Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 60 (1990)

Artikel: Semi-rigid composite connections

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DOI: https://doi.org/10.5169/seals-46516

Semi-Rigid Composite Connections

Assemblages mixtes à rigidité partielle

Verschiebliche Verbindungen in Rahmentragwerken mit Verbundträgern

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SUMMARY

Semi-rigid composite connections are traditional steel frame connections in which the additional strength and stiffness provided by the floor slab has been incorporated by adding shear studs and slab reinforcement in the negative moment regions adjacent to the column. Ten full-scale semi-rigid connection subassemblies were recently tested under both monotonic and reverse cyclic loads. The results of those tests indicate that these connections can provide adequate lateral stiffness for buildings of up to ten stories in areas of low to moderate winds.

RÉSUMÉ

La présence de connecteurs à cisaillement et d'armature dans les zones de moments négatifs dans les dalles de planchers en béton à proximité des poteaux augmente la résistance et la rigidité dans les assemblages simples d'ossatures métalliques. Récemment dix assemblages mixtes à rigidité partielle de grandeur nature ont été mis à l'épreuve en les soumettant à des charges monotones croissantes et à des charges cycliques. Les résultats de ces expériences démontrent que de tels assemblages peuvent être utilisés dans des édifices jusqu'à dix étages situés dans des zones de vents d'intensité modérée.

ZUSAMMENFASSUNG

Verschiebliche Verbindungen in Rahmentragwerken mit Verbundträgern sind gewöhnliche Stahlverbindungen welche weiter mit geschweissten Dübeln und Betonarmierung verstärkt und versteift werden, so dass die Deckenplatten und die Stahlkonstruktion im Verbund wirken. Zehn Versuche an Verbindungskörpern mit verschieblichen Verbindungen wurden mit monotoner und wiederholter Belastung durchgeführt. Die Ergebnisse dieser Versuche wurden dann in einer parametrischen Studie von Rahmentragwerken angewandt, und es wurde gezeigt, dass diese Verbindungen für bis zu zehnstöckige Rahmen in Gegenden moderater Windkräfte genügend biegesteif sind.

1. INTRODUCTION

The strength, ductility and stiffness of ordinary steel frames can be significantly improved by composite action. This can be achieved by providing a few continuous reinforcing bars across the column lines and insuring full composite action through the use of sufficient shear studs in the beams. The connections can range from very weak to almost rigid, and represent variations of typical steel connections used in the U.S.A. The data generated in this study indicates that unbraced frames up to 10 stories can be erected in zones of low to moderate seismicity , and that significant economies in steel can be achieved by utilizing this structural system.

2. SEMI-RIGID COMPOSITE CONNECTIONS

Ten full-scale semi-rigid composite connection subassemblies have been tested under both monotonic and reverse cyclic loads. Semi-rigid composite connections are traditional steel frame connections in which the additional strength and stiffness provided by the floor slab has been incorporated by adding shear studs and slab reinforcement in the negative moment regions adjacent to the columns. Four different types of connections have been tested (Figure 1):

(1) Seat and web angles (Type I): These consists of a typical seat angle connection, where significant moments can be transmitted at the column face by the slab reinforcement as the tension part of the couple and the angle as the compression member. Under reversed cyclic loading this type of connection is weak if positive moments are present because the angle will pullout at relatively low loads. The strength of the connection is controlled in negative moment by the amount of slab steel and in positive moment by the size (primarily thickness) of the angle. Very stiff connections can be obtained by using large friction bolts and thick angles in the web and seat. Figure 2 shows a typical moment-rotation curve for one such connection under reversed cyclic loads.

(2) Bottom welded plate plus web angles (Type II): This represents the stiffest economical semi-rigid connection possible. The bottom angle, which in Type I connections was the weak link, has been substituted with a welded plate. This plate carries the tensile and compressive forces basically as axial loads, resulting in a very stiff and non-degrading connection. The same results can be achieved by welding the plate to the column in the shop and bolting it to the beam with high strength friction bolts in the field. The welds must be detailed to insure full transfer of moment and to eliminate the possibility of weld fracture. This connection offers very large initial stiffnesses and symmetrical behavior under cyclic loading.

(3) Seat angle (Type III): This connection is similar to Type I, except that the web angles are missing. This results is a connection with a much flatter inelastic region, because there are no web angle to provide additional restraint once the seat has yielded. In addition careful attention must be paid to the stability of the bottom angle and the beam web. As for Type I, the thickness of the seat angle is the controlling parameter.

(4) Web angles (Type IV): This connection is a variation on the simplest shear connection used in all-steel frames. Although the angles are relatively weak, the moment capacity of the composite connection can be substantially improved by increasing the thickness of the angle and lowering its position towards the bottom of the beam web. Since the web angles are carrying both

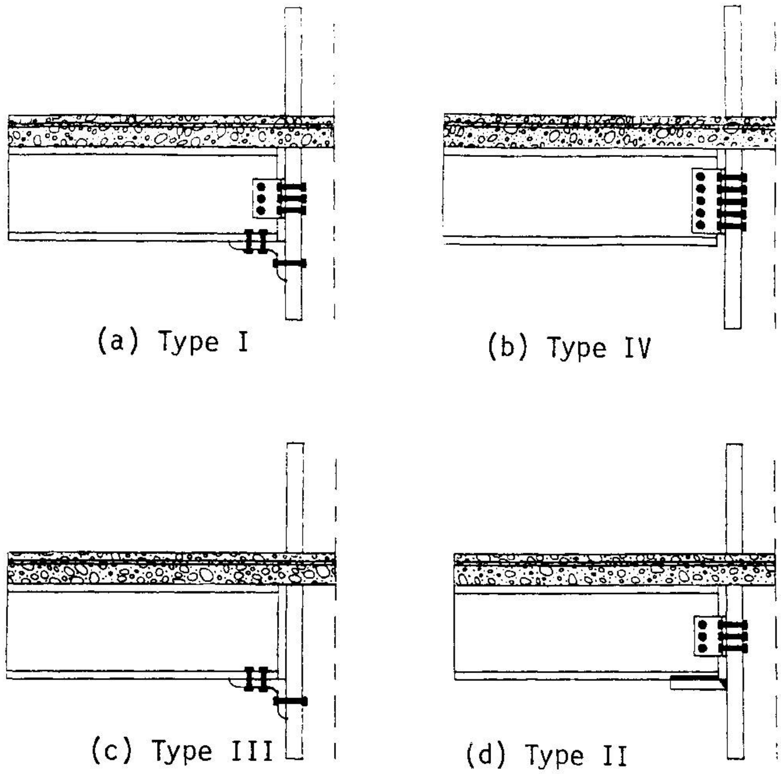


Figure 1 - Different semi-rigid composite connections.

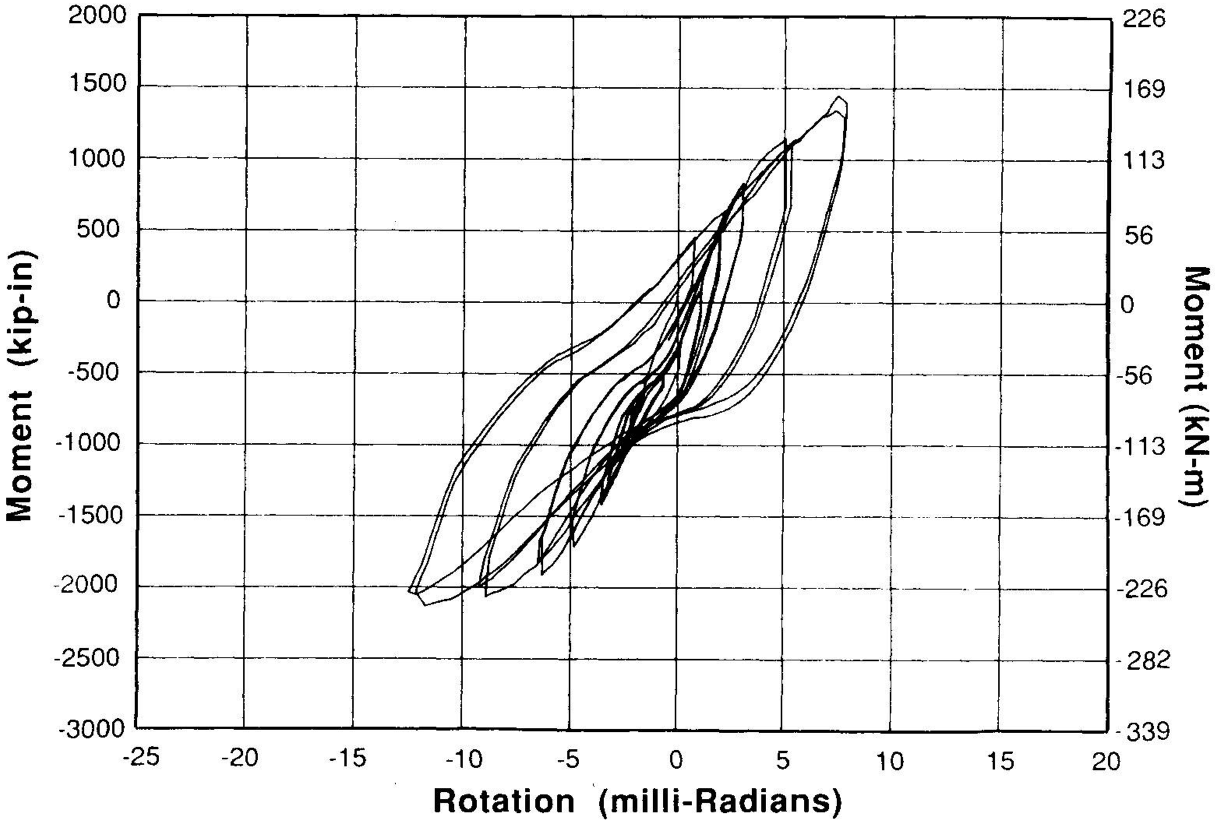


Figure 2 - Typical cyclic response for a Type I connection.

shear and moment care must be taken to prevent any type of block shear failure.

3. ANALYTICAL STUDIES

For analysis and design purposes, it is imperative to have a good knowledge of the moment-rotation characteristics of the connection. This is not a simple task, since the non-linear behavior of the connection must be carefully modelled. In order to obtain a parametric equation for Type I connections, relating the size of the angle, amount of slab steel, beam depth, and web angle size, a large number of non-linear finite element models of semi-rigid composite connections were run. These computational studies led to the development of a complete moment- rotation equation, and for the ultimate and yield moments. For the case of Type I connections the equations for the yield and ultimate moment take the form:

$$M_y = 0.170 \ [\ (4 \times A_{rb} \times F_{yrb}) + (A_{sL} \times F_{ysL})\ ]\ (d + y_3\ )$$

$$M_u = 0.245 \ [\ (4 \times A_{rb} \times F_{yrb}) + (A_{sL} \times F_{ysL})\ ]\ (d + y_3\ )$$

where M_y is the yield moment, M_u is the ultimate moment, A_{rb} is the area of the slab reinforcement, F_{yrb} is the yield stress fro the slab reinforcement, A_{sL} is the area of the seat angle, F_{ysL} is the yirld stress for the seat angle, d is the depth of the beam and y_3 is the height of the shear studs. All units in these equations are in kips and inches. These two equations allow the use of a bilinear approximation to the exact curve, considerably simplifying the computational task of analyzing a large frame.

4. DESIGN EXAMPLES

To show the potential of the semi-rigid composite system and to illustrate extremes of behavior, two frames were designed as both rigid and semi-rigid. The first frame was a four-story, three-bay frame with 9.60 m bays and 4.27 m story heights. The second frame was an eight-story, three-bay frame with 8.53 m bays and 4.27 m story heights. The frames were designed for ultimate strength, and checked under live loads for lateral deflection. Figure 3 shows the entire load-deflection curves to collapse, normalized to a load factor of 1.00 (full wind load = 1.00) and a lateral deflection of L/400 (maximum commonly allowed in the U.S.). Figure 4 shows the sequence of plastic hinge formation for both frames.

For the case of the four-story frame, the beams for both the rigid and semi-rigid frames were the same. Thus the only differences are the rigidity of the connection and the composite action. For these frames the behavior is very similar up to collapse, with the semi-rigid frame being slightly more flexible. This demonstrates that semi-rigid composite frames possess good stiffness and strength characteristics.

For the case of the eight-story frame, the beams in the semi-rigid composite frame were proportioned to be about 2/3 of the weight of those in the rigid frame. For this case the rigid frame can attain a load factor of over 3.5, while the semi-rigid composite frame only attains about 2.0. Moreover the deflections in the semi-rigid composite frame are much bigger, and it barely meets the L/400 drift criteria. From inspection of Figure 4, it is clear that as soon as the first level hinges in the semi-rigid composite frame the lateral deformations become very large and P-Δ take over.

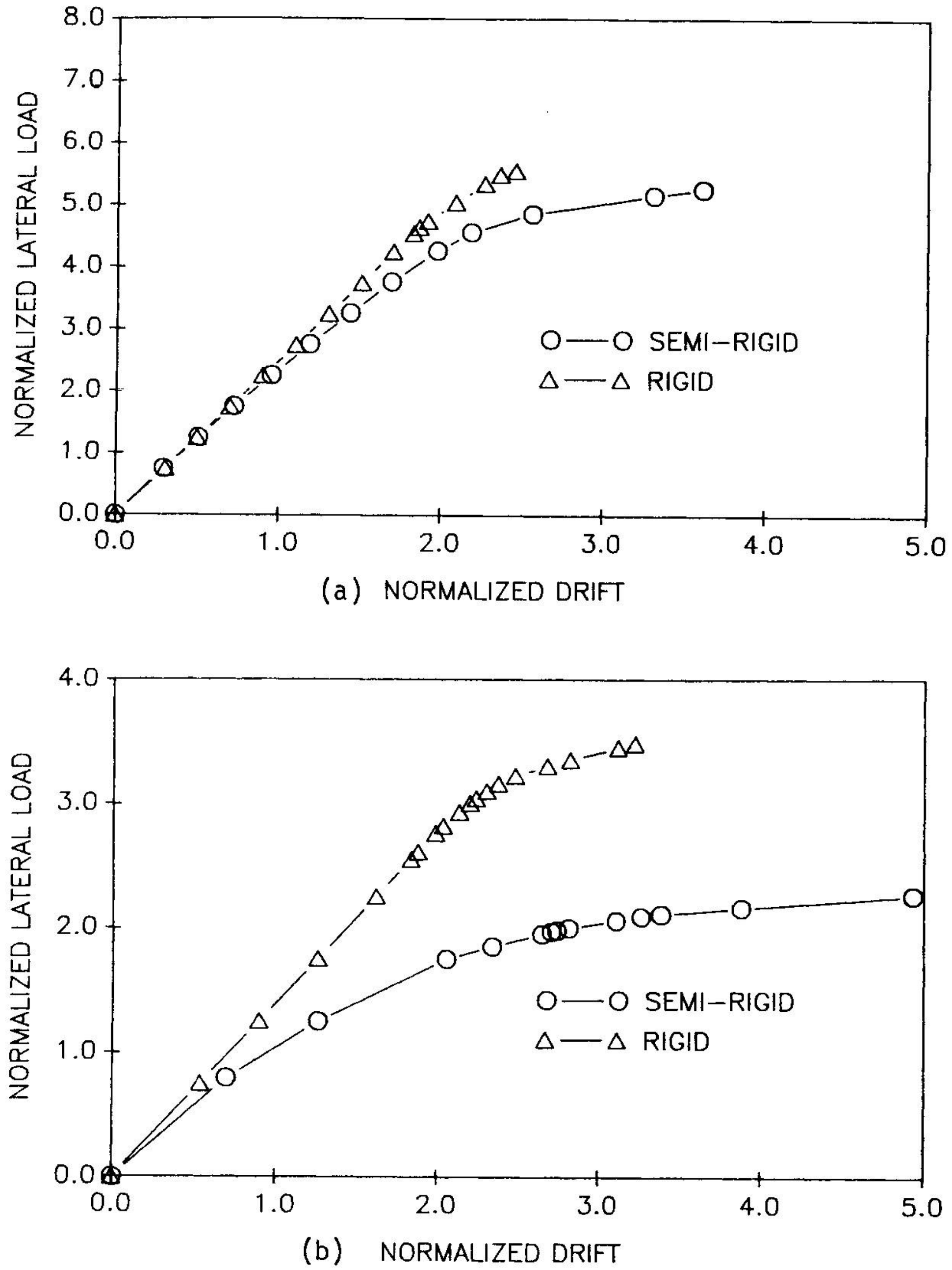


Figure 3 - Normalized drift vs. lateral load for (a) four-story and (b) eight story structures.

5. SUMMARY

The work reported here indicates that :

[1] Significant increases in strength and stiffness of simple connections can be achieved by providing some continuous slab reinforcement over the column lines. The added cost is only that of a few supplementary slab bars, since no additional shear studs or bolts are required.

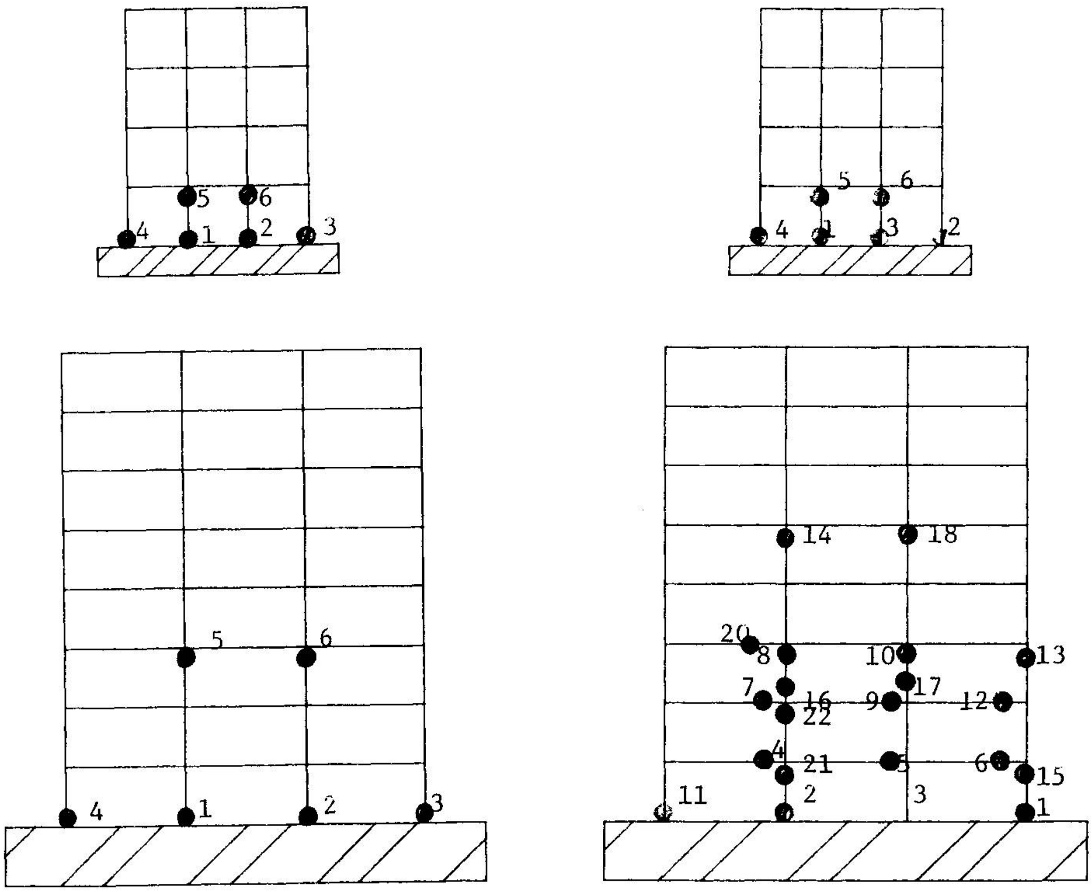
[2] Semi-rigid composite connections provide a large amount of ductility and extra reserve capacity. A structure designed as semi-rigid composite would provide a very high degree of protection against progressive collapse failures.

[3] The failure mechanisms for these connections are well understood and design procedures to prevent them have already been developed.

[4] A semi-rigid composite system can be very economical in braced frame construction if the design live loads exceed the dead loads by a factor of at least two.

6. ACKNOWLEDGEMENTS

The work described in this paper was carried out under the generous sponsorship of the American Institute of Steel Construction and the National Center for Earthquake Engineering Research at the State University of New York at Buffalo.



(a) Semi-rigid composite (b) Rigid

Figure 4 - Sequence of plastic hinge formation for example frames.