear Lag

Reinforcing steel in the composite slab was instrumented with strain gages so that the load in each reinforcing bar could be determined. This information was then used to determine what shear lag effects were present in the 60-in. wide composite slab. A shear lag coefficient was used to measure the amount of shear lag present from the center of the specimen to the edges of the specimen. The coefficient is the load in the reinforcing steel near the center line of the beam divided by the load in the reinforcing steel nearest the edge of the composite slab (a shear lag coefficient of one indicates no shear lag). The shear lag coefficient vs. the connection moment is plotted in Figure 17 for Connections #1 through #4. Data from the dead load moment up to the moment where the reinforcing steel yielded (if it yielded) has been included.

As seen in the plot, an initial region of wide scatter exists. This scatter is the result of non-uniform concrete cracking across the width of the slab. As the cracking propagates with increasing moment the scatter tends to be reduced and reasonable trends in the data start to appear. For Connections #2 through #4 these trends suggest that the shear lag coefficient was typically between 1 and 1.2. For Connection #1 the coefficient started just above 1 and then increased to a maximum of about 1.5. The writers believe that the reason for the higher shear lag seen in Connection #1 was the result of over reinforcing the composite slab (12 #4 bars). Overall the typical values seen for the shear lag coefficient suggest that shear lag was not significant for the 60-in. wide composite slab.

Snug Vs. Fully Tightened Bolts

To determine what effect fully tensioned bolts vs. snug tight bolts had on the steel connection behavior, Connection #3 was loaded and unloaded twice prior to the composite slab being constructed. During the first loading all the bolts in the connection were left snug tight. During the second loading the bolts were fully tightened using the turn-of-the-nut method. The resulting moment-rotation behavior of the two load trials is presented in Figure 18. The initial response of the fully tightened connection was much stiffer than the initial response of the snug tight connection, although the fully tensioned connection started to exhibit a reduced stiffness at approximately 600 k-in., while the snug tight connections seemed to exhibit an increased stiffness. It is believed that the bolts in the fully tightened connection had started to slip while the bolts in the snug tight connection had gone into bearing.

Additional loading was not conducted to ensure the connection was not permanently damaged. After these comparison loadings the bolts of the connection were loosened, the specimen was returned to its original position, the bolts were re-tightened, and the composite slab was constructed before the typical loading sequence for the composite connections was carried out.

Individual Connection Test History

Connection #1

Connection #1 was loaded in three different stages: preload,

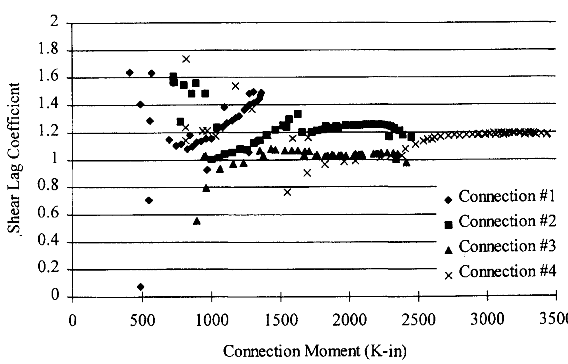

Fig. 17. Shear lag coefficient vs. connection moment.

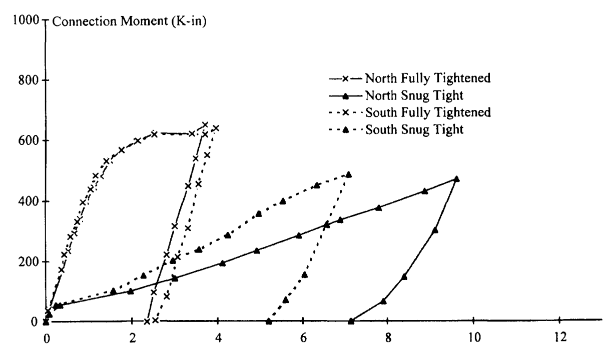

Fig. 18. Moment-rotation behavior of steel connection #3 under trial loads.

dead load, live load. The moment-rotation behavior for the entire load history is presented in Figure 19. The reason for the preloading stage was to ensure the test setup was working properly before the concrete was cast.

Preload

- Load was applied using dead load frames in one to two kips increments.
- The south side of the specimen was not as rotationally stiff as the north side. This led to a non-symmetrical response about the girder centerline.
- Load was increased to 15 kips (the design dead load) then the specimen was unloaded and reloaded.
- The specimen was unloaded and the composite slab was constructed.

Dead Load

- Immediately after the composite slab was cast the dead load frames were used to load the specimen up to 15 kips.
- After 28 days the specimen was unloaded.

Live Load

- Live load frames were used to load the specimen rapidly back up to the dead load moment (490 k-in.). Subsequently load was increased in one kip increments.
- First cracks were noticed at 830 k-in. and were parallel to the girder. One crack over the centerline of the girder and one crack approximately 12 in. on each side of the centerline of the girder.
- Specimen was unloaded and reloaded at 970 k-in. Response was basically linear but not as stiff as the initial stiffness (242 k-in./mrad vs. 432 k-in./mrad).
- Yielding around base of south connection plate was noticed at 1,200 k-in.
- Distortional buckling of the north beam was noticed at 1,360 k-in. The bottom flange of the beam started to rotate about a yield line that developed in the web of the beam near the bottom bolt in the single plate connection. This occurred before any reinforcing steel yielded.

Connection #1 had an excessive amount of reinforcing steel. Based on calculations using the AISC Specification[18] the nominal strength of the connection plate and bolts should have been able to handle the nominal yield force developed by the reinforcing steel. In addition, the beam web met the compact criteria for combined bending and axial load. The two factors that are believed to have led to the distortional buckling before the reinforcing yielded are: 1) the reinforcing yield strength was around 70 ksi rather than the nominal 60 ksi used in the calculations, 2) the compact criteria assumes a bending distribution through the entire web which is clearly not possible at the end of the beam with this type of connection. The results of this test indicate an upper limit to the amount of reinforcing steel that would be expected to yield in a connection of this type before distortional buckling of the section would occur.

Connection #2

Connection #2 was loaded in two different stages: dead load and live load. The moment-rotation behavior for the entire load history is presented in Figure 20.

Dead Load

- Immediately after the composite slab was cast the dead load frames were used to load the specimen up to the 15 kip design dead load. The moment-rotation behavior was very linear up to this load.
- After 28 days the specimen was unloaded.

Live Load

- Live load frames were used to load the specimen rapidly back up to the dead load moment. Subsequently load was increased in one kip increments.
- The first cracks occurred at 890 k-in. The cracks were parallel with the girder and were approximately located above the connections (bolt line in the single plate). The moment-rotation behavior remained fairly linear after cracking but the stiffness was reduced.

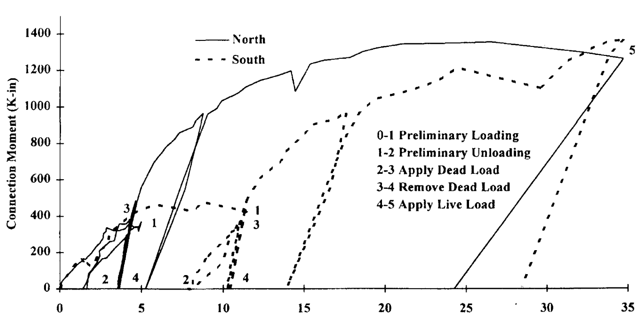

Fig. 19. Moment-rotation behavior of connection #1.

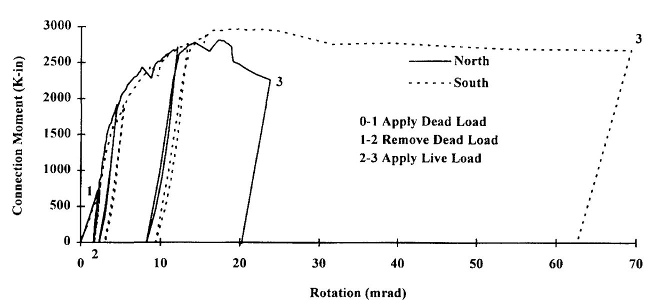

Fig. 20. Moment-rotation behavior of connection #2.

- A crack parallel to and over the girder centerline occurred at 1,700 k-in.
- Yielding around bolts attaching the beam to seat angle occurred at 1,850 k-in.
- The specimen was unloaded and reloaded in an attempt to correct problems with uneven deflections between the two sides of the specimen. Unloading and reloading stiffness was 881 k-in/mrad.
- First yielding of reinforcing steel at 2,500 k-in.
- Yield patterns around top bolts in the single connection plates occurred at 2,700 k-in.
- The specimen was unloaded and reloaded in an attempt to correct problems with uneven deflections between the two sides of the specimen. Unloading and reloading stiffness was 717 k-in./mrad.
- Cracking in the composite slab right around the points where the load was applied started occurring around 2,800 k-in.
- A shear stud failed above the south connection at 2,900 k-in. Despite the shear stud failure the connection still maintained substantial moment capacity (approximately 2,700 k-in) and maintained this moment through rotations in excess of 60 mrad. This deformation was accommodated through a combination of shear stud deformation and bolt bearing deformations in the beam web.
- The test was finally ended because of excessive uneven deflections between the two sides of the specimen. If the test had continued it was apparent that bolt tearout through the web of the south beam would have occurred.

Connection #2 was exhibiting excellent behavior before the shear stud failure. It is apparent that the addition of the seat angle increased rotational stiffness, moment capacity and stability of the connection compared to Connection #1. Because it was not possible to determine the load in the composite slab after the reinforcing steel yielded (strain gages on the reinforcing steel no longer gave reliable readings) it is unclear whether the shear studs were overloaded. The writer believes the most likely reason for the shear stud failure is a poor quality stud weld. The reasoning for this conclusion is as follows. The reinforcing steel in Connection #3 (which is the same as Connection #2) also fully yielded but no shear studs failed despite the fact that Connection #3 had one less shear stud than Connection #2 on each side of the specimen. Because the shear studs in Connection #3 must have been loaded more heavily than the shear studs in Connection #2 it stands to reason that there must have been one or two poor quality stud welds in Connection #2.

Connection #3

Connection #3 was loaded in three different stages: snug vs. fully tightened bolts comparison loadings, dead load and live load. The moment-rotation behavior for the entire load history, except for the comparison loading, is presented in Figure 21. The comparison loading was described previously.

Dead Load

- Immediately after the composite slab was cast the dead load frames were used to load the specimen up to the 17 kip design dead load.
- After 28 days the specimen was unloaded.

Live Load

- Live load frames were used to load the specimen rapidly back up to the dead load moment. Subsequently load was increased in one to two kip increments.
- The first crack occurred at 1,340 k-in. The crack was parallel to and over the girder centerline .
- Additional cracks in the composite slab above the connections as well as some yielding around the toe of the south beam occurred at 1,700 k-in.
- Tension plates started to yield at 1,750 k-in.
- The specimen was unloaded and reloaded in an attempt to prevent problems with rotation measurement. Unloading and reloading stiffness was 1,349 k-in./mrad.
- First yielding of the reinforcing steel occurred at 2,440 k-in.
- Yielding around the bolts connecting the beam flange to the seat angle developed at 2,800 k-in.
- The tension plate on the south side of the specimen failed in net section tension rupture. The failure occurred through the bolt hole closest to the girder web.

The moment-rotation behavior of Connection #3 was excellent up until the tension plate started to neck down and finally rupture. Necking at the net section appeared to start occurring at 20 mrad and rupture occurred around 40 mrad. There was a substantial loss in moment capacity between when necking started and rupture occurred. One possible way to increase the ductility of this type of connection would be to design the tension plate such that a significant amount of bolt bearing

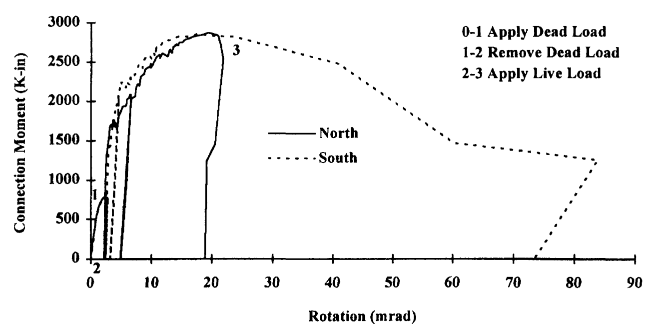

Fig. 21. Moment-rotation behavior of connection #3.

deformation occurs prior to any sudden failure such as tension rupture or bolt shear.

Connection #4

Connection #4 was loaded in two different stages: dead load and live load. The moment-rotation behavior for the entire load history is presented in Figure 22.

Dead Load

- Immediately after the composite slab was cast the dead load frames were used to load the specimen up to the 17 kip design dead load. The moment-rotation behavior should have been linear up to this load. The writers believe the jumps seen in the initial data might have been caused by a lack of sensitivity of the potentiometers used to measure the connection rotation.
- After 28 days the specimen was unloaded.

Live Load

- Live load frames were used to load the specimen rapidly back up to the dead load moment. Subsequently load was increased in two to three kip increments.
- The first crack occurred at 1,300 k-in. The crack was parallel to the girder centerline and was located approximately 8–10 inches south of the girder centerline.
- Yielding patterns near the toe of the beams adjacent the girder started appearing at 1,400 k-in.
- Tension plates started forming plastic hinges at 2,600 k-in. as the beam end started to drop vertically. The hinges were located adjacent to the top flange of the girder and the top flange of the beams.
- Yielding patterns around the beam toes started to spread into the web at 2,800 k-in.
- At approximately 3,300 k-in. two hinges formed in the north seat angle, one located along the toe of the seat angle and the other along the line of the erection bolts. It was noticed that the steel deck above the south beam had started to uplift in the region just above the connection.

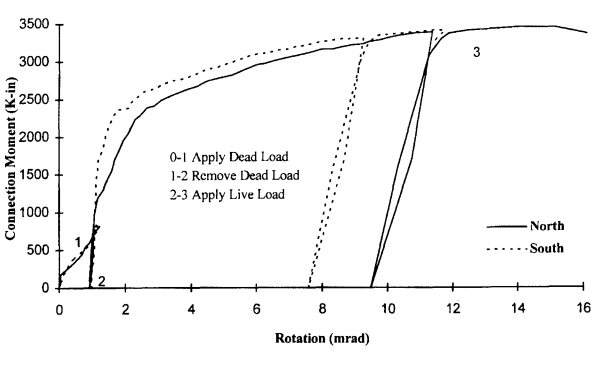

Fig. 22. Moment-rotation behavior of connection #4.

- The specimen was unloaded and reloaded. Unloading and reloading stiffness was 1,773 k-in./mrad.
- As a result of the hinges in the tension plate and the seat angle the north beam was dropping vertically as additional load was applied.
- The north beam web started crippling at 3,400 k-in. As the beam web started to cripple the top of the beam started dropping vertically rapidly. Finally the welds holding the tension plate to the top beam flange ruptured and the test was ended.

Connections #1 through #3 all failed because of the moment resisted by the connections. Connection #4 was the first connection to fail because of the shear load. The design of a seated connection like this for shear has not been investigated to the best of the writers knowledge. If a connection like this is to be used additional research on seat angle design will be required.

SUMMARY AND CONCLUSIONS

Advancements in design methods and construction materials are resulting in shallower and lighter composite floor systems. As a result, serviceability issues, rather than strength considerations, often control designs. Partial continuous composite floor systems may be one method by which some serviceability characteristics of floor systems can be improved. PR beam-to-girder connections are being investigated as a means by which this partial continuity can be developed. The development of these connections is an extension of the PR composite beam-to-column connection research that has been conducted in recent years.

Four PR composite beam-to-girder connections (Connection #1 through Connection #4) were tested experimentally. The behavior exhibited by these connections indicated that simple steel beam-to-girder connections, some with and some without any significant rotational stiffness or moment capacity of their own, can be turned into very stiff connections with moment capacities near the plastic capacity of the steel beam by adding a reinforced composite slab.

Additional conclusions include:

1. Most composite beams are constructed using unshored construction techniques. This leads to two distinct stages of connection behavior. It is currently unclear how connections with two stages of behavior should be classified.
2. For the 60-in. wide composite slabs used on the four specimens shear lag was not significant.
3. Connections using fully tensioned bolts will typically be characterized by a higher degree of rotational restraint than connections which use only snug tight bolts.
4. A semi-rigid connection created by combining a simple steel connection, such as a shear tab, and a reinforced composite slab may lead to an instability such as lateral

buckling of the beam bottom flange as seen in Connection #1. To ensure that instability does not occur, and at the same time increase the rotational stiffness of the connection, a seat angle or a plate needs to be attached to the bottom flange. If the angle or plate is not provided, then detailing of the reinforcing steel should be given careful consideration (i.e., the amount of reinforcing steel should be detailed so that the reinforcing steel yields prior to the occurrence of any local instabilities in the connection).

5. To ensure that the connection has sufficient ductility, the details of the steel connection need to be given careful consideration. If the steel connection is too stiff and does not allow the reinforcing steel to yield (as in Connections #4), then the connection will likely fail at relatively low rotations compared to a connection in which the reinforcing steel fully yields (as in Connections #2 and #3).
6. Accounting for the rotational restraint provided by these connections will lead to decreased deflections and moments. This, in turn, should allow more efficient designs and possibly an eventual reduction in costs.

FUTURE STUDIES

The work described in this paper represents the initial results of a multi-year study of PR composite beam-to-girder connections. Based on the work to date the plan for future studies will focus on a limited number of connections (the thought at present being two connections like Connections #2 and #3). These connections will be evaluated and developed in detail, ensuring that they possess significant rotational stiffness both prior to and after concrete placement. The evaluation will include finite element modeling, parametric studies, and experimental verification. Following these aspects of the work, design recommendations will be formulated.

In addition to the investigation of the specific connections, a general economic study will be conducted. This study will address issues such as when should semi-rigid beam-to-girder connections be considered, what parameters influence the economy, what design constraints limit or enhance the efficiency of the design. Evaluating the composite floor system in this manner should result in efficient application of the semi-rigid composite beam-to-girder connections and in general increase the performance and efficiency of steel framed floor systems.

ACKNOWLEDGEMENTS

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CORRECTIONS

A Practical Approach to the "Leaning" Column

Paper by LOUIS F. GESCHWINDNER
(4th Quarter 1994)

The following corrections should be made in the paper as published. On page 146, second column, the equation should be

$$K_n^2 = \frac{I_i}{57}\pi^2 \frac{207 + 0.0724(57)}{2.40 I_i} = 15.23$$

In Figure 11, the spring stiffness in terms of G should be as follows

$$K = \frac{6EI_c}{G_B L_c}$$

This correction in stiffness yields a different deflection in Example 1, page 147, line 11 where $\Delta_{oh} = 2.1307$. The results of this change and correcting the value of ΣQ in the equation as published yields, from Equation 16

$$K_i = \pi\sqrt{\frac{20{,}000(238)}{(16(12))^3}\frac{207}{57}\frac{2.1307}{5}\left[1 + \frac{0.216(207 - 105)}{207}\right]} = 3.972$$

and from Equation 17, $K_i = 3.942$. Carrying out the correction for spring stiffness in Example 3, page 149, column 1, yields $\Delta_{oh} = 1.096$ and from Equation 16, $K_l = 2.166$ and $K_2 = 1.028$ while Equation 17 yields $K_l = 2.130$ and $K_2 = 1.011$.

The author wishes to thank those readers who took the time to communicate some of these corrections.