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Behavior of open-web steel joists in moment-resisting frames

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ABSTRACT

Many lightweight steel structures are constructed using open-web steel truss joists as part of the gravity load carrying system. Recently, these joists have also been used as part of moment-resisting frames. These frames are relatively new adaptations of older gravity framing systems and are unusual in that they forego the use of standard rolled steel shapes in favor of proprietary structural elements. The behavior of the typical moment-resisting connections between vertical and horizontal framing members of these systems are complex and bear little resemblance to the connections in more standard systems. Since these systems have little or no history of seismic resistance, little is known about their actual behavior during earthquakes. Frame drifts and ductility are affected by many construction features not found in more typical systems. This paper presents analytical studies of open web moment-resisting frames that explore features of these systems related to vertical and lateral loading. Experimental data on the behavior of components is also presented. Recommendations are made regarding detailing of the connections. The analysis presented in this paper is based on reviews of several structures which have specified steel truss joists in moment frames in both single-story and multi-story buildings.

SYSTEM DESCRIPTION

The moment frames which are the subject of this study have several typical features: open-web steel truss joists, square tubular columns and welded connections. Open-web steel joists are generally proprietary items which are ordered from a manufacturer. Framing plans usually specify truss joists at regular spacing in one direction and truss joists as girders in the other direction. Truss joists used as girders are larger versions of the same system

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and will be described as truss girders for this study. The details characterized below apply to both truss joists and truss girders.

The sizes of the components and the details of the fabrication are not available to the engineer. The designer usually supplies the loads, spans and other design criteria to the manufacturer and the sizes and details of the members are then determined by the manufacturer. Many of these joists are composed of steel angles for the top and bottom chord member and steel angles or round bars as the diagonal web members as shown in Fig. 1. To achieve the

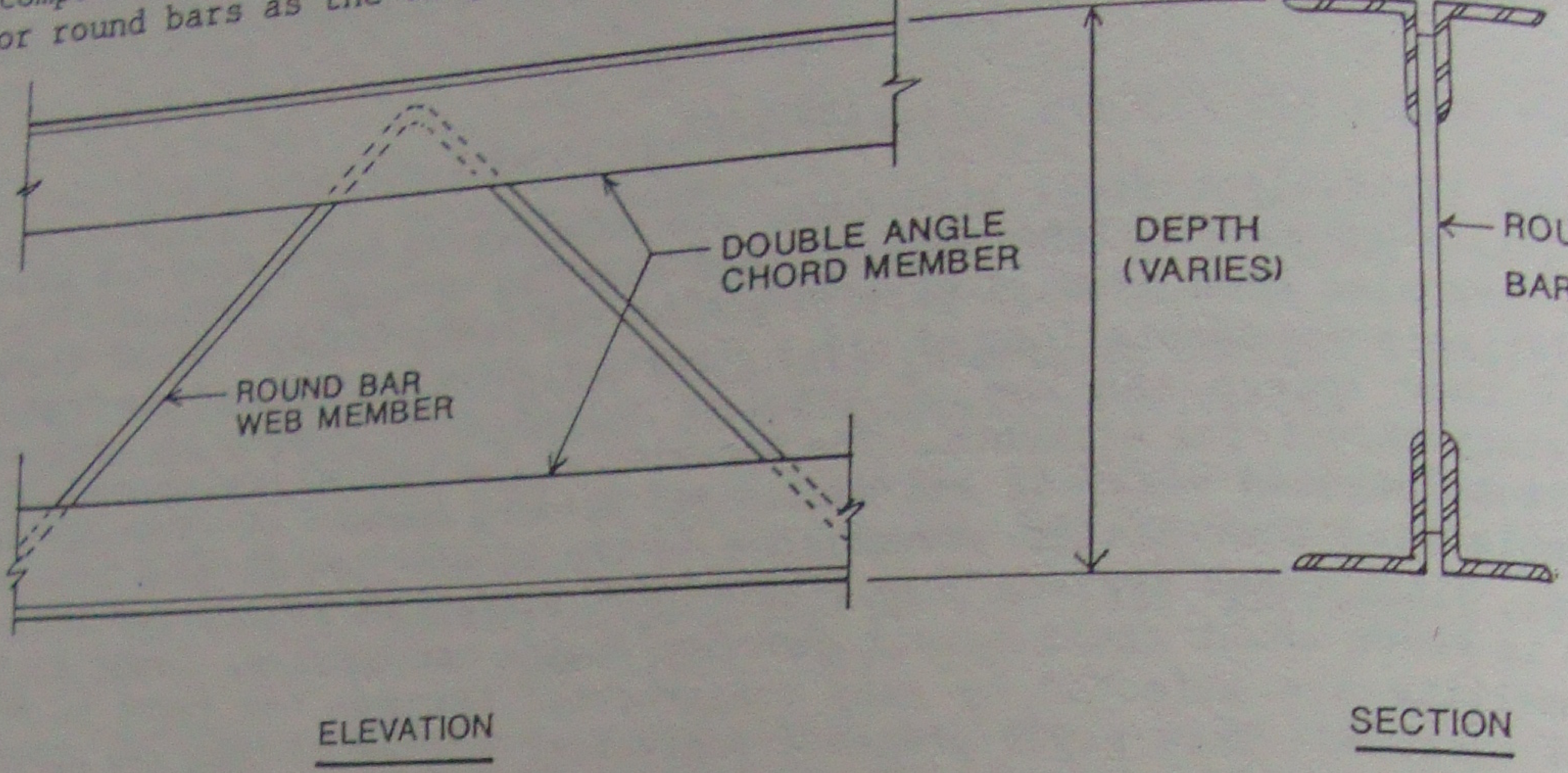

Figure 1. Detail of Open-Web Joist

greatest economy, the chord members are usually very light sections, with thicknesses as thin as 1/8 inch.

Square tubular columns are typically used for the vertical elements of these structural systems. This allows the column to be part of the lateral force resisting system in both directions and also provides efficiency in axial load carrying capacity. The tubes are supported on a steel base plate which can be set below the level of the floor slab. Where this occurs, the base plate may act with the floor slab to provide fixity at the base of the column. If crack control joints are provided in the slab around the column, the slab may not provide supplemental fixity.

CONNECTIONS

The connection details are critical elements of the design of truss joists within moment frame structures. In contrast to wide flange frames, separate connections are provided for the top and bottom chords of the truss joist. A force couple develops between the top and bottom chord connections. The design of the connections can be provided by either the engineer or the fabricator. When designing a building using this system, the responsibility for the design of the connections should be clearly specified in order to insure that the connections will be designed for all anticipated loads.

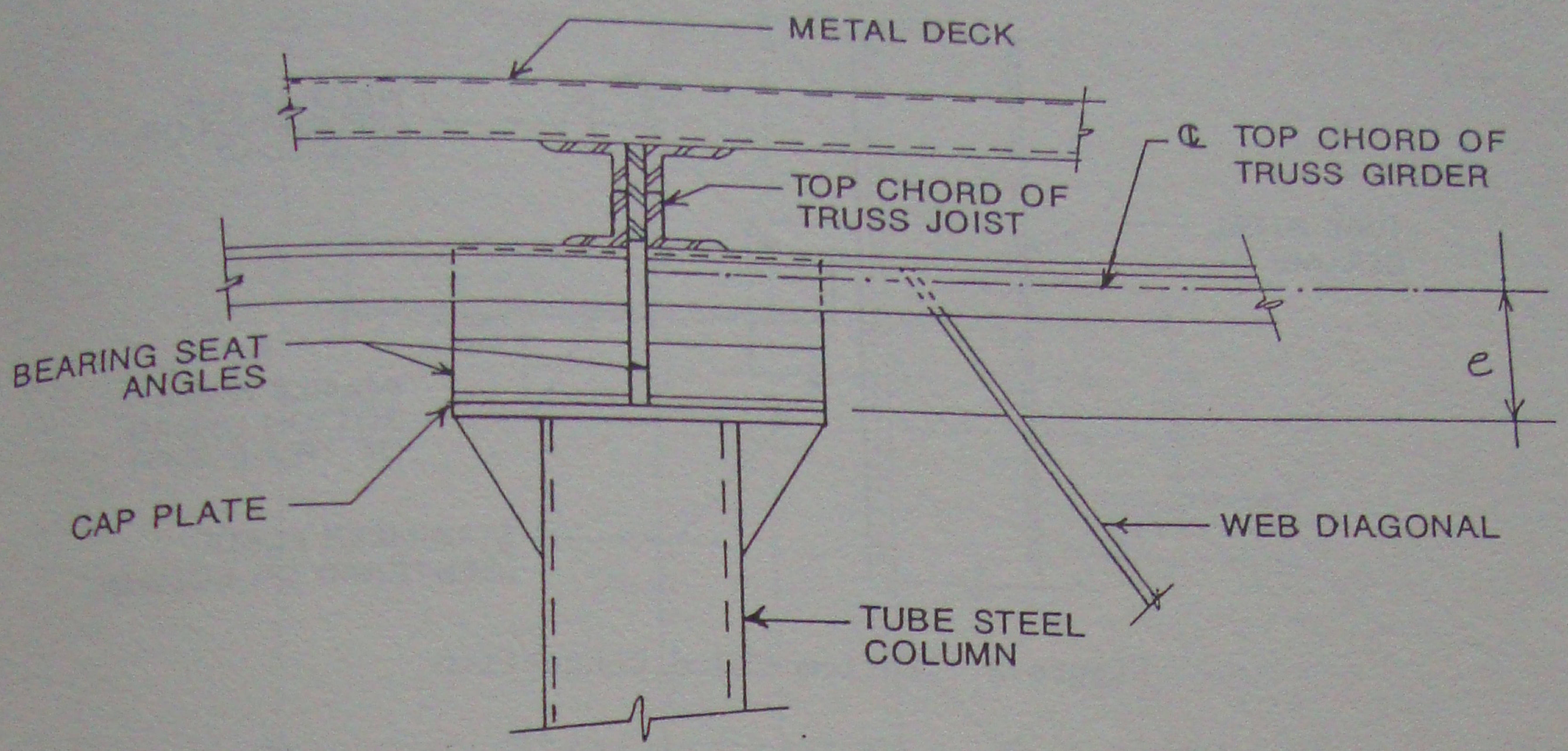

Figure 2. Top Chord Connection

In single story structures, or the top floor of multi-story structures, the top of the columns are typically fitted with a cap plate. The end of the truss-joist is supported vertically by inverted bearing seat angles which rest either on truss girders or on top of this cap plate. An example of this connection is shown in Fig. 2. In this connection, there is an eccentricity which is developed between the force in the top chord of the joist and the weld to the bearing seat angle and the column cap plate. It is important that the design of the welds of the joist bearing angles and the girder bearing angles account for this eccentricity.

In addition, the eccentricity of the top chord connection also causes bending in the top chord member of the truss joist. The bending moment is greatest in the length of the top chord between the end of the joist chord and the first panel point of the diagonal web member. Although the truss joists are proprietary and are not designed by the engineer, the engineer should confirm that the fabricator understands the behavior of the truss resulting from this eccentricity.

A typical connection of the bottom chord to the column consists of a flat stabilizer plate, oriented vertically and welded to the column face. The double angle bottom chord is field welded to this plate, usually after the application of all dead loads, as shown in Fig. 3. This allows the joist to act as a simply supported beam for resisting dead loads, and is intended to provide a fixed end condition for resisting live and lateral loads. Under the influence of lateral load, this connection develops a concentrated force from the chord on the middle of the wall of the tube where the stabilizer plate is attached. Where this concentrated force would cause excessive bending stresses in the wall of the tube, a doubler plate may be added to the face of the tube.

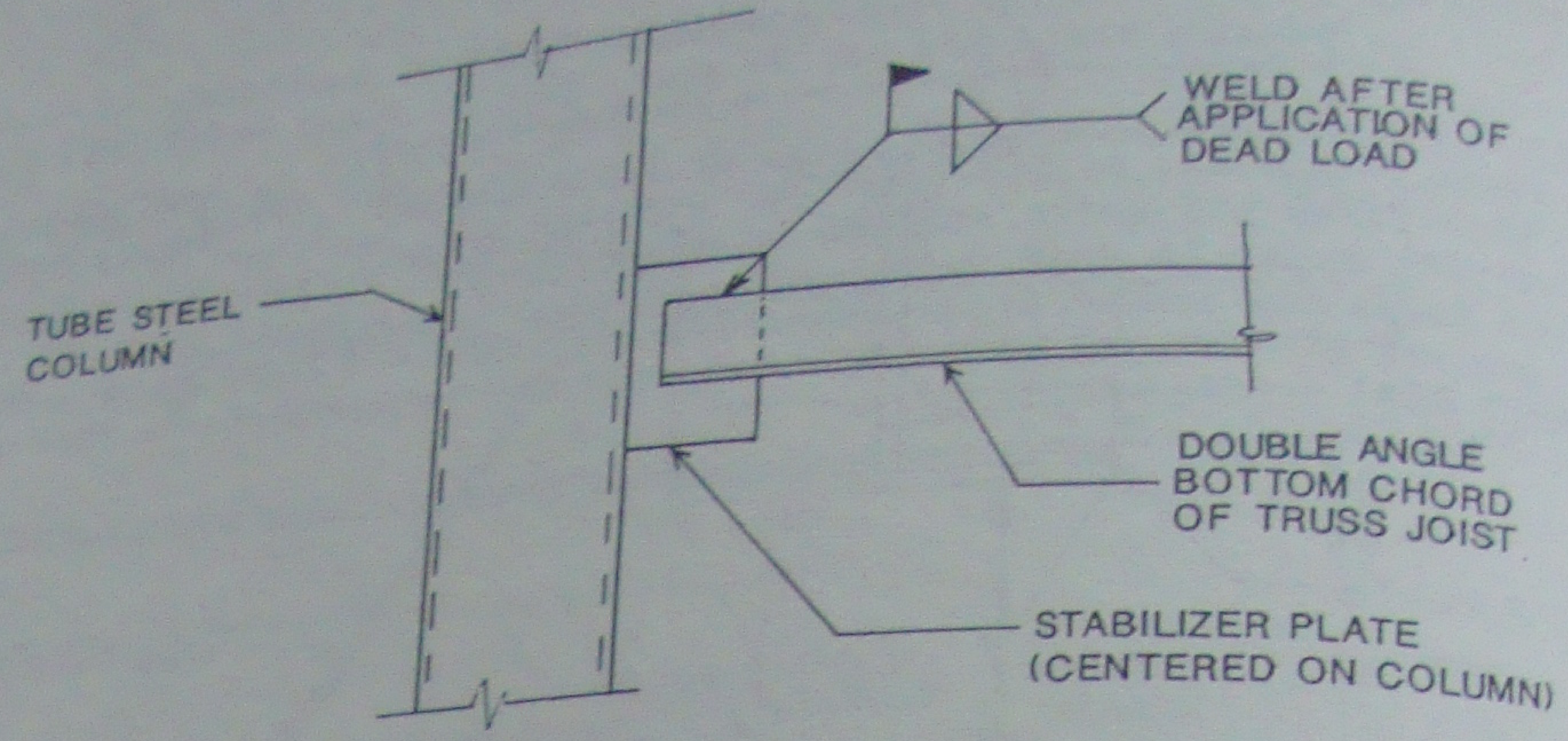

Figure 3. Bottom Chord Connection

The allowable and ultimate loads for this type of bottom connection cannot be easily determined by conventional analysis procedures. Various approximations of the effective width of the wall of the tube suggest substantially different allowable loads for the tube. Finite element analysis of this connection has been used to approximate the load at which first yielding could occur and also to indicate the initial stiffness of the connection. This analysis, however, does not indicate how much reserve strength is in the connection nor the stiffness of the connection after initial yield. Yield line methods provide indications of total strength but cannot identify first yield or the stiffness characteristics of the connection.

To verify the actual strength and stiffness of this connection, two test specimens were fabricated and tested. Two 40 inch long tube steel columns, each 12 inch square with 1/2 inch thick walls were fitted with stabilizer plates, as shown in Fig. 4. One the specimens was fitted with a 1/2 inch thick doubler plate to strengthen the tube wall to which the stabilizer plate was welded, as shown in Fig. 5. The testing procedure called for the application of monotonically increasing load to each specimen until first local yield was indicated by strain gages. The load was then removed. Each specimen was subsequently cycled 6 times to the load corresponding to first local yield to assess the response of the welds and the tube face to pseudo-seismic loading. Finally, the specimens were loaded to failure. A plot of the load-defection curve for the specimen with the doubler plate is shown in Fig. 6 and the plot for the specimen without the doubler plate is shown in Fig. 7. The load at which first yield was measured is 26.5 kips and 17.8 kips for the two specimens, as shown on the load-deflection plots.

The load-displacement curves for the test specimens indicate that these connections have a considerable ultimate capacity. However, the connections are very soft and substantial deflections are required in order to engage the

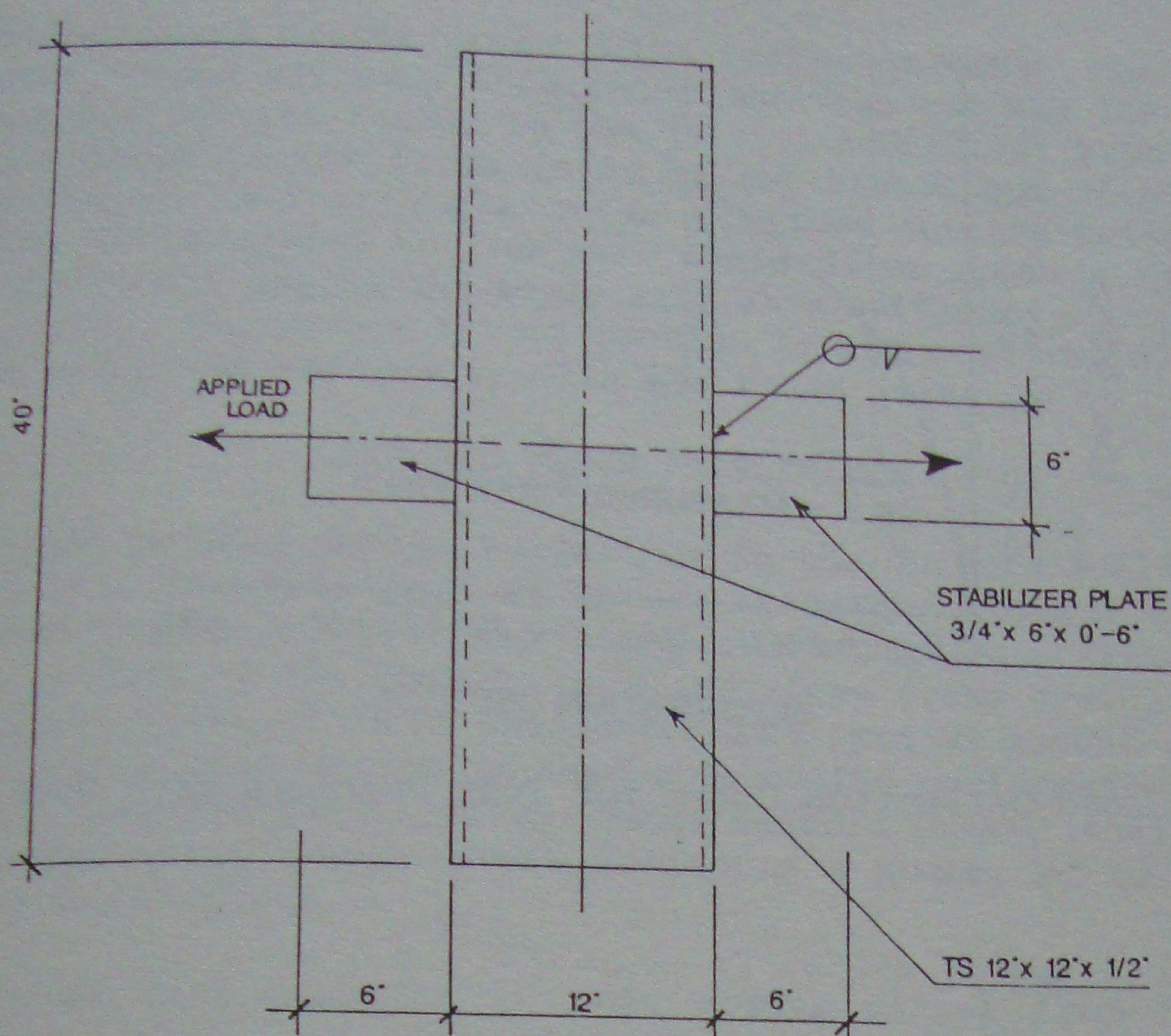

Figure 4. Test Specimen

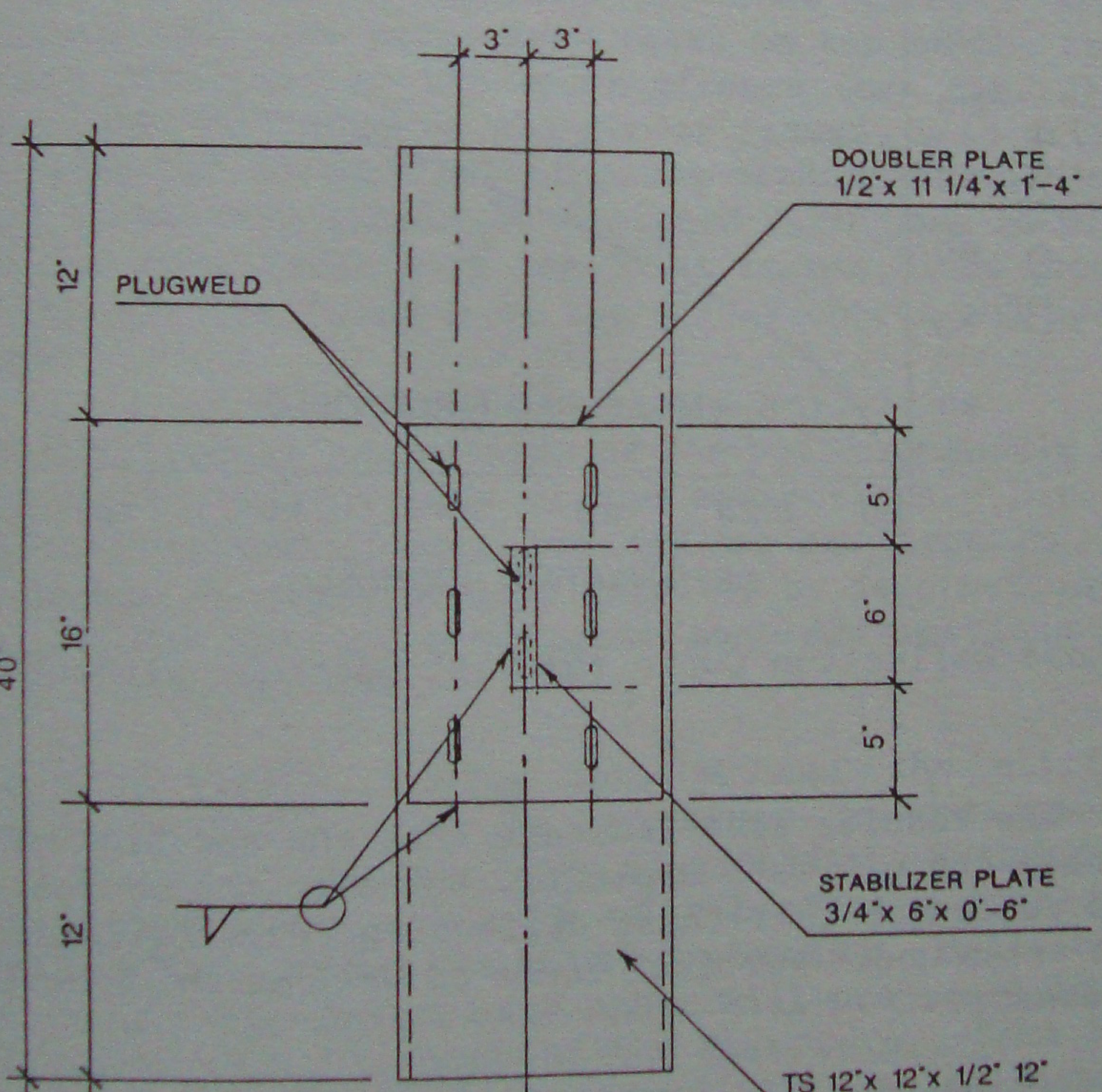

Figure 5. Test Specimen with Doubler Plate

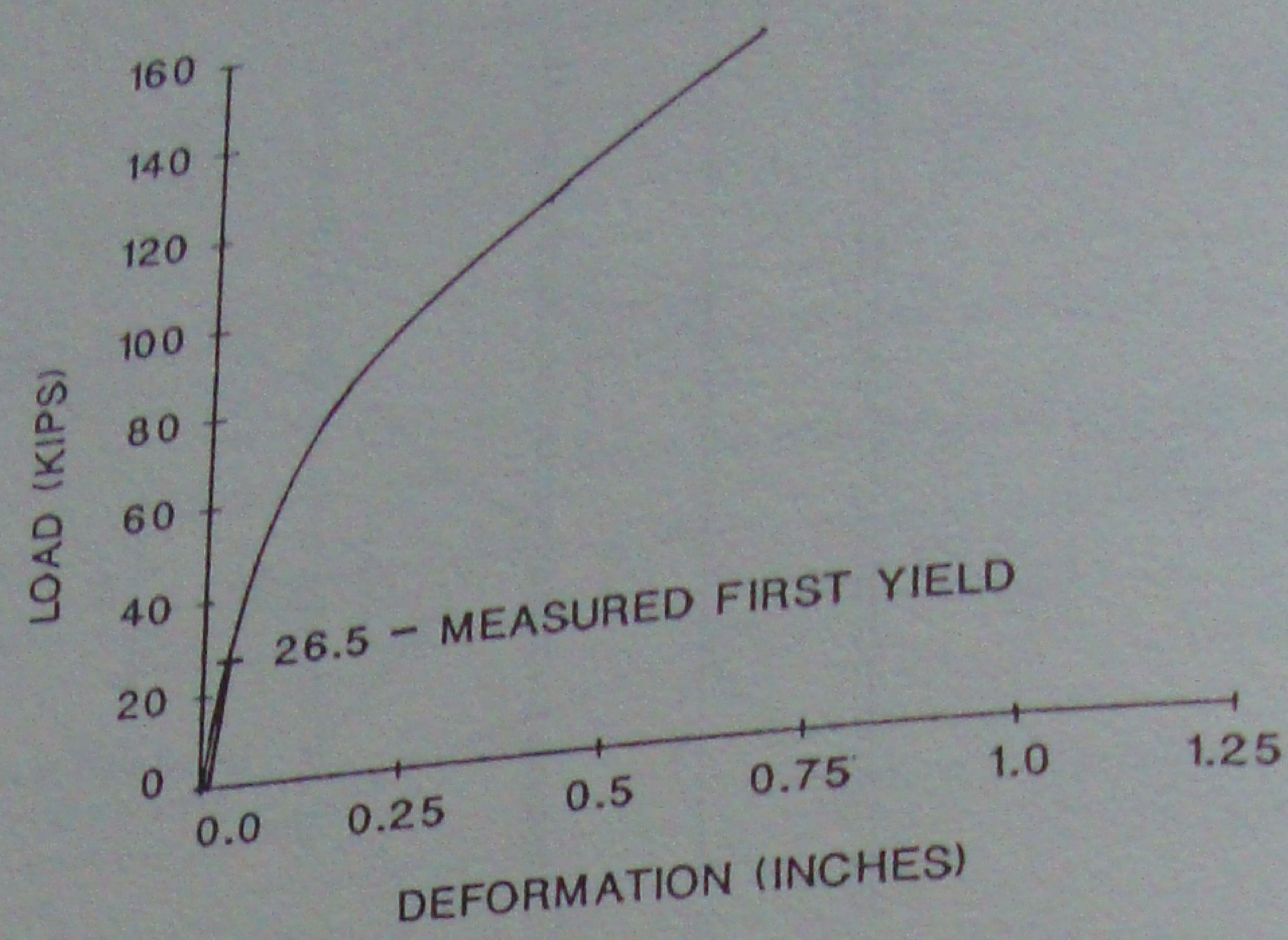

Figure 6. Load-Deflection Curve for Test Specimen with Doubler Plate

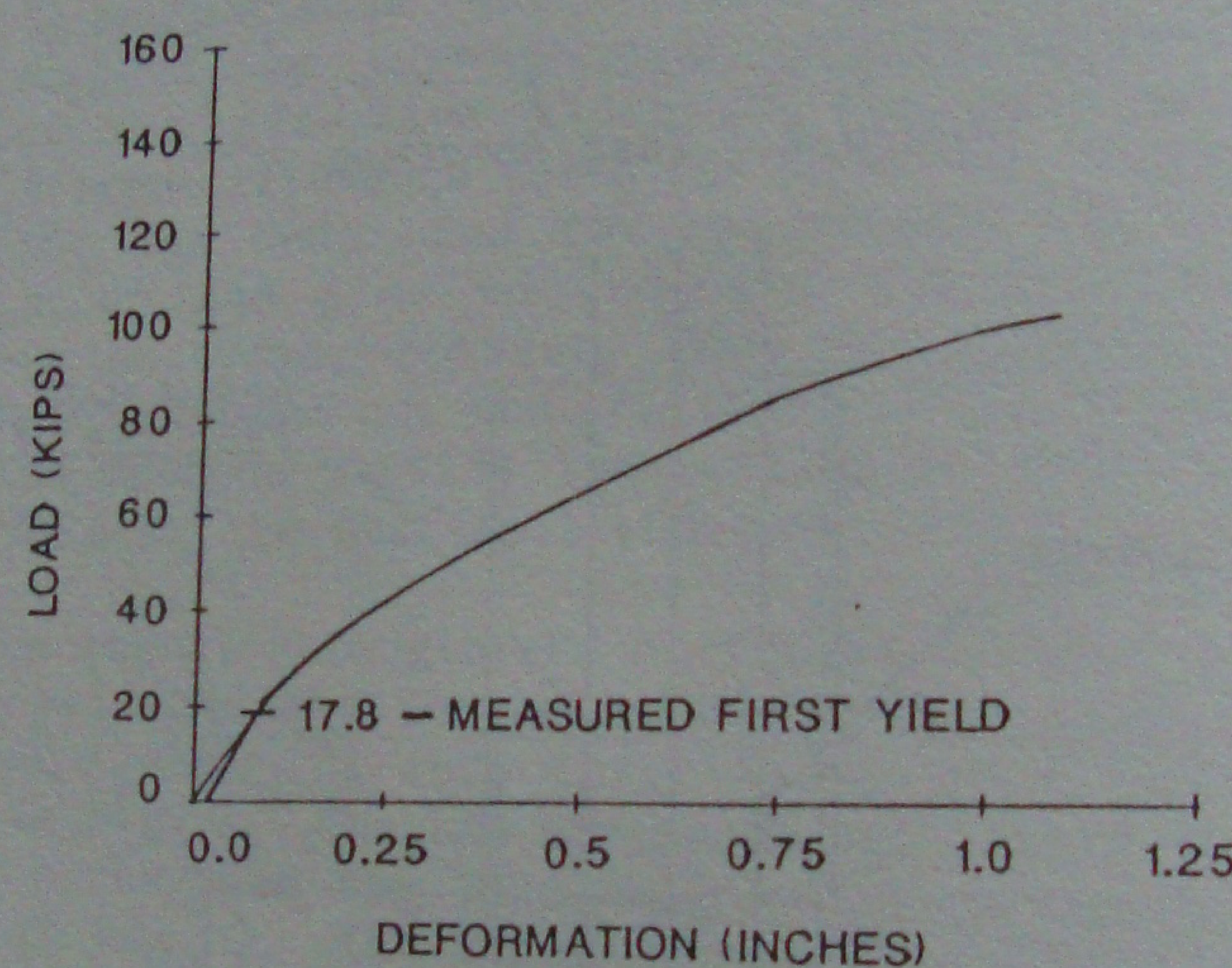

Figure 7. Load-Deflection Curve for Test Specimen without Doubler Plate

full strength. The results also indicate that the addition of doubler plates not only increases the ultimate capacity, but also the overall stiffness. It is important to recognize that these deflection characteristics influence the behavior of the truss under vertical loads as well as the behavior of the frame under lateral loads.

Other connection configurations for the bottom chord can be used which will increase the stiffness and strength of the connection. One method of reducing the force to the bottom chord connection is to use a diagonal "kicker" from the bottom chord of the truss joint to the column. The moment in the joist, however, drops off away from the column too quickly for the kicker to be of any benefit for the forces in the joist. Additional problems can arise if the bottom chord of the truss joist is not braced out-of-plane where the kicker is attached. A seat angle can be provided for the bottom chord. The chord is then field welded to the seat angle after application of the dead load. The horizontal leg of the angle acts as a stiffener for the wall of the tube. This detail can pose a fit-up problem if the seat angle, which is shop welded, does not align vertically with the truss joist.

SEISMIC FORCE PERFORMANCE

The current seismic design guidelines in the Uniform Building Code (1988 Edition), provide for two types of moment-resisting frames: Special moment-resisting space frames and ordinary moment-resisting space frames. The use of truss joists as part of the moment resisting space frame typically do not meet the ductility requirements to be considered as a special moment-resisting space frame. Very little information is available on the performance of the lateral load resisting open-web joist systems under seismic loads. Analysis of moment-resisting frames using truss joists indicates a number of critical elements affecting the performance.

The lateral load transfer in this system varies from typical moment-resisting frame systems. In most cases, truss joists span in one direction and are supported at the ends by bearing on truss girders spanning in the perpendicular direction. The metal deck rests on top of the truss joists and is not be directly connected to the truss girders (see Fig. 2). For lateral loading aligned in the direction of the girder frames, load originating in the deck must be transferred through out-of-plane bending of the joist ends. The top chord of the joist ends should be designed with this in mind. Similarly for lateral load in the direction of the joist frames, load from the deck must travel through out-of-plane bending of the girder chords and bearing angles to reach the columns.

Analysis of the lateral deflections of the building should account for the deformations due to any flexibility of the connections. Since the distance between the top and bottom connections can be as large as 36 inches, the response of the system is significantly affected by deformations in this area. This is similar to the inclusion of panel zone deformations when analyzing typical moment framed structures.

An eigenvalue analysis, taking into account the stiffness and mass properties of the building, can be performed to determine the dynamic characteristics of the structure. The flexibility of the frame gives the structure a longer fundamental period of vibration compared to a similar building using wide flange members as bending elements. Building code formulas for determining the seismic design base shear will produce a lower design base shear, and thus lower forces throughout the structure. Some building codes, such as the Uniform Building Code, specify limits on the computed period to avoid underestimating the actual seismic forces. The additional flexibility of the system that results from deformation within the joint is difficult to incorporate into a global building analysis.

Because of the excessive flexibility of the structure, drift can also be a problem in this system. Approximate analysis methods may not yield accurate estimates of lateral deflection. Excessive drift can cause extensive nonstructural damage during an earthquake. Nonstructural elements, which may not have been considered as part of the lateral force resisting system, can become engaged because of the flexibility, and act to resist forces. This can result in a change in the anticipated force distribution.

CONCLUDING REMARKS

Open-web steel truss joists are being used as bending elements in moment resisting frames. Little information is presently available on the performance of this type of system for resisting lateral loads due to earthquakes. Until such information has been gathered, the designers who specify this system should be aware of the potential problems associated with this system.

The critical feature of the truss joist system is the connections of the truss joists to the columns. At the top connection, an eccentricity between the chord member and the attachment to the column can occur when a bearing seat is used at the end of the joist. The eccentricity can also cause bending in the chord member of the joist or girder. Stabilizer plates used for the bottom chord connection result in concentrated forces being developed at the wall of the tube. These details must be analyzed to determine the stiffness and capacity of the connections. The stiffness of the bottom connections can also have an effect on the vertical capacity of the joists. Flexibility of the bottom connection may not provide the anticipated fixity at the ends of the joists or girders, thereby allowing them to behave as simply supported beams.

Structures which utilize truss joists as part of moment resisting frames are more flexible than typical systems. Excessive drifts can result from the flexibility of the moment frames. The flexibility of the connections must be included in an lateral analysis of the building. The designer should evaluate the effect of the increased drift on the building to verify that the nonstructural elements are capable of withstanding the anticipated lateral deflections.

Finally, the responsibility for the performance of the overall structural system rests with the design engineer. Although the design of the truss joists is provided by the fabricator, the engineer should be aware of the significance of the features used in the system. The engineer must also clearly define the responsibilities for the design of connections and other details. The design engineer should work closely with the fabricator to ensure that all of the anticipated forces are considered.

The concepts presented herein were generated during structural reviews of existing structures. Details of construction identified as typical may not be for structures other than those reviewed by the authors. Other details may result in seismic resisting systems of greater or lesser adequacy.