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Paper No. 1113. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

SEISMIC BEHAVIOR OF SEMI-RIGID MOMENT FRAMES OF COLUMN TREE STRUCTURES

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ABSTRACT

Analytical studies using nonlinear DRAIN-2DX software were performed on building frames modeled as semi-rigid steel frames. These frames were designed to be constructed according to the column tree method of construction. Field splices were located in the beams at approximately 1/4 of the span length. The semi-rigid parameters of these connections were varied to allow for the study of the seismic response to various recorded ground motions. Global response characteristics of period, base shear and lateral story deflection were recorded. Local response characteristics for the beam-to-column assembly and the field splice assembly were also studied. The effectiveness of the semi-rigid connections to improve the seismic response of the building was demonstrated.

KEYWORDS

steel frame, inelastic analysis, semi-rigid steel connections, seismic building design, ductility, connection rotation, base shear, earthquake, steel, bolted connection, welded connection

INELASTIC STUDIES OF SEMI-RIGID FRAMES

A column tree erection system allows for convenient fabrication and erection of a steel frame. In the column tree system the field splice is located at a significant distance from the face of the column. The moment demands on this connection are much lower than the strength of the steel beam. The connection capacity can be limited and the stiffness varied so that it will yield in a ductile manner before any other portion of the steel frame reaches a yield state. Figure 1 shows a typical frame built by using the column tree fabrication process. Figure 2 shows a field beam splice connection that can easily be altered to develop a semi-rigid connection behavior.

In recent earthquakes it has been seen that the field-welded connection at the face of the steel column is exposed to severe loading and rotational demands, and can often fracture during a large seismic event. The column-tree system allows this connection to be fabricated entirely in the shop, thus providing high quality control. Design procedures have been developed for field-bolted splices (Astaneh-Asl, 1995). The objective of this study was to show the effect of the stiffness, strength, and strain-hardening parameters of the semi-

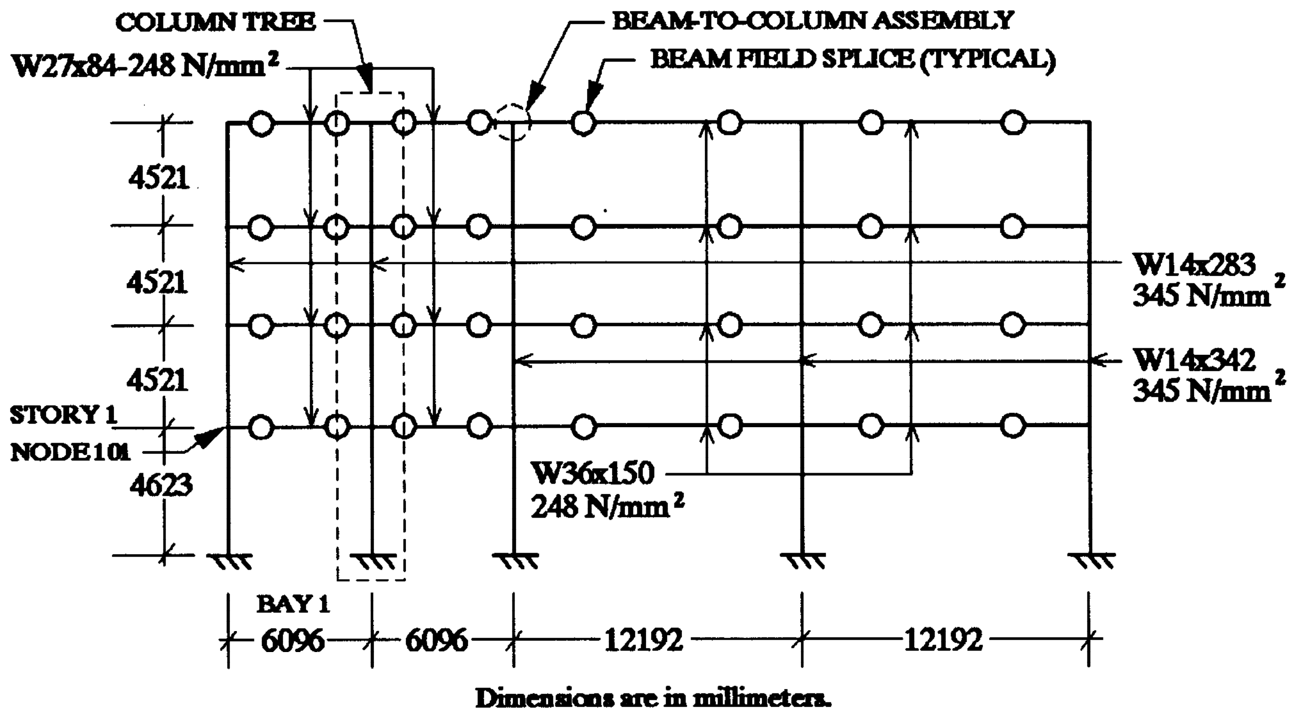


Fig. 1. Column tree frame

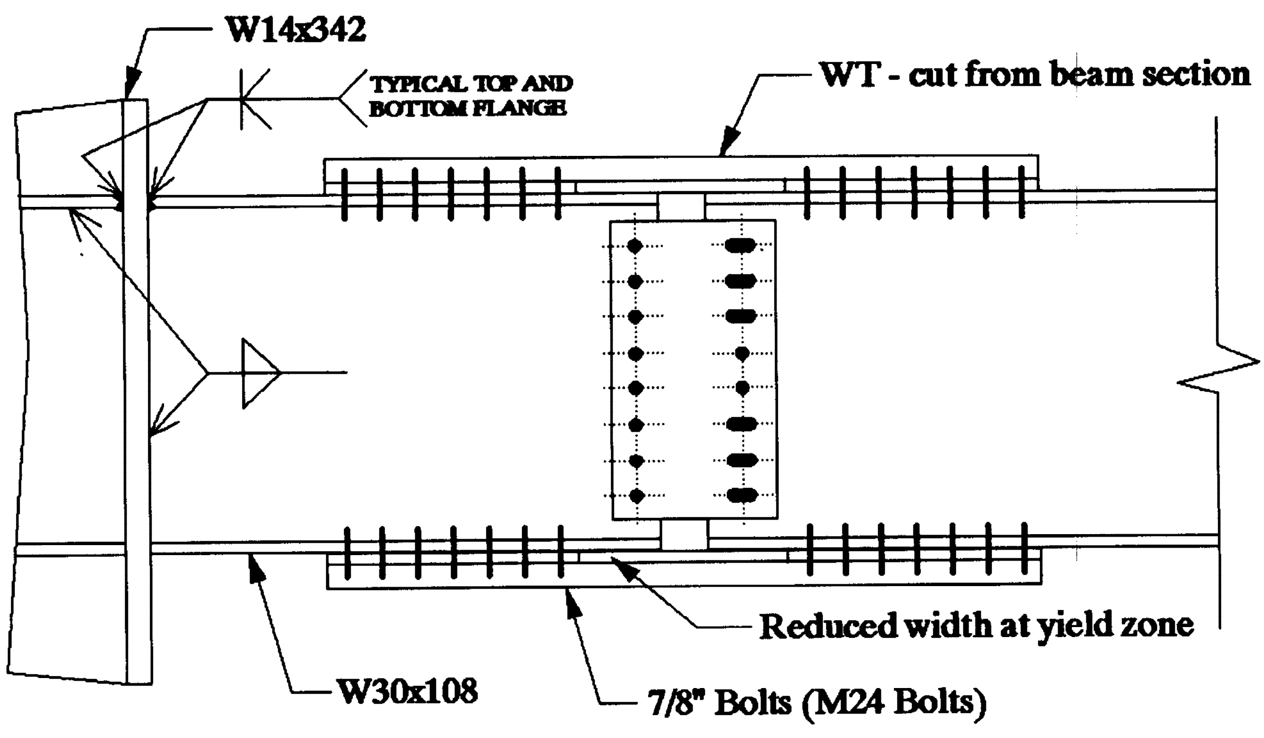


Fig. 2. Semi-rigid connection

rigid connection. It was desired to see what combination of parameters would provide the optimal response to a range of earthquake ground motions.

Methods

This study is a continuation of a program of study into semi-rigid frame behavior at the University of California at Berkeley. Past studies have shown the effectiveness of semi-rigid frames through analytical and shaking table experimental work (Nader, 1992). Another study explored the effect of using semi-rigid connections in column trees (McMullin *et al.*, 1993). That study compared global response of two frames to different ground motions. The study was performed by using the DRAIN-2DX nonlinear two-dimensional analysis program. Two steel frames were considered. A four story frame was designed by the authors to represent a typical steel lowrise frame. A 24 story frame design was obtained from an actual building designed and built in Japan. Ground motions recorded from El Centro, Taft, and Miyagiken-Oki were scaled to represent both service level and ultimate level earthquakes. The peak velocity of the ground motion was scaled according to the Japanese building code.

The connection parameters for rotational stiffness before and after yielding were varied. Global parameters such as base shear and roof drift were studied. Additionally, local parameters of plastic rotation demands for connections were tabulated. The general behavior between the 4 story and 24 story frames was similar. This paper discusses the response from the 4 story frame.

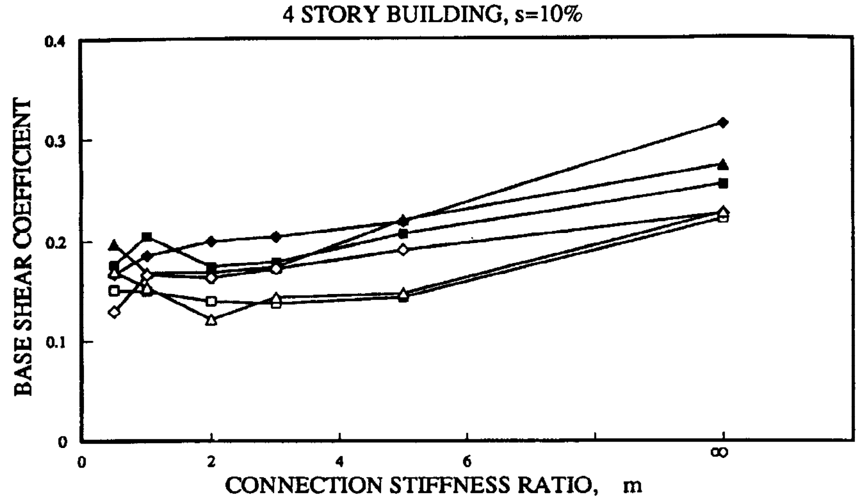
Semi-rigid connections can be described reasonably well by three parameters. Alpha, α, is the ratio of the yield strength of the connection to the plastic strength of the beam. A ratio, m, defines the initial stiffness of the connection to the stiffness of the beam:

$$m = \frac{k_{elastic-conn}}{(EI/L)_{beam}} \tag{1}$$

Finally, the strain-hardening stiffness of the connection divided by the connection's initial stiffness is defined as the ratio s. In this paper the ratio s was used as a constant value of 5%.

Global Response Parameters

Two primary global response factors are considered in building design. First is the base shear that is developed in a building during an earthquake. Changing the stiffness of the connection increases the fundamental period of the building. This relates to a reduction in the base shear, as can be seen in Figure 3. Generally this base shear reduces until the stiffness ratio is approximately three. As the stiffness is reduced below this point, the building response becomes difficult to predict.



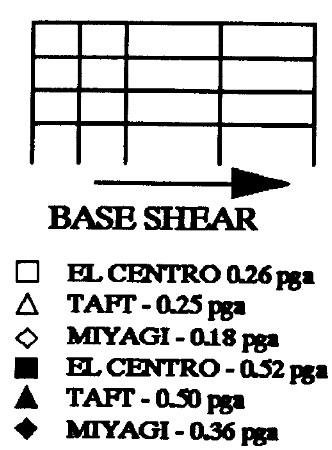


Fig. 3. Building base shear of four story semi-rigid frame.

Lateral story deflection is the other global response parameter of concern. The lateral drift of the roof is shown in Fig. 4. There is a general feeling that semi-rigid moment frames will have larger displacements than rigidly connected frames. All of the inelastic models had lower displacements than the elastic model. Considering Figs. 3 and 4, an optimal stiffness ratio appears to be three.

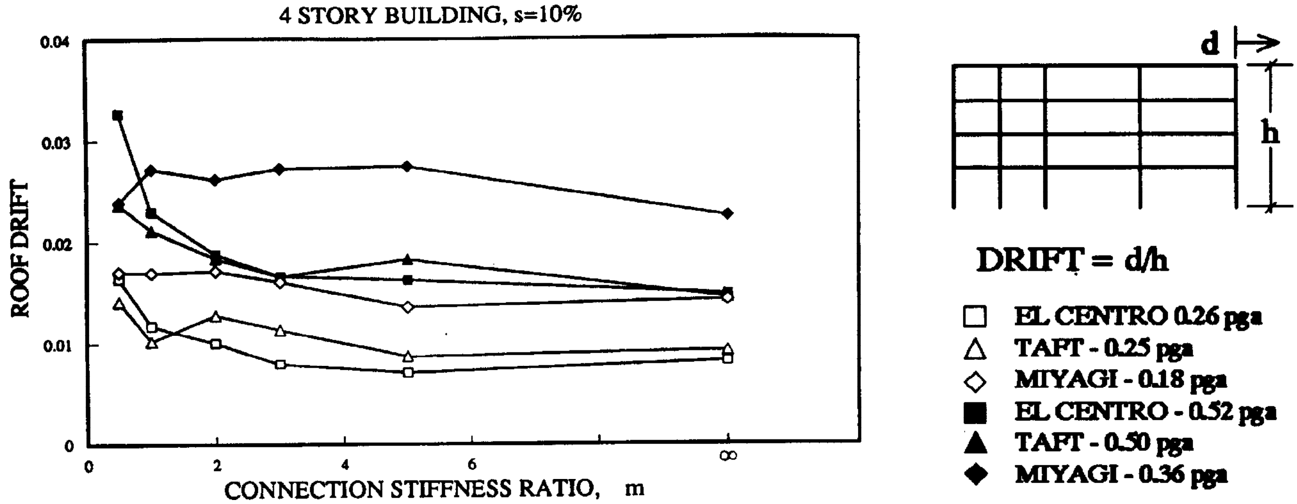


Fig. 4. Lateral story deflection of four story semi-rigid frame.

Demands on Connections

To study the effect of the connection parameters on local connections, the 4 story frame model shown in Fig. 1 was modeled using DRAIN-2DX nonlinear software. Beam elements with local plasticity were used for the beams and columns, nonlinear rotational elements were used for the semi-rigid beam field splice connections. The Miyagi Ken Oki 1978 ground motion record was used, but the ground motion was scaled to a peak ground acceleration of 0.6g to simulate a major earthquake. Time histories were plotted for the first eight seconds for the connections in bay 1 of story 1. Figure 5 is the time history of the lateral drift of the first floor for a semi-rigid frame and a frame where the connections are completely rigid. As can be seen, the peak drift is comparable for both cases, but occurs at a different time and in a different direction.

The primary local demand parameters are on the connections between various members. Since the severe damage that resulted from the Northridge earthquake, primary interest has been focused upon the beam-to-column assembly. The primary philosophy of earthquake engineers has been to develop connections in special moment resisting frames that will develop the strength of the beam, hence causing all inelastic behavior to occur in the steel beam. The moments at the beam-to-column assembly and the field splice are related, as shown in Fig. 6. The top drawing is the moment time history for the semi-rigid frame, the lower drawing represents a rigidly connected frame. The philosophy of semi-rigid column tree frames is that the moment at the beam-to-column assembly can be controlled by specially designing the field splice to yield at moments low enough to limit the damage to the beam-to-column assemblage. The maximum moment occurring in the semi-rigid frame is 1.5 Mp, whereas the rigid frame has moments near 2.0 Mp.

Local Connection Rotation Demand and Global Drift Relationship

One area of concern is the prediction of rotation required from steel assemblies. In traditional plastic analysis, the drift of the building is considered to be equal to the rotation of the beam-to-column assembly. In actual application, several regions of the frame will contribute to the rotation of the assembly. These regions include the panel zone, the column outside the panel zone, the beam-to-column connection, the beam, and the column-tree field splice. These regions will contain both elastic and inelastic rotation. In large earthquakes, the inelastic rotation will far exceed the elastic rotations provided by most buildings.

The rotation demand for various regions of the beam are shown in Fig. 7 for a typical analysis. This is an accumulation of all the concentrated rotation demands at the left end of the bay. It includes the plastic hinging at various locations in the beam and the elastic and inelastic rotational requirements at the semi-rigid field splice. The total rotation of the rigidly connected frame is almost exactly the same as the drift of the story. The rotation of the semi-rigid frame is larger than the drift, indicating the effect of having the rotation occur further from the column centerline. In actuality, all the rotation that occurs in the DRAIN beam elements must develop from near the column face in a proof test. The rigid frame requires rotations at the face of the column to be over 0.04 radians, whereas the semi-rigid frame requires maximum beam rotations of less than 0.02 radians.

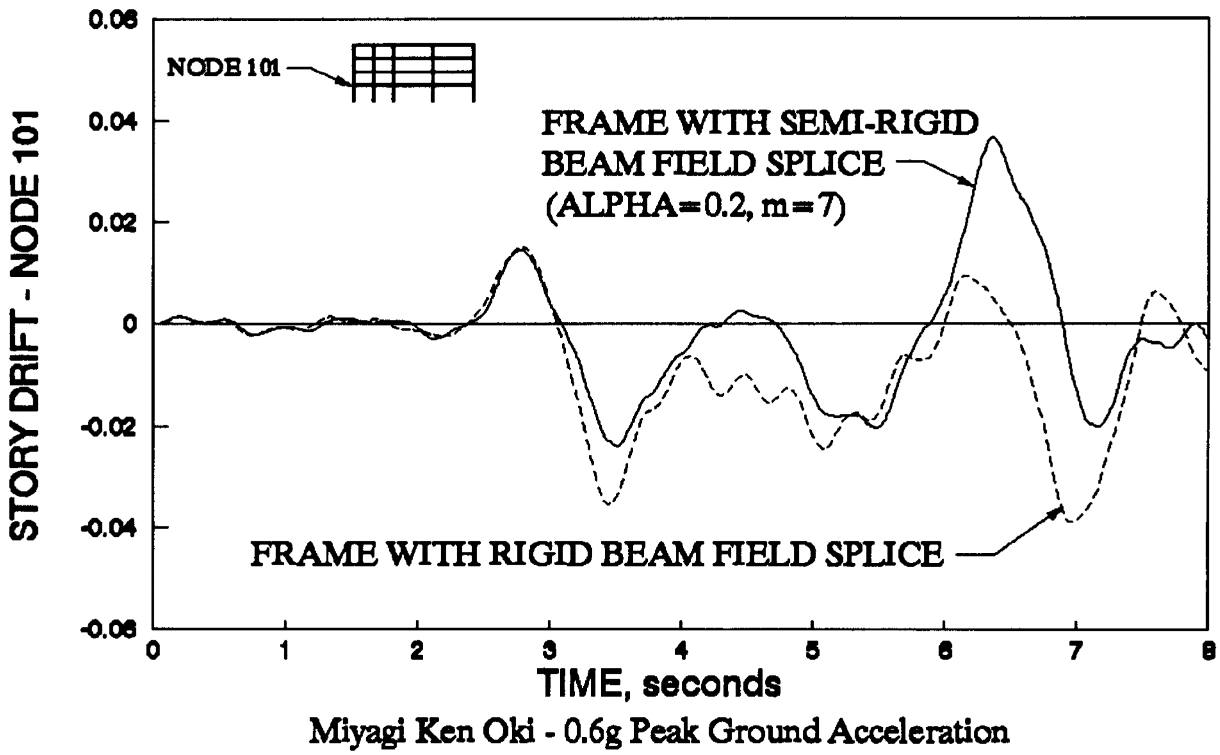


Fig. 5. Time history of first floor drift for four story frame with rigid and semi-rigid connections.

Figure 8 shows the rotational demands for different beam field splice connection parameters. The graphs include the concentrated rotations from the 4-story building analysis. In comparison is shown the first floor drift. The graphs are drawn for the peak displacements located near the 6.40 second point of the Miyagi Ken Oki ground motion. This point is usually the maximum displacement that occurs in the analysis, but occasionally is not.

Several aspects can be seen from Fig. 8. First the behavior is very similar between the two ends of the beam. The left end is an exterior beam-to-column assembly, whereas the right end is an interior beam-to-column assembly. As seen before, the drift of the building generally increases as the semi-rigid connection is designed to be more flexible. As the connection strength decreases, the contribution of the column-tree field connection is greatly increased. The demand on the beam-to-column connection is generally constant, but decreases slightly as the field connection is made more flexible and weaker.

Also the total rotation demand increase significantly depending upon the rotation provided by the field connection. When this connection easily rotates, the total rotation increases, similar to the estimated increase in rotation expected when the plastic hinges of a frame move away from the beam-to-column assembly.

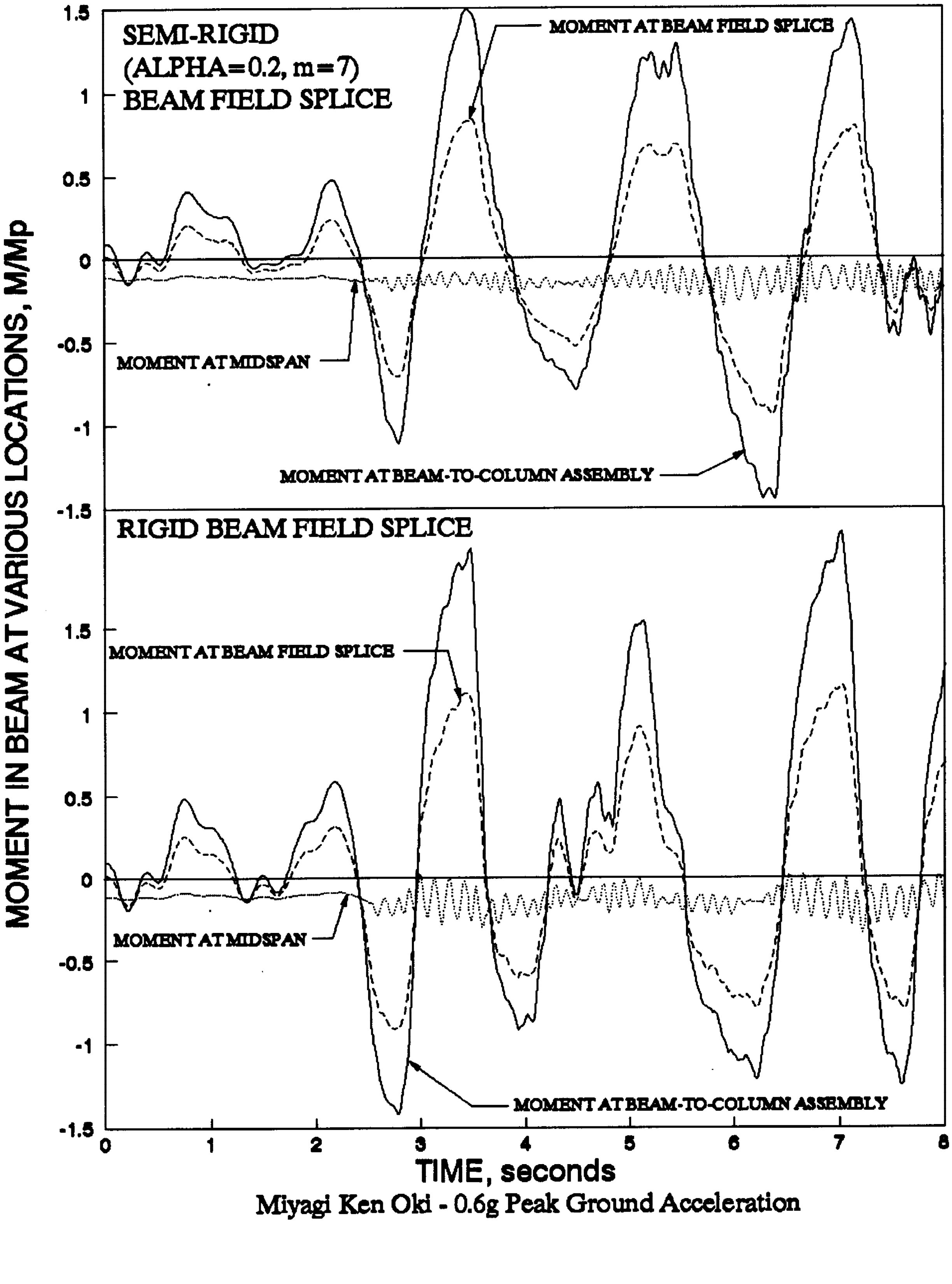


Fig. 6. Time history of moment demand for four story frame with rigid and semi-rigid connections.

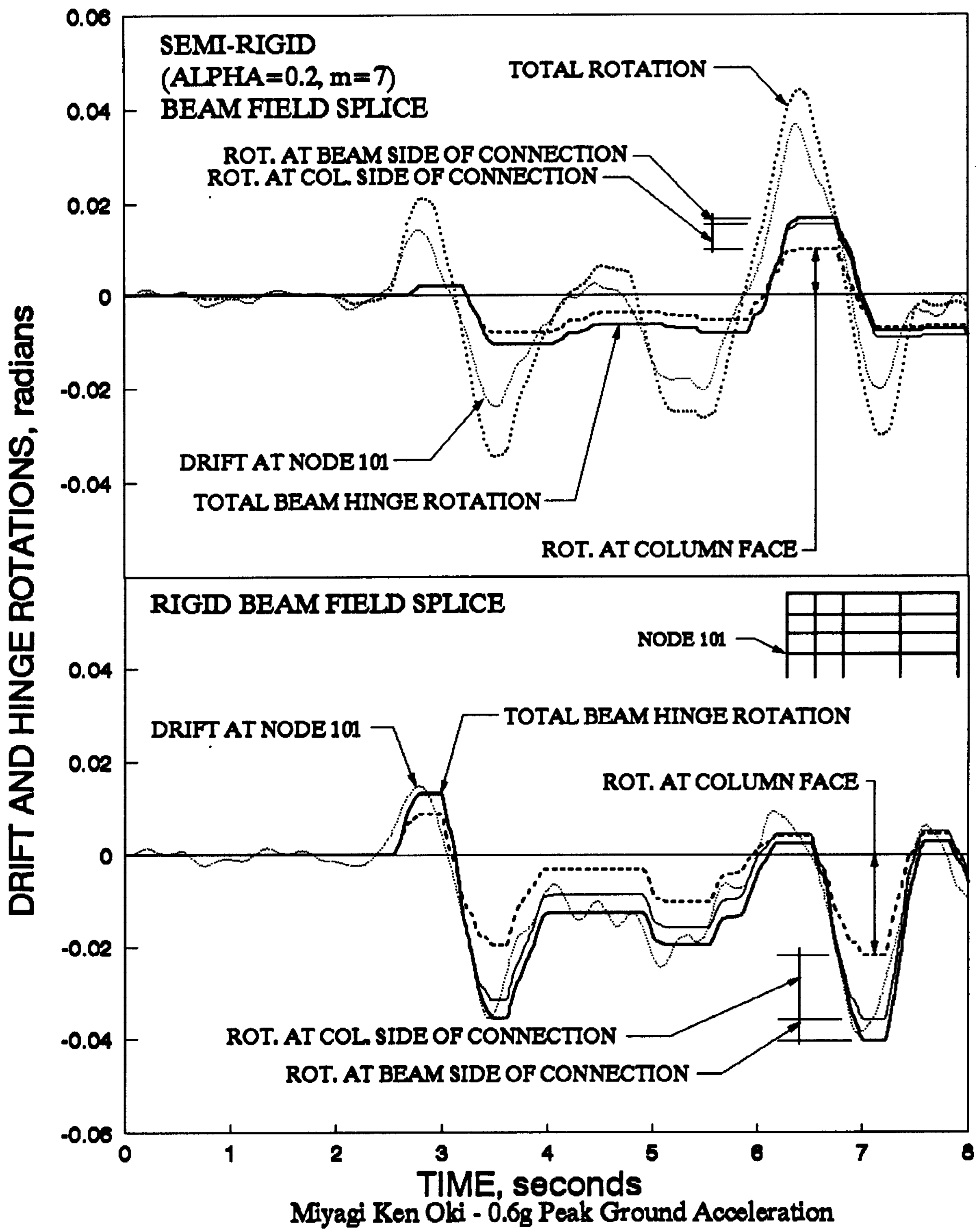


Fig. 7. Time history of rotation demand for four story frame with rigid and semi-rigid connections.

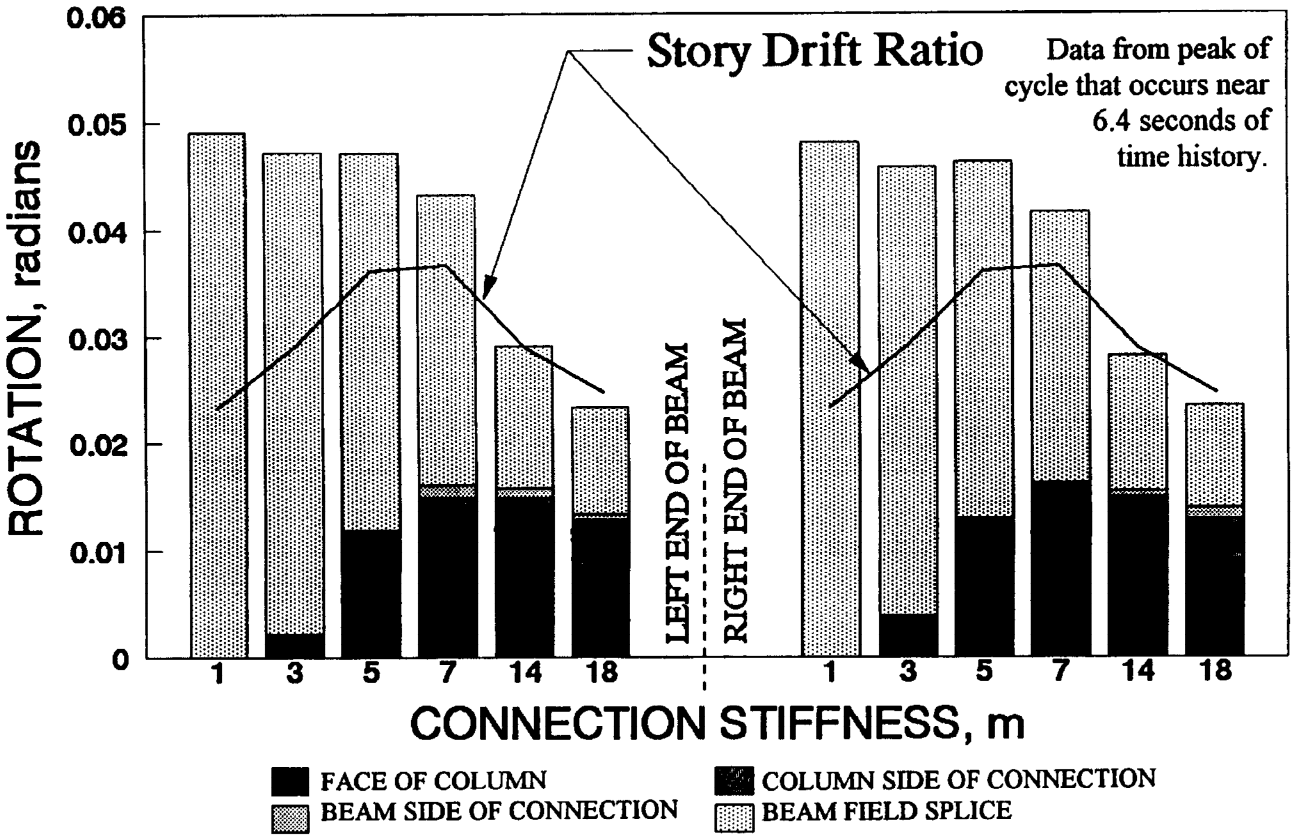


Fig. 8. Rotation demand for four story frame with semi-rigid connections with varying elastic stiffness.

CONCLUSIONS

The study concluded that semi-rigid moment connections can help control the seismic response of steel frames. The base shear and roof displacement can be reduced by varying the stiffness and strength of the connection. Rotation demands were also minimal. Semi-rigid connections appeared to be more beneficial for tall buildings than for low rise buildings.

The column tree form of construction is especially adapted to semi-rigid frame design. This construction technique places the field connection in regions of low bending moment. The maximum moment applied to the beam-to-column connection could be controlled and limited.

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