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**EXPERIMENTAL TESTING AND ANALYTICAL MODELING OF
STEEL SEMI-RIGID CONNECTIONS**

A Thesis

Presented to

The Faculty of the Department of

Civil Engineering

San Jose State University

In Partial Fulfillment

Of the Requirements for the Degree

Masters of Science

By

Adam V. White

August 2003

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ABSTRACT

EXPERIMENTAL TESTING AND ANALYTICAL MODELING OF STEEL SEMI-RIGID CONNECTIONS

By Adam V. White

The semi-rigid connection considered in this thesis is a column tree connection, an innovative departure from steel moment connections used in modern building construction. There were four objectives of this thesis: experimentally measure the moment rotation relationship of a full scale weak axis specimen, develop modeling input parameters, compare performance of semi-rigid versus simple frames using a standardized model, and quantify global and local performance of the connection using a series of ground motions.

Specimen testing showed acceptable rotation capacity. However, the specimen fractured at the beam column weld connection due to poor weld penetration. Modeling input parameters were derived from a comparable strong axis test results and a simple bilinear curve was developed. Overall performance of the semi-rigid connection, based on analytical modeling, showed that the column tree connection forced inelastic behavior away from the column face. Analytical modeling using DRAIN 2DX software showed acceptable global and local performance.

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Chapter 1 Introduction

1.1 Background

After the Northridge Earthquake of 1994, it was discovered that typical moment frame connections subjected to the ground motion fractured at the interface between the beam and column. As a result, the engineering community began to investigate connection modifications to avoid this type of connection failure. One variation of the common moment frame, the column-tree connection, has been investigated as an alternative for use in steel building construction as a means of providing stable energy dissipation while maintaining displacements within tolerable limits (McMullin & Astanek-Asl, 1996).

The column tree connection provides several distinct advantages as a viable alternative for beam to column connections. Compared to common moment connections, column-tree connections have advantages which include: use of shop welding during column fabrication to provide a better quality weld, field bolting to provide quality control, and ease of replacement for post earthquake repair. In addition, the connection configuration allows inelastic behavior to occur away from the column face due to plastic rotation occurring at the field splice connection. A typical bay built using the column-tree connection considered in this project is shown in Figure 1 of Appendix A.

1.2 Project Goals and Objectives

The overall research goal of this project is to determine the performance characteristics of the column-tree connection for evaluating its suitability for use in actual steel frame construction in high seismic zone regions. Connection characteristics were determined through experimental testing of a full-scale prototype. Capacity input parameters of an analytical model were based upon results of experimental testing. By subjecting the model to simulated ground motions derived from historical earthquakes, a determination of frame demand was obtained. The modeling results provided needed performance characteristics of the connection. These demands were used to compare performance of a rigid connection with that of the semi-rigid column-tree field splice.

Project objectives specific to this thesis were:

- To experimentally measure the moment-rotation relationship of a full-scale test specimen during cyclic loading.
- To develop a procedure to define the bi-linear parameters for analytical modeling input from full scale specimen testing results.
- To compare the performance of the analytical model of a building using either rigid or semi-rigid connections for a suite of ground motion records.
- To quantify the global and local demand performance parameters for the semi-rigid connection.

1.3 Scope of Work

This project is part of a larger investigation of column-tree connections. The experimental portion of this project is a continuation of previous testing of similar connections. In addition, modeling was conducted to quantify the demands of semi-rigid structures and to compare with the response of rigid structures in the hope that information could be obtained to allow simulation of similar connections for use in high seismic zone areas to accurately determine performance. This phase of the research initiative has lasted approximately 15 months.

In addition to the specimen considered in this thesis, two comparable specimens in this sequence have been previously tested but the results have yet to be published. The specimen tested for this project consisted of a beam connected to the weak axis of a column. Frames were modeled with the semi-rigid connection in one direction only. Future expansion of the work may involve developing modeling input parameters for a weak axis connection and conducting analytical modeling of semi-rigid connections in both directions.

The objectives were accomplished by testing one full-scale specimen and performing analytical modeling of a standard structure. It is hoped that the data obtained will provide a reference for engineers to design structures using the column-tree connection.

Chapter 2 Literature Review

2.1 Traditional Moment Frame Design Research

The column tree system of this project is one of several variations of the reduced beam section concept. This section discusses available literature on these variations.

Steel moment resisting frame (SMRF) structures are designed to resist lateral loading by bending of members and connections and are common in major commercial and institutional structures. Architects and owners prefer a steel moment frame because it provides unobstructed open space, clear of braces or shear walls. Structural engineers have supported steel moment frame design because it was expected to provide ductile behavior in high seismic zone areas. Historically, steel moment frames were used in tall buildings. Since the 1960's, engineers designed steel moment frames using bolted or welded connections. Bolted and welded connections provided ease of fabrication and design, and were desirable for economic reasons.

Engineers designed frame connections to absorb substantial energy and provide major contributions to displacement ductility demand. Common practice has been to either bolt or weld the beam web to the column shear plate, and to weld the beam flanges to column flanges using a complete joint penetration groove weld. The common assumption was that beam shear would be transferred to the column by the beam web connection and the moment would be transferred through the beam flanges (Astaneh-Asl, 1995).

This assumption of shear transfer was called into question after these connections developed problems during the 1994 Northridge earthquake. Steel moment frame buildings, designed and built to code requirements at the time, exhibited brittle behavior at the connections. Investigation of these systems after the earthquake revealed numerous failures of bolted-web welded-flange connections, and column flange fractures. In some cases, weld failure was attributed to low quality. The intensity of ground acceleration also contributed to connection failures. However, these actions alone did not adequately explain the nature and extent of brittle connection behavior (Maranian, 1997). The engineering community undertook an investigation to resolve the reason for connection failure.

As part of the investigation, SAC Joint Venture was formed. The SAC Joint Venture is a partnership of the Structural Engineers Association of California, the Applied Technology Council, and the California Universities for Research in Earthquake Engineering. Work is conducted under contract with the Federal Emergency Management Agency to understand, quantify, and improve the seismic behavior of SMRF structures with welded connections.

The results of the investigation showed that in some cases, connections designed prior to the earthquake were not strong enough to develop the strength of the beam. That is, the connections were not designed to sufficiently develop inelastic capacity to assure ductile behavior (Shipp, 1994).

Investigation of possible causes of failure in these connections found that use of a backing bar provided an artificial crack at the juncture of the column flange and beam flange backing bar (Popov, Yang, and Chang, 1996). The interface between the backing bar and the column/beam interface provided an area for crack initiation. During beam bending, tension in the beam flange opens this artificial crack and initiates rupture. This type of rupture is considered serious as fracture proceeds at high speeds without exhibiting ductile behavior. One solution to reduce the deleterious effect of backing bar crack initiation could be to remove the backing bar after erection and provide a fillet weld. However, this method is expensive and can damage the main weld.

2.2 Post-Northridge Moment Frame Innovations

Less costly improvements of the connection resulted from changes to the detailing. A revised approach would be to design connections to remain essentially elastic at the column face and force elastic deformation of the frame to occur in the portion of the beam, away from the connection. A typical revised connection design locates the plastic hinge (i.e., the point at which inelastic deformation is concentrated) away from the column flange. In doing so, inelastic behavior occurs within the beam. This condition is desirable over column inelastic behavior as it provides the potential for inherent post earthquake stability, less costly repair potential, and provides for easier post-earthquake inspection.

Moore, et. al. (1999) and Zekioglu, et. al. (1997) conducted testing of the Reduced Beam Section (RBS) connection. In an effort to force the inelastic behavior

away from the beam column interface, the RBS connection relies on selective material removal of the beam flange adjacent to the beam-column connection. Typically, material is removed from both top and bottom flanges. Reduction of the cross sectional area reduces the moment capacity of the beam at a pre-designated location. The RBS connection forces yielding and plastic hinge formation within the reduced section and therefore limits the moment that can be developed at the column face. By reducing demands on the beam flange groove weld and surrounding base metal, the RBS connection reduces the possibility of fracture in this vulnerable region. The RBS connection essentially weakens the overall beam strength, however its impact is small on lateral strength and stiffness of the steel moment frame. A smaller moment is generated at the column face, reducing weld stress levels, providing advantages for strong-column weak-beam requirements, and minimizing column doubler plate requirements.

Fabrication and erection of the RBS connection reduced the need for additional strengthening plates and special weldments typically required for steel moment frame connections. These considerations make the RBS connection cost effective. Some of the disadvantages include additional detailing requirements and shop fabrication procedures.

Popov and Takhirov (2001) published testing results for a revised bolted connection. This connection involves attaching T-beam sections to the column and beam flanges and a web plate connection between the beam web and column. Beam stresses due to shear and flexure are transferred to the column through bolted brackets located at the top and bottom of the beam flanges. The behavior of the connection can be

controlled by different modes related to relative proportions of the connection components. Connections are designed to control inelastic behavior either by flexural yielding of the beam in combination with shear yielding of the column panel zone, or by flexural yielding of the beam alone. Connection dimensions are proportioned such that the moment at the column face, as limited by the controlling inelastic behavior, is less than the moment at the face in relation to other behavioral modes (FEMA 350, 2000).

The slotted web connection is a proprietary connection (Seismic Structural Design Associates, Inc. US patent No. 5,680,738 issued October 28, 1997) similar to field-welded, field-bolted “Pre-Northridge” moment frame connections. The distinct difference is slots cut through the beam web separate the beam flange from the beam web in the region of the connection. These slots provide a discontinuity in areas otherwise subject to large stress/strain gradients across the connection at the flange/column weld interface. By providing these longitudinal slots, slotted web connections have been shown to reduce vertical beam shear forces in beam flange welds by restricting vertical shear to the beam web and shear plate to the column. The slot also forces the beam web to resist its portion of the applied moment that would otherwise be transferred to the connection welds. Beam web slots also allow out of plane bending of the web to column flange weldments, thereby reducing stress/strain gradients across and through the beam flanges. By applying a slot at the web, fatigue life of the connection is enhanced by reducing the beam flange prying moment and accumulated plastic strain and ductility demand under cyclic loading. The slot allows the flanges and web to buckle

independently and concurrently, eliminating the twisting mode of buckling at the connection (FEMA 350, 2000).

2.3 SAC Joint Venture Steel Research Project

The SAC Joint Venture study included consideration of differing behavior of welded beam-to-column connections and attempted to determine the seismic demands for SMRF structures.

In order to assess building performance, several tasks were completed to provide the reference for future study. One of the tasks necessary to conduct an assessment of building performance was to develop model buildings with which to compare demands for varying ground motions. Nine model buildings representing a 3-story, 9-story, and 20-story buildings located in Los Angeles, Seattle, and Boston were designed by leading consulting engineering companies to represent typical design practices followed in different parts of the United States. Basic parameters (e.g., configuration, loading, etc.) were common to all locations. Therefore, seismic evaluation of these structures provides a frame of reference for estimating seismic demands for typical structures.

Another task was to develop suites of ground motions that were representative of different hazard levels. Seven suites of ground motions were developed by Somerville (1997) representing hazard levels with a 72 year to 2475 year return period. The ground motion suites developed addressed behavior and response issues over a wide range of seismic hazard levels. With both tasks completed, additional study was done to address

the long-term objective of the SAC Joint Venture study, i.e., development of performance based seismic design and evaluation.

The primary objective of performance based seismic engineering is to design, build, and maintain facilities such that they have predictable performance in accordance with pre-defined performance objectives. The basic performance objective of designing structures is to provide adequate life-safety while keeping design economically feasible.

Additional pre-defined performance objectives can include special consideration for building use (i.e., critical facilities). Typically, a non-technical definition of building use is defined by the owner and is translated into a facility design based on cost-benefit evaluation. The specific structural technical requirements are then related to seismic hazard levels (i.e., desired performance) and the structure is evaluated and modified accordingly.

A primary consideration in determining building performance is defining building seismic demand (i.e., the response of a structure during a seismic event). Evaluation of typical SMRF demand was conducted by Gupta and Krawinkler (1999), as part of the SAC Joint Venture study. The study focused on the behavior and demands of ductile SMRF structures with no strength and stiffness degradation in element characteristics, disregarding the potential for connection weld fractures, near-fault effects, and soft soil effects.

The Gupta and Krawinkler study considered, among others, the SAC Joint Venture model 3-story frame designed for the Los Angeles region. This was a traditional

centerline based model of a bare moment resisting frame, where beams and columns extend from centerline to centerline for all beam and column elements. Strength, stiffness, dimensions, and panel zone shear distortions were neglected. Moments in beams and columns were computed at the connection centerline.

The Gupta and Krawinkler model used 3% strain hardening, strength properties based on expected strength of material, and a 2% Rayleigh damping enforced at the first mode period. The moment resisting frame was oriented in the north-south direction resulting in uniformly distributed gravity loads on beams of the moment resisting frame and concentrated loads on the columns from orthogonal beams. Uniformly distributed loads were applied to the model as fixed end effects. The concentrated loads transferred from the orthogonal beams to the columns were modeled as nodal loads. Seismic excitation was applied at all the laterally restrained degrees of freedom simultaneously.

The Gupta and Krawinkler model is a two-dimensional model representing only perimeter frames and ignores the presence of simply connected interior gravity frames. Included were P-delta effects from vertical loads tributary to interior frames that are transferred to the moment resisting frame through the rigid floor slab. Geometric and material nonlinear static and dynamic analysis was performed on the various models.

A similar frame configuration was considered in this thesis. In the interest of reducing complexity and computational effort, the model considered in this thesis was altered or omitted the following:

- Gravity loading was altered to concentrate loads at end nodes and mid-span nodes along each floor level.
- Panel zone simulation at the beam-column connection was omitted.
- P-delta effects were omitted.
- Non-linear static analysis was not considered.

The model considered in this thesis was similar to the basic model, as described above. Building mass, beam and column properties, and dimensions were similar. Geometric non-linearity was not considered. The basic model provided a standard by which the rigid and semi-rigid connection could be simulated and was consistent with configurations used for SAC Joint Venture study and were used to compare rigid and semi-rigid behavior, within the context of this thesis.

Chapter 3 Experimental Design and Procedure

3.1 Connection Mechanics

The intent of this project was to both define the ability of a connection to maintain strength while providing rotational ductility during cyclic loading and compare this capacity to the required demands during a seismic event. A moment frame structural system subjected to strong ground motion responds by developing plastic hinges. Plastic hinging is one inelastic mechanism that dissipates energy from the system during an earthquake. This section provides a basic explanation of the elastic and plastic theory of bending.

3.1.1 Elastic Bending and Ductility

During the initial stages of bending, a generic beam and connection experiences elastic bending, which is often as a linear response to beam loading. In theory, a beam member will exhibit elastic bending up to the point at which the elastic limit is reached. This point, the yield moment, can be defined for a beam of known dimensions. Elastic bending occurs during the initial loading and does not result in permanent deformation.

Ductility is defined as a materials ability to deform plastically, thereby absorbing energy prior to fracture. Steel is one material that exhibits large post-yield strain capacity prior to failure. This quality allows the material to convert strain energy to thermal energy during deformation.

3.1.2 Inelastic Bending

When the bending moment applied exceeds the yield moment, the beam section experiences inelastic beam bending, meaning the beam experiences permanent deformation or yielding within the cross section. In theory, if the applied bending moment is increased further, the beam will reach its plastic moment representing the theoretical maximum moment the beam can sustain. Once reaching this state, additional deformation can occur with no additional increase in bending moment resistance.

3.1.3 Plastic Hinge Development

From a practical standpoint, it is desirable to not only define the plastic moment, but to ensure the plastic moment occurs at a point away from the column face within the beam section. Brittle failure at the welded connection of the beam and column can occur when the applied moment exceeds the flange weld strength. The intent of all RBS connections is to limit the moment applied to the column face and ensure the connection plastic moment is reached before the failure limit of brittle components are reached. In this way, the system remains intact while energy dissipation occurs during a seismic event. The point of eccentricity is important in design and is defined as the distance away from the column face along the beam section in which the plastic hinge of the connection is meant to occur.

3.2 Specimen Design

The following design considerations were made:

- The couple created by the force in the flange plates was assumed to provide all the moment capacity of the connection.
- Force in the flange plate was assumed to be equally resisted by all bolts.
- For the web plate, design of the bolts was based on the necessary shear capacity at the column face.
- The web plate shear demand was multiplied by a factor of 1.4 to assure the gravity carrying capacity of the connection would survive plastic hinge formation of the flange plates.

3.2.1 Reduced Area Connection Design

Figure 2 of Appendix A shows the dimensions and configuration of the weak axis prototype test specimen. Design of the prototype specimen involved determining the desired maximum plastic moment at the column face from beam properties defined in the AISC Design Manual. The plastic moment at the column face was determined by multiplying the plastic modulus of the beam by its nominal material yield strength. The plastic moment at the column face was reduced by 80 percent to protect the flange weld from fracture. Based on the reduced column face plastic moment, the desired plastic moment of the beam section at the splice connection was determined by multiplying the moment by a linear ratio, with a zero moment assumed at beam mid-span. From the

determined splice connection plastic moment, the required area of the splice plate was determined. The half section flange and web areas were then calculated from the W16X40 beam geometry. An example calculation for the design of the splice connection of the test specimen is provided in Appendix B.

A method to provide the reduced section for varying beam sizes was developed. Percentages of the reduced section dimensions for the half beam section area are necessary to provide the required splice connection plastic moment. These percentages apply to the connection size for different beams that may be considered. Assumed values of thirty five percent of the half beam section flange height and sixty percent of the half beam section flange width provide consistent dimensions for the design of different beam section members, while providing the required splice connection plastic moment.

The required web plate shear resistance was calculated by dividing the determined splice connection plastic moment by the beam span half-length. Web plate shear resistance was increased by 1.4 to assure a significantly stronger connection. The amplification factor of 1.4 is recommended from the SAC interim guidelines for Grade 50 steel. This increase is to assure the web connection does not fail, which would likely lead to the loss of gravity support of the beam. Beam bending is assumed to be resisted solely by the flange plates.

3.2.2 Eccentricity

The point of eccentricity is defined as the middle of the connection. This point is the assumed location of the plastic hinge that forms during severe cyclic loading. FEMA

350 recommends the location be at least half of the beam depth from the column face for any connection.

To allow for minimal interaction between the splice connection and the connection at the face of the column, the distance from the point of eccentricity to the face of the column was set equal to the beam depth. This locates the splice at a distance sufficient to accommodate connection bolts along the splice flange.

3.2.3 Bolt Specifications

All bolts used were 7/8" A325-N type with threads excluded from the shear planes. The 1994 AISC Manual of Steel Construction (Vol. 2, Table 8-11) provides a shear strength of 21.6 kips and was used to determine the number of bolts for both the flange plate splice and the web plate. Bolt spacing for web and flange plate connections were set at three inches. The 1994 AISC Manual (Vol. 2, Table 8-18) also provides for the number of bolts based on bolt group eccentricity and spacing.

3.2.4 Weld Specifications

The nominal strength of all welds was assumed to be 70 ksi. Welds between the stub and column web were made in the flat position. Welds to the column flange were made in the vertical position. All welds used E-70 wire welding electrodes with 5/16-inch single-sided fillet welds. Welding was preformed in the laboratory by a local certified commercial welder.

3.3 Specimen Construction

3.3.1 Connection Description

The test specimen was fabricated and tested at the Structural Mechanics Laboratory at San Jose State University. The specimen was assembled from a W12X120 column and a W16X40 beam. Test specimen configuration and instrumentation layout are shown in Figure 3.

The reduced beam section was fabricated from a segment of the same hot-rolled stock section as the beam. Coupon tests were conducted for all steel components. Based upon these results, the beam and column members were determined to be ASTM Grade 50 steel. Table 1 lists mechanical properties for both pieces of the specimen.

The main components for the test were the beam, stub, column, actuator, and support blocks. The actuator provided a single fixed location force that developed rotation in the beam. Support blocks consisted of steel frames bolted to the floor of the laboratory. The beam stub was constructed from plate, with the flange width increased to allow for welding to the weak axis of the column (column flange). The specimen was tested in a horizontal position and out-of-plane bracing for torsional control was provided for the beam near the actuator and near the connection.

Figure 4 shows the field splice connection member details. The reduced section flange plates connect both the top and bottom flanges of the beam to the stub and a web plate connects the webs of the two members. These three components are bolted to both

the beam and the stub to allow for quick erection in the field. Shop welds connect the stub to the column.

3.4 Experimental Variables

This phase of the research initiative covers the last experiment in a series of three. The first two tests were completed by prior researchers. The first test involved a strong column orientation with a splice connection located at the face of the column. The second test involved a strong column orientation with a splice connection located away from the face of the column very similar to the connection of the current phase. Testing considered in this thesis involved a weak column orientation with a splice connection located away from the face of the column.

3.5 Test Procedure

The intent of the experimental program was to quantify the moment versus rotation relationship for the bolted connection. Testing was conducted over several days due to the length of time necessary to complete cyclic loading. Readings of the instrumentation for force and displacement were taken at two-second time intervals. The splice plate connection was subjected to a combination of shear, moment, and displacement, according to a pre-determined displacement controlled cyclic loading history. The loading sequence was based on the recommended guidelines for loading history specified by the AISC Manual. A figure showing an idealized loading protocol is shown in Figure 5 and the actual loading, which occurred during testing, is shown in

Figures 6 and 7. Loading was applied with a single hydraulic actuator located near the tip of the beam, away from the column.

3.6 Instrumentation

The data collected was the applied load, displacement at various locations, and time for each data point. One load cell measured the force in the actuator. Wire potentiometers placed at various locations, as shown in Figure 3, were used to measure displacements at various locations of the beam, stub, and connection. The data acquisition system consisted of an IBM-PC based system with real time data collection capability. Prior to testing, specimen geometry was measured and measurements were recorded. Initial voltages in each channel were also recorded. When a significant event occurred, such as weld fracture, loading was stopped and photographs of the specimen were taken. The specimen was displaced until a significant loss of moment capacity was observed and the maximum displacement allowed by the test arrangement was reached.

The displacement controlled loading protocol consisted of positive and negative displacement of the beam by the actuator. Beam tip displacement was monitored visually by a dial gauge and a scale was used to retain a permanent reference to the initial position. Five channels were used to record test data and the location of each is shown in Figure 3. Channel 2 monitored the actuator force applied to the beam during each cycle. All other channels were displacement recordings. Channel 3 was placed at the tip of the beam to record transverse beam displacement.

Channel 6 was placed approximately twice the eccentricity from the column face to record transverse beam displacement near the connection to provide for an additional way to calculate the hinge rotation. Data was monitored and recorded at two-second increments using an IBC-PC based system. Data collected was recorded in volts and then converted to kips and inches using previously determined calibration factors.

Channels 4 and 5 were placed approximately twice the eccentricity length from the column face to record the axial elongation of the flange plates parallel to the beam. Channels 4 and 5 were used to calculate the rotation of the splice (hinge) during each cycle.

3.7 Data Collection and Reduction

3.7.1 Data Format

Voltage data was originally recorded as ASCII text and imported into a Microsoft Excel Spreadsheet. The appropriate calibration factor was multiplied to each cell after either adding or subtracting the initial voltage reading. Readings for the actuator were multiplied by a calibration factor.

3.7.2 Data Reduction

Due to the limited maximum stroke of the actuator, spacing blocks were placed on each side of the beam and repositioned in order to allow for the large displacements required for testing. Each time the blocks needed to be relocated, the beam was unloaded to zero force. At zero force, the blocks were disassembled, the actuator repositioned, the

blocks replaced, and the test continued with additional displacement applied in the original direction of loading.

This repositioning resulted in several short unloading and reloading excursions where the force drops and rises back near to the original point on the curve. These excursions showed very little hysteretic energy dissipation due to their essentially elastic behavior. Because this repositioning of the blocks is not representative of the expected behavior of the connection and showed little effect on the total energy dissipated during the experiment, these excursions were deleted from the data to produce a cleaner hysteretic graph. Raw data after removal of these zero-energy excursions are shown in Figure 8.

During testing, wire potentiometers at Channels 3 and 6 required adjustment to accommodate beam displacement during large displacements. The wire length was doubled by looping the wire through the beam attachment and connecting it back to the potentiometer. Data collected from these channels were adjusted by doubling recorded values. For clarity, all remaining discussion and figures in this report pertain to the data after deleting and adjusting, as shown in Figure 9.

3.8 Predicted Results

Predicted linear displacements were derived from beam theory calculations based on the beam, column, and connection physical parameters. Using the AISC loading protocol described earlier, the actuator load is considered as a cantilever. The predicted deflection was determined by calculating the deflection of portions of the total system.

Two components of deflection were calculated as the deflection due to elastic bending and deflection due to hinge rotation.

Elastic bending deflection due to cantilever bending was based on elastic beam theory relating deflection to force applied. The sum of the two deflections for each method of hinge rotation considered, predicted the total deflection and the general equation for total deflection is provided as Equation 1.

$$\Delta_{\text{Total}} = \Delta_{\text{Cantilever}} + \Delta_{\text{Hinge}} \quad \text{Equation 1}$$

Two methods were considered to determine hinge rotation. Method No. 1 provides hinge rotation values for each data point by dividing the displacement at the channel location (Channel 6) by a distance from column centerline along the beam to the point at where displacement readings were taken, approximately twice the eccentricity.

$$\Theta_{\text{Hinge (M.1)}} = \Delta_{\text{Ch6}} / (L_{\text{Ch6}} + 1/2 * d_1) \quad \text{Equation 2}$$

Method No. 2 provides hinge rotation values and are determined for each data point by calculating the difference in relative displacement of the beam stub in relation to the beam divided by the distance between the two instruments.

$$\Theta_{\text{Hinge (M.2)}} = \Delta_{\text{Ch 4}} + \Delta_{\text{Ch 5}} / d_2 \quad \text{Equation 3}$$

Total rotation values considering each method are shown in Equations 4 and 5.

$$\Delta_{\text{Total}} = PL_t^3/3EI + \Theta_{\text{Hinge (M.1)}} * L_h \quad \text{Equation 4}$$

$$\Delta_{\text{Total}} = PL_t^3/3EI + \Theta_{\text{Hinge (M.2)}} * L_h \quad \text{Equation 5}$$

Predicted actuator loading and total deflection values are shown in Table 2. These predicted values are based upon a model developed by McMullin (1997). A schematic drawing of the structural model is shown in Figure 10.

Chapter 4 Experimental Test Results

4.1 Test Specimen Behavior

4.1.1 General

Several interesting events occurred during the experiment that illustrated the behavior of the specimen. At smaller loading forces, the specimen behaved within the expected linear elastic performance as determined by simple beam equations, with no observable yielding. At primary cycle Number 20, the specimen showed minor bolt slip with some yielding observed of the stub plate at the face of the column flange. A few excursions later at primary cycle Number 23, significant bolt slip was observed and the weld return at the column flange cracked. The recorded data at this point was 22.20 kips, while total displacement at the beam tip was 1.8 inches.

At primary cycle Number 27, the longitudinal weld on the bottom of the flange stub fractured along the full length and the transverse weld started to tear across the web of the column. At this point, the recorded value was 28.01 kips, while total displacement at the beam tip was 3.6 inches. Sliding of one of the support blocks became evident at this point and the test was temporarily halted while the bolts securing the support frame to the floor were tightened before testing was continued. After the longitudinal weld on the bottom of the flange stub broke, loss of strength in the connection was noted thereafter during negative cycles.

Thirty-five complete cycles of loading were applied during testing. During tension loading of primary cycle Number 36, the maximum travel inside the beams testing arrangement was reached and since the welds had fractured, the test was ended.

The maximum force recorded during any positive excursion was 33.42 kips during cycle Number 31. The maximum force recorded during any negative excursion was 28.00 kips and the test at conclusion of negative excursions could only develop a force of 5.10 kips due to extensive fracturing of the weld connecting the stub to the column. At that point, a total displacement of 9.6 inches was recorded.

Loss of the flexural resistance at the column face prevented the flange plate from developing large forces. Due to the lower forces in these plates, limited bolt slip and plate yielding or buckling were seen. Hence, the splice plate connection did not perform in the semi-rigid manner intended.

4.2 Observed Failure Modes

4.2.1 Weld Fracturing & Yielding

The observed failure mode was weld fracture initiating at the weld return of the column flange of the bottom flange stub and progressing along the weld between the column and the beam stub during negative cycles. Complete separation in the weld between the column and the beam stub was noted. It appeared that insufficient weld penetration into the column had occurred in this region. Fracture initiation was likely due to concentrated strain at the terminus of the weld between the column and the beam stub.

This fracture was observed only at the bottom of the beam and was not apparent at the top of the beam. Since strain patterns at both flanges should be similar, the non-symmetric behavior suggests an inferior weld along the one connection.

Other portions of the connection remained intact. During positive rotations the top flange showed yielding at the beam stub flange. Buckling of the splice connection along the flange was observed. Bolts did not deform and the web plate remained intact throughout the test.

4.3 Graphs of Cyclic Behavior

Data was converted to engineering terms used to produce graphs showing different relationships of force and displacement of the test specimen during testing. The areas of interest for the specimen include the total rotation at the beam tip versus the moment at the centerline of the column, the rotation and moment at the splice connection, and displacement and force at the flange plate.

Figure 6 shows the displacement time history during the experimental testing and Figure 7 shows the force time history. A maximum force of 33.42 kips was resisted during primary cycle Number 31. Strength degradation occurred in subsequent cycles after weld fracture initiated in this cycle.

Figure 9 shows force displacement at the beam tip and illustrates the capacity of the connection during cyclic testing. Figure 11 shows the force actuator displacement at the hinge while Figure 12 shows moment at the column centerline versus rotation of the

beam. It should be noted that these figures include rotation after weld fracture had occurred.

Figure 13 shows moment rotation at the splice connection and was determined from displacement values at Channel 6 (Method No.1). Figure 14 also shows moment rotation at the splice connection and was determined from displacement values at Channels 4 and 5 (Method No.2).

One issue of interest was the ability of the web plate to minimize shear deformation during loading. Figures 15, 16, and 17 show the displacement effects due to different portions of the connection. Each graph represents a portion of the tip displacement during loading. Equation 6 is provided showing the different portions of displacement.

$$\Delta_{\text{Total}} = \Delta_{\text{Cant.}} + \Delta_{\text{Hinge}} + \Delta_{\text{Sh.}} + \Delta_{\text{Misc.}} \quad \text{Equation 6}$$

$$\Delta_{\text{Total}} = \Delta_{\text{Ch. 3}} \quad \text{Equation 7}$$

$$\Delta_{\text{Cant.}} = PL_t^3/3EI \quad \text{Equation 8}$$

$$\Delta_{\text{Hinge}} = (\Delta_4 - \Delta_5) * L_h/d_2 \quad \text{Equation 9}$$

$$\Delta_{\text{Sh}} = \Delta_{\text{Total}} - \Delta_{\text{Cant.}} - \Delta_{\text{Hinge}} \quad \text{Equation 10}$$

$$\Delta_{\text{Misc.}} = \text{Assumed Zero} \quad \text{Equation 11}$$

Figure 15 shows percent displacement due to cantilever bending of the beam versus shear and was determined from the classic beam theory for cantilever bending. Figure 16 shows percent displacement due to hinge rotation at the splice connection versus moment and was determined from displacement values at Channels 4 and 5.

Figure 17 shows percent displacement due to shear of the web plate. Values for web plate shear were determined by subtracting the displacement due to cantilever bending of the beam and the displacement due to hinge rotation from the total beam tip displacement recorded during testing.

These graphs show the different percent displacements that each component contributes to the overall displacement. The graphs show percentages for larger positive actuator forces (greater than 20 kips) because smaller forces tend to distort percentage results. These distortions may be the result of relatively large hinge or web plate shear displacements during small applied actuator forces. Only the positive quadrant of each graph is shown for clarity.

The graphs show that the largest percentage of displacement was due to hinge rotation and the smallest percentage of displacement was due to web plate shear. For example, during the largest applied actuator force, hinge rotation resulted in 60 percent of the total displacement, while displacement due to cantilever beam bending resulted in 28 percent and web plate shear resulted in 12 percent. In terms of actual displacement, during the applied actuator force of 33.42 kips, the recorded total beam tip displacement was 6.2 inches, while hinge rotation using Method No. 2 was 3.7 inches. Displacement due to cantilever beam bending was calculated as 1.7 inches and displacement as a result of web plate shear was 0.7 inches. Displacement resulting from other miscellaneous factors, such as bolt slip or support movement was assumed to be insignificant.

Table 3 provides specimen performance information for Specimen S-3 testing. Performance results for S-2 testing (from previous researchers) are provided for general information. The overall energy dissipated during testing was 530 kip-in. The maximum moment achieved during testing was 3900 kip-in. with a peak hinge rotation of 0.047 radians. The yield strength moment was 2100 kip-in. A strength degradation of 20 percent was observed at a maximum displacement of 9.55 inches.

Chapter 5 Analytical Model of Building Frame

5.1 Analytical Model

5.1.1 DRAIN-2DX Program

Analytical studies of a model building frame were performed using a common nonlinear structural analysis software, DRAIN-2DX, Version 1.10 (Allahabadi and Powell, 1988). The program allows for two-dimensional structural models assembled with nonlinear elements to be subjected to ground acceleration input records. Use of this modeling program was selected due to the ability to simulate frames with rigid and semi-rigid connections. This was necessary to allow for suitable comparison of realistic modeling of seismic events.

5.1.2 SAC Project 3-Story Frame Model

The model structure selected for analysis was a benchmark structure developed by the SAC Joint Venture. Model structures for the Los Angeles area were designed by Brandow & Johnson Associates, Consulting Structural Engineers, Los Angeles, California. A description of the model building is described in detail in Gupta and Krawinkler (1999). A three-story structure was selected for analytical modeling since the author believes low-rise commercial construction is well suited for semi-rigid column-tree construction. The benchmark model was originally designed with a three-bay partial perimeter moment resisting frame in both directions. The frame considered for this

analysis is in the transverse direction. In the longitudinal building direction, a similar three-story three-bay rigid frame is used for lateral resistance. It is expected that little change in building behavior would result from the choice of the building direction modeled. Figure 18 shows beam and column sizes and Figure 19 provides a plan and elevation of the model building.

5.1.3 Loading

Due to building symmetry, each frame was assumed to support one half of the seismic mass of the entire structure. Accidental eccentricity defined by building codes would slightly increase the mass supported by each frame but such issues were considered minor for this preliminary level analysis. For modeling purposes, mass was lumped at each end node and mid-span. Building mass values are described in Gupta & Krawinkler (1999). Building dead loads are determined based on an assumed composite slab construction and appropriate code dictated live loads. The building was designed with a typical floor dead load of 96 pounds per square foot and a live load of 20 pounds per square foot.

5.1.4 Ground Accelerations

Six different ground acceleration time history records, as listed in Table 4, (Somerville et. al, 1997) were selected for evaluating the building response. These time history records were all developed from three ground motion records:

- 1995 Kobe Earthquake, Fault Normal & Fault Parallel

- 1989 Loma Prieta Earthquake, Fault Normal & Fault Parallel
- 1994 Northridge Earthquake, Fault Normal & Fault Parallel

These ground motions are recorded from significant earthquakes in high seismic zones.

All records are from regions comparable to Zone 4 in the Uniform Building Code.

Somerville et. al. (1997) developed the six records using the following process.

Each original motion recorded was rotated 45° with respect to the fault in order to minimize directivity effects. Each record represents a ground motion with a return period of 2475 years (2% probability of being exceeded in 50 years). The set of records used in this project were assembled by the SAC steel research project to represent the geographic location designated as Los Angeles (an area of high seismic activity). Each record was scaled such that, on average, the spectral values match with a least squares error fit to United States Geological Survey (USGS) mapped value. The USGS target spectra were modified to be representative of stiff soil (soil type S_D per UBC).

The SAC steel research project altered the original historical ground motion pair in the following ways:

- The two horizontal components recorded during each earthquake were resolved into vector fault-normal and fault-parallel directions.
- The records are adjusted in the frequency domain to have characteristics appropriate for National Earthquake Hazard Reduction Program (FEMA 350) S_o soil sites.
- The amplitude of the each acceleration record is scaled so that the average of the two horizontal spectra match the target spectrum developed from the USGS data.

- Each individual component was rotated 45 degrees away from the fault-normal fault-parallel axis orientation to in order to minimize directivity effects.

5.1.5 Member Specifications

One goal of this project was to show the benefit of semi-rigid connection construction. To allow for direct comparison of rigid frame and semi-rigid frame behavior, the cross-sections used in the analysis were chosen to be the same as the original SAC benchmark model. Cross-section properties for the columns and beams used for the model are listed in Table 5. Beam and column sizes and locations are shown in Figure 18. Beams were assumed to be 36-ksi steel, while columns were of 50-ksi steel. Beams vary in size, with lighter beams toward the top floor and heavier beams towards the ground. Column sizes vary also with lighter columns located at the end of the frame and heavier specified in the middle.

5.1.6 Eccentricity

Eccentricity of a semi-rigid building frame is defined as the distance of the field splice centerline to the centerline of the column. It is recommended that the eccentricity be at least equal to the beam depth plus half the column depth (FEMA 350, 2000). The DRAIN-2DX software analyzes column and beam elements connected by nodes. The eccentricity distance was determined using the largest column and beam sections (i.e., lowest floor) and was constant for all bays and floors. In order to simulate a semi-rigid connection, it was necessary to locate two nodes at the field splice centerline location.

5.1.7 Node Assignment

Figure 20 provides a key to assist in identification of nodes. There are 62 nodes in the model and each node is numbered according to its relative position horizontally and vertically. The first two digits represent the floor while the second two represent the node in the horizontal direction starting at 01 with the first node on the first floor being 0101 and the last node on the third floor is 0319. The bottom floor was considered as 00. There are 19 nodes at each floor level along the three bays. This configuration was chosen to allow additional nodes to be added for future analysis of additional floors and bays. No additional floors or bays were considered in this thesis.

Node 401 was added mid-span of the top leftmost column. This node was added to allow relative displacement information to be obtained. Relative displacements of Nodes 301, 302, 304, 401 were used to determine rotation values for post-analysis evaluation of the LA21 and LA 26 ground motions. Output from the analysis was used to develop moment rotation curves shown in Figures 21 and 22.

5.1.8 Rigid and Semi-Rigid Configurations

Two different models were developed for inelastic analysis. The rigid frame model (Model FR) consisted of three bays of a rigid frame having moment connections at each beam-column joint and at each floor and the field splices. The semi-rigid frame model (Model PR) was the same configuration, with the only difference being bilinear, semi-rigid connections were assumed at the field splice connections.

The base plate of both the rigid and semi-rigid models were fixed. Gravity loads from building dead weight were applied to the nodes at each beam-column joint as well as mid span of each beam.

5.2 Development of Semi-Rigid Model Parameters from Experimental Data

5.2.1 S-2 Specimen Testing

This thesis included experimental testing of a weak axis connection (Specimen S-3) as discussed in Chapters 3 through 5. Earlier testing conducted as part of an overall study of column tree connections involved testing of a strong axis connection similar to the design of Specimen S-3. Data from this previous experiment (Specimen S-2) was utilized in this thesis to provide appropriate results to compare with the work conducted by the SAC project. Figure 23 shows hinge rotation versus moment at the hinge from testing of Specimen S-2 (comparable to Figure 13 for Specimen S-3).

The Specimen S-2 field splice connection dimensions and materials were similar to dimensions and materials used in the Specimen S-3. Bolts, welds, and test setup were similar. However, because the Specimen S-3 was connected to the weak axis, a direct correlation for use in analytical modeling was deemed unsuitable.

5.2.2 Model of Semi-Rigid Connection

Control of the behavior of rotational connections during seismic analysis is accomplished by assigning connection parameters that allow rotation of the spring when the plastic moment of the beam is reached. By assigning slaving, the two nodes at each

splice connection are locked and therefore, not allowed to rotate. Bi-linear rotational spring elements were required in the DRAIN-2DX program based on stiffness and moment capacities for beam members of the SAC model frame. The modeling of the rotational connection was intended to represent a semi-rigid connection similar to the one tested experimentally and discussed in Chapters 3 through 5.

In order for the semi-rigid model to simulate inelastic behavior, the rotational degree of freedom at the splice location is released and relative rotation of the two nodes at the splice is allowed. In this semi-rigid model, only the bilinear rotational spring will provide flexural stiffness representing the splice connection.

5.2.3 Spring Constant

Spring constants used as input were based on a best fit of the available experimental data for Specimen S-2 shown in Figure 24. The best fit was determined by visual means and spring constants for each floor. Normalized moment rotation information was used to provide a best-fit bilinear curve for model input. The first portion of the curve represents elastic behavior and the yield point is shown is at 70% of the normalized moment at 0.005 radians of rotation. The ultimate plastic moment was estimated at 150 % of the normalized moment at 0.08 radians of rotation.

Table 5 shows model input parameters. The plastic stiffness represents the second portion of the bi-linear curve. The yield moment represents the applied moment at the transition from elastic to plastic behavior for each of the various beams at each floor.

The elastic stiffness represents the yield moment divided by the rotation at yielding.

Each of the values shown in Table 5 were calculated by the following equations:

$$k_{el} = 0.7 * M_p / 0.005 \quad \text{Equation 12}$$

$$k_{pl} = (1.5M_p - 0.7M_p) / (0.08 - 0.005) \quad \text{Equation 13}$$

$$S_{pl} = k_{pl} / k_{el} \quad \text{Equation 14}$$

5.2.4 DRAIN-2DX Input File

A representative analytical input file is provided in Appendix C. Input parameters for cross-sections of beams and columns for the DRAIN-2DX program are from the AISC Manual. Parameters for the rotational connection include the ultimate plastic moment, plastic stiffness, and yield moment. An explanation of the various input categories is described in Table 6.

5.3 Parametric Study

Variables considered during analysis were the different ground motions and the rigidity of the field splice connection. Ground motions considered were for the LA21 through LA 26 seismic events, as defined by SAC (Somerville, 1997). The author selected these ground motions because they represent a diverse sample of earthquakes that have recently occurred and which have resulted in significant damage to steel moment frames. Due to excessive amount of computational time, additional ground motions were not considered.

Chapter 6 Analytical Results

6.1 Overview

Inelastic demand of the connection was quantified by conducting analytical runs on the two model structures. The connection demand for both the rigid (Model FR) and semi-rigid (Model PR) frames allowed for direct comparison of both global and local behavior. Output from the analysis provided nodal displacement, element forces, and hinge elastic rotations for each model frame for each ground motion. Output from Model FR provides information on how a rigid connection would behave during a given seismic event and provides a benchmark for comparison. The Model PR represents a semi-rigid column tree frame and corresponding connection behavior.

6.2 Global Model Behavior

6.2.1 Period of Vibration

Table 7 lists the period of vibration and mass participation for the two models. The period of vibration increased from 1.00 second for Model FR to 1.29 seconds for Model PR. This increase shows that the semi-rigid field splices result in a significantly more flexible structure. However, model mass participation does not significantly change.

6.2.2 Global Building Behavior

Maximum values for global results are listed in Table 8. Peak base shear was reduced in the semi-rigid frame for all ground motions. For example, in the LA26 ground motion, maximum base shear was reduced from 3300 kips for the rigid frame to 2870 kips for the semi-rigid frame.

Another significant global result was the source of energy dissipation. In Table 8, results for element groups show that the beam elements provided the greatest energy dissipation for the rigid frame, as expected. Results for the semi-rigid frame show energy dissipation by the beam elements was essentially eliminated, with the semi-rigid field splice connection elements dissipating the energy during seismic events. Semi-rigid connections dissipated up to 98 percent of the energy of the structure.

Likewise, for the LA26 ground motion, the maximum lateral roof displacement for the rigid frame was 14.36 inches and the semi-rigid frame had a slightly larger displacement of 14.85 inches. The LA26 ground motion produced the largest lateral displacement of all analysis conducted.

Figure 25 represents maximum base shear and roof drift for both models for all ground motions considered. The semi-rigid model base shear was reduced between 13 to 39 percent with an average reduction of 21 percent. The semi-rigid model roof drift varied between a 64 percent increase for the LA 24 ground motion to reduction of 11 percent for the LA 21 ground motion. On average, the semi-rigid model increased roof

drift by 15 percent. The LA 21 ground motion was the only ground motion which resulted in a simultaneous reduction in both base shear and roof drift.

6.3 Local Model Behavior

6.3.1 Local Connection Maximum Rotation

Table 9 provides the maximum rotation recorded for any hinge in each of the rigid and semi-rigid models analyzed. Values show a consistent increase in rotation for the semi-rigid frame model for all seismic events. Rotation values for the rigid frame primarily consist of beam rotation at the column face, while semi-rigid values primarily consist of splice connection rotation.

6.3.2 Local Connection Performance

Table 10 provides the elastic stiffness, yield moment and ultimate moment from a best fit backbone curve from moment rotation curves shown in Figures 21 and 22 for the ground motions considered. For the two ground motions considered (LA 21 & LA 26), the elastic stiffness, yield moment and ultimate moment values were reduced for the semi-rigid connection model. Rotations were calculated from the relative displacements of nodes 301, 302, 304 and 401 during each seismic event. Translation in the horizontal direction was calculated by dividing the difference of nodes 301 and 401 by half the story height of the top floor. Translation in the vertical direction was calculated by dividing the difference of nodes 302 and 304 by the distance from the point of eccentricity to

middle of the beam. These translations were used to determine rotation of the beam in relation to the column during analysis.

Figures 21 and 22 show that the S-2 connection was able to achieve rotations values greater than those experienced by the model connection during seismic analysis. During experimental testing, the S-2 connection experienced rotation values in excess of 0.08 radians while model connection splice rotation values were about half that value.

6.3.3 Local Connection Behavior

Figures 26 through 31 show the rotation values determined from analysis of each model during each seismic event. The column, panel zone, stub, and beam rotations are shown in smaller boxes at each element and the overall rotation is shown in the larger box above the element. Figure 32 provides a key for the location of each element rotation and its corresponding location on Figures 26 through 31.

Moment rotation graphs for both the rigid and semi-rigid connections for the LA21 ground motion and LA26 ground motion are shown in Figures 21 and 22. These two events were more closely reviewed because they had the highest peak acceleration values for all seismic events considered. Values for the bending moment were normalized by the plastic bending strength of the beam where the hinge was located.

Chapter 7 Comparison of Experimental Capacity and Analytical Demand

7.1 Discussion of Model Performance

This thesis provided for a comparison of experimentally determined capacities and analytically determined demands in an attempt to define the suitability of semi-rigid splices in moment resisting frames.

7.2 Comparison of Graphs

The hysteresis loops for both the semi-rigid and rigid connections were superimposed on the hysteresis loop for the normalized S-2 specimen testing results. Moments were obtained from analytical output and rotations were calculated from relative displacements of nodes 301, 302, 304, and 401, as previously described. These comparisons are shown in Figures 21 and 22.

The analytical results showed moments and rotations greater than the experimental model (Specimen S-2). Maximum rotation for the semi-rigid frame exceeded the maximum rotation of the rigid frame while maximum moment was greater for the rigid frame. Although the experimental specimen did not achieve rotations and moments sufficient to carry demands imposed by the ground motions analyzed, values shown in Figures 21 and 22 suggest the rigid frame would have significant difficulty

carrying the seismic loads as well. This suggests that reconsideration of the base frame model beam and column sizes may be necessary.

Rotation of Specimen S-3 as tested showed a maximum rotation of 0.047 radians. This value is comparable with a maximum local rotation of 0.050 radians for the LA 26 ground motion analyzed for the semi-rigid frame, as listed in Table 9. The elastic stiffness of the specimen tested showed a value of 2.1×10^5 kip-in. per radians which was comparable with the elastic stiffness of the 2.0×10^5 kip-in. per radians for the LA 21 ground motion analyzed for the semi-rigid frame. The yield moment of the specimen tested showed a value of 2100 kip-in. The plastic stiffness of the specimen tested showed a value of 3.8×10^4 kip-in. per radians. The strength degradation of the specimen tested showed a value of 20 percent at a maximum displacement of 9.55 inches.

7.3 Web Plate Shear

Figure 16 shows that displacement due to rotation at the splice connection contributed to approximately 60 of the overall beam displacement. Figure 17 shows that displacement due to shear at the splice connection was limited to approximately 12 of the overall beam displacement. Specific conclusions relating model hinge shear deformation to experimental testing results cannot be made, as analysis was not conducted to determine actual splice shear deflection during analysis of the model.

7.4 Accuracy of Bilinear Model

The bilinear curve used for development of model input parameters was based on a best-fit assumption. A determination of accuracy is difficult to make given the many uncertainties associated with the connection. In addition, no mechanics based model was found suitable for predicting hinge behavior.

7.5 Normalized Base Shear and Roof Drift Angle

Table 11 provides values for the normalized maximum base shear and roof drift angle. Base shear coefficient values were calculated by dividing the maximum base shear by the building weight resulting in a seismic coefficient. Roof drift angle was calculated by dividing the top floor maximum top floor displacement by the building height. Figure 33 is a graph of each of these peak values for each ground motion. Also shown is a line representing the maximum base shear versus roof drift angle from the non-linear static analysis portion of the Gupta and Krawinkler (1999) study. These values represent a typical pushover analysis result and are shown for reference. The LA23 and LA 24 motions closely follow the pushover curve. However, the other four motions show significantly higher base shear values than those expected by static analysis. This may relate to discrepancies in post yield behavior assumed by the two analysis models

7.6 Rigid versus Semi-Rigid Rotation

Table 9 provides maximum rotation values determined from the analysis. These values represent the maximum beam column connection rotation for the rigid model and maximum rotation at the splice connection for the semi-rigid model. These are the maximum inelastic rotation that the hinge received at any point during the time history.

For the rigid frame, the hinge forms at the ends of the beam elements, with a large inelastic rotation at the end of the bay and smaller inelastic rotations at nodes closer to the middle of the bay. For example, the leftmost bay of the top story has nodes 301, 302 and 303 representing a rigid connection (41” inches from the column centerline). These nodes received a combined inelastic rotation of 0.0372 radians during the LA26 ground motion. The other end of the element, at nodes 305, 306 and 307 receives a combined inelastic rotation of 0.0385 radians. At the same location for the semi-rigid connection, the connecting element received 0.0464 radians and 0.0493 radians of inelastic rotation at the end closer to the column and no inelastic rotation at the mid-span node. There is no concentrated elastic rotation in the rigid model, as the beam elements model elastic rotation as a distribution over their length. In an actual steel beam, yielding and inelastic bending is going to be distributed over a length of beam that is perhaps twice the depth of the beam. During analytical modeling, this distributed inelastic behavior is idealized as a concentrated inelastic hinge rotation.

On the other hand, for the semi-rigid frame, almost all inelastic rotation occurs in the splice connection. This connection experiences elastic rotation up to the yield point

and then inelastic rotation thereafter. The total rotation of the connection is then a combination of both concentrated elastic rotation and inelastic rotation. The values in Table 9 show a consistent increase in the semi-rigid splice connection rotation.

Chapter 8 Conclusions and Recommendations

8.1 Results of Original Research Objectives

The research objectives of this thesis were stated in Chapter One. The first objective was achieved by conducting one experiment testing a full-scale specimen under cyclic loading. Assumptions for the original design produced a connection that maintained strength at large rotations. A design procedure was developed that can be used to size connections for different size frame beams. Unfortunately, the desired failure mode was not seen during testing of Specimen S-3. Instead, cracking of the welds between the stub and column allowed for large rotations. While weld fracture was not intended, the connection did maintain flexural and shear strength. It is certainly believed that the complicated geometry of attaching to the weak axis of the column contributed significantly to the actual failure mode.

The second objective was not fully achieved because specimen testing results were inconsistent, to the point where a proposed multi-linear prediction curve was deemed unacceptable. The author recommends additional study in this area, with a specific investigation of the mechanisms contributing to connection behavior being considered. In lieu of a mechanics-based prediction method, a simple bi-linear curve fit of experimental data was used to determine input parameters for the analytical model.

This curve may not accurately describe the behavior of the connection during elastic and inelastic behavior due to complex nature of the connection, but can be used in

preliminary analysis to understand connection behavior. In addition, its' accuracy for larger connections has not been experimentally verified.

The third objective was achieved by conducting analytical modeling of both a rigid and semi-rigid frame structure using a collection of ground motions. The analytical model provided output for global and local performance, quantifying both elastic and inelastic behavior. The rigid frame model provided benchmark results, allowing for comparison of the semi-rigid connection.

The fourth objective was achieved by analytical modeling to quantify performance of local and global parameters for the semi-rigid connection. As expected, the semi-rigid connection limited inelastic behavior at the column face. The field splice connection dissipated up to 98 % of building energy during seismic loading. Base shear was reduced, however roof drift was increased.

8.2 Conclusions

This thesis presents results on the plastic and inelastic behavior of an innovative reduced section column-tree connection. The reduced section column-tree connection represents a reliable and practical alternative to conventional steel moment frame systems to enhance earthquake resistance.

Specific conclusions drawn from this thesis indicate:

1. The column-tree connection is a viable alternative providing stable energy dissipation while maintaining acceptable structure displacement.

2. Weld quality is of paramount importance for beam column connections, especially when connecting beams to the weak axis of a column.
3. The semi-rigid column-tree connection provides intended energy dissipation, limiting the inelastic demand at the column by absorbing it into the field splice.
4. Model results showed the semi-rigid connection provides improved behavior by reducing base shear while not significantly increasing roof drift.
5. Modeling results showed base shear was reduced on average by 21 percent while displacements increased on average by 15 percent.
6. Installation of the semi-rigid field splices limited the beam elements to only providing 2 percent of the inelastic energy dissipation.

8.3 Recommendations for Continuation of Research

Due to observed weld fracture, the author recommends that further experimental testing be conducted. In addition, quality control of all welds should be ensured to enable accurate performance during testing. Further analytical modeling should be conducted to assess the performance of weak axis connections, with attention given to their contribution in assisting reduction of building seismic displacement.

Assessment and further study of geometric inelastic behavior of a weak axis specimen is recommended to understand the performance of the connection during severe ground motions. P-delta effects should also be considered, particularly for high-rise buildings. Refinement of the bilinear rotational spring element is also suggested.

This part of a larger investigation of column-tree connections considered replacing rigid bays with semi-rigid column-tree connection frames. Future study should investigate other options including:

- Replacing all rigid and pinned bays with semi-rigid column-tree connection frames,
- Replacing all pinned connections with semi-rigid column-tree connection frames to supplement the rigid bays.

Results of analysis of these alternative configurations should be compared to the traditional rigid bay design.

This thesis considered a limited number of ground motions representing building located in areas with severe seismic activity. Additional study should be conducted to assess connection performance in areas with less severe seismic activity, which may receive more benefit from semi-rigid behavior.

Notations

AISC	American Institute of Steel Construction, Chicago, IL
EI/L	Flexural Stiffness of Beam
FEMA	Federal Emergency Management Agency
F_y	Yield Strength
K	Stiffness
L	Beam Length
P	Force
RBS	Reduced Beam Section
SMRF	Steel Moment Resisting Frame
UBC	Uniform Building Code, 1997
Δ_x	Deflection
Θ_{Conn}	Connection Rotation
d_1	Distance along the beam from Channel 6 to column centerline (34.25")
d_2	Distance between Channels 4 and 5 (26.00")
L_t	Total length of beam from column to actuator (130.88")
L_h	Length to actuator to center of hinge (96.63")
k_{el}	Stiffness – Elastic
k_{pl}	Stiffness – Plastic
S_{pl}	Ratio of post yield stiffness to elastic stiffness

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Recommended Seismic Design Criteria for New Steel Moment Frame Buildings (FEMA 350).

Beam & Column ($F_y = 36$ ksi)	Area (in. ²)	Moment of Inertia (in. ⁴)	Section Modulus (in. ³)	Plastic Moment (kip-in.)
W16X40	11.8	518	72.9	3645
W12X120	35.3	1070	186	9300

Table 1. Test Specimen Mechanical Properties

Set No.	No. of Cycles	Predicted Load (Kips)	Beam Tip Deflection (in.)
1	6	4.66	0.45
2	6	6.37	0.6
3	6	9.33	0.9
4	4	13.57	1.2
5	2	18.96	1.8
6	2	21.37	2.4
7	2	21.61	3.6
8	2	22.54	4.8
9	2	23.32	6.0
10	2	24.09	7.2
11	2	24.87	8.4
12	2	25.80	9.6
13	2	26.58	10.8

Table 2. Test Specimen Loading Protocol

Specimen	Weak Axis Specimen S-3	Strong Axis Specimen S-2
Energy Dissipated (kips-in.)	530	343
Initial Stiffness (kips-in./ radian)	210,000	150,000
Rotation @ Yield (radians)	0.01	0.01
Yield Strength (kips-in.)	2100	1500
Plastic Stiffness (kips-in./ radian)	38,000	26,000
Peak Rotation (radians)	0.047	0.080
Maximum Moment (kips-in.)	3900	3600
Strength Degradation	20% at 9.55" Displacement	14% at 11.05" Displacement

Table 3. Test Specimen Performance

G.M. No.	Record	Mag.	Dist. (km)	Scale Factor	Dur. Pt.	Step (sec)	PGA (cm/sec ²)	PGA (g's)
LA21	Kobe -FN	6.9	3.4	1.15	3000	0.020	1258	1.28
LA22	Kobe -FP	6.9	3.4	1.15	3000	0.020	902	0.92
LA23	Loma Prieta -FN	7.0	3.5	0.82	2500	0.010	409	0.42
LA24	Loma Prieta -FP	7.0	3.5	0.82	2500	0.010	463	0.47
LA25	Northridge -FN	6.7	7.5	1.29	2990	0.005	851	0.87
LA26	Northridge FP	6.7	7.5	1.29	2990	0.005	925	0.94

Table 4. 2-50 SAC Ground Motions Used in Analysis

Notes:

G.M.-Ground Motion Record Designation

FN- Fault Normal Direction

FP- Fault Parallel

Mag.- Earthquake Magnitude

Dist.- Distance from Epicenter to Recorder Location

Step- Time Increment of Data Collection

Beams ($F_y = 36$ ksi)	Area (in. ²)	Moment of Inertia (in. ⁴)	Section Modulus (in. ³)	Plastic Moment (kip-in.)
W24X68	20.1	1830	177	6,372
W30X116	34.2	4930	378	13,608
W36X118	34.7	5900	415	14,940
Columns ($F_y = 50$ ksi)				
W14X257	75.6	3400	487	24,350
W14X311	91.4	4330	603	30,150

Spring Constants

Beam Cross- Section	Elastic Stiffness (kips-in./radian)	Plastic Stiffness Constant	Yield Moment (kips-in.)
W24X68	550,000	0.0762	2,745
W30X116	110,000	0.0762	5,515
W36X118	120,000	0.0762	5,882

Table 5. Model Element Properties

Model Input Categories		
Category	Description	
NODECOORDS	Defines the coordinates of each node in the model	
RESTRAINTS	Defines the degrees of freedom for specific nodes (typically supports)	
SLAVING	Fixes nodes at specific points (Rigid model fixes degrees of freedom at springs)	
MASSES	Assigns mass values representing building gravity dead loads	
ELEMENTGROUP	Beam	Assigns beam element properties
	Column	Assigns column element properties
	Connections	Assigns connection element properties
SECTION	Defines a plane through a section of the frame for determining results	
NODALOAD	Assigns a modification factor representing specific loads at specific nodes	
ACCNREC	Defines the input acceleration record	
PARAMETERS	Establishes the output parameters	
ACCN	Initiates the analysis of the defined acceleration record	

Table 6. Model Input Categories

Mode	Period of Vibration		Frame Mass Participation	
	Rigid	Semi-Rigid	Rigid	Semi-Rigid
1	1.00	1.29	0.82	0.80
2	0.32	0.37	0.14	0.16
3	0.17	0.17	0.04	0.05

Table 7. Model Period of Vibration and Mass Participation

Ground Motion Record		Base Shear (Kips)	Roof Disp. (in.)	Energy Dissipated by Element Group (Kips-in.)		
				Beams	Columns	Splice
LA21	Rigid	3307, -2215	8.49, -13.93	31567	49	0
	Semi	2758, -1946	10.09, -12.36	27	12	18600
LA22	Rigid	2745, -2489	7.87, -11.42	22370	35	0
	Semi	2277, -2032	7.00, -12.19	23	6	11990
LA23	Rigid	1312, -1156	3.34, -3.30	1358	7	0
	Semi	731, -802	4.14, -2.32	21	11	722
LA24	Rigid	1490, -1388	4.38, -4.39	5226	8	0
	Semi	1190, -1099	6.32, -7.18	21	17	6111
LA25	Rigid	2159, -2866	12.24, -6.88	13246	63	0
	Semi	2096, -2314	9.95, -12.46	24	65	7671
LA26	Rigid	2420, -3300	14.36, -10.41	22902	108	0
	Semi	2100, -2870	14.85, -12.67	65	121	14842

Table 8. Model Global Response (Rigid vs. Semi-Rigid)

Model	Ground Motion					
	LA21 (Radians)	LA22 (Radians)	LA23 (Radians)	LA24 (Radians)	LA25 (Radians)	LA26 (Radians)
Rigid	0.0411	0.0311	0.0042	0.0078	0.0348	0.0422
Semi- Rigid	0.0420	0.0406	0.0129	0.0210	0.0398	0.0497

Table 9. Model Frame Connection Maximum Rotation

Model Specimen	Elastic Stiffness (kips-in./radian)	Yield Moment (kips-in.)	Ultimate Moment (kips-in.)
LA21-Rigid	210,000	6,900	10,700
LA21-Semi	200,000	3,900	6,100
LA26-Rigid	190,000	5,700	9,500
LA26-Semi	100,000	4,100	6,000

Table 10. Model Connection Performance

Model		Roof Drift Angle	Normalized Base Shear
LA21	Rigid	0.030	0.51
	Semi	0.026	0.42
LA22	Rigid	0.024	0.42
	Semi	0.026	0.35
LA23	Rigid	0.007	0.20
	Semi	0.009	0.12
LA24	Rigid	0.009	0.23
	Semi	0.015	0.18
LA25	Rigid	0.026	0.44
	Semi	0.027	0.36
LA26	Rigid	0.031	0.51
	Semi	0.032	0.44

Table 11. Model Frame Roof Drift Angle vs. Normalized Base Shear

Appendix A - Figures

Figure 1. Typical Coulmn Tree Connection Moment Frame

Figure 2. S-3 Specimen Dimensions

Figure 3. Specimen Test Setup

Figure 4. Specimen Details

Figure 5. Loading Protocol

Figure 6. Time vs. Displacement at Cantilever Tip

Figure 7. S-3 Time vs. Actuator Force

Figure 8. S-3 Force vs. Displacement at Beam Tip (Raw & "Clean" Data)

Figure 9. S-3 Force vs. Displacement at Beam Tip (Channel 3)

Figure 10. Schematic Drawing of Hinge

Figure 11. S-3 Force vs. Displacement at Hinge (Channel 4)

Figure 12. S-3 Total Rotation vs. Moment at Column Centerline

Figure 13. S-3 Hinge Rotation vs. Moment at Hinge (Method No. 1)

Figure 14. S-3 Hinge Rotation vs. Moment at Hinge (Method No. 2)

Figure 15. Cantilever Bending Percent Displacement vs. Shear at Connection

Figure 16. Hinge Rotation Percent Displacement vs. Shear at Connection

Figure 17. Connection Shear Precent Displacement vs. Shear at Connection

Figure 18. Beam & Column Sizes, SAC L.A. 3-Story Model Frame

Figure 19. Plan & Elevation, SAC L.A. 3-Story Model Frame

Figure 20. Model Node Number Sequence

Figure 21. Beam-Column Subassembly Behavior, LA21 Ground Motion, Experimental and Analytical Results

Figure 22. Beam-Column Subassembly Behavior, LA26 Ground Motion, Experimental and Analytical Results

Figure 23. S-2 Hinge Rotation vs. Moment at Hinge

Figure 24. Bi-Linear Curve Fit

Figure 25. Maximum Base Shear vs. Maximum Roof Drift

a) Rigid Frame

b) Semi-Rigid Frame

Figure 26. Maximum Plastic Rotation Values - LA 21 Ground Motion

a) Rigid Frame

b) Semi-Rigid Frame

Figure 27. Maximum Plastic Rotation Values - LA22 Ground Motion

a) Rigid Frame

b) Semi-Rigid Frame

Figure 28. Maximum Plastic Rotation Values - LA23 Ground Motions

a) Rigid Frame

b) Semi-Rigid Frame

Figure 29. Maximum Plastic Rotation Values - LA24 Ground Motion

a) Rigid Frame

b) Semi-Rigid Frame

Figure 30. Maximum Plastic Rotation Values - LA25 Ground Motion

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Figure 32. Key for Reading Rotation Output, Figures 26-31

Figure 33. Maximum Normalized Base Shear vs. Roof Drift Angle

Appendix B - Connection Design Procedure

Semi-Rigid Connection Design

This section provides procedures for design of a reduced section bolted splice plate utilizing a web plate and column welds. All welds are E70 fillet type, all bolts are 7/8" diameter, ASTM A325-N type. The various parameters used in equations are defined in the 1994 AISC-LRFD Design Manual.

The following example for a connection design of a W16X40, Gr. 50, beam.

Define Input Variables Units

$$F_y := 50 \text{ ksi}$$

$$A_g := 11.8 \text{ in}^2$$

$$d := 16.01 \text{ in}$$

$$T := 13.625 \text{ in}$$

$$t_w := 0.305 \text{ in.}$$

$$t_f := 0.505 \text{ in.}$$

$$b_f := 6.995 \text{ in.}$$

$$Z_x := 72.9 \text{ in.}^3$$

$$\Phi_{shear} := 0.75$$

Step 1: Determine Eccentricity

Eccentricity is determined as a percentage of beam span. The location of “e” is at the center of the semi-rigid splice connection. FEMA 350 recommends its location be at least half of the depth of the beam from the column face. Consideration of the length of the splice plate from column face is necessary to accommodate the proposed number of bolts.

$$l := 113.5 \text{ in.}$$

$$\Phi \ l := 0.15$$

$$e := \Phi \ l$$

$$e = 17 \text{ in.}$$

Step 2: Determine Plastic Moment and Shear

The maximum bending strength of the beam is found as a function of the plastic modulus and yield strength of the beam material. The maximum shear is found by dividing by the half beam span length.

$$M_{pu} := F_y \cdot Z_x$$

$$M_{pu} = 3645 \text{ kip-in.}$$

$$V_{pu} := \frac{M_{pu}}{l}$$

$$V_{pu} = 32.1 \text{ kips}$$

Step 3: Design Reduced Area Splice Plate

The moment and shear at the connection should be reduced by a factor (an assumed value of 80%) to minimize potential weld fracture at the column face. Shear is also reduced by this factor. The value for $X1$ is assumed and represents the eccentricity from the top of the flange to the centroid of the splice plate.

$$\Phi_2 := 0.8$$

$$x1 := 1 \text{ in.}$$

$$mp := \Phi_2 \cdot (M_{pu} - V_{pu} \cdot e)$$

$$mp = 2479 \text{ kip-in}$$

$$Fp := \frac{mp}{\left(\frac{d}{2} + x1\right) \cdot 2}$$

$$Fp = 137.6 \text{ kips}$$

$$Ar := \frac{Fp}{F_y}$$

$$Ar = 2.75 \text{ in.}^2$$

The required area of the reduced section is found as a percentage of the half beam section. The splice plate is fabricated from beam stock and the web height is 35% of the half height and the beam flange is 60% of the original flange width.

$$\Phi_3 := 0.35$$

$$\Phi_4 := 0.6$$

$$dr := \Phi_3 \cdot \left(\frac{d}{2} - tf \right)$$

$$dr = 2.63 \text{ in.}$$

$$br := \Phi_4 \cdot bf$$

$$br = 4.20 \text{ in.}$$

$$Ap := (dr - tf) \cdot tw + (br \cdot tf)$$

$$Ap = 2.77 \text{ in.}^2$$

A bolt shear strength value is obtained from Table 8-11, AISC Manual, Vol. 2 (page 8-24) and the required number of bolts is determined by dividing the force in the splice plate by shear strength per bolt.

$$Rn := 21.6 \text{ kips/Bolt}$$

$$Ru := \frac{Rn}{\Phi_{shear}}$$

$$Ru = 28.8 \text{ kips/Bolt}$$

$$N_{bolts} := \frac{Fp}{Ru}$$

$N_{bolts} = 4.78$ (Use Cross Section per d_r (reduced section depth), b_r (reduced section width), & A_p (reduced section area), with 6 Bolts per Side)

Step 4: Design Web Plate

The web plate is designed to resist beam shear at the connection. In order to assure the connection is significantly stronger than intended (so as to not lose gravity carrying capacity), the shear force is multiplied by a amplification factor β_1 . A recommended value was obtained from the SAC Interim Guidelines for Grade 50 steel. Shear forces act through the center of the connection without moment. Beam bending is assumed to be resisted by flange plates only.

$$\beta_1 := 1.4$$

$$V_{web} := \Phi \cdot V_{pu} \cdot \beta_1$$

$$V_{web} = 35.97 \quad \text{kips}$$

$$C_{reqd} := \frac{V_{web}}{R_n}$$

$$C_{reqd} = 1.7$$

(Consult Table 8-18, AISC Vol. 2 (page 8-40), $e = 2.5"$, $s = 3"$, Try 3 Bolts)

The number of bolts is determined using the AISC Design Manual. The same type of bolts are used in the web plate that are used in the splice plate for simplicity. The

eccentricity of the web bolts is the distance from the center of the bolt line to the end of the beam, plus half the gap length between the beams at the connection.

$$N_{boltsweb} := 3$$

$$C := 1.99$$

$$P_u := C \cdot R_n$$

$$P_u = 42.98 \text{ kips}$$

The depth of the web plate is determined based on a allowable bolt separation of 3 inches times the number of bolts in the bolt line.

$$dw := 3 \cdot N_{boltsweb}$$

$$dw = 9 \text{ in.}$$

The web plate thickness is determined based on the web shear and ultimate strength of the material. The strength provided is checked against the design web shear.

$$t_{plate} := \frac{V_{web}}{dw \cdot F_y}$$

$$t_{plate} = 0.08 \text{ in.}$$

(Use 1/4" thick 9" x 8" Web Plate, 3 Bolts per Side)

Step 5: Design Column Welds

In order to avoid brittle failure of welds at the column face, welds at the flanges and web are designed to resist a maximum loading. The moment is distributed to the flange and

web welds by a ratio of the plastic moment. The flange weld is designed as a double fillet weld with pure axial loading from the flange. The weld size should be adjusted, if necessary, to be as wide as the column web thickness.

$$Z_{fl} := b_f \cdot t_f \cdot (d - t_f)$$

$$Z_{fl} = 54.8$$

$$P_{fl} := \left(\frac{Z_{fl}}{Z_x} \right) \cdot \left[\frac{\beta \cdot 1 \cdot M_{pu}}{(d - t_f)} \right]$$

$$P_{fl} = 247.3$$

$$FEXX := 70$$

$$D_{weld} := \frac{P_{fl}}{0.7 \cdot FEXX \cdot (b_f - t_w) \cdot \frac{2}{\sqrt{2}}}$$

$$D_{weld} = 0.53 \text{ in.}$$

(Use 1/2" Double Fillet Flange Weld at Column)

The web weld is designed as a double fillet weld with the applied moment and shear force. The flange weld size, in sixteen's of an inch, is obtained from Table 8-38, AISC Vol. 2 (page 8-163).

$$M_w := (\beta \cdot 1 \cdot M_{pu}) - [P_{fl} \cdot (d - t_f)]$$

$$M_w = 1269 \text{ kip-in.}$$

$$ew := \frac{Mw}{V_{pu} \cdot \Phi^2}$$

$$ew = 49.39$$

$$Lw := 2 \cdot T$$

$$Lw = 27.25$$

$$a := \frac{ew}{Lw}$$

$$a = 1.8$$

(Consult Table 8-38, AISC Vol. 2 (page 8-163), a = 1.8, k = 0, Try C = 0.544)

$$C := 0.544$$

$$C1 := 1.0$$

$$Dw := \frac{V_{pu}}{C \cdot C1 \cdot Lw}$$

$$Dw = 2.17 \text{ in.}$$

(Use 3/16" Fillet Web Weld at Column)

Appendix C - DRAIN 2DX Input File

```

*STARTXX
  A1_F          0 0 1 1   F          3 STORY FRAME - SMRF PROJECT
! SAC model with three story frame, three bays
! Eccentricity is 41" from centerline of column
! SAC LA21 Ground Motion - Rigid Frame
!
! MAR.15, 2003
!
*NODECOORDS
C      0001      0.      0.
C      0007      360.     0.      -1
C      0013      360.     0.      -1
C      0019      360.     0.      -1
C      0101      0.      156.     0001
C      0102      41.      0.      -1
C      0103      0.      0.      -1
C      0104      139.     0.      -1
C      0105      139.     0.      -1
C      0106      0.      0.      -1
C      0107      41.      0.      -1
C      0108      41.      0.      -1
C      0109      0.      0.      -1
C      0110      139.     0.      -1
C      0111      139.     0.      -1
C      0112      0.      0.      -1
C      0113      41.      0.      -1
C      0114      41.      0.      -1
C      0115      0.      0.      -1
C      0116      139.     0.      -1
C      0117      139.     0.      -1
C      0118      0.      0.      -1
C      0119      41.      0.      -1
C      0201      0.      312.     0001
C      0202      41.      0.      -1
C      0203      0.      0.      -1
C      0204      139.     0.      -1
C      0205      139.     0.      -1
C      0206      0.      0.      -1
C      0207      41.      0.      -1
C      0208      41.      0.      -1
C      0209      0.      0.      -1
C      0210      139.     0.      -1
C      0211      139.     0.      -1
C      0212      0.      0.      -1
C      0213      41.      0.      -1
C      0214      41.      0.      -1
C      0215      0.      0.      -1
C      0216      139.     0.      -1
C      0217      139.     0.      -1
C      0218      0.      0.      -1
C      0219      41.      0.      -1
C      0301      0.      468.     0001
C      0302      41.      0.      -1
C      0303      0.      0.      -1
C      0304      139.     0.      -1
C      0305      139.     0.      -1
C      0306      0.      0.      -1

```

```

C      0307      41.      0.      -1
C      0308      41.      0.      -1
C      0309       0.      0.      -1
C      0310     139.      0.      -1
C      0311     139.      0.      -1
C      0312       0.      0.      -1
C      0313      41.      0.      -1
C      0314      41.      0.      -1
C      0315       0.      0.      -1
C      0316     139.      0.      -1
C      0317     139.      0.      -1
C      0318       0.      0.      -1
C      0319      41.      0.      -1
*RESTRAINTS
! Fixed supports at foundations
S 111      0001      0019
*SLAVING
! Make connections translationally flexible
S 111      0102      0103
S 111      0105      0106
S 111      0108      0109
S 111      0111      0112
S 111      0114      0115
S 111      0117      0118
S 111      0202      0203
S 111      0205      0206
S 111      0208      0209
S 111      0211      0212
S 111      0214      0215
S 111      0217      0218
S 111      0302      0303
S 111      0305      0306
S 111      0308      0309
S 111      0311      0312
S 111      0314      0315
S 111      0317      0318
*MASSES
S 110      8.2      0101      0119      6      12.0
S 110      8.2      0201      0219      6
S 110      8.875     0301      0319      6
*ELEMENTGROUP
      2      1      0      0.      BEAM ELEMENTS
      4      0      4
! MRF Beam Sections (Fy = 36 ksi)
! W33X118 Beam (first floor)
      1      29000.      .10      34.7      5900.      4.      4.      2.
! W30X116 Beam (second floor)
      2      29000.      .10      34.2      4930.      4.      4.      2.
! W24X68 Beam (third floor)
      3      29000.      .10      20.1      1830.      4.      4.      2.

! Pinned Frame Beam Sections (Fy = 36 ksi)
! Not used in analysis
! W21X44
      4      29000.      .10      13.0      843      4.      4.      2.
! yield strength
      1      2      14940      14940      1249      1249      1      0      1      0

```

2	2	13608	13608	1231	1231	1	0	1	0
3	2	6372	6372	724	724	1	0	1	0
4	2	3434	3434	468	468	1	0	1	0

! elements

1	0101	0102	1	1	0	1	1
2	0103	0104	1	1	0	1	1
4	0106	0107	1	1	0	1	1
6	0109	0110	1	1	0	1	1
8	0112	0113	1	1	0	1	1
10	0115	0116	1	1	0	1	1
12	0118	0119	1	1	0	1	1
13	0201	0202	1	2	0	2	2
14	0203	0204	1	2	0	2	2
16	0206	0207	1	2	0	2	2
18	0209	0210	1	2	0	2	2
20	0212	0213	1	2	0	2	2
22	0215	0216	1	2	0	2	2
24	0218	0219	1	2	0	2	2
25	0301	0302	1	3	0	3	3
26	0303	0304	1	3	0	3	3
28	0306	0307	1	3	0	3	3
30	0309	0310	1	3	0	3	3
32	0312	0313	1	3	0	3	3
34	0315	0316	1	3	0	3	3
36	0318	0319	1	3	0	3	3

*ELEMENTGROUP

2	1	1	1.	COLUMN ELEMENTS
3	0	3		

! MRF Column Sections (fy = 50 ksi)

! W14X257

1	29000.	.10	75.6	3400.	4.	4.	2.
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! W14X311

2	29000.	.10	91.4	4330.	4.	4.	2.
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! Pinned Frame Column Sections (fy = 50 ksi)

! Not used in analysis

! W14X68

3	29000.	.10	20.0	723.	4.	4.	2.
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! yield capacities

1	2	24350	24350	3780	3780	1	0	1	0
2	2	30150	30150	4570	4570	1	0	1	0
3	2	5750	5750	1000	1000	1	0	1	0

! elements

1	0001	0101	100	1	0	1	1
4	0007	0107	100	2	0	2	2
7	0013	0113	100	2	0	2	2
10	0019	0119	100	2	0	2	2
12	0219	0319	1	1	0	1	1

*ELEMENTGROUP

! Springs located at splice connection

4	1	0	0.	ROTATIONAL CONNECTIONS
3				
1	5.5e5	.0762	2745.	2745.
2	1.1e6	.0762	5515.	5515.
3	1.2e6	.0762	5882.	5882.
1	0102	0103	6	3
4	0105	0106	6	3

7	0202	0203	6	2
10	0205	0206	6	2
13	0302	0303	6	1
16	0305	0306	6	1
18	0317	0318	6	1

*SECTION
! Section to get base shear and overturning moment.

0.				SECTION THROUGH BASE
2	1	-540.	0	
	1.	0.	0.	
	0.	1.	0.	
	0.	0.	1.	
	0.	0.	0.	
	0.	0.	0.	
	0.	0.	0.	
2	4	-180.	1	
2	7	180.	1	
2	10	540.	1	

*RESULTS
! node displacement
NSD 001 0302
NSD 001 0301
NSD 001 0201
! elements
E 001 1 25
! all sections
!S 001

*NODALOAD
VERT

				UNIT VERTICAL LOAD		
S	0.	-1.	0.	0101	0301	100
S	0.	-1.	0.	0104	0304	100
S	0.	-1.	0.	0107	0307	100
S	0.	-1.	0.	0110	0310	100
S	0.	-1.	0.	0113	0313	100
S	0.	-1.	0.	0116	0316	100
S	0.	-1.	0.	0119	0319	100

*NODALOAD
HORZ

				UNIT HORIZONTAL LOAD		
S	1.	0.	0.	0101		
S	1.	0.	0.	0201		
S	1.	0.	0.	0301		

*ACCNREC
! LA21 Ground Motion considered during run

LA1	LA21(8F22.0)		1995 KOBE-FN		
3000	1	0	2	1.0	0.3937 0.020 0.0
! LA2	LA22(8F22.0)		1995 KOBE-FP		
! 3000	1	0	2	1.0	0.3937 0.020 0.0
! LA3	LA23(8F22.0)		LOMA PRIETA-FN		
! 2500	1	0	2	1.0	0.3937 0.010 0.0
! LA4	LA24(8F22.0)		LOMA PRIETA-FP		
! 2500	1	0	2	1.0	0.3937 0.010 0.0
! LA5	LA25(8F22.0)		NORTHRIDGE-FN		
! 2990	1	0	2	1.0	0.3937 0.005 0.0
! LA6	LA26(8F22.0)		NORTHRIDGE-FP		
! 2990	1	0	2	1.0	0.3937 0.005 0.0

*PARAMETERS
! print only

```

OS      0      0      1      0      0
OD      0      0      0.0      0      0.0      1      0.02      0      0.00      0
0.0
DC  1      0      0      -99
DT  0.005
DA      3.0      0.0      0.5      0.3      0.0      2      2.0
!*MODE                                MODE SHAPE AND PERIOD ANALYSIS
!      2      0.001      0
*GRAV                                APPLY FULL GRAVITY LOAD
N      VERT      32.2
!*STAT                                APPLY STATIC HORIZONTAL FORCE
!N      HORZ
!D      101      201      1      1      5
*ACCN                                Test input data
      60.0 9999      1      0.020
1      LA1      1.15
!1      LA2      1.15
!      25.0 9999      1      0.010
!1      LA3      0.82
!1      LA4      0.82
!      14.95 9999      1      0.005
!1      LA5      1.29
!1      LA6      1.29
*STOP

```