On the Seismic Performance of Skewed Special Moment Frame Reduced Beam Section Connections

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On the Seismic Performance of Skewed Special Moment Frame Reduced Beam Section Connections

A dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Engineering

by

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ABSTRACT

Special Moment Frames (SMFs) are frequently used in high seismic areas for architecturally constrained designs, as they provide lateral system stiffness without the use of braces which often obstruct views and architectural features. Reduced beam section (RBS) connections are popular SMF connection details developed following the Northridge earthquake to limit brittle fractures within connection welds. Current American Institute of Steel Construction (ASIC) provisions (i.e. AISC 341-16) provide prequalified SMF RBS connection details (including welding requirements); however, all prequalified details only consider orthogonal connections between the beam and column. This dissertation investigates the effect of adding skew within SMF RBS connections and provides insights into allowable skew levels for design, widening the application of SMF RBS connections.

The study presented herein involves parametric component-level analyses and system-level dynamic time-history analyses of skewed SMF RBS connections. The component-level parametric study involves detailed finite-element analysis of 64 SMF RBS connections and 48 SMF Welded Unreinforced Flange-Welded Web (WUF-W) connections (as a typical connection alternative to the RBS). The component-level investigation considers 3 skew angles, 4 column axial load levels and 3 beam-to-column connection geometries (shallow, medium, and deep column geometries). Connection capacity, column twist/yielding, connection response and fatigue performance are all investigated. Additional component-level composite (concrete-steel) connection simulations are conducted to investigate the effects of composite concrete slabs on the behavior of the skewed connections.

In addition to the component-level analyses, system-level time-history analyses are used to investigate skewed SMF RBS connection demands during dynamic seismic loading. To
investigate system-level effects on skewed connection behavior, a six-story building containing
various levels of skew at the SMF connection is designed, simulated using detailed finite element
procedures, and loaded using a scaled earthquake ground motion to represent both design basis
and maximum considered earthquake demands.

In addition to the detailed finite element investigations, an experimental testing program
is designed and initiated to allow prequalification of skewed SMF RBS connections within the
AISC provisions. Specimen fabrication, experimental setup (including instrumentation, loading,
and boundary conditions), and initial results for the prequalification testing are described herein.

Results from the component-level parametric research work indicate that SMF RBS
connection capacity decreases when increasing the skew angle; however, all performance levels
achieved would satisfy current AISC requirements for prequalification. Additionally, as skew
angle is increased within the SMF RBS connection, the resulting column twist increases. Column
flange-tip yielding is also observed at beam bottom-flange levels of the skewed geometries, and
this yielding does increase for skewed connections having medium and deep column geometries
when increasing the skew angle; however, the yielding is rather localized on the column flange.
Local damage (indication of low-cycle fatigue fracture susceptibility) within the RBS section
decreases when increasing the column axial load but does increase when increasing the skew
angle. When a concrete slab is included, the connection’s positive moment capacity increases
due to composite action, but the result is increased column twist for medium and deep column
geometries at rather large skew (30° skew relative to the column). A column twist prediction
formula is developed and proposed.
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LIST OF TERMS

ATC  Applied Technology Council

β    Weighting term for plastic strain under compressive triaxiality

C    Initial kinematic hardening

C_d  Deflection Amplification Factor

C_s  Seismic Response Coefficient

CUREe California Universities for Research in Earthquake Engineering

E    Modulus of Elasticity of Steel

F_y  Yield Strength of Steel

F_u  Ultimate strength of Steel

PEEQ Equivalent Plastic Strain

PGA Peak Ground Acceleration

R    Response Modification Factor

R_up Closest distance to the rupture

RBS  Reduced Beam Section

SEAOC Structural Engineers Association of California

S_DS Short-period design response spectral acceleration

S_D1 One-second design response spectral acceleration

SF   Scale Factor

SMF  Special Moment Frame

S-Mises Mises stresses

SW   Seismic weight

SP   Principal Stresses
<table>
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<tr>
<td>ULCF</td>
<td>Ultra-Low Cycle Fatigue</td>
</tr>
<tr>
<td>V</td>
<td>Base shear</td>
</tr>
<tr>
<td>$V_p$</td>
<td>Plastic Shear at reduced beam section location</td>
</tr>
<tr>
<td>$Z_x$</td>
<td>Section Modulus</td>
</tr>
<tr>
<td>$Z_{RBS}$</td>
<td>Section Modulus of Reduced Beam Section</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Resistance Factor</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Curve rate of departure from C</td>
</tr>
<tr>
<td>$\alpha_1$</td>
<td>Backstress</td>
</tr>
<tr>
<td>$\varepsilon^{pl}$</td>
<td>Material coefficient from cyclic coupon testing</td>
</tr>
<tr>
<td>$\lambda_{DSPS}$</td>
<td>Damageability parameter</td>
</tr>
<tr>
<td>$\alpha$</td>
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<tr>
<td>$\varepsilon_{p,t}$</td>
<td>Integration of plastic strain under tensile triaxiality</td>
</tr>
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<td>$\varepsilon_{p,c}$</td>
<td>Integration of plastic strain under compressive triaxiality</td>
</tr>
<tr>
<td>$\varepsilon_p$</td>
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CHAPTER 1. INTRODUCTION

1.1 Overview

Steel special moment-resisting frames (SMFs) are common in high seismic areas for architecturally constrained designs because they can offer open areas (areas without obstructions from braces) and accommodate many architectural features. For non-rectangular building envelope designs, skew between the beam and column within a beam-to-column connection may occur; however, how much skew should be allowed in design is not clear, as present SMF connection design procedures do not consider skew. Skew in steel beam-to-column connections can be considered as either in-plane or out-of-plane as is shown in Figure 1(a) and (b), respectively. A distinction between the two skew categories is made in this proposal considering in-plane skew as “sloped” connections, while connections with out-of-plane skew are named “skewed” connections. Figure 1 shows a reduced beam section (RBS) in combination with the sloped and skewed connection geometries. Although acceptable performance of RBS steel connections without skew has been corroborated in several experimental investigations [1, 2, 3, 4], it is not clear if the existing RBS design methods are adequate when skew is present.

Figure 1. (a) Sloped and (b) skewed SMRF connection configurations[5].
For engineers dealing with skewed architectural constraints, guidance for designing sloped and skewed SMF connection geometries would be useful in design. It is implicit in AISC 358-16 that current prequalified moment frame connection criteria provide an orthogonal connection between the beam and column [6], but in common field situations, this orthogonal condition must be broken to accommodate architectural requirements. Unfortunately, AISC 358-16 does not provide procedures to determine the limits or design contemplations to deal with those connections.

This research project will include both full-scale experimental component testing of skewed SMFs and comprehensive parametric finite element analyses at both the system and component levels (including detailed low-cycle fatigue submodel investigations). Dynamic time-history analyses at the system level will provide an understanding of skewed SMF response during earthquake loading, while static parametric component analyses will identify key design features affecting performance. It is anticipated that design recommendations and prequalification of skewed SMF RBS connections will result from this research.

The following sections describe relevant background related to the proposed work, as well as the detailed tasks proposed to achieve the described objectives.

1.2 Background

The capacity of a skewed connection can be affected by column size. Deep columns are frequently used in SMF connections; according to Chi and Uang [3] and Zang and Ricles [7] they can display larger column twisting than shallow columns due to the increased eccentricity from lateral movement of the RBS compression flange.
Some experimental and computational work on in-plane skew has been done by Ozkula et al. [8] and Mashayekh and Uang [9], in which in-plane skew up to 25 degrees has been explored, but no experimental work has been done regarding to out-of-plane skew.

Sloped moment connections have been investigated more deeply than skewed connections, with interesting behavior related to brittle weld fractures having been observed. A recent study by Kim et al. [10] investigating the performance of full-scale sloped connections indicates the possibility of accommodating large slopes and skews in SMF connections using existing design procedures; however, the exploratory natures of these studies carry limitations that need to be addressed in further research prior to implementation in design practice. The investigation by Kim et al. [10] showed that after reaching the minimum rotation for connection qualification (4% drift), consistently brittle fractures of the top flange weld and significant yielding at the top flange (close to the column face) led to considerable strain increments on the top flange, but the strain demand was smaller at the bottom flange. This level of potential brittle fracture modes at the beam-to-column welds highlighted new weld quality control needs between the beam web and column flange connection [10]. These sloped SMF connection fractures observed in the experimental testing by Kim et al. could not be predicted using simulations alone. In this study, two RBS configurations for slope in SMF were presented, as shown in Figure 2, but not all were validated experimentally. Therefore, a deeper investigation needs to be addressed in further research for this type of skewed connection.
A recent publication by Prinz and Richards [5] investigated the performance of skewed RBS moment connections (see Figure 1(b)). Detailed finite element models were used to study skew effects in this skewed moment connection research [5]. Simulations of two types of models were presented: one type simulating classic laboratory beam-to-column connection testing (half-story column), and the other type representing building boundary conditions closer to reality (3-story columns). The results of the study mentioned herein this paragraph demonstrated an intricate relationship between out-of-plane skew, column twisting/yielding, and strain demands in the RBS section. Increments of column twisting were obtained due to out-of-plane skew, which resulted in some yielding in the column flange ends. While the model results presented by Prinz and Richards [5] included some axial loads (on columns) introduced through beam shear, it is still not clear whether substantial column twisting would arise in medium and high-rise structures where columns are exposed to large axial loads. Additionally, this research did not consider local weld geometry/material effects due to fracture propensity. Abaqus computer models did not capture those effects, and it is not clear if unwanted beam-to-column connection failure mechanisms would happen during seismic events.
A publication by Desrochers et al. [11] about out-of-plane SMF connections concluded that connection moment capacity is not affected significantly by beam skew. The column twist can be increased by beam skew angle and column depths, but axial loads less than 25% of $\phi_c P_n$ (25% column axial capacity) have minor effects on column twist. Similar to Prinz and Richards [5], column flange tip yielding is increased by the out-of-plane skew.

1.3 Research Needs and Objectives

Currently, there is a need for seismic systems and connection designs that enhance the architectural flexibility of steel structures.

The Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications (AISC 358-16) [6] do not currently have guidance for considering out-of-plane skew effects on SMF connections. Although orthogonal RBS connections have been widely tested in the experimental and computational field, there is limited understanding of how out-of-plane skew will affect important design parameters such as twisting, yielding and connection capacity. Out-of-plane skew can increase column twisting and the column flange tip yielding due to the nature of connection configurations. In addition, the connection capacity can be reduced because part of the beam moment at the column face should be resisted by the column weak axis.

The purpose of this study focuses on building up extensive seismic design procedures for skewed RBS moment connections by

1) broadening the exploratory research of Prinz and Richards [5],
2) establishing skew limits and helpful detailing strategies for skewed SMF connections,
3) understanding behavior of composite skewed SMF connections,
4) providing confidence in expected system-level dynamic moment frame response,
5) developing an experimental program for skewed SMF connection prequalification.

1.4 Research Approach

The dissertation is divided into four parts, as follows: 1) a system-level dynamic investigation into skewed SMF response, 2) a component-level parametric investigation into skew effects, 3) a component-level analytical study into composite frame effects (with steel-to-concrete composite action considered), and 4) an experimental investigation into skewed connection demands through prequalification-type testing. The next sections give more details of the mentioned divisions.

1.4.1 System-level Investigation: Design and Modeling of 6-Story Prototype Buildings

Five 6-story building plans on the U.S. West Coast were designed following the equivalent lateral force method presented in ASCE 7-16 [12]. Los Angeles was selected for this study because it is one the most seismically active areas in the U.S. The sizes of the SMF members were selected based on drift requirements. Reduced beam sections, which are needed for ductility requirements, were used in the design of the SMF beams. To address insufficient strength at the column face due to the applied moment from the beam and shear requirements at the panel zone (PN), continuity plates and doubler plates were designed following AISC Seismic Provisions and Seismic Design Manual [13, 14]. For calculations of all elements mentioned above, see Appendix A.

Finite element models of each previously designed building were developed to study the behavior of the SMF structure under complex seismic loading. Shell elements from ABAQUS CAE [15] were used to model the connection regions.
1.4.2 Component-Level Parametric Study into Skewed Connection Demands

AB AQUS simulation of skewed connections (having varied skew angles of 10°, 20°, and 30°) were developed to study the effects of skew angles in SMF with RBS connections. Among the choices for modeling skewed RBS connections, the traditional specimen size to study the beam-to-column connection, is a 13 ft-column connected at mid-height with a 15 ft-beam. This type of model is normally used to simulate the conditions of a full-scale experiment as well. In this model, the bottom and top of columns are pinned, the beam is laterally braced and only vertical displacements are allowed at the beam tip. More details about degrees of freedom (DOF) are provided in Chapter 3.

Another option, which is less common, is to model several stories of a building with beams at each floor to simulate field conditions. For this research project, 3-story ABAQUS frame models are developed because they better replicate the torsional boundary conditions in real steel moment frames. The second option is uncommon (for experimental investigations) because it is difficult to find large enough facilities to accommodate the testing. Previous works presented by Prinz and Richards [5] and Desrochers [16] have used those two types of finite element models to study the beam-to-column SMF RBS connections without experimental validation, but they can be referred for comparison.

Three-story frame models of Welded Unreinforced Flange-Welded Web (WUF-W) moment connections are also considered in this chapter for a direct comparison with RBS moment connections. Because out-of-plane WUF-W moment connections have not been tested analytical or experimentally, it would be interesting to have comparison with RBS skewed connections.
1.4.2 Component-Level Investigation into Skewed Composite Connection Demands

The effects of a concrete slab on the 3-story frames with RBS SMF connections from chapter three were evaluated through finite element models. Slab parameters from Jones et al. [4] are considered for modeling. The same frame geometry and mesh refinements from chapter 3 were used here. Studs and rebar are modeled using one-dimensional element and the slab was created with solid elements.

1.4.3 Experimental Investigation of Skewed Connection Demands

The experimental part of this research project was done at the Steel Structures Research Laboratory (SSRL) at the University of Arkansas where various beam-to-column assemblies of SMRF connections having skewed arrangements will be tested. In this research stage, the main activity includes specimen cycling loading [13] to investigate demands in skewed RBS moment connections and local flange yielding at the beam and column.

The purpose of this research section was to study undesirable failure modes within the skewed beam-to-column weld regions. The experimental investigation is also verified by finite element models developed especially for this chapter. The work presented by Prinz and Richards [5] and Desrochers [16] can be compared with these experiment results. In addition, these experiments help to address limitations in existing AISC literature providing guidance for designing skewed SMF connections.

1.5 Organization of the Dissertation

This dissertation covers the four main parts described above. The outline of the dissertation chapters is presented as follows:

Chapter 2 focuses on investigating the dynamic system-level performance in skewed RBS SMF connections. ABAQUS finite element model of a building under seismic loading in a
high seismic area is used to study realistic skewed connection strain demands, connection capacity, strain demands within RBS and beam-to-column welds, and column twisting.

Chapter 3 investigates skewed connection demands through a parametric study for two types of SMF connections such as RBS and WUF-W moment connections. Three column sizes (shallow, medium and deep) are included for every connection type. Analysis of connection moment capacity, column twisting, column flange yielding, connection global and local response, and connection fatigue life are presented.

Chapter 4 looks into the effects of composite action provided by a concrete slab on the skewed RBS connections. Three column sizes (shallow, medium and deep) are included in the study, and two concrete strengths are considered as well. Similar to chapter 3, an analysis of connection moment capacity, column twisting, column flange yielding, connection global and local response, and connection fatigue life are presented. A modified AISC formula for twisting prediction of skewed RBS moment composite connections is presented.

Chapter 5 focusses on the experimental testing of skewed RBS moment connections. Cyclic loading is applied at the beam tip to evaluate connection capacity, column flange yielding, connection global and local response and demands within the RBS cut.

Chapter 6 summarizes the research findings, presents conclusion related to skewed SMF connection and discusses areas for future research and further study.
CHAPTER 2. SYSTEM-LEVEL DYNAMIC TIME-HISTORY INVESTIGATION INTO SKEWED SMF CONNECTION BEHAVIOR

2.1 Background Information

This chapter presents the design of five prototype buildings located in Los Angeles, CA, (high seismic area) and numerical modeling of one selected prototype design. Different building plan shapes accommodate a diverse set of skew connections and provide a variety of structural element sizes, offering an assortment of research results.

The parameters considered for the seismic design (seismic acceleration parameters and ground motion selection), include loads, structure geometry, steel properties, and design approach. A table is presented listing ground motions considered for modeling, which included name of the recording station, peak ground acceleration (PGA), distance to the rupture ($R_{rup}$) and scale factor. Then ABAQUS finite-element analysis modeling considerations are presented to study the response of the system under complex seismic loading and evaluate the performance (dynamic analysis) of a building subjected to diverse levels of seismic demands.

2.2 Seismic Acceleration Parameters

Two period response parameters define the response spectrum for a particular site, one for short periods (0.2 s) and one for long periods (1.0 s). The spectral acceleration parameters for the location in this study were calculated using the method in the ASCE 7-16 code. Those factors known as the short period response acceleration parameter ($S_{DS}$) and the 1-s period response acceleration parameter ($S_{DL}$) are presented in Table 1. To calculate these parameters, a risk category II of buildings was considered.

The site class for all the places was defined as class C. The default site class classification for this type of studies is D, but this site class (D) led to structural element sizes (beams and columns) considerably larger than those intended to be used for the parametric study.
In order to keep similar structural sections in the parametric study and the dynamic analysis, a site class C was selected.

Calculation equations of the seismic acceleration parameters are presented below:

\[ S_{DS} = \frac{2}{3} S_{MS} \]  
Equation (1)

\[ S_{D1} = \frac{2}{3} S_{M1} \]  
Equation (2)

where

\[ S_{MS} = Fa \cdot S_s \]  
Equation (3)

\[ S_{M1} = Fv \cdot S_l \]  
Equation (4)

\[ S_s \text{ and } S_l \text{ are, respectively, the lesser of} \]

\[ S_s = \begin{cases} C_{RS} \cdot S_{SUH} \\ S_{SD} \end{cases} \]  
Equation (5)

\[ S_l = \begin{cases} C_{RI} \cdot S_{1UH} \\ S_{1D} \end{cases} \]  
Equation (6)

The variables listed above are defined as:

\[ S_{DS} \] = design, 5\% damped, spectral response acceleration parameter at short periods (0.2 s)

\[ S_{D1} \] = design, 5\% damped, spectral response acceleration parameter at a period of 1 second

\[ S_{MS} \] = the MCE, 5\% damped, spectral response acceleration parameter at short periods (0.2 s)

\[ S_{M1} \] = the MCE, 5\% damped, spectral response acceleration parameter at a period of 1 second

\[ S_s \] = mapped MCE, 5\% damped, spectral response acceleration parameter at short periods (0.2 s)

\[ S_l \] = mapped MCE, 5\% damped, spectral response acceleration parameter at a period of 1 second

\[ Fa \] = short period site coefficient at 0.2 s-period (Table 11.4-1, ASCE 7-16)

\[ Fv \] = long period site coefficient at 1.0 s-period (Table 11.4-2, ASCE 7-16)

\[ C_{RS} \] = mapped value of the risk coefficient at short periods (0.2 s) [17]
\( C_{RI} \) = mapped value of the risk coefficient at a period of 1 second [17]

\( S_{SUH} \) = mapped uniform-hazard, 5% damped, spectral response acceleration parameter at short periods (0.2 s) [17]

\( S_{1UH} \) = mapped uniform-hazard, 5% damped, spectral response acceleration parameter at a period of 1 second [17]

\( S_{SD} \) = mapped deterministic, 5% damped, spectral response acceleration parameter at short periods (0.2 s) [17]

\( S_{ID} \) = mapped deterministic, 5% damped, spectral response acceleration parameter at a period of 1 second [17]

Equation 9 is a formulation from ASCE 7-16 [12] used in this research to calculate the building period (T).

\[
T = C_u \times C_t \times h_n^x
\]  
Equation (9)

From the previous equation, the variables are defined as:

\( C_u \) = Coefficient for upper limit on calculated period; (Table 12.8-1, ASCE 7), \( C_u = 1.4 \) for \( S_{D1} > 0.4 \) and \( C_u = 1.7 \) for \( S_{D1} \leq 0.1 \)

\( C_t \) = (Table 12.8-2, ASCE 7), for steel moment-resisting frames: \( C_t = 0.028 \)

\( x \) = (Table 12.8-2, ASCE 7), for steel moment-resisting frames: \( x = 0.8 \)

\( h_n \) = structure height: 78 feet

A calculation example of the building period and seismic acceleration parameters is presented in Appendix A and Appendix C, respectively.

<table>
<thead>
<tr>
<th>Place</th>
<th>Spectral Acceleration Parameters</th>
<th>Building Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( S_{DS} )</td>
<td>( S_{DI} )</td>
</tr>
<tr>
<td>Los Angeles, CA</td>
<td>1.583g</td>
<td>0.658g</td>
</tr>
</tbody>
</table>
Table 1 shows acceleration parameters for Los Angeles. There is a direct relationship between acceleration’s parameters and the expected level of damage after a seismic event. It can be stated that the higher the design parameters, the more damage is expected in the structure. Therefore, the seismic demands are higher for a building in Los Angeles compared to those for other buildings in the east coast, for example. A response spectrum, which is built up from seismic accelerations parameters, is a plot of the peak response at a specific period. The USGS defines earthquake spectrum as a curve showing amplitude and phase as a function of frequency or period [18]. The design response spectra for the location under consideration is presented in Figure 3, which a fundamental parameter when scaling earthquakes for finite element modeling.

The severity of seismic demands stated in the previous sentences leads to structural section sizes considerably larger in Los Angeles.

![Figure 3. Design Response spectra for Los Angeles.](image-url)
2.3 Fault Types

This section was included because the definitions presented here are important for understanding the following chapter part (ground motion selection).

The majority of earthquakes worldwide are caused by a quick slip (move) on a fault. There are three types of faults: strike-slip, normal and reverse. For clarification, these types of faults are presented in Figure 4, accompanied for a brief definition [18].

![Figure 4. Types of faults a) strike-Slip b) normal c) reverse (modified from [18]).]

The USGS defines the faults presented above as follows:

- Strike-slip faults are vertical (or nearly vertical) fractures where the blocks have mostly moved horizontally.
- Normal faults are inclined fractures where the blocks have mostly shifted vertically and the rock mass above an inclined fault moves down.
- Reverse faults are similar to normal faults but the rock above the fault moves up.

2.4 Ground Motion Selection

2.4.1 Introduction

In seismic design, the selection and scaling of ground motions are fundamental for having an appropriate load definition in the model. Selecting inappropriate ground motions for modeling can produce some bias leading to a different controlling earthquake scenario. Probabilistic seismic hazard analysis (PSHA) is widely used in structural dynamic analysis and geotechnical engineering for ground motion selection. To compute the seismic hazard for a specific site,
PSHA couples earthquake magnitude and distance with probabilities of several earthquake scenarios, taking uncertainties in ground motion predictions into account by using several ground motions prediction models [19]. The ground motions prediction models currently used are (Campbell & Bozorgnia (2014), Abrahamson, Silva & Kamai (2014), Boore, Stewart, Seyhan & Atkinson (2014), Chiou & Youngs (2014), Idriss (2014)). The PSHA method used in this research for ground motion selection is called deaggregation.

2.4.2 Deaggregation

Deaggregation uses magnitude and distance to select contributing events for a given spectral acceleration ($S_a$).

The United States Geological Service (USGS) has compiled all the methods (prediction models) described previously in a web tool called the Unified Hazard Tool (UHT) to provide deaggregation outputs for places in the U.S [20]. This web page was used to perform deaggregation analysis at the site considered in this investigation (Los Angeles). This deaggregation analysis provides key information for an adequate earthquake motion selection such as fault name, earthquake magnitude and distance to the fault as shown in Table 2-4. The faults that give the major contribution to hazard for the site under consideration are considered as major source of hazard and therefore used for ground motion selection. The spectral period for deaggregation is taken between 0.2 sec and 2.0 sec because of the building height (78 feet) considered for this project. This consideration is based on a formulation from ASCE 7-16 (see Equation 10), which states that the approximate building period for this research project is around 0.6 s, knowing that the building is 6-story height. Therefore, there is no need to use spectral periods far above from 0.6 s (such as 5.0 s).

$$T_a = 0.1 \times N$$

Equation (10)
where N is the number of stories above the base

Figure 5 shows deaggregation analysis results for downtown Los Angeles for different spectral accelerations. This figure provides a visual idea about what is happening at the site regarding hazard, but the exact values from deaggregation are presented in Table 2. Here it is clear that the expected magnitude for an earthquake is between 6.5-7.5 and the closest distance to the rupture ($R_{rup}$) is around 6 km.

It is important to clarify the contribution percentage in Table 2 through Table 4. The total contribution (TC) to hazard of the system of faults is 37.56% for 0.2 $Sa$ with 12.39% coming from the Elysian Park (Upper) fault, 5% coming from the Puente Hills (LA) fault, and 3.29% coming from the Newport-Inglewood alt 1 fault as listed in Table 2. Other minor contributions are also listed in Table 2, but they will not be presented in the other tables because they are not major contributions to hazard.

![Figure 5. Deaggregation analysis for downtown Los Angeles a) 0.2 sec. Sa b)1.0 sec. Sa c)2.0 sec. Sa.](image)
Figure 5. Deaggregation analysis for downtown Los Angeles a) 0.2 sec. Sa b) 1.0 sec. Sa c) 2.0 sec. Sa. (Cont.).

Table 2. Deaggregation analysis for downtown Los Angeles at 0.2 Sa (TC: 37.56%)

<table>
<thead>
<tr>
<th>Fault name</th>
<th>$R_{rup}$ (Km)</th>
<th>Magnitude</th>
<th>Contribution (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elysian Park (Upper) [1]</td>
<td>5.94</td>
<td>6.60</td>
<td>12.39</td>
</tr>
<tr>
<td>Puente Hills (LA) [4]</td>
<td>5.82</td>
<td>7.13</td>
<td>5.00</td>
</tr>
<tr>
<td>Newport-Inglewood alt 1 [8]</td>
<td>11.93</td>
<td>6.67</td>
<td>3.29</td>
</tr>
<tr>
<td>Hollywood [0]</td>
<td>9.39</td>
<td>7.34</td>
<td>1.58</td>
</tr>
<tr>
<td>Newport-Inglewood alt 1 [6]</td>
<td>13.41</td>
<td>7.57</td>
<td>1.51</td>
</tr>
<tr>
<td>Sierra Madre [5]</td>
<td>20.79</td>
<td>7.66</td>
<td>1.17</td>
</tr>
</tbody>
</table>
Table 3. Deaggregation analysis for downtown Los Angeles at 1.0 $S_g$ (TC: 47.18%)

<table>
<thead>
<tr>
<th>Fault name</th>
<th>$R_{rup}$ (Km)</th>
<th>Magnitude</th>
<th>Contribution (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elysian Park (Upper)</td>
<td>5.94</td>
<td>7.09</td>
<td>11.64</td>
</tr>
<tr>
<td>Puente Hills (LA)</td>
<td>4.31</td>
<td>7.18</td>
<td>9.36</td>
</tr>
<tr>
<td>Compton</td>
<td>14.21</td>
<td>7.36</td>
<td>4.30</td>
</tr>
</tbody>
</table>

Table 4. Deaggregation analysis for downtown Los Angeles at 2.0 $S_g$ (TC: 49.58%)

<table>
<thead>
<tr>
<th>Fault name</th>
<th>$R_{rup}$ (Km)</th>
<th>Magnitude</th>
<th>Contribution (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elysian Park (Upper)</td>
<td>5.94</td>
<td>7.16</td>
<td>11.90</td>
</tr>
<tr>
<td>Puente Hills (LA)</td>
<td>4.34</td>
<td>7.20</td>
<td>9.32</td>
</tr>
<tr>
<td>Compton</td>
<td>14.21</td>
<td>7.39</td>
<td>4.12</td>
</tr>
</tbody>
</table>

2.4.3 Pacific Earthquake Engineering Research Center (PEER) Next Generation Attenuation (NGA) database

Appropriate information about the expected ground motions at a specific place is the key in current approaches for evaluating seismic performance of structures.

In 1996, the Pacific Earthquake Engineering Research Center (PEER), was established as a group formed by nine universities of the West Coast, and one year later obtained a status of National Science Foundation. PEER works as education and multi-institutional research center located at the University of California, Berkeley. Since its creation, PEER has collected, processed important ground motions around the world. Related information to those ground motions such as earthquake magnitude, distance to rupture, type of fault and recording stations was also gathered. All this information has been stored in a web-based, searchable database of ground motion records called the Pacific Earthquake Engineering Research Center (PEER) Next Generation Attenuation (NGA)-West2 database. This database works as a research tool for ground motion selection in the U.S. western states [21].

Recently and due to different geological conditions, PEER has created the Pacific Earthquake Engineering Research Center (PEER) NGA-East database for ground motion selection in Central and Easter parts of the U.S. PEER Next Generation Attenuation (NGA)-West2 database was used after deaggregation analysis for ground motion selection in this study.
Having deaggregation results, to search for appropriate ground motions in PEER web tool, earthquake magnitude and distance to rupture are fundamental input parameters to consider. In PEER database is important to specify type of fault, which is also provided by UHT. From deaggregation analysis at Los Angeles, the larger contribution to hazard comes from Elysian Park (Upper) and Puente Hills (LA) faults at all $S_a$. These faults are classified as Reverse (Thrust) faults. With these three important input parameters (earthquake magnitude, distance to rupture and type of fault), it is possible to search for appropriate ground motion in the PEER Next Generation Attenuation (NGA)-West2 database [22].

For our location in the Western U.S. (Los Angeles), ten ground motions will be considered for modeling (dynamic analysis), and they are representative of a 2% of being exceeded in the next 50 years. It is important to note that final ground motion selection includes different records, in an effort to avoid some bias if we use different stations for the same earthquake. The ground motions selected through deaggregation for Los Angeles are listed in Table 5; all of them come from PEER NGA-West database and Northridge-01 was considered.

<table>
<thead>
<tr>
<th>Event Name</th>
<th>Year</th>
<th>Mw$^a$</th>
<th>Station Name</th>
<th>PGA(g)$^b$</th>
<th>$R_{rup}$(km)$^c$</th>
<th>SF$^d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Simeon, CA</td>
<td>2003</td>
<td>6.52</td>
<td>Templeton - 1-story Hospital</td>
<td>0.482</td>
<td>6.22</td>
<td>1.17</td>
</tr>
<tr>
<td>Nigata, Japan</td>
<td>2004</td>
<td>6.63</td>
<td>NIG017</td>
<td>0.476</td>
<td>12.81</td>
<td>1.25</td>
</tr>
<tr>
<td>Northridge-01</td>
<td>1994</td>
<td>6.69</td>
<td>LA Dam</td>
<td>0.426</td>
<td>5.92</td>
<td>1.25</td>
</tr>
<tr>
<td>Gazli, USSR</td>
<td>1976</td>
<td>6.80</td>
<td>Karakyr</td>
<td>1.698</td>
<td>5.46</td>
<td>0.99</td>
</tr>
<tr>
<td>Iwate, Japan</td>
<td>2008</td>
<td>6.90</td>
<td>IWTH26</td>
<td>1.069</td>
<td>6.02</td>
<td>0.44</td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>1989</td>
<td>6.93</td>
<td>Los Gatos - Lexington Dam</td>
<td>0.443</td>
<td>5.02</td>
<td>1.30</td>
</tr>
<tr>
<td>Loma Prieta</td>
<td>1989</td>
<td>6.93</td>
<td>Gilroy Array #2</td>
<td>0.370</td>
<td>11.07</td>
<td>1.41</td>
</tr>
<tr>
<td>Cape Mendocino</td>
<td>1992</td>
<td>7.01</td>
<td>Cape Mendocino</td>
<td>1.494</td>
<td>6.96</td>
<td>0.46</td>
</tr>
<tr>
<td>Montenegro, Yugoslavia</td>
<td>1979</td>
<td>7.10</td>
<td>Ulcinj - Hotel Olimpic</td>
<td>0.461</td>
<td>5.76</td>
<td>1.48</td>
</tr>
<tr>
<td>Tabas, Iran</td>
<td>1978</td>
<td>7.35</td>
<td>Dayhook</td>
<td>0.409</td>
<td>13.94</td>
<td>1.66</td>
</tr>
</tbody>
</table>

$^a$ Earthquake Magnitude  
$^b$ Peak Ground Acceleration  
$^c$ Distance to fault rupture  
$^d$ Scale Factor
2.5 Design of 6-story Prototype Buildings

Five 6-story buildings were designed considering SMF in North-South direction. These five types of building floor plans (Plan A, B, C, D and General) for the location previously mentioned are considered in this research. Floor plans A, B, C, D have different skewed connection configurations to investigate the performance of those connection geometries in a building frame subjected to seismic loading. A floor plan (called General plan) with straight connection (orthogonal) was also designed for comparison purposes. All the plans considered for this research project are presented in Appendix B.

A SAC (SEAOC, ATC, CUREe) study [13] was considered as a reference for structural dimensioning (beam and column lengths) as well as floor loads. SAC is the join of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC) and the Consortium of Universities for Research in Earthquake Engineering (CUREe). The seismic weight varies depending upon the slab shape (perimeter). The strong axis of the frames is oriented in the N-S direction as in [23], except top and bottom beam raw, which strong axis is oriented in the E-W direction.

In addition to the seismic acceleration parameters previously obtained, to calculate the lateral loads some other considerations need to be stated here: seismic importance factor $I_e = 1.0$ (Risk category II), seismic design category D ($S_{DS} > 0.5$ and $S_{D1} > 0.2$). Seismic base shear was calculated using the equivalent lateral force procedure from ASCE 7-16 (Section 12.8) [12].

Designed members for the special moment frames are presented in Table 6 through Table 8, which as stated earlier, were calculated under drift requirements. All the sections considered for beams and columns are compact [13, 14].
Building shape differences create some element size changes for this research project even though all designs consider the same seismic place.

As common practice after the 1994-Northridge earthquake for SMRF, portions of the beam flanges are removed in the region close to the beam-to-column joint. These types of connections, which are called Reduced Beam Section (RBS) connections, were designed following the procedure established in AISC 358-16 [6] and are presented in Table 6 through Table 8.

Columns are the most important members of a structure because they support the weight of the structure above of them, while beams just carry the loads on a specific floor. Column collapse is more critical than the failure of a beam, resulting in a possible total breakdown of the entire building. Aware of this fundamental concept, the strong column/weak beam principle was considered when sizing beams and columns for the study [13].

The stress and strain at the connection zone are very high due to the transfer of moments at the beam-to-column joint. Two types of high stresses are generated in the column portion of the connection: normal stresses are created at the column flanges and shear stresses generated in the panel zone [23]. Significant consequences can emerge due to low strength at the panel zone (PN). A weak PN can result in reduction of strength and stiffness of the structure and substantial increment is story drift. Due to those high stresses mentioned before, yielding can occur in this part of the column, and consequently, plastification may arise. Therefore, doubler plates and continuity plates are also included in the building design following the procedure established in AISC seismic provisions [13]. Doubler plate and continuity plate dimensions are presented in Table 9 through Table 11.
The beams and columns in this project were A992 steel, which is the most common for rolled wide-flange sections in the U.S. Moreover, steel A992 is similar to A572 Gr 50 steel (E=29000 ksi, F_y =50 ksi) from which several necessary properties for modeling in Abaqus are known.

Table 6. Member size for Los Angeles building (Plans A and D)

<table>
<thead>
<tr>
<th>Level</th>
<th>Structural Element</th>
<th>RBS Dimensions (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Beam size</td>
<td>Column size</td>
</tr>
<tr>
<td>1-2</td>
<td>W24 × 306</td>
<td>W 33 × 387</td>
</tr>
<tr>
<td>3-4</td>
<td>W24 × 279</td>
<td>W 33 × 354</td>
</tr>
<tr>
<td>5-6</td>
<td>W21 × 201</td>
<td>W 33 × 241</td>
</tr>
</tbody>
</table>

Table 7. Member size for Los Angeles building (Plan B)

<table>
<thead>
<tr>
<th>Level</th>
<th>Structural Element</th>
<th>RBS Dimensions (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Beam size</td>
<td>Column size</td>
</tr>
<tr>
<td>1-2</td>
<td>W24 × 335</td>
<td>W 33 × 387</td>
</tr>
<tr>
<td>3-4</td>
<td>W24 × 279</td>
<td>W 33 × 354</td>
</tr>
<tr>
<td>5-6</td>
<td>W21 × 201</td>
<td>W 33 × 241</td>
</tr>
</tbody>
</table>

Table 8. Member size for Los Angeles building (Plan C)

<table>
<thead>
<tr>
<th>Level</th>
<th>Structural Element</th>
<th>RBS Dimensions (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Beam size</td>
<td>Column size</td>
</tr>
<tr>
<td>1-2</td>
<td>W24 × 335</td>
<td>W 33 × 387</td>
</tr>
<tr>
<td>3-4</td>
<td>W24 × 279</td>
<td>W 33 × 354</td>
</tr>
<tr>
<td>5-6</td>
<td>W24 × 176</td>
<td>W 33 × 241</td>
</tr>
</tbody>
</table>

Table 9. Doubler Plate and Continuity Plate size for Los Angeles building (Plans A and D)

<table>
<thead>
<tr>
<th>Level</th>
<th>Doubler Plate Thickness (in.)</th>
<th>Continuity Plate Thickness (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Interior Column</td>
<td>Exterior Column</td>
</tr>
<tr>
<td>1-2</td>
<td>1.500</td>
<td>None</td>
</tr>
<tr>
<td>3-4</td>
<td>1.375</td>
<td>None</td>
</tr>
<tr>
<td>5-6</td>
<td>1.125</td>
<td>0.625</td>
</tr>
</tbody>
</table>

Table 10. Doubler Plate and Continuity Plate size for Los Angeles building (Plan B)

<table>
<thead>
<tr>
<th>Level</th>
<th>Doubler Plate Thickness (in.)</th>
<th>Continuity Plate Thickness (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Interior Column</td>
<td>Exterior Column</td>
</tr>
<tr>
<td>1-2</td>
<td>1.625</td>
<td>0.625</td>
</tr>
<tr>
<td>3-4</td>
<td>1.375</td>
<td>None</td>
</tr>
<tr>
<td>5-6</td>
<td>1.125</td>
<td>0.625</td>
</tr>
</tbody>
</table>
Table 11. Doubler Plate and Continuity Plate size for Los Angeles building (Plan C)

<table>
<thead>
<tr>
<th>Level</th>
<th>Doubler Plate Thickness (in.)</th>
<th>Continuity Plate Thickness (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Interior Column</td>
<td>Exterior Column</td>
</tr>
<tr>
<td>1-2</td>
<td>1.625</td>
<td>0.625</td>
</tr>
<tr>
<td>3-4</td>
<td>1.375</td>
<td>None</td>
</tr>
<tr>
<td>5-6</td>
<td>1.000</td>
<td>None</td>
</tr>
</tbody>
</table>

For comparison purposes, a traditional building without skew (just orthogonal connections) was considered in this study. The floor plan for this building is also presented in Appendix B. Results from this design are shown in Table 12 and Table 13.

Table 12. Member size for Los Angeles building (General Plan)

<table>
<thead>
<tr>
<th>Level</th>
<th>Structural Element</th>
<th>RBS Dimensions (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Beam size</td>
<td>Column size</td>
</tr>
<tr>
<td>1-2</td>
<td>W24 x 335</td>
<td>W 33 x 387</td>
</tr>
<tr>
<td>3-4</td>
<td>W24 x 306</td>
<td>W 33 x 354</td>
</tr>
<tr>
<td>5-6</td>
<td>W24 x 176</td>
<td>W 33 x 241</td>
</tr>
</tbody>
</table>

Table 13. Doubler Plate and Continuity Plate size for Los Angeles building (General Plan)

<table>
<thead>
<tr>
<th>Level</th>
<th>Doubler Plate Thickness (in.)</th>
<th>Continuity Plate Thickness (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Interior Column</td>
<td>Exterior Column</td>
</tr>
<tr>
<td>1-2</td>
<td>1.625</td>
<td>0.625</td>
</tr>
<tr>
<td>3-4</td>
<td>1.625</td>
<td>0.625</td>
</tr>
<tr>
<td>5-6</td>
<td>1.000</td>
<td>None</td>
</tr>
</tbody>
</table>

2.6 Investigation into Dynamic System-Level Performance through Modeling of Prototype Buildings

As a first step to explore the system-level dynamic performance of SMFs having skewed connections, a detailed 3-D finite element simulation is conducted here. The simulation, conducted through 3-D finite element modeling, provide realistic demands trigger by seismic excitations. One of the prototypes SMF systems (Plan B) at a 6-story height and at three skew levels (10, 20, 30°) is considered for modeling. For more details of skew configurations, Appendix B shows the floor plans considered for the research project. The prototype building design for the location considered (Los Angeles) was uni-directionally loaded using the recorded
earthquake ground motions presented in Table 5, which are scaled to represent both design basis earthquakes (DBE, 10% probability in 50 years) and maximum considered earthquakes (MCE, 2% probability in 50 years). Figure 6 shows an example set of a prototype building containing skewed SMFs. Modeling of the 3-D system-level enables determination of realistic column twist demands during earthquake loading and provide strain demands within RBS and beam-to-column connection zones. Modeling detailed finite element models is necessary for determining local stress and strain fields and is required for use of micro-mechanics based fracture models.

![Figure 6](image)

**Figure 6.** Representative system-level model to investigate the influence of skewed SMFs during dynamic loading.

Beam and column connections are modeled as shell elements with a length of $d_c/2$ for the columns (above and below the beam flange) and $d_b/2$ (beyond the RBS cut) for the beams as shown in Figure 7. For the previous dimension definitions, $d_b$ and $d_c$ are the depth of beams and columns, respectively. The rest of the element will be modeled as a one-dimensional beam element (B31 in ABAQUS) to reduce computational costs. This approach was considered because it is not expected to have yielding far from the connections. The joint between the shell elements and the 1-D element is connected with a rigid-body node, which is marked with an X in
In addition, far more computational cost can be reduced considering that the building structure is symmetric. Therefore, the building model considers one-fourth of the structure seismic mass (one-half of the seismic resisting frame) and is distributed as presented in Figure 8. At the column of every building story (beam top flange level), seismic masses (red dots) are lumped (considering tributary areas) in order to simulate real building conditions during a seismic event.

Figure 7. Structural elements considered for modeling.

A particular continuous column is joined to the model, which represents all gravity columns within one quarter of the structure gravity columns, in order to account for P-Δ effects. This gravity column joins the building frame by rigid links with pin connections to avoid moment transfer between them (dashed line in Figure 8). The sum of the weak axis bending inertia of each gravity column within the selected one quarter of the building equals the P-Δ gravity column stiffness (inertia and polar moment of inertia) [24, 25, 26, 27, 28, 29]. A generalized profile is used in ABAQUS to model the gravity column. This Abaqus particular
profile allows modification of some column properties for proper modeling of the gravity column. For P-Δ effects on the building, gravity column inertia is the critical property to be specified; however, gravity column area and density are modified in order to avoid gravity column buckling and extra P-Δ effects due to gravity column mass inertial effects. For this specific part of the model, the gravity column area is 10000 in² for levels 1 and 2, 9000 in² for levels 3 and 4, and 8000 in² for levels 5 and 6; the gravity column density is 2.836E-10 kip/in³. The load of one quarter of the building floor plan is placed at the corresponding level in the gravity column as depicted with red arrows in Figure 8.

For the computer model, rigid foundations are considered when including rigid-body nodal constraints. All beams are laterally braced at $d_b/2$ beyond the end of the reduced beam section farthest form the face of the column [6]. At the beam top flange level, every column has a constraint to simulate the restrictions provided by the slab. Because only one quarter of the building is used for modeling, the column profile on the right side of Figure 8 is just half of the corresponding column at each level. To account for the continuity of the frame at this side, a special boundary condition (YASIMM in ABAQUS), which is marked with xxx in Figure 8, is placed at the right edge of the column. In addition to lateral accelerations, factored gravity loads corresponding to load combination $1.2D + 0.5L + E$ [12] were applied to the gravity column at each level story. The load applied to each gravity column level is the combination of the dead load (D), live load (L) and the seismic load (E), which is applied to the column base.

Damping should be considered for building modeling. Five percent (5%) Rayleigh damping is described from the first and third vibration modes of each model frame. Frequency analysis is executed of every SMF model to obtained modal frequencies, which are used to calculate Rayleigh damping parameters ($\alpha$ and $\beta$). The Rayleigh damping relationship is stated in
Equation 11. Table 14 shows damping parameters obtained after an ABAQUS frequency analysis.

\[ \xi_i = \frac{\alpha}{2\omega_i} + \frac{\beta\omega_i}{2} \quad \text{Equation (11)} \]

Table 14. Damping parameters for study buildings.

<table>
<thead>
<tr>
<th></th>
<th>Plan A</th>
<th>Plan B</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\omega_1) ,(rad/s)</td>
<td>5.1809</td>
<td>5.4763</td>
</tr>
<tr>
<td>(\omega_3) ,(rad/s)</td>
<td>5.4766</td>
<td>5.6437</td>
</tr>
<tr>
<td>(\alpha)</td>
<td>0.266232</td>
<td>0.277937</td>
</tr>
<tr>
<td>(\beta)</td>
<td>0.009383</td>
<td>0.008993</td>
</tr>
</tbody>
</table>

Element size is crucial when working with finite elements. Proper capture of those investigated variables depends on selecting a suitable element size. Therefore, in this research project, four-node quadrilateral elements will be used, and the general size of mesh elements is 2 in. In special moment frames, the RBS section is the critical part of the connection, in which yielding is expected; consequently, the mesh will be refined at that location considering 0.5 in. as an element size as it was done for Prinz (2007) [30].
Material data for this cyclic nonlinear analysis was taken from cyclic coupon testing of A572 Gr 50 steel up to 8% [31], which as stated earlier is similar to A992 steel. This material data has been also used in previous research [5, 32, 33, 34], and it has led to realistic results of plastic strains for A992 steel.

### 2.6.1 Earthquake Scaling and Modeling

For modeling, dynamic time history analysis is used. To apply this method, a set of earthquake accelerations, as artificially simulated earthquakes, are imposed at the building base. Those artificial time histories should match real ground motions (time and amplitude) as much as possible. There are two approaches to modify earthquake time histories: scaling and spectral matching. In this project, we work with linear scaling, which means multiplying the original time history by a factor in order to get a match between the design spectrum and scaled time history.
Getting a scale factor is a procedure based on diminishing the differences between the target spectrum (a spectrum for the site where our project is located, Figure 3) and the earthquake acceleration time history (ground acceleration time history) [12, 35]. Scaling should be performed in the same period range, and it frequently is between 0.2T and 1.5T (ASCE 7) [12]. The main purpose of scaling ground motions is to develop acceleration time histories consonant with the ground-shaking hazard for the structure in the selected place [36].

For scale factor calculation, mean spectral acceleration from the two horizontal (orthogonal) components is used. For scaling, we will use a formulation presented by Makrup and Jamal [35], which is stated in Equation 12.

The ground motions selected and listed in Table 5, which are used for computer modeling and dynamic analysis, were scaled, so they can match design spectra for the locations presented in Section 2.2. The scale factor for each ground motion are also presented in Table 5.

\[
SF = \frac{\sum_{T=T_B}^{T_B} S_{a}^{target}}{\sum_{T=T_A}^{T_A} (S_{a}^{actual})^2} \tag{12}
\]

Equation (12)

The parameters used in the previous equation for spectral matching are defined as:

- \( S_{a}^{target} \) = target acceleration response spectrum from seismic code spectrum (ASCE 7)
- \( S_{a}^{actual} \) = acceleration response spectrum of the given earthquake
- \( SF \) = Scale Factor
- \( T \) = Period of the structure
- \( T_A \) = lower period response spectra (0.2 T)
- \( T_B \) = upper period response spectra (1.5 T)

Figure 9 shows time history accelerations (horizontal and vertical components) for Loma Prieta Earthquake registered by the Hollister City Hall Station. Peak ground acceleration and the corresponding time in each direction are pointed out in this figure. In ABAQUS, those
accelerations are included as Amplitude. As stated earlier, these Amplitudes multiplied by the scale factor are be applied at the building base to study building connection seismic demands.

Figure 9. Time History Accelerations for Loma Prieta Earthquake (Hollister City Hall Station).

2.7 Results and Discussion

The dynamic system-level performance of the skewed SMF building (called Plan B in Appendix B) is presented in three parts. In the first part, peak inter-story and residual drift are presented to evaluate relative lateral deformation demands for the structure under a real scaled ground motion. The second part of this section depicts the level of column twist during the ground motion length. Lastly, an evaluation of column flange plastic strain levels at all building stories is presented. All the results shown hereafter were extracted from analyzing the structure under the 1994 Northridge earthquake, which is depicted in Figure 10. Columns identified as A, B, C and D in Figure 8 are used to explain the results.
Figure 10. Northridge earthquake acceleration plot.

Peak and residual inter-story drift for the building due to the Northridge ground motion are presented in Figure 11. The skewed SMF design responded with maximum drift values at the lower and upper stories.

Figure 11a corresponds to drift plots for columns A (the exterior column), and Figure 11b, c and d correspond to columns B, C and D (interior columns) in Figure 8. Drift values at the first floor is above 4%, which demonstrates the connection ability to accommodate the story drift
angle (0.04 rad) suggested in AISC 341-16 [13]. Residual drift plots show similar pattern as peak inter-story drift values. At the upper and lower stories, residual drift is higher than 0.5% (Figure 11), which according to McCormick et al. [37] is the maximum permissible residual drift were it is cheaper to repair than reconstructing the structure.

The twisting behavior of exterior columns is presented in Figure 12. It is clear that the column supporting the first level is the most twisted column, followed by the columns at the upper stories (5 and 6), which agrees with the drift plot in Figure 11a. Out-of-plane skew for exterior columns is 10 degrees. Then it is expected to have less twist for exterior columns than for interior columns.

![Figure 12. Twist plot for exterior columns (Columns A).](image)

A considerable increment of twist was obtained for the interior columns (columns B in Figure 8). For this group of columns, which twist plot is presented in Figure 13, out-of-plane skew angles are 10 and 20 degrees. Because two beams are acting over the column, it was expected to have higher twist values for this set of interior columns in comparison with exterior columns. Although significant column twist is found again at the lower level (level 1), column at
the upper level (level 6) shows more twist than at other stories at several points during the earthquake.

![Twist plot for interior columns (Columns B).](image)

The highest column twist from this model is found at the second group of interior columns (Columns C), and the plot is presented in Figure 14. As expected, the column twist is the highest because out-of-plane angles are 20 and 30 degrees and it is an interior column as well. Normally there is a delay to reach the twist peaks when comparing two or more successive stories. For this case, the first story shows the highest twist (0.0065 rad) of all column groups at 6 seconds of the ground motion event.
Figure 14. Twist plot for interior columns (Columns C).

Yielding at the column flange is another important feature presented to evaluate skewed SMF performance. Figure 15 shows equivalent plastic strain (PEEQ) distribution for column flange at the beam bottom flange level on the column right side. PEEQ values on the column left side are not plotted because they were zero almost always, which could be due to initial imperfections included in the model. Based on this figure, yielding at the first floor is the highest at all group of columns, and the next stories with more PEEQ are levels 5 and 6. The effect of this high level of yielding at these specific stories matches perfectly the elevated amount of residual drift presented in Figure 11. In other words, structure stories 1, 5 and 6 have the highest residual drift due to the increased amount of yielding presented at these floor levels. PEEQ at column flange-tip is almost not affected at column groups A and B (see Figure 8 and Figure 15a and b) at all floor levels, but at floors 1 and 2 of column group C (Figure 15c), there is a considerable amount of yielding at the column flange edge.
Figure 15. PEEQ distribution along the normalized column flange at the beam bottom flange level for all stories.

Due to boundary conditions, column flange yielding at the beam top flange level behaves completely different compared with column flange yielding the beam bottom flange level. At the beam top flange level, the majority of column yielding happens at the column flange-tip as presented in Figure 16. At all column groups (A, B and C in Figure 8), normally the upper stories (5 and 6) face more yielding than the lower levels. Yielding at exterior columns is smaller than for interior columns, and this might be related to the fact that links connect the gravity column and the frame at that specific level. This also could explain that at the beam bottom flange level happens the opposite regarding to column flange yielding. As it is clear in Figure 15, exterior columns present more yielding at the beam bottom flange level than interior columns.

It should be pointed out that at the beam top flange-to-column weld, having out-of-plane skew of 30 degrees, the amount of yielding was higher than within the RBS section. This happens specifically at the second and third levels. Certainly, RBS beam connections with skew of 30 degrees have less yielding than all other RBS connection in the building.
2.8 Summary and Conclusions

In this chapter, an investigation into dynamic system-level performance was conducted. A finite element model of a real-scale building located in Los Angeles, CA, designed according to AISC 341-16 [13] and under the Northridge earthquake ground motion, was used to evaluate the seismic performance of skewed RBS SMF connections. Ground motion selection was done using deaggregation. The six-story building was created from four-node linear shell elements (S4R in ABAQUS) and one-dimensional beam elements (B31 in ABAQUS), considering material damping and gravity loads applied to the leaning column to account for P-Δ effects. This leaning column represents one quarter of the building gravity columns and its stiffness equals the sum of gravity column weak-axis inertia. Three levels of skew (10, 20 and 30 degrees) were considered when creating the building geometry.
Drift at the first floor is over the limit (4%) established in AISC 341-16 due to increased yielding at this specific story. Residual drift at these (1, 5 and 6) stories is also higher than the suggested limit (0.5%) for repairing.

Twist plots reflect similar behavior to drift results. Generally, twist is higher at the first and upper (5 and 6) levels. However, the intermediate stories (2, 3 and 4) show less twist and drift, which is consistent with the amount of yielding at these intermediate stories.

Although peak drift at the first story is high, no plastic hinge was developed within the building beam RBS sections. Yielding within the RBS for beams with out-of-plane skew of 30 degrees was the smallest among all the out-of-plane skewed beams.
CHAPTER 3: COMPONENT-LEVEL PARAMETRIC INVESTIGATION INTO SKEWED SMF CONNECTION RESPONSE

Out-of-plane skew connections can be studied using finite element models, which offers options for modeling several beam-to-column connection configurations. Previous finite element models have provided realistic data about fundamental parameters of steel design, such as stress, demands, buckling and fracture [1, 5, 7]. A 13 ft-column and a 15 ft-beam connection model, which constitutes the conventional moment frame assembly for modeling and experimental testing, are not be considered here to create finite element models because the column boundary conditions do not reflect the torsional boundary conditions of the columns in a real structure. Instead, a 3-story frame is used for modeling because the column boundary conditions represent those in the real interior frame. The boundary conditions and applied loads used for modeling, are be the same used by Prinz and Richards and Desrochers [5, 16] in order to have a result bank for comparison.

3.1 Background

Since that key concept of weakening the beam that frame into the connection was proposed after the 1994 Northridge earthquake in order to move the plastic hinge away from the column face, trimming the beam flanges was the most accepted solution to avoid potential fragility at the connection welding [38]. The idea of cutting off part of beam flanges for improvement of steel connection in seismic zones was first investigated experimentally in 1990 [39]. However, it was in 1994 when the effectiveness of this type of connection was confirmed [40]. The first trimming strategy was established intending to follow the moment diagram profile (beam flanges tapered) [41, 42], but a circular profile was also proposed by Engelhardt et al. [43] in 1996. Flange trimming following a circular profile for RBS connections is the current prequalified approach in the AISC prequalified connections [6].
The capacity of a RBS SMF connection is defined as the ability to achieve 4% drift with neither fracture nor strength degradation below 80% [13] of the sample nominal capacity [44] under cyclic loading.

Experimental results have been used to validate finite-element models of RBS connections. Finite element models of RBS connections have exhibit well accuracy to predict important parameters of steel connection analysis such as local buckling, fracture locations and local stress [1, 7, 45]. Besides moving the plastic hinge away from the column face, cutting off the beam flanges delays local buckling, but increases the possibility of web buckling and lateral torsional buckling [38]. Jones et al.[4] and Deierlein et al. [46], through finite element RBS models, found that substantial decrease in inelastic strain demands can be achieved at the CJP beam flange welding for RBS connections. The column section modulus as well as torsional rigidity have great influence on fracture potential and column twist in RBS moment connections. RBS connection ductile fracture potential can be smaller for shallow columns than for the same connection type with deep columns [44].

After the Northridge earthquake, the SMF connection design trend was to move the plastic hinge away from the column face as for RBS connection, but for the case of WUF-W connections, the plastic hinge is not moved away from the column face. For achieving SMF performance without fracture, WUF-W moment connections use special design and detailing features such as welding the shear tab to the beam web and different access hole dimensions. In general, WUF-W moment connections have good performance satisfying the minimum prequalification requirements for SMF, although some researchers [47, 48] have reported concerns about the failure pattern due to the access hole geometry. A geometry change improvement to avoid access hole issues was presented by Han et al. [49].
Generally, WUF-W moment connections provide more moment capacity than RBS moment connections for SMF, but panel zone plastic rotations could be higher for WUF-W connections. It was reported that RBS moment connections have less potential for ductile fracture at the connections region than WUF moment connections [44].

3.2 Analytical Investigation

The sensitivity of results to parameter changes can be investigated through finite element simulations to study larger subassemblies. While the beam-column subassemblies proposed for experimental testing in current lab facilities are a convenient and reasonable method to represent the flexural conditions in the column and investigate potential failure modes at the connections, constraints associated with the component-level testing make it difficult to represent realistic column torsional restrictions. Reduced Beam Section (RBS) and Welded Unreinforced Flange-Welded Web (WUF-W) prequalified moment connections for SMF are included in this research. With these two types of connections, an ample scope of the component-level analysis in kept in the investigation. The response of RBS and WUF-W connections having skew could be largely affected by the column torsional restraints. More realistic column conditions can be achieved using larger and more expensive experimental subassemblies with complex loading systems; however, experimental investigations at this scale are often not practical. The models to be developed in this research will provide augmented information on the response of skewed SMF connections having realistic column boundary conditions and realistic column axial loads. Moreover, the measured response from experiments performed in Chapter 5 of this investigation will be used to validate modeling procedures for the computer models in the current chapter.
3.2.1 Parametric Investigation of Column Axial Loads on Skewed Connection Response

Four different RBS models (T2R-14, T2R-18, T2R-24 and T2R-33) and three WUF-W models (T2W-14, T2W-24, T2W-33) having various beam-to-column connection configurations are analyzed to determine whether column axial loads have a negative effect on skewed SMF response (moment-rotation capacity, column twisting/yielding, etc.). The geometries for the RBS connections consider deep (W33×291), medium (W18×143 and W24×131) and shallow (W14×193) column configurations, with four levels of beam skew (0, 10, 20 and 30 degrees). Similarly, WUF-W models include deep (W33×354), medium (W24×162) and shallow (W14×257) column configurations. The model columns are subjected to four levels of column axial load (0, 10, 25, and 50% of the column axial capacity) in addition to the qualifying cyclic loading protocol applied to the model beam tips. Here, column axial capacity is also called $\phi P_n$.

Table 15 and Table 16 show the analysis matrix of beam and column geometries considered, along with several degrees of skew and column axial loads. The beam sizes are intentionally selected to have flange width-thickness ratios that barely satisfy the seismic compactness requirements outlined in AISC [13, 14], therein considered critical cases. A total of 112 advanced non-linear finite element simulations are conducted (7 beam-column geometries x 4 skew angles x 4 column axial load levels = 112 Abaqus analyses).

The strong column/weak beam principle was considered when sizing beam and columns for the parametric study [13] as it was contemplated in the seismic design to have a more uniformly distributed drift and localized yielding at RBS sections.
Computer models consider a 3-story SMF structure such that the middle beam-to-column connection represents a realistic column condition (being detached from imposed boundary constraints), which hereafter is called Connection of Interest (COI). Because there is a need to
study real demands on interior frame joints, all simulations are performed on double-sided 3-story skewed SMF.

Figure 17 shows a sketch of the 3-story frame, the representative connection (COI), and boundary constraints for computer simulations. Having beams at both column sides represents real conditions of moments and shear of an interior column in the actual structure. In Figure 17, available degrees of freedom (DOF), rather than applied constraints, are shown. A node placed at centroid of the cross section, which is tied to all edges in the transversal section, is used to apply all DOFs.

These boundary conditions will be the same as those used in the exploratory research study presented by Prinz and Richards and Desrochers [5, 16]. In order to have an agreement regarding to boundary conditions, the model column ends (bottom and top support) are considered pinned to match those conditions in experimental study.

Building story drifts are caused by flexural and shear strain, and by shear deformations at the joint of beams and columns. Shear deformations in panel zones cause a shear mode of drift [50], which need to be considered in the design of the structural elements. To control shear deformation at the panel zone, doubler plates and continuity plates were designed according to AISC seismic provisions [13] and their sizes are presented in Table 17 (for the RBS connections), and Table 18 shows continuity plate and doubler plate dimensions for the WUF-W connections.

<table>
<thead>
<tr>
<th>Element</th>
<th>Column sizes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>W14×193</td>
</tr>
<tr>
<td>Doubler plate thickness (in.)</td>
<td>0.750</td>
</tr>
<tr>
<td>Continuity plate thickness (in.)</td>
<td>0.625</td>
</tr>
</tbody>
</table>

Table 17. Doubler Plate and Continuity Plate sizes for the parametric study (RBS).
Figure 17. Boundary conditions for simulation of column response in SMRF connections.
Likewise, in previous sections, the RBS connections for the parametric study models considered removing fractions of the beam flanges to reduce the moment capacity of the beams. Reduced Beam Sections (RBS) in the parametric study were designed for each beam size according to AISC 358-16 [6], and they are shown in Table 19.

Table 19. RBS dimensions for the parametric study.

<table>
<thead>
<tr>
<th>Beam</th>
<th>RBS Dimensions (inches)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>W24 x 76</td>
<td>5.5</td>
<td>18.0</td>
<td>2.0</td>
</tr>
<tr>
<td>W36 x 150</td>
<td>9.0</td>
<td>23.0</td>
<td>2.5</td>
</tr>
</tbody>
</table>

All SMRF computer model geometries were created using four-node linear shell elements with reduced integration (S4R in ABAQUS) in order to capture local buckling and obtain local stress and strain measures in the connection regions. This type of shell element (S4R), which has six degrees of freedom (three rotations and three translations) at each node, is appropriate for computational efficiency and it is free from shear locking when the bending strains are developed in the beams and columns, especially at the RBS section [16]. Previous related research [2, 5, 16] had considered a structured mesh size of 0.5 in. x 0.5 in. for the shell elements at the connection region and the size for the rest of the structural elements is 2 in. x 0.5 in. (see Figure 18). Although reasonable results have been reported from these studies [2, 5, 16], a mesh sensitivity study was done to make sure results of this investigation are not skewed due to inappropriate grid refinement. The study was performed using a model T2R-14×-30 (see Table...
Von Mises stresses were extracted from the COI, at top and bottom beam-flange to column-flange interface (right side) to evaluate the stress capturing ability of each grid size. Figure 19 shows a final plot of the mesh sensitivity study from where it is clear that the change in stress for all grid refinements diminish (have less variability) considering result plots backwards and forwards at the mesh 0.5 in x 0.5 in.

Figure 18. General view of the shell element size in the model.

The models for simulations consider fabrication tolerances by including initial geometric imperfections in ABAQUS. More details about initial imperfections are given in the following section.

For all the steel SMRF, elastic properties for the computer model consider a Young’s modulus of 29000 ksi and a Poison’s ratio of 0.30. Steel plastic properties (nonlinear properties) such as strain hardening are fundamental in this analysis. Because of that, a combined nonlinear isotropic and kinematic material model is used to describe strain hardening in ABAQUS [15] (Equation 12). Equation 12 was developed to use several backstresses, but here just one backstress is used leading to $\alpha_1 = 0$. 

46
\[
\alpha = \frac{C}{\gamma} \left(1 - e^{-\gamma \varepsilon_{pl}}\right) + \alpha_1 e^{-\gamma \varepsilon_{pl}} \tag{13}
\]

The variables presented in Equation 11 are named as follows:

- \(C\): Initial kinematic hardening
- \(\gamma\): curve rate of departure from \(C\)
- \(\alpha_1\): backstress
- \(\varepsilon_{pl}\): material coefficient

For A572 Grade 50 steel (plates, beams and columns), the values for the previous equation are: \(C = 406.18\); \(\gamma = 37.175\); and the value of yield stress is 63.5 ksi [51] because considerable plastic strains are expected. Cyclic coupon test data of A572 Grade 50 steel was used to define plastic hardening through linear kinematic hardening law [31]. Among the three prestrain choices presented in Kaufmann et al. [31], 8% was used as strain range for stabilized cycles of the calibration test because it gave reasonable results in previous works presented by Richards and Prinz, Richards and Uang [32, 33]. Physical properties of A572 Grade 50 steel such as weight density is considered as 0.2836 lb/in³ [52] for modeling purposes.

![Figure 19. Mesh refinement analysis graph.](image-url)
3.2.1.1 Initial Imperfections

Element out-of-straightness can affect the capacity of the member. These imperfections, which result mostly because of construction tolerances, will be considered for modeling through the Abaqus imperfection option. Normally geometric imperfections are introduced in the model by perturbations of the geometry using a linear overlapping of buckling modes. The mode shapes are obtained from one preliminary run analysis using the corresponding Abaqus model without loads. For modeling purposes herein, the first (the lowest) mode shape is used because it provides the most critical imperfections. In other words, the buckling loads of the systems decrease as the amplitude of the mode shape increases, and this reduction of buckling loads is more significant when having lower imperfection amplitudes. Usually the geometric imperfections are scaled using a factor of $L/1000$ [5, 16], where $L$ is the length of the column between adjacent supports.

Twenty-eight finite element models (one for each connection geometry) were developed to obtained frequency analysis on SMF connection specimens. Those outputs were used as initial geometric imperfection for complete finite element simulations.

3.2.1.2 Loading Protocol

Frame/building design method used for this project is based on drift calculations because of the severe seismic conditions presented in the place selected for analysis, which means that axial loads (due to gravity loads) are considerably small compared with lateral loads. The columns in each model should be modeled with axial loads to simulate real conditions in the structure, however. Those axial loads will be a percentage of the column axial capacity, which is also presented in Table 15. This percentage is not small because the lateral loads used for designing the frame are significantly larger, which means that column axial capacity is high.
In addition to the axial loads, all finite element models are loaded with the last version of the loading protocol established by the AISC seismic provisions [13]. The loading protocol presented in Figure 20 establishes the number of cycles and the amount of rotation that should be applied to the model beam end. Loading sequence for beam-to-column moment connections (loading protocol) are applied through vertical displacements based on the beam length and the angles for qualifying cyclic tests of beam-to-column moment connections in SMF described in the AISC seismic provisions [13]. Loading protocol has been used successfully for experimental testing by Chi and Uang and Tsai et al. [3, 53], for analytical simulations by Prinz and Richards [5], and for experimental and analytical work by Kim et al. [10], among others. Although the AISC seismic provisions [13] outline story drift increments of 0.01 rad beyond 0.04 rad, the prequalification requirement for ductility is 0.04 rad [13], as it is pointed out in Figure 20. However, several increments of 0.01 rad are considered in the loading protocol (see Figure 20) in order to study connection capacity at more severe deformations.

Figure 20. Loading Protocol [13].
3.3 Application of Low-Cycle Fatigue Damage Models for Investigating Potential System-Level

3.3.1 Low-Cycle Fatigue Fractures

Potential for unexpected local damage during seismic loading, which could lead to material fractures, was one of the stunning observations found after the 1994 Northridge earthquake building damage investigations. Moment frame connections with low quality performance were found in several buildings after the Northridge earthquake. An assessment of connection concerns not visible through typical tests having cyclic loading protocols is possible when developing computer models capable of capturing these deleterious effects in simulations having more practical boundary conditions or during more realistic seismic loading.

For this research project, the overall performance of the model is of interest, but also the detailed behavior of COI. An advantageous alternative to do that is to use submodeling. The submodeling technique is used to study a local part of a model with a refined mesh based on interpolation of the solution from a global model. This technique is more useful when it is necessary to obtain an accurate, detailed solution in a local region [54]. With refined submodels of the beam-to-column connection region and RBS zone for selected connections in the parametric study models, it is possible to identify tendencies between skew angle and the potential for low-cycle fatigue damage. Not only relationships between skew and fatigue damage can be acquired from submodeling, but also complete calculations for micromechanics-based damage (crack initiation) because entire stress and strain states could be obtained from them.

Reasonable predictive results have been obtained when using micro-mechanics based fatigue and damage models [34, 55, 56, 57, 58, 59]. Figure 21 shows a common geometry of a submodel for the beam-to-column connection and RBS regions to be developed in this research.
For submodeling, eight-node hexahedral elements with size of 0.2 in. are used. This element size was selected based on the good results obtained by Prinz and Richards [5].

From coupon tests on materials extracted from the completed experimental testing, steel specific parameters required for calibration of the micro-mechanics based damage models are obtained. Comparison between damage model predictions and observations during testing could be done creating models simulating the experimental tests, validating the previously described damage calculations, simultaneously.

Several parameters can be extracted from Abaqus to study fracture initiation due to large plastic strains in the model. Because structural steel systems under earthquake loading can fail due to Ultra-Low Cycle Fatigue (ULCF), an investigation of this phenomenon needs to be performed, and Abaqus provides us with useful outputs to evaluate crack initiation. Among the values used here to study fracture initiation, Von Mises stresses (S-Mises), Principal Stresses (SP in ABAQUS), Equivalent Plastic Strain (PEEQ in ABAQUS) are the most important. Those values are extracted at a point where the major strain is expected. In other words, Abaqus is set to extract data values at the RBS cut region and at the connection weld.

Figure 21. Submodel to study stress/strain states for micro-mechanic analysis.
To predict crack initiation, some studies [5, 30, 34] used the failure index for ductile fracture prediction. Having the data previously mentioned (S-Mises and PEEQ), the failure index can be calculated (see Equation 15) dividing PEEQ by a critical plastic strain calculated from the Stress Modified Critical Strain (SMCS) criterion presented by Hancock and Mackenzie [60]. The critical plastic strain is calculated as:

$$\varepsilon_{p, \text{critical}} = \alpha \cdot e^{-1.5\left[\frac{\sigma_m}{\sigma_e}\right]}$$

Equation (14)

Failure Index = \frac{\text{PEEQ}}{\varepsilon_{p, \text{critical}}}

Equation (15)

From Equation 14, $\sigma_m$ is the mean stress, $\sigma_e$ is the effective stress (S-Mises), and $\alpha$ was defined previously. The quotient $\sigma_m/\sigma_e$ is called triaxiality ($T$); the larger the triaxiality, the higher the potential for fatigue fracture [10]. Then, the condition of failure is reached when the value of PEEQ is greater than $\varepsilon_{p, \text{critical}}$.

This method is relatively easy to apply, but it is based on two fundamental assumptions that we want to avoid in this research project. The SMCS model assumes that triaxiality does not vary substantially regarding to increasing plastic strain. However, steel is a very ductile material in which considerable geometry changes are expected. The other important factor to be considered is the fact that SMCS models were developed using monotonic testing (tension load is applied). However, when dealing with seismic loading, the load is cyclic. For this reason, the potential for fatigue failures is evaluated using the Degraded Significant Plastic Strain (DSPS) model, which is an extension of the SMCS [61], and was developed considering cyclic loading. For the DSPS model, the criterion to predict failure is based on a relationship between the significant plastic strain ($\varepsilon^*_t$), the degraded critical plastic strain ($\varepsilon^*_{p,cr}$) and the characteristic length $l^*$. The equation for the significant plastic strain is shown in Equation 16, and the complete form of the DSPS model (degraded critical plastic strain) is presented in Equation 17.
The characteristic length ($l^*$) is introduced in this model to collect multiple single-material points failures [61], and its value is $l^* = 0.0079$ in. for steel A572 Grade 50 [55, 61]. For this model, when the significant plastic strain ($\varepsilon_t^*$) exceeds the degraded critical plastic strain ($\varepsilon_{p,cr}^*$) over characteristic length $l^*$, failure is expected.

$$\varepsilon_t^* = \varepsilon_{p,t} - \beta * \varepsilon_{p,c}$$  
Equation (16)

$$\varepsilon_{p,cr}^* = e^{(-\lambda_{DSPS}*|\varepsilon_p|)} * \alpha * e^{-1.5[\frac{\sigma_m}{\sigma_e}]}$$  
Equation (17)

From the previous equations, the plastic strains are Abaqus outputs, and the other variable values and names are defined as follows:

$\beta = 0.6$ [5], a constant

$\lambda_{DSPS} = 0.38$ for A572 Grade 50 [55, 61], called the damageability parameter, and it was calculated from cyclic test of notched bars

$\alpha = 2.9$ [5, 57, 59], is a material specific parameter

$\varepsilon_{p,t}$ = the integration of plastic strain under tensile triaxiality

$\varepsilon_{p,c}$ = the integration of plastic strain under compressive triaxiality

$\varepsilon_p$ = the accumulated equivalent plastic strain.

For submodels to be developed here, the smallest grid refinement is 0.2 in., but it is larger than the characteristic length ($l^*$). However, this is not a concern according to Kanvinde and Deierlein and Fell et al. [55, 56] because we are just looking for an estimation of the place and time of crack initiation.
3.4 Results and Discussion

Two types of finite element models were generated to accomplish this chapter. They are
called global models and submodels. The global model was used to perform analysis of a
connection subassembly in order to evaluate the global response, such as connection capacity,
column yielding, panel zone plastic rotation and column twist response. The sub-model was
utilized to perform a local analysis of the connection in the region of a beam
tension/compression flange.

3.4.1 RBS Results Analysis

Sixty-four RBS Abaqus global models were developed to investigate skew effects on
connection capacity and column twisting. Table 20 summarizes results of peak moment at the
connection, beam rotation at \(0.8M_p\) and column twist at 4\% drift for all the RBS models.

Table 20. RBS analysis matrix and results

<table>
<thead>
<tr>
<th>#</th>
<th>Model</th>
<th>Column</th>
<th>Beam</th>
<th>Skew (deg)</th>
<th>Axial Compression Force (% of (f_{Pn}))</th>
<th>Peak Moment @ Connection (k-ft)</th>
<th>Rotation @ 0.8M_p (rad)</th>
<th>Column Twist @ 0.04 rad drift (deg)</th>
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<tbody>
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<td>1</td>
<td>W14x193_0_0%</td>
<td>W14x193</td>
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<td>883.230945</td>
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<td>2</td>
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54
Table 20. RBS analysis matrix and results (Cont.).
#

Model

Column

Beam

26
27
28
29
30
31
32
33
34
35
36
37
38
39
40
41
42
43
44
45
46
47
48
49
50
51
52
53
54
55
56
57
58
59
60
61
62
63
64

W18×143_20_10%
W18×143_20_25%
W18×143_20_50%
W18×143_30_0%
W18×143_30_10%
W18×143_30_25%
W18×143_30_50%
W24×131_0_0%
W24×131_0_10%
W24×131_0_25%
W24×131_0_50%
W24×131_10_0%
W24×131_10_10%
W24×131_10_25%
W24×131_10_50%
W24×131_20_0%
W24×131_20_10%
W24×131_20_25%
W24×131_20_50%
W24×131_30_0%
W24×131_30_10%
W24×131_30_25%
W24×131_30_50%
W33×291_0_0%
W33×291_0_10%
W33×291_0_25%
W33×291_0_50%
W33×291_10_0%
W33×291_10_10%
W33×291_10_25%
W33×291_10_50%
W33×291_20_0%
W33×291_20_10%
W33×291_20_25%
W33×291_20_50%
W33×291_30_0%
W33×291_30_10%
W33×291_30_25%
W33×291_30_50%

W18×143
W18×143
W18×143
W18×143
W18×143
W18×143
W18×143
W24×131
W24×131
W24×131
W24×131
W24×131
W24×131
W24×131
W24×131
W24×131
W24×131
W24×131
W24×131
W24×131
W24×131
W24×131
W24×131
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W33×291
W33×291
W33×291
W33×291
W33×291
W33×291
W33×291
W33×291
W33×291
W33×291
W33×291
W33×291
W33×291
W33×291
W33×291

W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W24×76
W36×150
W36×150
W36×150
W36×150
W36×150
W36×150
W36×150
W36×150
W36×150
W36×150
W36×150
W36×150
W36×150
W36×150
W36×150
W36×150

Skew
(deg)

20

30

0

10

20

30

0

10

20

30

Axial
Compression
Force (% of
 Pn)
10
25
50
0
10
25
50
0
10
25
50
0
10
25
50
0
10
25
50
0
10
25
50
0
10
25
50
0
10
25
50
0
10
25
50
0
10
25
50

Peak
Moment @
Connection
(k-ft)
850.405433
853.507449
858.752962
844.736918
843.752761
846.740837
846.285077
879.639622
888.694602
893.347563
889.995581
871.134194
871.344481
870.088364
869.756110
863.112427
864.037692
864.487707
863.978811
853.688743
854.033614
854.378486
853.083115
2782.62383
2770.92246
2770.93608
2777.32484
2709.29615
2711.78900
2714.18648
2732.41283
2680.98947
2682.12010
2687.07854
2687.80051
2657.09633
2658.39043
2661.33280
2655.97932

Rotation
@ 0.8Mp
(rad)
0.051353
0.052546
--0.049082
0.049746
0.050120
0.054200
0.049751
0.051471
0.052465
0.051895
0.048083
0.048235
0.049085
0.046971
0.047239
0.047563
0.048207
0.050013
0.046569
0.046758
0.047034
0.047042
0.043809
0.043784
0.043710
0.044324
0.042877
0.042621
0.042707
0.042836
0.041918
0.042269
0.042432
0.042514
0.041947
0.041815
0.042174
0.041930

Column
Twist @
0.04 rad
drift (deg)
0.561202
0.566478
0.578992
0.712816
0.726358
0.757678
0.921945
0.162098
0.138474
0.105471
0.158990
0.336091
0.339126
0.342399
0.363475
0.482061
0.486982
0.497412
0.534688
0.618821
0.629543
0.644524
0.698373
0.242610
0.248276
0.252763
0.252313
0.378374
0.382134
0.390034
0.410236
0.467838
0.470915
0.474583
0.476298
0.535398
0.542879
0.552980
0.588286

3.4.1.1 Moment capacity of Skew RBS Connections
Backbone curve plots of skewed RBS connections indicate that connection capacity is not
affected considerably by the skew presence. Although there is always a moment capacity
reduction, connection strength overcomes the minimum requirements established in AISC for
prequalification [5]. As depicted in Figure 23, at 4% beam drift, the moment capacity does not

55


degrade below $0.8M_p$ for any connection configuration. It is true that out-of-plane skew reduces connection moment capacity because part of the beam moment must be resisted by the column weak axis (as shown in Figure 22), but still that capacity reduction meets the AISC minimum prequalification requirements. Figure 23 shows results for 10% column axial capacity. Results for some other levels of column axial capacity (0%, 25% and 50%) are presented in Appendix F, all of them with similar outcome as in Figure 23.

In comparing total connection rotation at $0.8M_p$, shallow (W14×193) and medium (W18×143 and W24×131) columns reached higher connection rotations. At $0.8M_p$, connection rotation for shallow and medium columns could reach more than 0.06 rad, but connection rotation for deep columns is barely above 0.04 rad. This behavior was observed by Ozkula and Uang [8] when performing full scale experimental testing for some W24 sections. They found there that deep sections are prone to develop less plastic rotation. Table 20 presents a complete list of connection rotations at $0.8M_p$ for all axial loads and out-of-plane skews considered in this study.

![Figure 22. Moment components resisted by column axes due to out-of-plane skew.](image)
3.4.1.2 **Twisting Response of Column Axial Load on Skew RBS Connections**

Column twisting is increased by the beam skew due to the increased out-of-plane bending. Figure 24 depicts the amount of column twist from all RBS models at 0.04-rad beam rotation. Twist versus skew for different beam rotation levels (2, 3 and 5% drift) are presented in Appendix F for more illustrations.
A comparison (at the same skew level) between all RBS models evidences that as the column size increase, the twist increases as well. This behavior can be explained looking at properties of plane areas such as polar moment of inertia \(J\) and column depth \(d\). For the three first sections (W14x, W18x and W24x) in Table 21, the shallow column (W14x193) has the highest polar moment of inertia, which explain why this column experiences less twist. Column depth can affect the amount of twist. Although the polar moment of inertia of the deep column (W33x291) is twice the polar moment of inertia for the shallow column, the deep column depth \(d\) is also double, which leads to more twist. Even for orthogonal connections (no skew), there is always a twisting angle for the medium (W18x and W24x) and deep columns (W33x) [2, 5]. Deeper columns present more twist, but the twist angle is less than 1° for all RBS models at 4% drift. Column twists due to skew angle at 25% axial load, for all skew connections, are presented in Figure 25. This figure shows a considerable amount of twist at the beginning of the cyclic loading (when connection moment capacity is over 80% of \(M_p\)) for the skewed connections (especially for 20° and 30° skew), yet for orthogonal connections, the highest twist occurs after the plastic hinge develops. However, this amount of twist decreases when connection moment capacity starts decreasing as well because the plastic hinge development reduces the torque applied by the beam bottom flange to the column.

<table>
<thead>
<tr>
<th>Column</th>
<th>d (in)</th>
<th>J (in(^4))</th>
<th>(\phi M_{px}) (k-ft)</th>
<th>(\phi M_{py}) (k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W14x193</td>
<td>15.5</td>
<td>34.8</td>
<td>1331.25</td>
<td>675.000</td>
</tr>
<tr>
<td>W18x143</td>
<td>19.5</td>
<td>19.2</td>
<td>1207.50</td>
<td>320.250</td>
</tr>
<tr>
<td>W24x131</td>
<td>24.5</td>
<td>09.5</td>
<td>1387.50</td>
<td>305.625</td>
</tr>
<tr>
<td>W33x291</td>
<td>34.8</td>
<td>65.1</td>
<td>4350.00</td>
<td>847.500</td>
</tr>
</tbody>
</table>

In addition to column properties, the beam skew also leads to increase the column twisting and it can be explained with the sketches presented in Figure 26. In this figure, it is clear
that the lateral displacement due to the buckled RBS affects the column torsion conditions. The lateral displacement of the buckle RBS behaves different from the experiment performed by Chi and Uang [3] because of the prequalification limits established in AISC Prequalified Connections [6]. Among all the limitations in AISC 358-16, lateral bracing is fundamental to prequalify a SMF connection. For SMF RBS connections, supplemental lateral bracing should be provided at a distance no greater than $d/2$ beyond the end of the RBS, where $d$ is the beam depth. Due to lateral bracing, the beam always keeps its alignment from the end of the RBS to the beam tip. Then when RBS buckle, the column moves back causing important differences regarding to twisting conditions. In Figure 26b (an orthogonal connection), torsional drift is negligible before buckling happens at RBS. However, torsional drift starts increasing considerably after the RBS buckle. In other words, $e_x$ starts increasing and keeps that way, causing a growing twist angle as it can be seen in Figure 27a.

![Figure 24. RBS column twist versus skew angle at 4% beam drift.](image-url)
Figure 25. Column twist comparison for all RBS models at 25% $\phi_c P_n$.

On the other hand, in Figure 26a (a skewed connection), before the RBS starts buckling, the beam flange force and the geometric center of the column are experiencing the highest possible eccentricity ($e_{xb}$), but when the buckling starts at RBS, the eccentricity in x-direction ($e_{xa}$) starts decreasing causing less twisting over the column cross-section. Figure 27b shows the cyclic twisting of a skewed connection where the maximum capacity of the connection is
reached before the plastic hinge and immediately, after the RBS buckle, the twist angle starts decreasing.

![Figure 26. RBS column torsion and weak axis bending produced by a) out-of-plane skewed and b) lateral torsional buckling of RBS.](image)

3.4.1.3 Column flange stresses on Skew RBS Connections

Previously, it was shown that even for orthogonal connections, columns experience some level of twist. Out-of-plane skew and flange local buckling contribute to increase the column twist, which produces column flange yielding. The higher levels of column flange yielding are presented at the column flange-tip and at the lower beam-flange to column-flange interface, as presented in Figure 28. To investigate yielding distribution along the column flange, equivalent plastic strain (PEEQ) was retrieved at the two shaded locations which were mentioned hereafter.
Out-of-plane skew and applied boundary conditions are the main source of column twist increments. Those increments of column twist lead to higher levels of column flange-tip plastic strain. Figure 29 shows the effects of beam skew and column axial load on PEEQ at the column flange-tip at 4% for all RBS models. Because the deepest column (W33×291) faces more twist and the applied moment by the beam is higher, these columns always have more yielding than the other column sections for all the considered skew angles. The level of yielding at the column flange-tip is also increased by the skew angle as depicted in Figure 29. The axial load has not significant impact on column flange-tip yielding.

Figure 29. Effects of beam RBS skew and column axial load on PEEQ at column flange-tip at 4% drift.

Figure 30 clearly depicts location for the extraction of PEEQ values at lower beam-flange to column-flange interface. Equivalent Plastic Strain result plots in Figure 31, at the beam-flange to column-flange interface, show that by increasing the column axial load, PEEQ results are also
enlarged, especially close to beam-to-column center line. Increment of skew angle does not affect much the yielding for the shallow column; however, for the medium and deep columns, the influence of the skew angle on increasing column flange yielding is significant. Figure 31 also shows that similar to PEEQ results at column flange-tip, the medium and the deep columns experience more yielding at the column flange edge for the most severe out-of-plane skew angles.

Figure 30. Location on beam-flange to column-flange interface for PEEQ extraction.
Figure 31. PEEQ distribution along the normalized column flange with varying skews and axial loads at 4% drift a) W14×193, b) W18×143, c) W24×131, d) W33×291 RBS models.
Figure 31. PEEQ distribution along the normalized column flange with varying skews and axial loads at 4% drift a) W14×193, b) W18×143, c) W24×131, d) W33×291 RBS models (Cont.).

3.4.2 WUF-W Results and Discussion

Forty-eight WUF-W ABAQUS global models were developed to add comparisons with RBS models as well as investigate skew effects on column flanges. Table 22 summarizes results of peak moment at the connection, beam rotation at 0.8\(M_p\) and column twist at 4% drift for all the WUF-W models. The model configurations consider three levels of skew and four level of axial loads.

Table 22. WUF-W analysis matrix and results

<table>
<thead>
<tr>
<th>#</th>
<th>Model</th>
<th>Column</th>
<th>Beam</th>
<th>Skew (deg.)</th>
<th>Axial Compression Force (% of (\phi P_n))</th>
<th>Peak Moment @ Connection (k-ft)</th>
<th>Rotation @ 0.8 (M_p) (rad)</th>
<th>Column Twist @ 0.04 rad drift (deg.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W14×257_0_0%</td>
<td>W14×257</td>
<td>W24×76</td>
<td>0</td>
<td>1166.37712</td>
<td>0.074814</td>
<td>0.000000</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>W14×257_0_10%</td>
<td>W14×257</td>
<td>W24×76</td>
<td>10</td>
<td>1150.94667</td>
<td>0.076812</td>
<td>0.000000</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>W14×257_0_25%</td>
<td>W14×257</td>
<td>W24×76</td>
<td>25</td>
<td>1141.45002</td>
<td>0.079652</td>
<td>0.000000</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>W14×257_0_50%</td>
<td>W14×257</td>
<td>W24×76</td>
<td>50</td>
<td>1127.79596</td>
<td>0.083990</td>
<td>0.000000</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>W14×257_10_0%</td>
<td>W14×257</td>
<td>W24×76</td>
<td>0</td>
<td>1157.55129</td>
<td>0.071636</td>
<td>0.152543</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>W14×257_10_10%</td>
<td>W14×257</td>
<td>W24×76</td>
<td>10</td>
<td>1140.45002</td>
<td>0.080352</td>
<td>0.153048</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>W14×257_10_25%</td>
<td>W14×257</td>
<td>W24×76</td>
<td>25</td>
<td>1136.48936</td>
<td>0.083990</td>
<td>0.150586</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>W14×257_10_50%</td>
<td>W14×257</td>
<td>W24×76</td>
<td>50</td>
<td>1124.7228</td>
<td>0.086991</td>
<td>0.152543</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>W14×257_20_0%</td>
<td>W14×257</td>
<td>W24×76</td>
<td>0</td>
<td>1154.82480</td>
<td>0.066938</td>
<td>0.296329</td>
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<tr>
<td>10</td>
<td>W14×257_20_10%</td>
<td>W14×257</td>
<td>W24×76</td>
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<td>1140.45002</td>
<td>0.066938</td>
<td>0.296329</td>
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<td>11</td>
<td>W14×257_20_25%</td>
<td>W14×257</td>
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<td>0.304661</td>
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<tr>
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<td>14</td>
<td>W14×257_30_10%</td>
<td>W14×257</td>
<td>W24×76</td>
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<td>0.436561</td>
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<tr>
<td>15</td>
<td>W14×257_30_25%</td>
<td>W14×257</td>
<td>W24×76</td>
<td>25</td>
<td>1148.93275</td>
<td>0.064118</td>
<td>0.436561</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>W14×257_30_50%</td>
<td>W14×257</td>
<td>W24×76</td>
<td>50</td>
<td>1140.93275</td>
<td>0.064118</td>
<td>0.499235</td>
<td></td>
</tr>
</tbody>
</table>
Table 22. WUF-W analysis matrix and results (Cont.).

<table>
<thead>
<tr>
<th>#</th>
<th>Model</th>
<th>Column</th>
<th>Beam</th>
<th>Skew (deg.)</th>
<th>Axial Compression Force (% of $P_n$)</th>
<th>Peak Moment @ Connectio (k-ft)</th>
<th>Rotation @ 0.8 $P_m$ (rad)</th>
<th>Column Twist @ 0.04 rad drift (deg.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>17</td>
<td>W24x162_0_0%</td>
<td>W24x162</td>
<td>W24x76</td>
<td>0</td>
<td>1176.92130</td>
<td>0.067782</td>
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<tr>
<td>18</td>
<td>W24x162_0_10%</td>
<td>W24x162</td>
<td>W24x76</td>
<td>10</td>
<td>1175.73052</td>
<td>0.067019</td>
<td>0.004194</td>
<td></td>
</tr>
<tr>
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<td>W24x162_0_25%</td>
<td>W24x162</td>
<td>W24x76</td>
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</tr>
<tr>
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<td>W24x162_0_50%</td>
<td>W24x162</td>
<td>W24x76</td>
<td>50</td>
<td>1163.91099</td>
<td>---</td>
<td>0.010061</td>
<td></td>
</tr>
<tr>
<td>21</td>
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<td>W24x76</td>
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<td>0.068405</td>
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<td></td>
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<tr>
<td>22</td>
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<td>W24x76</td>
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<tr>
<td>23</td>
<td>W24x162_10_25%</td>
<td>W24x162</td>
<td>W24x76</td>
<td>25</td>
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<tr>
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<tr>
<td>28</td>
<td>W24x162_20_50%</td>
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<td>W24x76</td>
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<td>1165.34693</td>
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<tr>
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<td>W24x162</td>
<td>W24x76</td>
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<td>0.060482</td>
<td>0.978250</td>
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<td>W24x76</td>
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</tr>
<tr>
<td>31</td>
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<td>25</td>
<td>1161.97212</td>
<td>0.062222</td>
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</tr>
<tr>
<td>32</td>
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<td>W24x76</td>
<td>50</td>
<td>1165.65271</td>
<td>---</td>
<td>0.997600</td>
<td></td>
</tr>
<tr>
<td>33</td>
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<td>W36x150</td>
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3.4.2.1 Moment capacity of Skew WUF-W Connections

Similar to backbone curve plots for RBS connections, strength reduction for WUF-W connections is not severe when having some degree of skew. Out-of-plane WUF-W connections are in good agreement with AISC minimum requirement for prequalification because the moment capacity at 4% beam drift does not degrade below $0.8M_p$ for all the studied models. Figure 32 shows comparison backbone curves for the three WUF-W models at 10% axial load. In this figure, it is clear that just deep column connections present some moment capacity reduction caused by out-of-plane skew at 4% drift. Because there are no RBS cuts, the peak
moment at the connection is higher than RBS peak moments. Table 22 also shows that WUF-W connections have more moment capacity at 4% drift in comparison with RRS models. Appendix G shows an additional complete set of backbones curves for 0%, 25% and 50% column axial loads.

Figure 32. Moment capacity comparison for WUF-W models at 10% $\phi_cP_n$.

RBS rotations at $0.8M_p$ were higher for shallow and medium column connections than for deeper column connections. Likewise, deeper WUF-W column sections experience less rotation when moment capacity is equal to 80% of $M_p$. 
3.4.2.2 Twisting Response due to Column Axial Load and Skew on WUF-W Connections

For SMF WUF-W skewed connections, beam plastic hinges lead to column torsional conditions similar to those in Figure 26a. The beam buckles (allowing lateral displacement) between the column face and the beam lateral brace, resulting the column buckling shape in Figure 33 and the twist graph presented in Figure 34b. Lateral braces for SMF WUF-W connections shall be located at a distance of \(d\) to \(1.5d\) from the face of the column [6], where \(d\) is the beam depth. Different from RBS connections where the beam buckling is forced to happen at RBS cut (away from the column face), for WUF-W connections the beam buckles very close to the column flange. The plastic hinge is created after the beam web buckles, which lead to web lateral displacement at the buckling site, but the connection alignment does not change significantly as shown in Figure 33.

![Figure 33. Plastic hinge location for WUF-W connections.](image)

The twist angle for the WUF-W orthogonal connections is very small when comparing it with the twist angle for all skewed WUF-W connections. Figure 34 shows the different amount of twist for orthogonal connections and for 30-degree skew connections. Polar moment of inertia
and column depth (Table 23) play an important role for twist developed by WUF-W columns as explained for the RBS models (large column depth and small polar moment of inertia lead to ample column twist).

![Figure 34. WUF-W column twist vs moment for a) orthogonal connection (0 degree) b) skewed connection (30 degrees).](image)

Similar to RBS connection twist, WUF-W column twisting is increased by out-of-plane skew. Figure 35 shows the amount of column twisting from all WUF-W models at 0.04-rad beam rotation, in which the amount of axial load slightly increases the level of twist. Figure 36 depicts twist differences at 25% axial load for all the WUF-W models. Twist versus skew plots for different beam drift levels (2%, 3% and 5% drift) are presented in Appendix G for more illustrations.

<table>
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<tr>
<th>Column</th>
<th>d (in)</th>
<th>J (in^4)</th>
<th>(\phi M_{px} (k\text{-ft}))</th>
<th>(\phi M_{py} (k\text{-ft}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>W14×257</td>
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<td>79.1</td>
<td>1826.25</td>
<td>922.50</td>
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<tr>
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<td>18.5</td>
<td>1755.00</td>
<td>393.75</td>
</tr>
<tr>
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<td>35.6</td>
<td>115.0</td>
<td>5325.00</td>
<td>1057.50</td>
</tr>
</tbody>
</table>
Figure 35. WUF-W column twist versus skew angle at 4% beam drift.

Figure 36. Column twist comparison for WUF-W models at 25% $\phi_{Pn}$. 
3.4.2.3 Column flange stresses on Skew WUF-W Connections

The total amount of twist produced by orthogonal WUF-W connections is smaller than that for RBS orthogonal connections as shown in Figure 25 and Figure 36. However, the amount of column flange yielding obtained from WUF-W connection could be higher (especially for deep columns) than the yielding on RBS columns. Figure 37 presents PEEQ result plots from the lower beam-flange to column-flange interface (Figure 28 and Figure 30) for all skew angles and axial loads at 4% drift. A direct comparison between RBS and WUF-W for shallow and deep columns (Figure 31 and Figure 37) shows a considerably high yielding level for the WUF-W columns. As expected, this is more significant for 30-degree skew connections.

Figure 32 shows the effects of beam skew and column axial load on PEEQ at the column flange-tip at 4% for all WUF-W models. The variation of column flange PEEQ observed in this plot is similar to the RBS models at the flange-tip. In other words, the shallow column (W14×257) faces less twist and accordingly this column should have less yielding. In addition, the level of yielding at the column flange-tip is also increased by the skew angle as depicted in Figure 38 and the axial load has not significant impact on column flange-tip yielding.

Figure 37. PEEQ distribution along the normalized column flange with varying skews and axial loads at 4% a) W14×257, b) W24×162, c) W33×354 WUF-W models.
Figure 37. PEEQ distribution along the normalized column flange with varying skews and axial loads at 4% a) W14×257, b) W24×162, c) W33×354 WUF-W models (Cont.).

Figure 38. Effects of beam WUF-W skew and column axial load on PEEQ at column flange-tip at 4% drift.
3.4.3 Twist comparison between RBS and WUF-W connections

A direct twist comparison between RBS and WUF-W shallow columns is done next. As stated previously, the beam buckling location is different for RBS and WUF-W connections. Having a plastic hinge close to the column flange exerts less torsion for orthogonal WUF-W connections than for orthogonal RBS connections, but normally skewed WUF-W connections excerpt more twist than skewed RBS connections. Out-of-plane skew angles start triggering more torsion as the angle increases due to the eccentricity between the plastic hinge and the column center. Bucking at RBS connections always happens at a distance $a + b/2$ from the column flange, which produces some amount of twist even for orthogonal RBS connections. Figure 39 shows beam buckling locations for both types of connections considered in this study. Twist comparison details are presented next for each of the investigated column size.

![Figure 39. Plastic hinge location for RBS and WUF-W connections.](image)
3.4.3.1 Shallow columns

Shallow columns having three skew levels and three level of axial loads are compared here. Figure 40 shows column twist comparison for shallow columns with RBS and WUF-W connections. For out-of-plane skew of 10 degrees, RBS connections exert more twist on the column than WUF-W connections at all axial load levels (except for 50% column axial load, due to the buckling). At 20- and 30-degree skew, WUF-W columns start getting more twisted than RBS joints. RBS connection twist does not change considerably (except for the 30-degree skew and 50% axial load), but WUF-W twist is always increasing when enlarging the skew angle and axial load, however. It is important to state that column local buckling is reported for all WUF-W connections having 50% axial load, and it is affecting the degree of column twisting as it can be noticed on the right side of Figure 40. When the axial load is 50%, the column buckling leads to a different twist plot shape as it is clear in Figure 40.
3.4.3.2 Medium columns

Reduce Beam Section medium column connections with 10 degree-out-of-plane skew get more twisted than WUF-W columns as it happened with shallow columns for all axial loads. Similar to shallow columns, at 20- and 30-degree skew, WUF-W connections start exerting more column twist than RBS joins. For medium column sizes, column twist grows as out-of-plane skew and axial loads increase. Also, important to note that almost all medium column presented local buckling for 50% axial load, and WUF-W connections with 20-degree skew presented excessive beam buckling at plastic hinge location.
3.4.3.3 Deep columns

Figure 42 clearly depicts column twist for deep columns with WUF-W and RBS connections. With the exception of the WUF-W connection with 10° skew and 50% axial load (column local buckling) presented in Figure 43, the twist behavior for deep columns is similar to medium column twist behavior. Reduce Beam Section column twist increases when enlarging axial loads and skew angle. For 20- and 30-degree skew, WUF-W column twist is always higher than RBS column twist for all axial loads considered. In general, the column twist for deep column is higher than for the shallow and medium columns.

Figure 41. Twist comparison for medium columns.
Figure 42. Twist comparison for deep columns.
Figure 43. WUF-W column buckling (W33x354, 10-degree skew, 50% axial load).

A comparison of column maximum twist for shallow columns is presented in Figure 44.

This graph confirms some of the abovementioned statements about the level of twist obtained by those columns. For orthogonal connections, higher twist is expected for RBS connections. Twisting increases with the skew and axial loads for WUF-W connections, but RBS connections kept constant twisting level for all axial loads without instability mechanisms (column local buckling).

Figure 44. WUF and RBS twist comparison for shallow columns.

Figure 45 depicts a comparison for RBS and WUF-W connections with medium columns. Orthogonal and 10-degree skew WUF-W moment connections are less twisted than
RBS connections with the same skew levels. However, twisting increased fast for WUF-W connections with 20- and 30-degree skew, which creates a significant difference between shallow and medium WUF-W connections. The values of maximum twist for medium columns are not significantly different from shallow column maximum twist results.

![Figure 45. WUF and RBS twist comparison for medium columns.](image1)

For deep column, Figure 46 shows the twist comparison for deep columns having RBS and WUF-W connections. Both RBS and WUF-W connections follow the same trend as for medium columns (Orthogonal and 10-degree skew RBS connections showed more twist than the corresponding WUF-W connection). Note that twist is not affected by axial load for RBS connections at all column sizes, but WUF-W connections presented some twist increments when increasing axial load.

![Figure 46. WUF and RBS twist comparison for deep columns.](image2)
3.4.4 Ultra-low Cycle Fatigue Results and Discussion

Low-cycle fatigue failure is sometimes a limitation for rotation capacity in experimental testing of SMF RBS connections. Low-cycle fatigue leads to material failure within the RBS before local buckling can cause excessive strength degradation [5]. Ultra-low cycle fatigue analysis of SMF RBS connections was used to investigate potential for ductile fracture, at connection welds and RBS cross section at the bottom and top flange, for all the models presented in Table 15.

The values of PEEQ, MISSES stress and pressure stress were extracted at the element with the highest PEEQ value at the RBS flange cut and the join of beam flange-to-column flange (weld). Those values are extracted at 4% drift, which is the minimum drift rotation for AISC prequalification. To investigate the level of damage, the degraded critical plastic strain (strain capacity) and the significant plastic strain (strain demand), which were defined in section 3.3, are calculated are the RBS flange cut and the connection weld (beam top and bottom flange). The strain capacity decreases due to the damage experienced by the connection (If PEEQ increases, strain capacity decreases) [5].

Figure 47 shows an ultra-low cycle fatigue plot for the shallow column (W14×193), with 0- and 30-degree skew and 0% axial load. The red dashed line represents 4% of total drift. Fracture is predicted when the stain capacity joins the demand capacity. From this graph, it is clear that there is more damage within the RBS for the out-of-plane skewed connection than for the orthogonal connection beyond 5% drift (analysis step ~130). Fatigue capacity at the connection (weld) is experiencing minor degradation in both cases. However, this study is focused on damage investigation at minimum prequalification requirements, and therefore the level of damage is calculated at 4% drift.
To calculate the damage ratio, the difference between the strain capacity and strain demand at 4% drift is divided by initial damage index. Graphs of damage ratio vs axial load are presented to depict the effects of increasing axial loads on fatigue capacity of SMF RBS connections.

Figure 47. Strain capacity vs strain demand at bottom RBS and weld for the shallow column at 0- and 30-degree skew at 4% drift.

For the shallow column models and for some other that are presented later, the plotted damage ratio at 50% axial load is not representative of real damage values because the columns experienced local buckling (instability mechanism) at early stages and the RBS connection could not reach a plastic hinge. However, all damage ratios are plotted for completeness.

The damage ratio at the shallow column (W14×193) bottom flange for all skew levels is depicted in Figure 48. Within the RBS, it is clear that damage increases with increments of the skew angle and decreases when increasing the column axial load. Comparing 0- and 30-degree skew, the RBS damage ratio difference is more than double between them. Different from the RBS section, at the bottom flange weld, the damage ratio is almost constant for 0, 10 and 25 $\phi_p P_n$ (column axial capacity). However, with skew of 30 degrees, the damage ratio increases more than 5% at the beam-to-column welding for all the axial loads. This graph shows that the RBS function is met because more damage in the connection weld was diminished compared to the RBS damage.
The shallow column damage ratio within the RBS and the connection weld at the top flange is smaller in comparison with damage at the bottom flange. Figure 49 depicts the damage ratio for the shallow columns at the top flange. Although those damage ratio values are smaller than those for the bottom flange, they follow the same trend as the bottom flange damage ratios. This means that the RBS damage growth when increasing the skew and diminish when increasing column axial load.

The medium column (W18×143) bottom flange presents more damage within the RBS section for the less severe skew angles (0 and 10 degrees), but for 20- and 30-degree skew, the level of damage is similar when comparing it with the shallow column bottom flange damage ratio. The level of damage at the bottom flange weld replicates the damage ratio for the shallow columns at 0-, 10- and 20-degree skew. Similar to the bottom flange weld damage ratio for the

Figure 48. Damage ratio for the shallow column- beam bottom flange at 4% drift.

Figure 49. Damage ratio for the shallow column-beam top flange at 4% drift.
shallow column, an out-of-plane skew of 30 degrees causes more damage at the connection weld for medium columns, which is clearly depicted in Figure 50.

![Figure 50. Damage ratio for the W18x143 column-beam bottom flange at 4% drift.](image)

Beam top flange damage ratio for the column W18×143 is presented in Figure 51. The medium column top flange weld damage ratio replicates the same level of damage than the top beam flange shallow column (Damage ratio is constant for 0, 10 and 20 degrees and less than 10% for all skew angles). However, the beam top flange RBS damage ratio changes for 0- and 30-degree skew. When comparing damage ratio within RBS for the shallow column and medium column, the damage ratio for the medium orthogonal connection (top flange) is larger than the damage ratio for the shallow orthogonal connection (top flange), but it is smaller for the medium column connection with skew of 30 degrees. The damage ratio change for 10 and 20 degrees is very small when comparing both connection sizes.

![Figure 51. Damage ratio for the W18×143 column-beam top flange at 4% drift.](image)
The connection fatigue analysis for the column W24×131 depicts more damage ratio within the RBS section than all preceding connections. Figure 52 shows more than 50% damage ratio for skews of 20 and 30 degrees for less than 50% axial loads. This large amount of damage is predicted at 4% drift because fracture is predicted at the following rotation increment (5% drift) as presented in Figure 53. The damage ratio is also larger at the connection welding for the skews of 20 and 30 degrees when comparing it with previous column sizes. It is important to note that Figure 52 shows the first case where damage ratio is not affected by column local buckling. The column W24×131 with skew of 30 degrees and 50% axial load did not buckle during the test.

Figure 52. Damage ratio for the W24×131 column-beam bottom flange at 4% drift.

Figure 53. Strain capacity vs strain demand at RBS and weld for W24×131 bottom flange at 4% drift.
Figure 54 shows damage ratios at the beam top flange for the column W24×131. The level of damage at the RBS top flange is smaller than at the bottom flange, but the overall level of damage is higher for this connection than for the other column sizes. At the weld, the level of damage follows the same pattern (constant values and less than 10% damage ratio) as previous models. As in all previous models, RBS damage ratio increases when increasing the skew angle and reduces when increasing column axial loads.

Figure 54. Damage ratio for the W24×131 column-beam top flange at 4% drift.

The level of damage at the connection for the deep column (W33×291) is different from the other column sizes. Fracture is predicted at the RBS bottom flange at all axial load levels (for orthogonal connections) as depicted in Figure 55. Similar to the results reported by Prinz and Richards [5], fracture is predicted at or before 4% drift when there is no skew for the beam W36×150. However, the level of damage generally decreases when having an out-plane skew for this column connection. Figure 56 confirms the statement presented in Figure 55 where damage ratio is 100% for the orthogonal connection, but it decreases when having a skewed connection mostly because of excessive deformation within the RBS.
Figure 55. Strain capacity and strain demand at RBS and weld for orthogonal deep column bottom flange at 4% drift.

The damage ratio at the bottom flange weld increases for all the skew level in this connection. The strain demand for the skew of 30 degrees is considerably high (more than 60%), and therefore this is the highest damage ratio (for connection welds) among all the connection analysis performed in this investigation. This capacity reduction at the weld for the case of 50% axial load with skew of 30 degrees is depicted in Figure 57. From this figure, it is clear that fatigue capacity at connection weld has been reduced considerably. Normally, RBS damage ratio for orthogonal connections is the smallest among all skewed connections, but for deep column connections, orthogonal RBS damage ratio is the highest.

Figure 56. Damage ratio for the deep column-beam bottom flange at 4% drift.
Figure 57. Strain capacity vs strain demand at RBS and weld for a deep skewed connection (bottom flange) at 4% drift.

At the top flange there is no fracture indicated for all the skew angles and axial load levels as shown in Figure 58. Although the damage ratio is smaller at the top flange, it follows a similar trend as the RBS bottom flange (the orthogonal connection has a higher degraded fatigue capacity). The damage ratio at the top flange connection weld is constant for all the axial load levels. Similar to all top flange welds for the other column sizes, the 30-degree skew connection presents more damage at the welding than for the other skew levels.

Figure 58. Damage ratio for the deep column-beam top flange at 4% drift.

Ultra-low cycle fatigue analysis showed that damage ratio within RBS section for shallow (W14×) and medium (W18× and W24×) columns was amplified when increasing axial load at all skew levels (see Figure 48 through Figure 51). Increasing the column axial load
during the cyclic loading enlarges the column deformed shape as presented in Figure 59. Therefore, stresses at the connection due to the beam tip movement (up and down) are smaller for the more deformed shape of the column (high column axial load) than for less deformed shape of the column (small column axial load). For deep column connections, the level of damage ratio behaves differently. The orthogonal connections showed more damage than skewed connections because of less severe local buckling due to higher column twist. Similar findings were presented by Prinz and Richards [5] for deep column RBS skewed connection damage analysis.

![Deformed Shape Diagram](image)

**Figure 59. Model deformed shape.**

### 3.4.5 Bare Frame Panel Zone Performance

#### 3.4.5.1 Global Response

Appropriate performance of beam-to-column connections is vital in order to have adequate seismic response on moment resisting frames [38]. A comparison of panel zone plastic rotations between RBS and WUF-W moment connections is presented to evaluate the effects of different skew angles and axial loads on the connection. As stated before, the moment for
orthogonal connections is larger than for skewed connections. Also, the column depth \((d_c)\) is a fundamental parameter when calculating the panel zone strength. Considering these two aspects, it is expected that shallow orthogonal connections show more panel zone plastic rotation.

Figure 60 shows a comparison of panel zone plastic rotations for RBS and WUF-W shallow column connections. It clear that the higher moment coming from the WUF-W connection is exerting more plastic rotation than the moment for the RBS connection. The effect of axial loads is also visible in the figure. Increasing the axial load leads to more panel zone plastic rotation, and it could be explained from the point of view of second order effects depicted in Figure 59. In other words, the deformed shape of the column allows more plastic rotation at the panel zone connection. This effect of axial load on plastic rotation capacity was also observed by Ozkula et al. [8]. At 50% axial load, normally the shallow columns undergo column local buckling, which leads to a different panel zone plastic rotation behavior.

Figure 60. Panel zone plastic rotation for shallow RBS and WUF-W orthogonal connections.
Although WUF-W moment connections present larger plastic rotation at the panel zone than RBS moment connections, increment of skew angle will reduce those plastic rotations as presented in Figure 61. The axial load effect on panel zone plastic rotation continues the same trend (increasing axial load ≈ increasing plastic rotation). Panel zone plastic rotations at 50% axial load generally are not symmetric because the column presented local buckling.

Figure 61. Panel zone plastic rotation for shallow RBS and WUF-W skewed connections (10 degrees).

More panel zone plastic rotation reduction is evidenced when the skew angle is 20 degrees. Once again, panel zone plastic rotation is larger at 50% axial load than for the axial load levels. The same behavior was observed for skews of 30 degrees. Figure 62 shows panel zone plastic rotation comparison at 20-degree skew.
Panel zone plastic rotation for medium column (W24×) connections are considerably reduced compared with plastic rotation for shallow columns (W14). The formula to calculate the panel zone shear capacity, established in AISC 360-16 [62], is used to explained differences between panel zone plastic rotations for different column sizes. The panel capacity depends on column cross section geometry properties and the steel yield stress. The column depth is proportional to the panel zone capacity. In consequence, more column depth means more panel zone strength. This is why the plastic rotations for medium columns are smaller than for shallow columns even though the same beam size is connected to these two column sizes and the same yield strength is used. Figure 63 shows a comparison plot for medium columns panel zone plastic rotations without skew. Although the plastic rotations are smaller, the axial load tends to increase the panel zone plastic rotations as for the shallow column.
When having a skew angle, panel zone plastic rotation will continue to be reduced because of the combine effects of lesser applied moment at the connection, the skew angle and the higher column depth. The column axial load tends to increase rotation as in previous cases. Figure 64 presents a comparison of panel zone plastic rotations for the medium columns at 10-degree skew. The graphs for the other skew angles are not presented because they follow the same trend as for the case of 10 degrees, and the plastic rotation values are significant smaller for 20- and 30-degrees skew.
Panel zone plastic rotations for deep columns with orthogonal connections are smaller than for the shallow column but bigger than for the medium column. Although the W33× is the deepest column in this study (more panel zone capacity), the beam W36×150 is also applying a moment almost three times bigger than the moment for the medium and shallow columns. The other variables such as the skew angle and the column axial load kept the trend effects; column axial load increases plastic rotation, but the out-of-plane skew reduces the panel zone rotation.

Figure 65 shows a comparison of panel zone plastic rotations for orthogonal RBS and WUF-W connections for deep columns.

Deep columns with skew of 10 degrees present less plastic rotation than orthogonal connections (as expected), except for 50% axial load. As it was depicted in Figure 43, an excessive column buckling caused an elevated level of panel zone plastic rotation.
Figure 65. Panel zone plastic rotation for deep RBS and WUF-W orthogonal connections.

Figure 66. Panel zone plastic rotation for deep RBS and WUF-W skewed connections (10-degree skew).
3.4.5.2 Local Response

Panel zone shear strain for RBS and WUF-W moment connections is compared in this section. Shear strain behaves similar to panel zone plastic rotations. Figure 67 shows shear strain plots for orthogonal shallow columns. It can be noticed in this figure that shear strain increases when increasing column axial loads. Because the moment applied at the connection by WUF-W connections is higher, WUF-W shear strain is always larger than RBS shear strain. Column local buckling for WUF-W connection at 50% axial load leads to an uneven graph.

![Shear strain plots for orthogonal shallow columns](image)

Figure 67. Shear strain for orthogonal shallow columns (W14x).

When the connection presents skew, panel zone shear strain is reduced. Figure 68 shows shear strain plots for shallow columns skewed at 10 degrees. As stated before, WUF-W shear strain is larger than RBS shear strain.
Figure 68. Shear strain for shallow columns skewed at 10 degrees.

If the skew is equal to 20 degrees, more shear strain reduction is obtained as presented in Figure 69. Second order effects lead to more shear strain for larger column axial loads.

Figure 69. Shear strain for shallow columns skewed at 20 degrees.
Shear strain at panel zone is reduced more for 30-degree skewed connections, but it follows the same trend as the other skewed connections. Similar to the other skew levels, the axial load increases the shear strain.

Figure 70. Shear strain for shallow column skewed at 30 degrees.

Shear strain for medium (W24×) column is significantly smaller compared with shallow column shear strain. Shear strain for orthogonal connections is always smaller than for skewed connections. Figure 71 shows shear strain plots for the orthogonal W24× column connections. Because the medium column skewed connections have less shear strain they will be presented in Appendix J.
Deep columns (W33) present more shear strain than medium columns (W24), but this amount of shear strain is considerably smaller than shear strain for shallow columns (W14×).

Figure 72 shows shear strain plots for orthogonal deep columns. As it happened before, WUF-W shear strain is much larger than RBS shear strain for deep column connections. The plots for deep skewed connections are presented in Appendix J.
3.5 Summary and Conclusions

In this chapter, a parametric finite element study was used to investigate the effects of out-of-plane skew and different levels of axial load on RBS and WUF-W SMF connections. A total of 112 detailed finite element models were analyzed using the commercial software ABAQUS and considering 64 RBS finite element models (4 column sizes, 4 skew levels and 4 axial load levels), 48 WUF-W finite element models (3 column sizes, 4 skew levels and 4 axial load levels). These finite elements models were created from four-node linear shell elements (S4R in ABAQUS) for the frame. All the 3-story frames were designed in accordance with AISC 358-16 [6] and loaded using the loading protocol established in Chapter K in AISC 341-16 [13].

RBS and WUF-W connection capacity is negatively affected by the skew. When increasing the skew, connection capacity is reduced because part of the moment should be supported by the column weak axis as presented in Figure 22. On the contrary, column twisting...
is enlarged when the skew angle is augmented, especially for those connections with large column depth and small polar moment of inertia. Generally, for orthogonal and 10-degree skew RBS connections, columns are more twisted than for WUF-W connections. In contrast, columns for WUF-W moment connection with skew of 20 and 30 degrees are more twisted than those for RBS connections. Beam rotation at $0.8M_p$ tends to increase when increasing column axial, although sometimes this trend is broken because the column is affected by local buckling when having high axial load.

Column flange-tip (Figure 28) developed more yielding when increasing the out-of-plane skew. Lower beam-flange to column-flange interface for RBS shallow columns is not affected by skew increments, but RBS medium and deep column connections and all WUF-W connections got more yielding when the skew angle was increased.

Fatigue analysis for shallow and medium RBS column connections showed that damage ratio within the RBS decreases when increasing the column axial load, while the damage ratio is enlarged when the skew is amplified. At shallow and medium column welds, damage ratio is not affected considerably by increments of axial load, but skew increments generally lead to more damage at the welds. As it was stated before, second order effects are the dominant parameters leading to a reduced damage ratio for column with high axial load. Damage ratio behaves differently for deep RBS column connections. Orthogonal RBS connection presented more damage than skewed connections. Different from shallow and medium column RBS connection, high column twisting delays the material failure leading to a less intense local buckling within the RBS. At the deep column welds, the severe skew angle also caused more damage as for shallow and medium column connections.
Skewed connections for deep columns have less plastic rotation than medium and shallow columns because the applied moment over the panel zone is smaller and the panel zone capacity is larger. Normally panel zone plastic rotation is higher for shallow column with orthogonal connection. WUF-W moment connections usually have more plastic rotations at the panel zone than RBS moment connections.

Panel zone shear strain is also larger for shallow column with orthogonal connection than the other connections. Second order effects affect the amount of shear strain causing more shear strain for higher column axial loads.
CHAPTER 4. COMPONENT-LEVEL INVESTIGATION INTO COMPOSITE (STEEL-CONCRETE) SKewed SMF CONNECTION RESPONSE

Special Moment Frames (SMF) are designed to dissipate seismic loads by placing RBS cuts close to the column face. Because steel moment frames are typically connected to concrete slabs, the beam-to-slab composite action during an earthquake could affect the behavior (connection capacity, column twisting and yielding, panel zone plastic rotation) of the moment frame system. The composite action could also affect the level of flange or welding damage (specially at the bottom flange) due to the shift of beam neutral axis.

The force transfer at the interface between steel beam and concrete slab is vital for the steel-concrete composite section performance during dynamic loading [63, 64]. In these systems, the composite section carrying capacity is improved due to the shear resistance at the composite join due to stud connector dowel action [65]. Figure 73 depicts the three-level composite section model used to study the steel-concrete skewed connection demands.

Although boundary conditions for SMF under composite action are similar to those used for SMF without slabs, AISC Prequalified Connections [6] established that supplemental top and bottom flange bracing near the RBS is not required when having a structural slab connected to the beam.

Similar to the design for bare frames, steel-concrete skewed connections were designed under the strong column-weak beam principle, and section compactness was also verified. The design of RBS connections follows the specifications established in AISC 358-16 [6].

The main objectives of this composite section analysis are 1) evaluation of SMF RBS skewed connection capacity according to AISC 341-16 [13], 2) investigation of connection fatigue capacity and comparison with bare frame models, 3) performance of panel zone (global and local response).
4.1 Background

A previous publication [7, 66] about composite action on orthogonal connections indicates an increment on connection strength and a delay of strength degradation. Findings presented by Jones et al. (2002) [4] also pointed out the enhancement of connection strength provided by the concrete slab composite action. The effect (strength increment and RBS damage reduction) of concrete slab on the connection was higher for shallow sections (W14×398) compared to deeper sections (W27×194), which make sense because the slab thickness does not change. However, it is important to state that they [7, 66] were bracing the beam near the RBS section, but current AISC Prequalified Connections [6] establishes that supplemental top and
bottom flange bracing at the RBS section is not required when the beam supports a structural slab. For this study, the increment of maximum strength provided by the concrete floor is not more than 4%. Deeper columns experienced more twist, but the slab contributed significantly to reduce the column twist. A damage investigation was also performed in this research [7, 66]. It was found that the slab presence increases the level of damage especially at the beam web-to-column flange Complete Join Penetration (CJP) weld.

Column torsional behavior is also affected by the concrete floor. In a report published by Ricles et al. (2004) [44], it was found that the slab composite action provides a considerable column twist reduction compared to column twist for bare frame columns. According to this publication, column twist seems not to be functionally related to the polar moment of inertia $J$, nor the ration $h/t_c r^4$, but to the column elastic torsional stiffness.

The strain increment at the bottom flange was also noticed in other publications [67]. The presence of the concrete floor increases the stiffness and strength close to the beam top flange, which results in higher strain at the bottom flange. According to Bruneau et al. [38], during the Northridge earthquake, the composite action developed by the concrete slab presence might be responsible for the elevated number of beam bottom flange fracture when comparing that with the number of beam top flange fractures. They pointed out the effect of the neutral axis shift up due to the slab presence, which translates into larger axial deformation demands on the beam bottom flange.

This chapter expands the knowledge of previous chapters through a parametric analytical investigation of steel-concrete skewed connections demands. In this parametric study, the effects of varied skew angles, concrete strength, different beams and column sizes under the AISC
loading protocol are considered. Additionally, a fracture investigation is performed to study the concrete slab effects on the connection.

4.2 RBS Composite Section Parametric Study

This additional investigation about composite slab effects on the skewed SMF RBS performance will be conducted from the most interesting bare-steel assemblies (W14×, W24× and W33×). Twelve detailed composite section FE models are developed for studying the composite section contribution to skewed SMF connections. The geometries for the RBS composite models consider deep (W33×291), medium (W24×131) and shallow (W14×193) column configurations, with three levels of beam skew (10, 20 and 30 degrees). The scope of this research does not include column axial loads because it was noticed in the previous chapter that axial loads are mostly affecting column local buckling/panel zone plastic rotations and not RBS or welds demands. In addition, in chapter 3 it was found that higher connection damage ratio happens for 0% axial load in the majority of cases. Table 24 presents a summary of model properties for the study.

Table 24. Beam and column dimensions for the composite section analytical study

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Model ID</th>
<th>Column Section</th>
<th>Beam Section</th>
<th>Beam Skew Angle [deg]</th>
<th>Column Axial Load (% Capacity)</th>
<th>Concrete (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2</td>
<td>T2RC-14 × 10</td>
<td>W 14×193</td>
<td>W 24×76</td>
<td>10</td>
<td>0</td>
<td>4000, 6000</td>
</tr>
<tr>
<td>3/4</td>
<td>T2RC-14 × 20</td>
<td>W 14×193</td>
<td>W 24×76</td>
<td>20</td>
<td>0</td>
<td>4000, 6000</td>
</tr>
<tr>
<td>5/6</td>
<td>T2RC-14 × 30</td>
<td>W 14×193</td>
<td>W 24×76</td>
<td>30</td>
<td>0</td>
<td>4000, 6000</td>
</tr>
<tr>
<td>7</td>
<td>T2RC-24 × 10</td>
<td>W 24×131</td>
<td>W 24×76</td>
<td>10</td>
<td>0</td>
<td>4000</td>
</tr>
<tr>
<td>8</td>
<td>T2RC-24 × 20</td>
<td>W 24×131</td>
<td>W 24×76</td>
<td>20</td>
<td>0</td>
<td>4000</td>
</tr>
<tr>
<td>9</td>
<td>T2RC-24 × 30</td>
<td>W 24×131</td>
<td>W 24×76</td>
<td>30</td>
<td>0</td>
<td>4000</td>
</tr>
<tr>
<td>10</td>
<td>T2RC-33 × 10</td>
<td>W 33×291</td>
<td>W 36×150</td>
<td>10</td>
<td>0</td>
<td>4000</td>
</tr>
<tr>
<td>11</td>
<td>T2RC-33 × 20</td>
<td>W 33×291</td>
<td>W 36×150</td>
<td>20</td>
<td>0</td>
<td>4000</td>
</tr>
<tr>
<td>12</td>
<td>T2RC-33 × 30</td>
<td>W 33×291</td>
<td>W 36×150</td>
<td>30</td>
<td>0</td>
<td>4000</td>
</tr>
</tbody>
</table>

All frames consisted of a two-sided connection carrying a concrete slab with 96 inches effective width. The slab effective width was calculated using the formulation establishes in
American Concrete Institute (ACI 318-14) [68]. Figure 74 shows the elements to be considered for ACI effective width calculation. As it is depicted in the figure, \( h \) is the slab thickness plus the beam depth, \( Sw \) is the tributary width of the beam and \( Ln \) is the beam length between supports. The value of \( b_{\text{eff}} \) is the smallest of the following conditions: \( 8h, Sw/2, Ln/8 \). The effective width is twice \( b_{\text{eff}} \) plus the width of the beam flange (\( 2b_{\text{eff}} + bw \)).

![Figure 74. Sketch for slab effective width calculation.](image)

Boundary conditions for the composite models are similar to those used for bare-steel frames. However, according to AISC 358-16 [6], supplemental bracing close to the end of the RBS is not required if the beam supports a concrete structural slab and the slab is connected to the beam with shear studs.

Figure 75a and b show a case of the bare-steel assembly with stud spacing following the experimental work performed by Jones et al. [4]. In Figure 75c, all the constitutive elements of the composite slab section are presented; slab thickness is 5.5 in., and consists of 4000 psi concrete strength with steel reinforcement of bars #4 (Grade 60) spaced at 12 in. both directions. Different from Jones et al. [4], a 6000 psi concrete slab is considered to study its effects on the connection. A strong concrete floor could affect the demand at the beam bottom flange (strain), the connection strength, the panel zone plastic rotation and the column twist.

The studs are separated 12 in. (typically) from one another due to metal deck sheet specifications and rib dimensions. The studs are placed along the beam except at the RBS.
segment to avoid possible negative effects over the beam due to the heat generated during welding (see Figure 75 and Figure 76). Shear studs have a diameter of $\frac{3}{4}$ in., length of 3.5 in., Modulus of Elasticity of 29000 ksi and Poisson’s ratio of 0.3.

![Figure 75. SMF bare steel subassembly (a and b) and composite subassembly (c).](image)

![Figure 76. Plan view of skewed SMF connection and concrete deck.](image)

The reinforced concrete (RC) slab is 5.5 in. thick, and the profile deck (rib height) is 2 in. (see Figure 75). The assigned thickness to the RC slab solid elements is equivalent to one-half of profile metal deck plus the concrete thickness over the rib ridge. Therefore, an equivalent
thickness of 4.5 in. (one-half of rib height =1.0 in., and concrete thickness over the rib =3.5 in.) is used as the RC slab composite depth for ABAQUS modeling purposes. The density of normal weight reinforced concrete is approximately 150 lb/ft³ and the Poisson’s ratio is 0.2 [69].

4.3 Modeling Techniques and Material Properties

A versatile representation of the actual composite section geometry can be done by three-dimensional Finite Elements (FE) models in ABAQUS. Combination of solid, shell and linear beam (1-D) elements allow accurate modeling of the composite section behavior and capacity [65]. As for the bare steel frame used in the previous chapter, column and beams are modeled with four-node linear shell elements with reduced integration (S4R in ABAQUS). Mesh size for column and beams has the same refinement used for the steel bare frame models. Different from the bare frame, which was modeled using shell elements, concrete slab is modeled using solid elements (C3D8R in ABAQUS), which is the most efficient brick element in terms of computation and storage. Solid elements allow modeling the shear studs as beam elements embedded in the concrete, which is an easy task compared with modeling studs as longitudinal and transverse spring elements (a necessary approach when using shell elements to model the concrete slab). The concrete floor slab is modeled as an elastic-plastic material only for compression stress, with general mesh size of 5 in. as in Prinz and Castro-e-Sousa [29] (see Figure 78). Stud connectors are modeled as 1-dimensional Timoshenko beam elements (B31 in ABAQUS) with mesh size of 2 in.

For this part of the research, composite slab modeling techniques is verified using experimental test data from the cyclic composite beam test done by Jones et al. [4]. The ABAQUS modeling of the concrete slab will consider a Young’s modulus of 3605 ksi, which match a concrete strength of f’c=4000 psi.
To model the composite section, cracking models from ABAQUS are used to define concrete properties after yielding has occurred. Concrete elasticity is considered isotropic before yielding has taken place.

Selection of a cracking model for modeling concrete in ABAQUS depends in great manner on the type of loading used in the computer model and the type of concrete elements. Among the cracking models available in ABAQUS, Smeared Cracking Model (SCM), Cast Iron Plasticity (CIP) Model and Concrete Damaged Plasticity (CDP) are commonly used for concrete computer simulations. The SCM is not used here because it is most appropriated for use with shell elements, it does not consider compression damage, and it is more effective for monotonic loading (not for seismic loading) [70, 71]. The CIP Model is able of simulating tensile failure, but it does not consider degradation of the concrete stiffness due to cracks after the concrete has reached the limit of traction ($f_t$) or compression ($f'_c$). Figure 77 shows the representation of Cast Iron model behavior.

![Figure 77. Representation of Cast Iron Model.](image)

However, the CDP accounts for cyclic loading and relative concrete damage for concrete in tension and compression. For a CDP model several initial parameters related to microstructure of the concrete need to defined. The CDP model requires parameters such as dilation angle, eccentricity, biaxial loading ratio, viscosity. Those parameters and their values are presented in Table 25.
Table 25. Initial parameters for the CDP Model [72].

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dilation angle</td>
<td>$\psi = 30$</td>
</tr>
<tr>
<td>Flow Potential Eccentricity</td>
<td>$m = 0.1$</td>
</tr>
<tr>
<td>Initial biaxial/uniaxial ratio</td>
<td>$\sigma_{c0}/\sigma_{b0} = 1.16$</td>
</tr>
<tr>
<td>Ratio of the second stress invariant on tensile meridian</td>
<td>$K_c = 0.666$</td>
</tr>
<tr>
<td>Viscosity parameter</td>
<td>$\mu = 0.01$</td>
</tr>
</tbody>
</table>

For proper concrete plasticity modeling, concrete compressive and tensile relationship, cracking and crushing damage parameters (tension and compression input data) needs to be specified. Jankowiak and Lodygowski [72] established the values of tension and compression stresses and damage parameters for the CDP Model, which are presented in the next tables.

Table 26. Concrete Compressive Behavior

<table>
<thead>
<tr>
<th>Yield Stress (ksi)</th>
<th>Inelastic Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1756</td>
<td>0.000000</td>
</tr>
<tr>
<td>2.9294</td>
<td>0.000075</td>
</tr>
<tr>
<td>4.3512</td>
<td>0.000099</td>
</tr>
<tr>
<td>5.8456</td>
<td>0.000154</td>
</tr>
<tr>
<td>7.2530</td>
<td>0.000762</td>
</tr>
<tr>
<td>5.8358</td>
<td>0.002558</td>
</tr>
<tr>
<td>2.9350</td>
<td>0.005675</td>
</tr>
<tr>
<td>0.7626</td>
<td>0.011733</td>
</tr>
</tbody>
</table>

Table 27. Concrete Tensile Behavior

<table>
<thead>
<tr>
<th>Yield Stress (ksi)</th>
<th>Cracking Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.289921</td>
<td>0.000000</td>
</tr>
<tr>
<td>0.412198</td>
<td>0.000033</td>
</tr>
<tr>
<td>0.271194</td>
<td>0.000460</td>
</tr>
<tr>
<td>0.125128</td>
<td>0.000799</td>
</tr>
<tr>
<td>0.032815</td>
<td>0.004985</td>
</tr>
<tr>
<td>0.008206</td>
<td>0.006087</td>
</tr>
</tbody>
</table>
Table 28. Concrete Compression Damage

<table>
<thead>
<tr>
<th>Damage Parameter</th>
<th>Inelastic Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.000000</td>
<td>0.000000</td>
</tr>
<tr>
<td>0.000000</td>
<td>0.000075</td>
</tr>
<tr>
<td>0.000000</td>
<td>0.000099</td>
</tr>
<tr>
<td>0.000000</td>
<td>0.00154</td>
</tr>
<tr>
<td>0.000000</td>
<td>0.00076</td>
</tr>
<tr>
<td>0.195402</td>
<td>0.00255</td>
</tr>
<tr>
<td>0.596382</td>
<td>0.00567</td>
</tr>
<tr>
<td>0.894865</td>
<td>0.011733</td>
</tr>
</tbody>
</table>

Table 29. Concrete Tension Damage

<table>
<thead>
<tr>
<th>Damage Parameter</th>
<th>Cracking Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.000000</td>
<td>0</td>
</tr>
<tr>
<td>0.000000</td>
<td>0.000033</td>
</tr>
<tr>
<td>0.406411</td>
<td>0.000460</td>
</tr>
<tr>
<td>0.696380</td>
<td>0.000799</td>
</tr>
<tr>
<td>0.920389</td>
<td>0.004985</td>
</tr>
<tr>
<td>0.980093</td>
<td>0.006087</td>
</tr>
</tbody>
</table>

Figure 78. Typical finite element model mesh for the beam-to-concrete slab.
Rebar is modeled as wire elements (truss element in ABAQUS: T3D2), with steel bar diameter of 0.5 in. (similar to Jones et al. [4]). The values for Modulus of Elasticity and Poisson’s ratio are equal to those used in modeling shear studs. The rebar is placed (embedded) in the concrete slab (host element) as two layers of bars perpendicular to each other as is presented in Figure 79. Plastic properties for rebar and studs are presented in Table 30 and Table 31, respectively. Rebar mesh size is 5 in. as for the concrete slab. Shell and beam elements reduce the analysis computational time. A refined mesh is not used for studs, slab, rebar and column/beam shell elements out of the connection region because yielding is not expected there and numerical outputs are not valuable for this study.

![Frame-Slab modeling technique for composite section interaction.](image)

Table 30. Rebar Plastic Strain

<table>
<thead>
<tr>
<th>Yield Stress (ksi)</th>
<th>Inelastic Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>0.00</td>
</tr>
<tr>
<td>60.175</td>
<td>0.008</td>
</tr>
<tr>
<td>70</td>
<td>0.02</td>
</tr>
<tr>
<td>80</td>
<td>0.04</td>
</tr>
<tr>
<td>86</td>
<td>0.06</td>
</tr>
<tr>
<td>90</td>
<td>0.08</td>
</tr>
<tr>
<td>86</td>
<td>0.10</td>
</tr>
<tr>
<td>80</td>
<td>0.13</td>
</tr>
</tbody>
</table>
### Table 31. Stud Plastic Strain

<table>
<thead>
<tr>
<th>Stress (ksi)</th>
<th>Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>60.0</td>
<td>0.00</td>
</tr>
<tr>
<td>60.1</td>
<td>0.01</td>
</tr>
</tbody>
</table>

#### 4.4 Available Experimental data for comparison

Figure 80 shows the experimental setup used by Jones et al. [4] to study the composite effects of steel beams and concrete decks connected with shear studs. In this figure, the locations of beam supports represent the beam mid-span where it is expected to have inflection points (zero moment), allowing horizontal translation, but avoiding out-of-plane displacements. Likewise, the 3-story SMF ABAQUS beam models are extended up to beam mid-spans on both column sides.

From the investigation results presented by Jones et al. [4], story drift capacities and controlling failure modes, panel zone or RBS contribution to energy dissipation, and evidence of negative effects on strain demands due to composite action, are considered useful data or comparison with this research project.
To validate our models, Jones’ experimental test was modeled in ABAQUS. Figure 81 shows an overlap of Jones’ experimental results and ABAQUS results for Jones’ experiment. This plot depicts a very good match between both works until the minimum prequalification requirement (0.04 rad). Because of this good agreement, our composite section models can predict with enough accuracy the slab contribution to the performance of RBS skewed connections.
Next section presents comparisons between bare frame and composite section models to evaluate the slab contribution to connection strength, demands and column flange yielding.

4.5 Results and Discussion

The connection capacity for the skewed composite section is evaluated considering different response parameters such as rotation capacity, column twist, fracture analysis, panel zone plastic rotation and yielding at the column flange. The majority of results reported in this section are comparisons between bare frame response and composite response because the main purpose of this chapter is to study the slab effects on the SMF connections. Table 32 summarizes results of peak moment at the connection, beam rotation at $0.8M_p$ and column twist at 4% drift for all the composite section models. All the rotation values reported at 80% of $M_p$ are negative because the backbone curve does not degrade below $0.8M_p$ for positive moment at any case in this research.
As expected, the strength of the connection was improved for the composite section (for positive moment).

Table 32. RBS composite section analysis matrix and results.

<table>
<thead>
<tr>
<th>Model No.</th>
<th>Model ID</th>
<th>Column</th>
<th>Beam</th>
<th>Skew (deg)</th>
<th>Peak Moment @ Connection (k-ft)</th>
<th>Rotation @ 0.8 Mp (rad)</th>
<th>Column Twist @ 0.04 rad drift (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&lt;sup&gt;a,c&lt;/sup&gt;</td>
<td>W14×193_10_0%</td>
<td>W14×193</td>
<td>W24×76</td>
<td>10</td>
<td>900.6562</td>
<td>-0.0461</td>
<td>0.1486</td>
</tr>
<tr>
<td>2&lt;sup&gt;a,c&lt;/sup&gt;</td>
<td>W14×193_20_0%</td>
<td>W14×193</td>
<td>W24×76</td>
<td>20</td>
<td>878.2479</td>
<td>-0.0471</td>
<td>0.3113</td>
</tr>
<tr>
<td>3&lt;sup&gt;a,c&lt;/sup&gt;</td>
<td>W14×193_30_0%</td>
<td>W14×193</td>
<td>W24×76</td>
<td>30</td>
<td>877.6555</td>
<td>-0.0468</td>
<td>0.4511</td>
</tr>
<tr>
<td>4&lt;sup&gt;a,c&lt;/sup&gt;</td>
<td>W24×131_10_0%</td>
<td>W24×131</td>
<td>W24×76</td>
<td>10</td>
<td>923.4776</td>
<td>-0.0422</td>
<td>0.2163</td>
</tr>
<tr>
<td>5&lt;sup&gt;a,c&lt;/sup&gt;</td>
<td>W24×131_20_0%</td>
<td>W24×131</td>
<td>W24×76</td>
<td>20</td>
<td>892.1784</td>
<td>-0.0446</td>
<td>0.5026</td>
</tr>
<tr>
<td>6&lt;sup&gt;a,c&lt;/sup&gt;</td>
<td>W24×131_30_0%</td>
<td>W24×131</td>
<td>W24×76</td>
<td>30</td>
<td>888.9638</td>
<td>-0.0493</td>
<td>1.3253</td>
</tr>
<tr>
<td>7&lt;sup&gt;a,c&lt;/sup&gt;</td>
<td>W33×291_10_0%</td>
<td>W33×291</td>
<td>W36×150</td>
<td>10</td>
<td>2849.9305</td>
<td>-0.0377</td>
<td>0.1885</td>
</tr>
<tr>
<td>8&lt;sup&gt;a,c&lt;/sup&gt;</td>
<td>W33×291_20_0%</td>
<td>W33×291</td>
<td>W36×150</td>
<td>20</td>
<td>2794.8837</td>
<td>-0.0406</td>
<td>0.4352</td>
</tr>
<tr>
<td>9&lt;sup&gt;a,c&lt;/sup&gt;</td>
<td>W33×291_30_0%</td>
<td>W33×291</td>
<td>W36×150</td>
<td>30</td>
<td>2790.1568</td>
<td>-0.0444</td>
<td>0.8953</td>
</tr>
<tr>
<td>10&lt;sup&gt;b,c&lt;/sup&gt;</td>
<td>W14×193_10_0%</td>
<td>W14×193</td>
<td>W24×76</td>
<td>10</td>
<td>901.9665</td>
<td>-0.0462</td>
<td>0.1475</td>
</tr>
<tr>
<td>11&lt;sup&gt;b,c&lt;/sup&gt;</td>
<td>W14×193_20_0%</td>
<td>W14×193</td>
<td>W24×76</td>
<td>20</td>
<td>881.2533</td>
<td>-0.0472</td>
<td>0.3121</td>
</tr>
<tr>
<td>12&lt;sup&gt;b,c&lt;/sup&gt;</td>
<td>W14×193_30_0%</td>
<td>W14×193</td>
<td>W24×76</td>
<td>30</td>
<td>879.3535</td>
<td>-0.0474</td>
<td>0.4519</td>
</tr>
</tbody>
</table>

<sup>a</sup> Concrete strength=4000 psi  
<sup>b</sup> Concrete strength=6000 psi  
<sup>c</sup> Axial Load = 0% of \( \phi_c P_n \)

4.5.1 Composite Section Connection Capacity Analysis

Several connection capacity comparison plots are presented to depict the effects of concrete slab on the connection. Figure 82 is created to illustrated the composite action on the connection for the shallow column (W14×193) and the beam W24×76 with different skew angles. Differences between the bare and composite frame results are clearly noticeable, especially for positive moment, where the composite action is improving the connection capacity as presented in Figure 82. All skewed composite connections (skew of 10, 20 and 30 degrees) for shallow columns meet the minimum requirements for prequalification from AISC seismic provisions [13] because moment capacity did not degrade below \( 0.8 M_p \) at 4% story-drift. Due to the slab contribution, the composite section connection capacity does not fall under \( 0.8 M_p \) even beyond the minimum requirement (4% drift). However, the concrete slab does not contribute to connection capacity for negative moments (concrete slab under tension).
Due to composite action, the lower flange is more susceptible to local and lateral torsional buckling because the neutral axis moves up due to the slab presence. This flange is also not braced by the slab or any other beam cross section bracing system, so the capacity in negative moment is not improved by the composite action. More about this effect can be seen in the fatigue analysis section.

![Diagram](image.png)

Figure 82. Shallow column backbone comparison for bare frame and composite section at 10- and 30-degree skews.

For the medium column (W24×131), the composite section performance is very close to the shallow column results. The slab improved the connection capacity for positive moment, but no significant effects for negative moment are noticed. Figure 83 shows a comparison between bare frame and composite section skewed connections with 10- and 30-degree skew for medium columns. Both skewed composite connections meet AISC minimum prequalification requirements.
For deep columns (W33×291), the concrete slab improved the connection capacity (for positive moment) similar to the other column sizes (shallow and medium). Normally, for negative moment, there is not moment capacity increment for 10-degree skew, but for the medium and deep column with 30-degree skew, there is a slight increment of connection capacity at 0.8Mₚ as presented in Figure 84. Important to point out that composite section reduces the excessive RBS deformation that the bare frame with 10 degrees skew reported. This can be seen in Figure 84 for the skew of 10 degrees where the number of loops for the bare frame is substantially smaller.
Table 32 shows several results for the connection composite section when increasing concrete strength from 4000 psi to 6000 psi. A slab with stronger concrete barely changes the peak moment capacity, the rotation at $0.8M_p$ and the twist at 4% drift. This is also visible in Figure 85 where a comparison of backbone curves for shallow columns with the two types of concrete is presented. This figure shows that adding 2000 psi to the initial concrete strength (4000 psi) does change the general connection performance.

![Figure 85. Backbone comparison for shallow columns with 10- and 30-degree skews for 4000 psi and 6000 psi.](image)

### 4.5.2 Composite Section Column Twist Analysis

The column twisting conditions are affected by the slab composite action. The concrete slab in compression provides a higher moment at the connection which increases the twist. Increasing the skew angle causes an enlargement of the column twisting as well. Figure 86 and Figure 87 show the twist path for the shallow column at different skew levels. For a skewed angle of 10 and 20 degrees, the maximum column twist (0.003006 rad and 0.005838 rad, respectively) for the composite section is smaller than the maximum bare frame twist (0.006279 rad and 0.006612 rad). However, skew of 30 degrees causes more twist for the composite section (0.008481 rad) than for the bare frame (0.007075 rad). It can be stated that the bracing provides by the slab on the beam for positive moment can reduce the column twist for 10 and 20 degrees...
because the slab helps to control the beam LTB. In the contrary, the skew of 30 degrees is so severe that the slab contribution for beam bracing is not enough to keep the twist equal or smaller than for the bare frame.

Figure 86. Shallow column twist paths at 10-degree skew.

Figure 87. Shallow column twist paths at 20- and 30-degree skews.

The column twist directly depends on the column polar moment of inertia. Table 21 presented the polar moment of inertia for the columns used for RBS composite sections. The polar moment of inertia for the medium column (W24×131) is the lowest, then it is expected to have more column twist for those models. The twist grows when increasing the skew angle, showing path similarities to the shallow column twist, but having larger twist values for the
medium column. Figure 88 shows the twist path for the medium column with 10- and 20-degree skew.

Figure 88. Medium column twist paths at 10- and 20-degree skews.

The skew of 30 degrees is the worst case for column twist in medium column. In this case, the composite section column twist (0.02550 rad) is more than double the bare frame column twist (0.012034 rad). Figure 89 illustrates a comparison between the twist paths for the bare frame and composite section for the medium column.

Figure 89. Medium column twist paths at 30-degree skew.

Although the moment applied by the beam W36×150 and the slab is larger than the applied moment for the medium and shallow column, the deep column experiences less column twist that the medium column because of the polar moment of inertia. The deep column polar
moment of inertia is the largest of all the column of this research. Therefore, it is providing some control over the column twist even though parameters such as the column depth and the applied moment tend to increase the column twist. Figure 90 shows a comparison between the twist paths for the bare frame and composite section for the deep column.

![Figure 90. Deep column twist paths at 10- and 20-degree skews.](image)

The twist paths at 30-degree skew for the bare and composite bare frame is presented in Figure 91. The difference between the column twist for composite section and bare frame is considerable for the skew of 30 degrees in the deep column (similar to the medium column performance), but those values are smaller than for the medium column due to polar moment of inertia magnitude.

![Figure 91. Deep column twist paths at 30-degree skew.](image)
In an effort to predict twist for SMF skewed connections, a design formula proposed in the ASIC Design Guide 9 (Torsional Analysis of Structured Steel Members) is modified for twisting prediction in skewed RBS connections. The added term to the ASIC Design Guide 9 is highlighted in the black dashed box in the following formula. According to the boundary conditions established in Figure 17, Case 6 from Design Guide 9 is used for this formulation purpose. Because the Connection of Interest (COI) is at the middle of the frame to meet proper torsional boundary conditions such as in a real structure, the ends of the column considered for formulation are fixed. The formula accounts for the concrete slab contribution and the beam itself to calculate the applied moment on the connection. The formula is presented below.

\[
\theta_{twist} = \frac{M \cdot a}{(H+1) \cdot G \cdot J} \left( H - \frac{1}{\sinh \left( \frac{L}{a} \right)} + \sinh \left( \frac{\alpha \cdot L}{a} \right) - \frac{\cosh \left( \frac{\alpha \cdot L}{a} \right)}{\tanh \left( \frac{L}{a} \right)} \right) + \sinh \left( \frac{\alpha \cdot L}{a} \right) - 1.0 \left( \frac{L}{a} \right) - \sinh \left( \frac{\alpha \cdot L}{a} \right) \right)
\]

where

\[
H := \frac{1 - \cosh \left( \frac{\alpha \cdot L}{a} \right) + \left( \cosh \left( \frac{\alpha \cdot L}{a} \right) - 1 \right) + \sinh \left( \frac{\alpha \cdot L}{a} \right) - \alpha \cdot L}{\tanh \left( \frac{L}{a} \right)} - \frac{\cosh \left( \frac{L}{a} \right) \cdot \cosh \left( \frac{\alpha \cdot L}{a} \right) - \cosh \left( \frac{\alpha \cdot L}{a} \right) - 1.0}{\sinh \left( \frac{L}{a} \right)} + \frac{L}{a} \cdot (\alpha - 1.0) - \sinh \left( \frac{\alpha \cdot L}{a} \right)
\]

\( M \) = torsional moment applied by the beam and slab
\( G \) = shear modulus of elasticity of steel, 11600 ksi
\( L \) = column length
\( a = (E \cdot C_w / G \cdot J)^{1/2} \)
\( E \) = modulus of elasticity of steel, 29000 ksi
\( C_w \) = warping constant for the cross section
\( J \) = polar moment of inertia
\( \alpha = (L - d_b)/L \)
\( z = L - d_b \)
\( d_c \) = column depth
\( d_b \) = beam depth
\( \theta \) = skew angle
\( \theta_{twist} \) = column angle of rotation, rad
A comparison between the predicted twist values from this formulation and twist results extracted from ABAQUS is presented in Table 33.

<table>
<thead>
<tr>
<th>Column</th>
<th>Beam</th>
<th>Skew (deg)</th>
<th>Formula (rad)</th>
<th>ABAQUS (rad)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W14×193</td>
<td>W24×76</td>
<td>10</td>
<td>0.002813</td>
<td>0.003006</td>
<td>6.43</td>
</tr>
<tr>
<td>W14×193</td>
<td>W24×76</td>
<td>20</td>
<td>0.005651</td>
<td>0.005838</td>
<td>3.21</td>
</tr>
<tr>
<td>W14×193</td>
<td>W24×76</td>
<td>30</td>
<td>0.008877</td>
<td>0.008481</td>
<td>4.67</td>
</tr>
<tr>
<td>W24×131</td>
<td>W24×76</td>
<td>10</td>
<td>0.004689</td>
<td>0.004661</td>
<td>0.61</td>
</tr>
<tr>
<td>W24×131</td>
<td>W24×76</td>
<td>20</td>
<td>0.011101</td>
<td>0.010098</td>
<td>9.93</td>
</tr>
<tr>
<td>W24×131</td>
<td>W24×76</td>
<td>30</td>
<td>0.026645</td>
<td>0.025531</td>
<td>4.36</td>
</tr>
<tr>
<td>W33×291</td>
<td>W36×150</td>
<td>10</td>
<td>0.004468</td>
<td>0.004979</td>
<td>10.25</td>
</tr>
<tr>
<td>W33×291</td>
<td>W36×150</td>
<td>20</td>
<td>0.010134</td>
<td>0.010929</td>
<td>7.27</td>
</tr>
<tr>
<td>W33×291</td>
<td>W36×150</td>
<td>30</td>
<td>0.022261</td>
<td>0.022984</td>
<td>3.14</td>
</tr>
</tbody>
</table>

4.5.3 Composite Section Flange Yielding Analysis

Plastic strain distributions along the lower beam-flange to column-flange welding are analyzed here to study the effects of the concrete slab on column flange stresses.

Similar to the increments on column twisting demands, column flange Equivalent Plastic Strain (PEEQ) is slightly increased due to composite action. Figure 92 shows a comparison between the bare frame and the composite section for 4% and 5% drift at different skew angles. Because the applied moment is larger for the composite section, the stresses at the column flange increase as well, specifically close to the column center, but the column flange edge is not affected by the slab action. As it was noticed in the previous chapter, increasing the skew for the composite section models leads to a reduction of plastic strain at the column flange center. For example, at the flange center for 10-degree skew and 4% drift, the PEEQ = 0.057, but PEEQ = 0.043 at 4% drift for 30-degree skew for the same connection size.
For the medium column, the column flange strain values are completely different from the shallow column. Those values are larger all along the column flange, including the flange edge. The plastic strain at the column flange tip at 30-degree skew is larger than any other plastic strain value for all medium and shallow column models. Figure 93 depicts a comparison for the column flange plastic strain for all the skew angles. It is important to note that the medium column has the thinnest and narrowest column flange among all the column sizes used in this investigation. Then applied moment causes severe stresses on the column flange.
Figure 93. Column flange plastic strain comparison for medium columns at 4% and 5% drift.

In Figure 94 are depicted the predicted level of yielding for the deep column with different skew levels. Column flange plastic strain for deep columns is also increased by the skew angle. A skew of 30 degrees caused more yielding at the column flange edge than at any other place along the column flange. The medium and deep column PEEQ plots, with skew of 30 degrees, indicate a large amount of yielding at the column flange edge.
Increasing concrete strength for composite section frames does not affect the level of column flange yielding at the lower beam-flange to column-flange interface. As depicted in Figure 95, yielding at the column flange for the same type of skew connection (shallow column), having different concrete strengths, does not change at all. A stronger concrete slab does not cause more yielding at the column flange edge.

Figure 94. Column flange plastic strain for deep columns at 4% and 5% drift.
Figure 95. Shallow column flange composite section yielding comparison for 4000 psi and 6000 psi.

4.5.4 Composite Section Fatigue Analysis

The approach used to study low-cycle fatigue fracture in the previous chapter is applied here as well. Ultra-low cycle fatigue analysis [61] of SMF RBS composite connections was used to investigate potential for ductile fracture, at connection welds and RBS cross section at the bottom and top flange, for all the composite section models presented in Table 24. The slab presence affects considerably the demand (plastic strain) at the beam bottom flange [66]. The fact that AISC 358-16 [6] does not requires lateral bracing close to the RBS section and the plastic neutral axis shifting up (due to the slab inertia) normally cause an increment of the level of damage for the beam bottom flange. Less damage at the top flange is frequently found for the composite section connections. This behavior is explained in the following paragraphs.

For all the results presented here, the values of PEEQ, Misses Stress and Pressure Stress were extracted at the element with the highest PEEQ value at the RBS flange cut and the join of
beam flange-to-column flange (weld). Sometime the highest PEEQ value was found at the access hole, but it was not considered because experimental testing has demonstrated that connection performance is not highly sensitive to the weld access hole failure [6].

Figure 96 shows a comparison of damage level at the bottom flange for the shallow column (W14×193) for the out-of-plane bare and composite SMF connections. An out-of-plane angle of 10 degrees for composite and bare frame connections does not affect considerably the capacity at the connection weld (see reserve fatigue capacity in Figure 96). The RBS composite section gets such amount of damage that fracture is estimated to occur at about 7% drift. In consequence, the main purpose of RBS connections, which is to concentrate damage within the RBS section and not the connection (weld), was reached. However, the level of damage at 4% drift for both connections at the RBS section was almost the same (32.11% for the bare frame and 36.01% for the composite section), which means that both types of connections are satisfactory according to the AISC 358-16 prequalification requirements. The level of damage at 4% drift for the welding at the bottom flange is also small and the values are similar between them. For this particular case, welding damage for the composite section is 11.07% and 9.62% for the bare frame.

Figure 96. Effects of concrete slab on shallow column beam bottom flange demands for 10-degree skew (a) Bare Frame (b) Composite Section.
Demands at the beam top flange behave very different compared to the beam bottom flange. Figure 97 depicts the concrete slab effects on the top flange for the shallow column and an out-of-plane skewed angle of 10 degrees. The overall performance of the RBS section is improved (there is not fracture estimated) due to the slab composite action. Although there is a slightly reduction of capacity at the weld for the composite section, the connection meets widely the minimum requirements for prequalification. The composite section damage ratio (at 4% drift) within the RBS is 22.18% and 23.54% for the bare frame. For the welding damage ratio, the composite section damage is just 5.78% and 4.96% for the bare frame connection.

Figure 97. Effects of concrete slab on shallow column beam top flange demands for 10-degree skew (a) Bare Frame (b) Composite Section.

A 20 degrees out-of-plane skew reduces RBS fatigue capacity for the bare and composite sections SMF connections. Figure 98 shows the demand increments for the RBS section when increasing the skew angle. Fracture for the RBS composite section is estimated to occur earlier than for 10 degrees out-of-plane skew (around 6% drift). Reserve fatigue capacity at the connection welds has no variation for both bare and composite connections regarding to the skew of 10 degrees. Although there is a reduction in fatigue capacity (within the RBS section), the overall connection performance satisfies the minimum prequalification requirements. The demand for the top flange is very similar to the one presented in Figure 97, where the concrete slab helped to increase the fatigue capacity at RBS section.
Different from the other less severe skew angles, for 30 degrees out-of-plane skew, RBS fracture is estimated to occur at the top and bottom flange for the composite section. The increased skew enlarged demands within the RBS and speed up the material failure. However, the connection satisfies the minimum requirements for prequalification because fracture is predicted after 4% drift. Figure 99 shows the fracture estimation at the top and bottom flange due to the skew increment and the slab presence. In addition, this figure shows a high level of reserve capacity at the connection weld although the damage ratio within the RBS is the highest for shallow columns.

The medium column (W24×131) composite section connection with out-of-plane skew of 10 and 30 degrees shows a similar trend when comparing those damage ratio plots with the corresponding shallow column plots. In other words, the bottom flange of the composite section
is having excessive strength degradation leading to fracture within the RBS section, but the top flange increases its fatigue capacity (due to the slab) at the RBS as well. The only difference between the bare and the composite section is the fatigue capacity at connection weld for 10-degree skew, which was increased at the composite section top flange.

Similar effect is found at the top flange of the composite section with skew of 20 degrees. Figure 100 shows the damage index comparison at the bottom flange for the column W24×131 with 20 degrees skew. Although fracture is estimated to occur in the RBS for both the bare and the composite section bottom flange with out-of-plane skew of 20 degrees, it is important to point out the increment in reserve fatigue capacity at the beam-to-column weld, when the connection is under composite action. The slab effect is also noticeable because the RBS fracture is indicated to occur earlier than for the bare frame.

![Figure 100](image)

**Figure 100.** Effects of concrete slab on beam bottom flange (20-degree skew) demands for medium columns (a) Bare Frame (b) Composite Section.

The level of damage for the composite section with deep column is more critical than for the other composite connections. For both top and bottom flange, RBS fracture is indicated not far from the minimum prequalification requirement. Analysis for the bare frame with deep column was indicating RBS fracture just for the skew of 30 degrees, but for the composite section, RBS fracture is indicated at all skew levels. Figure 101 shows a comparison bare and composite frame connections at top and bottom flange for deep columns at 10-degree skew.
While there is a significant RBS damage ratio increment when considering the composite slab effects, damage ratio at the connection weld is not severely affected by the concrete floor because composite section weld fatigue capacity remains similar to reserve fatigue capacity at the connection weld for the bare frame.

![Diagram](https://via.placeholder.com/150)

**Figure 101.** Effects of concrete slab on RBS connections for deep columns a) Bare frame bottom flange b) Bare frame top flange c) Composite section bottom flange d) Composite section top flange.

Similar connection damage ratio behavior (RBS fracture indicated for the composite section) was found the connections with 20 and 30 degrees whereas the RBS fracture was indicated to happen earlier when increasing the skew angle.

Connection capacity, yielding and twisting are not highly affected by increasing the concrete strength form 4000 psi to 6000 psi, but damage ratio was enlarged for shallow column connections having 6000 psi and 10-degree skew. In addition, reserve fatigue capacity at top
flange connection weld is severely reduced at the test end (8% drift). Figure 102 shows a comparison for the shallow column connection with 10-degree skew for both concrete strengths.

![Figure 102](image)

Figure 102. Effects of different concrete strengths of RBS connections for shallow columns a) Bottom flange-4000 psi b) Top flange-4000 psi c) Bottom Flange-6000 psi d) Top Flange-6000 psi.

For more details, all damage index plots are presented in Appendix D.

Figure 103 shows a summary of fatigue damage for SMF RBS composite connections at beam bottom flange. This figure shows that damage ratio at RBS is always higher than at the beam-to-column weld (fulfilling the purpose of RBS connection design). It is also clear that deep column connections (W33×) experience more fatigue damage than medium (W24×) or shallow (W14×) column connections.
As it was mentioned before, the concrete floor helps to reduce the fatigue damage at the beam top flange. Although the level of damage at the beam top flange is smaller compared with the beam bottom flange, Figure 104 shows similar pattern between top and bottom flange where deep column connections undergo more fatigue damage.

4.5.5 Composite Section Panel Zone Performance

Plastic rotations are affected by the concrete slab presence. A comparison of bare frame plastic rotations and composite section plastic rotations shows clear composite action effects (mostly for positive moment). The results presented in the global and local response sections were derived using the data reduction procedure presented by Uang and Boundad [73]. All the comparison figures for this section are presented in Appendix H.
4.5.5.1 Global Response

Figure 105 shows the connection plastic performance for the shallow (W14×193) column, with 10 degrees skew, considering the composite and the bare frame. Panel zone plastic rotation is very small compared to the other rotations, which means that the panel zone remains essentially elastic. The moment applied by the composite section beam is larger than the moment applied by the bare frame beam, causing some increments in the panel zone plastic rotation. The maximum total plastic rotation is 0.073 rad (composite). Of this amount, a plastic rotation of 0.065 rad was developed at the beam. During positive incursions, the concrete slab helps to reduce the beam plastic rotation, but panel zone plastic rotation increased due to the slab when comparing the bare and the composite frames.

![Figure 105. Plastic rotation comparison for the shallow column frame with 10-degree skew.](image)

For the connections with 20- and 30-degrees skew, there is a slightly increment of plastic rotations. The maximum total plastic rotation for case of out-of-plane of 30 degrees (composite) is 0.074 rad; of this amount, a plastic rotation of 0.069 rad was developed at the beam. A reduction of composite section panel zone plastic rotation is noticeable for the skew of 30 degrees; it decreased from 0.00095 rad to 0.00069 rad. Because the moment component applied on the column web is smaller for 30-degree skew than for 10-degree skew, the maximum panel zone plastic rotation was reduced. Figure 106 illustrates the plastic rotations for the shallow column (with and without slab), considering an out-plane skew of 30 degrees.
Plastic rotation values for the medium column (W24×131) with skew of 10 degrees are similar to those for the shallow column. For example, the maximum total plastic rotation for the composite section is 0.075 rad and the amount of beam plastic rotation is 0.070 rad. However, the maximum panel zone plastic rotation (0.00085 rad) is smaller for this column than for the shallow column (0.00095 rad). Figure 107 illustrates the plastic rotations for the medium column with 10-degree skew (with and without slab).

Out-of-plane skew of 20 and 30 degrees (composite section) does not change considerably the plastic rotations when comparing them with 10-degree skew, and the panel zone plastic rotations are reduced when increasing the beam skew.

For the deep column (W33×291), plastic rotation comparisons are done with 30-degree skew because bare frame models with 10- and 20-degrees skew fails due to excessive buckling within the RBS region before reaching the test end. Although the values are similar, the maximum total plastic rotation (0.076 rad) for the deep column composite section is larger than those for the shallow (0.074 rad) and medium column (0.074 rad) at the same skew angle.
However, the maximum panel plastic rotation (0.00046 rad) is smaller than those values for the shallow (0.00069 rad) and medium column (0.00062 rad). Figure 108 shows a comparison of plastic rotations for the deep column with and without concrete slab for skew of 30 degrees.

Figure 108. Plastic rotation comparison for the deep column frames with 30-degree skew.

A high strength concrete slab does not change the outcome of plastic rotations. The models are predicting almost always the same plastic rotations for both types of concrete (4000 and 6000 psi). Figure 109 presents a comparison of plastic rotations for the shallow columns with two concrete strengths considered in this project.

Figure 109. Comparison of plastic rotations for 4000 and 6000 psi composite sections.

Table 34 presents a summary of composite section plastic rotations. All the values presented were calculated at 8% drift. In general, panel zone plastic rotations decrease when increasing the skew angle, and the total plastic rotation tends to increase. Those maximum values came from the negative incursion of the test because the slab is contributing to reduce rotations for positive moment.
Table 34. Maximum composite section plastic rotations

<table>
<thead>
<tr>
<th>Composite Section Model</th>
<th>Beam Plastic Rotation (rad)</th>
<th>Panel Zone Plastic Rotation (rad)</th>
<th>Total Plastic Rotation (rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T2RC-14 × -10&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.065</td>
<td>0.00095</td>
<td>0.073</td>
</tr>
<tr>
<td>T2RC-14 × -20&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.067</td>
<td>0.00081</td>
<td>0.074</td>
</tr>
<tr>
<td>T2RC-14 × -30&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.069</td>
<td>0.00069</td>
<td>0.074</td>
</tr>
<tr>
<td>T2RC-24 × -10&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.070</td>
<td>0.00085</td>
<td>0.075</td>
</tr>
<tr>
<td>T2RC-24 × -20&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.071</td>
<td>0.00073</td>
<td>0.075</td>
</tr>
<tr>
<td>T2RC-24 × -30&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.071</td>
<td>0.00062</td>
<td>0.074</td>
</tr>
<tr>
<td>T2RC-33 × -10&lt;sup&gt;a&lt;/sup&gt;</td>
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<td>0.00037</td>
<td>0.076</td>
</tr>
<tr>
<td>T2RC-33 × -20&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.074</td>
<td>0.00050</td>
<td>0.076</td>
</tr>
<tr>
<td>T2RC-33 × -30&lt;sup&gt;a&lt;/sup&gt;</td>
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<td>0.00046</td>
<td>0.076</td>
</tr>
<tr>
<td>T2RC-14 × -10&lt;sup&gt;b&lt;/sup&gt;</td>
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<td>0.00095</td>
<td>0.072</td>
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<td>T2RC-14 × -20&lt;sup&gt;b&lt;/sup&gt;</td>
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<td>0.00081</td>
<td>0.073</td>
</tr>
<tr>
<td>T2RC-14 × -30&lt;sup&gt;b&lt;/sup&gt;</td>
<td>0.067</td>
<td>0.00069</td>
<td>0.073</td>
</tr>
</tbody>
</table>

<sup>a</sup> Concrete strength=4000 psi
<sup>b</sup> Concrete strength=6000 psi

4.5.5.2 Local Response

Panel zone shear strain is computed for all the composite section models. The same trend noticed for the panel zone plastic rotation is applicable for the shear strain. The increasing skew reduced the panel zone shear strain. For example, the maximum shear strain for the shallow column is 0.0039 rad for the skew of 10 degrees and 0.0032 rad for the skew of 30 degrees. Typically, the maximum shear strain happens during the moment positive incursion. Also, important to note that shear strain decreases when increasing the column size. Figure 110 shows a comparison plot of shear strain for the bare frame shallow column (higher shear strain) and the composite section (skew of 10 degrees). A summary of all shear strain at panel zone is presented in Table 35.
Figure 110. Shear strain comparison for the bare frame and composite section in shallow column panel zone.

Table 35. Maximum composite section shear strain

<table>
<thead>
<tr>
<th>Composite Section Model</th>
<th>Panel Zone Shear Strain (rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T2RC-14 × -10&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.003959</td>
</tr>
<tr>
<td>T2RC-14 × -20&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.003490</td>
</tr>
<tr>
<td>T2RC-14 × -30&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.003152</td>
</tr>
<tr>
<td>T2RC-24 × -10&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.003113</td>
</tr>
<tr>
<td>T2RC-24 × -20&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.002928</td>
</tr>
<tr>
<td>T2RC-24 × -30&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.002680</td>
</tr>
<tr>
<td>T2RC-33 × -10&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.00271&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
<tr>
<td>T2RC-33 × -20&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.002556</td>
</tr>
<tr>
<td>T2RC-33 × -30&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.002537</td>
</tr>
<tr>
<td>T2RC-14 × -10&lt;sup&gt;b&lt;/sup&gt;</td>
<td>0.003967</td>
</tr>
<tr>
<td>T2RC-14 × -20&lt;sup&gt;b&lt;/sup&gt;</td>
<td>0.003500</td>
</tr>
<tr>
<td>T2RC-14 × -30&lt;sup&gt;b&lt;/sup&gt;</td>
<td>0.003151</td>
</tr>
</tbody>
</table>

<sup>a</sup> Concrete strength=4000 psi  
<sup>b</sup> Concrete strength=6000 psi

4.6 Summary and Conclusions

A parametric finite element study was performed in this chapter to investigate the effects of a concrete slab on SMF skewed connections. Commercial software ABAQUS was used to model a total of 12 finite elements models considering 3 skew levels and 2 concrete strengths. Those finite elements models were created from four-node linear shell elements (S4R in ABAQUS) for the bare frame, eight-node solid elements (C3D8R in ABAQUS) for the slab, 1-dimensional Timoshenko beam elements (B31 in ABAQUS) for the studs and rebar is modeled
as wire elements (truss element in ABAQUS: T3D2). Finite elements models were validated using the experimental data from Jones et al. [4] and all SMF connections were design according with ASIC 358-16 [6].

The concrete slab has a significant increasing effect on the connection capacity for positive moment for all the models in the parametric study, especially beyond 4% drift when the bare frame moment capacity starts decreasing. The moment capacity for negative moment is almost not affected for the slab because the concrete does not work in tension, but for medium and deep column, the connection moment capacity slightly increases for negative moment. Normally the composite action increases the column twist for skew angles beyond 20 degrees, but 10 degrees skew, the column twist is always reduced by the slab presence. Flange tip yielding at the bottom beam flange-to-column flange join is not affected by the composite action for the shallow columns, but the slab drastically helps to enlarges the flange yielding for the medium and deep column for the skew of 30 degrees. Fatigue analysis indicates that the RBS at beam bottom flange is negatively affected the composite action. Fracture is predicted to happen earlier than for bare frame connections, although it happens beyond the minimum requirement for prequalification. However, fracture within the RBS section at the top flange is delayed or suppressed due to the composite action. Reserve fatigue capacity at the connection (weld) is not affected by the composite action for shallow columns, but the fatigue capacity at the weld for medium columns is reduced considerably. Connection capacity, column flange yielding and column twist are slightly affected by incrementing concrete strength. Ultra-low cycle fatigue analysis for shallow column composite section with 6000 psi shows similar levels of damage than the same connections with 4000 psi, except for the top flange at 10-degree skew connection where (different from all other 4000 psi top flange composite connections) fracture is predicted.
Compared to bare frame connections, panel zone plastic rotations are always increased for composite action.
CHAPTER 5: EXPERIMENTAL INVESTIGATION INTO SKEWED SMF CONNECTION DEMANDS

This chapter details several steps for testing, including the setting of specimen geometry (fabrication), instrumentation, loading and detailed fractures investigations at the fracture plane.

Since current prequalified RBS in SMF in the Seismic Provisions for Structural Steel Buildings [13] are implicitly orthogonal, experimental testing of skewed beam-to-column connections helps to increase the knowledge about this type of connection. The key point in this section is the fact that there will be a validation of skewed connections simulations performed by Prinz and Richard [5]. A replica of each experimental sample was modeled in ABAQUS for a direct comparison of results.

5.1 Background

Current SMF RBS connections must satisfy strength and drift requirements established in AISC 341-16 [13]. To satisfy strength requirements, the connection flexural resistance, calculated at the column face, shall be equal to 80% of $M_p$ (beam plastic moment) when reaching a story drift angle of 4%. Under the loading sequence in Chapter K of AISC 341-16 [13], the connection shall accommodate a story drift angle of 0.04 rad in order to satisfy drift requirements.

While several studies have examined the experimental performance of orthogonal RBS connections for seismic applications in special and intermediate steel moment frames [2, 3, 53, 66], none of them have studied the experimental performance of out-of-plane RBS connections. It has been demonstrated that SMF RBS connections display ductile behavior under inelastic cyclic loading [2]. Orthogonal RBS connections have been successfully implemented after the 1994 Northridge Earthquake, but just few cases of in-plane skew have been tested experimentally. Mashayekh and Uang [9] tested two types of RBS configurations for an in-plane
skew of 25 degrees. Brittle fracture was found due to concentration of force starting at the end of
the beam web CJP weld and the access hole. Another publication about in-plane skewed RBS
connections presented by Kim et al. [10] considered a deviation angle of 28 degrees. As in the
previous case, brittle fracture followed after the connection met the minimum requirements for
prequalification established in AISC 341-16 [13]. The unusual RBS failure mode of these in-
plane RBS connection is pointed out in this paper. The increased level of yielding at some
specific locations (top flange for example) is another factor of concern in this type of SMF
connections.

5.2 Experimental Program

The experimental testing will be conducted at the Structural Steel Research Laboratory
(SSRL) at the University of Arkansas.

The primary objective of this chapter is to examine the RBS skewed connection issues
using large scale experimental samples.

The experimental program is focused on 1) evaluation of SMF RBS skewed connection
capacity according to AISC 341-16 [13], 2) investigation of connection fatigue capacity and
comparison with analytical models, 3) performance of panel zone (global and local response).

5.2.1 Test Specimen Geometry and Fabrication of Skewed SMF Connections

A total of three 1-story connection assemblies are considered in this research for
experimental testing. Those experimental results provide a complete data set for estimating
demands in skewed RBS SMF connections during cyclic loading. These steel connection
arrangements represent skewed configurations at three different levels of skew (10, 20, and 30
degrees), with a column size of W14×132. This column geometry is selected to examine the
column twisting and yielding at the flange tip. All test specimens will include W24×76 beams.
For the beams and columns, steel A992 was detailed, and A572 Gr 50 steel for the continuity plates (CP) and doubler plates (DP). Table 36 presents the proposed experimental test matrix to be tested in this research project.

Table 36. Beams and Column dimensions for RBS experimental study.

<table>
<thead>
<tr>
<th></th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SC-1</td>
<td>10</td>
<td>0</td>
<td>W 14×132</td>
<td>W 24×76</td>
<td>1/2</td>
<td>3/8</td>
</tr>
<tr>
<td>2</td>
<td>SC-2</td>
<td>20</td>
<td>0</td>
<td>W 14×132</td>
<td>W 24×76</td>
<td>1/2</td>
<td>3/8</td>
</tr>
<tr>
<td>3</td>
<td>SC-3</td>
<td>30</td>
<td>0</td>
<td>W 14×132</td>
<td>W 24×76</td>
<td>1/2</td>
<td>3/8</td>
</tr>
</tbody>
</table>

Beams and columns were fabricated by a commercial fabricator. All skewed connection samples will be manufactured using current moment frame fabrication procedures.

To connect beams and columns, the welding procedure followed the requirements outlined in AISC 358-16 and American Welding Society (AWS) along with CJP groove welds for both beam flanges. Doubler plates and continuity plates have CJP welds and fillet welds according to design specifications presented in AISC Steel Construction Manual [74] and AISC 358-16 [6]. After the 1994 Northridge earthquake, it was found that the bottom flange CJP weld is more prone to brittle fracture [75]. Therefore, realistic field conditions during SMF beam-to-column connections fabrication will be replicated removing the backing bar on the bottom flange, backgouging of the weld root, and posterior backwelding with a reinforcing fillet weld [6]. During welding procedures, the specimens were erected in the vertical position in the shop to simulate field welding conditions [10] as presented in Figure 111. Gas-Shielded, flux-cored arc welding with a E70T-1C electrode was used for the welding. Figure 112 presents details of welding connection design used for sample fabrication. Complete details for fabrication are presented in Appendix E.
Figure 111. Specimen upright position during welding work (Photo by author).

Figure 112. Welding connection details for sample fabrication.

Use electrode which provides minimum specified Charpy V-Notch toughness of 20 lb-ft @ -20° F.

The beam web weld access hole needs to be ground to a surface roughness of 500 micro-inches.
5.2.2 Test Configuration, Instrumentation and Loading

Size of the test specimen and SSRL current frame size fit appropriately to perform the testing series proposed for this research project. The experimental arrangement proposed for this investigation is presented in Figure 113.

The skewed SMF connection will be loaded with applied beam-tip displacements. Those displacements derive from the outlined story drift angles in the loading protocol (Chapter K of AISC 341-16) multiplied by the beam length. The displacements applied to the 15 ft-long beam specimen, by a 220 kips servo-hydraulic actuator, involves 6 cycles each at 0.00375 rad, 0.005 rad and 0.0075 rad. It will be continued by 4 cycles at 0.01 rad, and 2 cycles each at 0.015 rad, 0.02 rad, 0.03 rad and 0.04 rad (prequalification), and so on to investigate skewed SMF capacity.

The experimental setup shown in Figure 113 is placed parallel to the floor with the beam laterally braced at two locations, (1) outside the RBS cut at a distance $d/2$ (maximum) from the RBS end, and (2) at the beam tip. A critical case for study is considered from this experimental setup because the locations selected for lateral supports represents the minimum level of beam lateral bracing allowed in the AISC seismic provisions [13]. An angled connection between the reaction frame and column will remain during the test because the beam is always kept horizontal (parallel to the floor), as presented in Figure 113 (Section A–A). Equivalent connection rotations similar to other full-scale testing of SMF elements are used to relate the applied displacements to the beam tips [53, 76].

Column axial loads are considered zero (Table 36) for these set of experiments because it was found from the computer models that fundamental parameters in SMF connection design such as maximum connection capacity, twist at 4% drift and moment at 4% drift are not highly
affected by increasing the column axial load. On the contrary, column local buckling has more significant adverse effects.

Data from experimental testing will be collected using several equipment and devices (strain gauges, Linear Variable Differential Transformers (LVDTs), optical extensometers, etc.). Strain gauges strategically mounted to supply insight into skew effects on local stress/strain demands and to provide information (global and local responses) for finite element model

Figure 113. Experimental Setup.
validations will be used to collect strain demands within the RBS section and beam-to-column weld regions. Figure 114 shows proposed locations to place strain gauges to gather strain data.

Figure 114. Strain gauge locations for beams (Sections A-A, B-B) and columns (Elevation).

5.3 Experimental Results

For the performed test, the specimen SC-2 (W14x132 column and W24x76 beam with skew of 20 degrees) was tested under the setting explained previously in this chapter. The displacement based-control test was executed with the hydraulic actuator for the first 18 cycles (0.00375 rad, 0.005 rad and 0.0075 rad) form the AISC loading protocol without any apparent issues. The subsequent cycles (0.01 rad, 0.015 rad and 0.02 rad) were applied, but at the beginning of the 0.03 rad cycles a considerable displacement of the reaction frame was detected. Figure 115 shows a sketch of the reaction frame relative displacement which led the research team to terminate the test before reaching 4% drift. The channels, which act as braces to avoid the beam lateral movement, are able to stiff the frame in the direction parallel to the column. However, the frame is not stiff enough in the direction of the beam, allowing such reaction frame movement.
At the upper left corner of the reaction frame, one bolt was broken and the others were loose. The reaction frame member adjacent/parallel to the column (W14x132) did allow some twist which also affected the system stiffness. Under these circumstances, the reaction frame should be stiffened before proceeding with the experimental testing.

Yielding was found at the beam flanges between the RBS section and the connection welding. This is evident from the flaking of the whitewash as depicted in Figure 116.
5.4 Correlation of Analytical Predictions and Experimental Results

A model of the tested connection was created and evaluated using the finite-element program ABAQUS. The computer three-dimensional model used four-node linear shell elements with reduced integration (S4R in ABAQUS) in order to capture local buckling and obtain local stress and strain measures in the connection regions. Similar to the parametric study, material and nonlinearities geometric imperfections were considered in the model. Shear tab and beam access holes were not included in this model. Elastic properties for the computer model consider a Young’s modulus of 29000 ksi and a Poison’s ratio of 0.30. A combined nonlinear isotropic and kinematic material model is used to describe strain hardening in order to account for steel plastic properties. Mesh refinement is the same used in the parametric study (Chapter 3). Cyclic displacements at the beam tip are applied following the loading protocol established by the AISC seismic provisions [13].
Figure 117 shows the local buckling and the lateral-torsional buckling for the beam flanges in the analytical model.

For the experimental specimen, buckling was not detected at the connection at this early stage of the test. More yielding needs to happen in order to develop some buckling at the RBS section.

A backbone comparison plot of experimental testing and analytical model in Figure 118. From this plot, it is clear that a reaction frame without enough stiffness leads to a higher connection rotation for the same applied moment. In a similar way, at 2% drift, the analytical moment is almost twice the experimental moment.
5.5 Summary and Conclusions

In this chapter, one skewed SMF connection was partially tested under the AISC loading protocol. This skewed SMF connection represents the standard size used in previous experimental moment connection testing, with half-span beam and half-story column below and above.

Due to the lack of reaction frame stiffness explained before, a direct comparison of rotation capacity (based on strength degradation) between the experimental testing and the ABAQUS model cannot be completed.
CHAPTER 6: SUMMARY, CONCLUSIONS, AND FUTURE RESEARCH

6.1 Summary and Conclusions

In the current AISC 358-16 provisions covering Prequalified Connections for Special (SMF) and Intermediate Steel Moment Frames (IMF) for Seismic Applications [6], SMF connection details are prequalified for situations where the beams and columns are framed orthogonal each other. Little guidance is provided in the specifications for framing situations where a skew is required between the beam and column in an SMF connection, and currently no prequalified skewed SMF details exist. This dissertation presented the results of an extensive analytical investigation consisting of 112 component-level bare-frame finite element analyses, 12 component-level concrete-steel composite analyses, one system-level dynamic earthquake simulation, 152 detailed finite element submodels for fracture investigation, and one preliminary experimental test. These various simulations and the preliminary experimental test investigated the behavior of both skewed SMF RBS and WUF-W connections. The following conclusions, which include an out-of-plane skew angle limit suggestion for consideration in AISC 358-16, result from the extensive component-level parametric finite element investigations, the system-level dynamic earthquake analyses, and a preliminary experimental prequalification test.

1) Increasing connection skew angle results in decreased flexural capacity at the SMF connection; however, for all 112 skewed SMF analyses the connection flexural capacity exceeded the $0.8M_p$ requirement at 4% drift (0.04 rad) required by the AISC provisions.

2) Increasing connection skew results in increased column twisting and corresponding column flange-tip yielding within the connection. It is important to note, however, that the observed column flange tip yielding was isolated to a small region near the beam-to-column weld.
3) Increases in column section depth result in an increased eccentricity between the beam-
flange force-line and column centroid, leading to increased column twist for deeper column
sections. A column twist prediction equation as presented in Chapter 4, resulting in
predictions that are within 10% of the observed twist in the finite element simulations (for
all column depths).

4) For the medium and deep column geometries having large skews (i.e. W24× and W33×
column geometries at 30-degree skew), column twist was increased by the presence of a
composite slab. In all other connection skew configurations, the inclusion of a composite
slab resulted in column twist reductions due to increased torsional stiffness at the
connection. The twist increase within the large skew composite connections is the
combined result of a beam-line eccentricity increase with column section centroid, and a
beam-section neutral axis shift leading to an increased bottom flange force within the
composite section. Changes in concrete compressive strength had negligible effect on
composite connection behavior.

5) For torsionally stiff column sections (i.e. the shallow and medium column geometries
considered) increased skew resulted in an increased susceptibility to low-cycle fatigue
fracture at the acute side of the skewed beam-to-column connection. As column torsional
stiffness decreased with the deeper column sections, a reduction in fatigue damage was
observed. This is due the increased column twist relaxing the weld region strains. Note
however that all fatigue simulations indicated low-cycle fatigue fracture after the 0.04 rad
cycles (4% drift) of the AISC seismic protocol (indicating acceptable performance).

6) Adding a composite slab to the skewed SMF connections increases low-cycle fatigue
susceptibility at the bottom flange welded connection, again due to the beam section neutral
axis shift; however, all simulations still exceeded the 0.04 rad rotation cycles (4% drift) of the AISC protocol prior to a fracture initiation indication.

7) Including skew within the beam-to-column connection can result in panel-zone demand reductions. As skew angle increases, plastic rotation within the panel-zone decreases.

8) System-level dynamic analyses indicate the potential for higher residual frame drifts than would be anticipated for orthogonal connection configurations.

9) A recommended skew limit of 20 degrees is suggested based on the observed skewed SMF connection behavior within the analytical parametric and composite connection simulations.

### 6.3 Recommendations for Future Work

An interesting comparison can be done if the same building (Plan B) and ground motion (Northridge earthquake) used in Chapter 2 is modeled having orthogonal connections. Analysis of some submodels for the building connections could lead to a better understanding of fatigue damage at the beam-to-column weld in the dynamic system-level performance.

It is recommended that more experimental research be conducted on other column sizes to determine the skew effects on other SMF RBS connections. Results from this dissertation suggest that columns W24×131 and 33×291 are more prone to higher stresses at the column flange and therefore to more yielding. These columns are also more affected by twisting.

To validate the findings from Chapter 4, some steel-concrete skewed connections samples are recommended to be tested experimentally.

It is suggested to conduct detailed fracture investigations for the experimental testing (especially at welds) in order to validate the analytical fracture investigation from Chapter 3 and Chapter 4.
REFERENCES


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APPENDICES

APENDIX A. BUILDING DESIGN EXAMPLE CALCULATIONS FOR PROTOTYPE BUILDING

The following is an example calculation of the structural section sizes, doubler plates and continuity plates in the first prototype building considered. The same calculation process was followed for each of the prototype designs. The floor plan (Plan B) for the following calculations is presented in Figure B-2.

Design based on SAC 6-story building - Plan B

Building Codes/ Design Documents:
1 - Gravity Loads:

Floor Dead Load: \( D_1 = 85 \cdot \text{psf} \)

Floor Live Load: \( L_1 = 50 \ \text{psf} \)

Roof Dead Load: \( D_2 = 95 \cdot \text{psf} \)

Roof Live Load: \( L_2 = 20 \ \text{psf} \)

Weight of exterior walls: \( WL = 25 \cdot \text{psf} \)

Number of Floors: \( NF = 6 \)

Consider 2 ft slab overhang all around

Slab Area: \( SA = 33062.46 \cdot \text{ft}^2 \quad \text{Considering overhang} \)

Perimeter: \( P = 746.58 \cdot \text{ft} \)

Building Length: \( L = 180 \cdot \text{ft} \)

Height of the first floor: \( h_1 = 13 \ \text{ft} \)

Height of the other floors: \( h_2 = 13 \ \text{ft} \)

Beam length: \( L_b = 30 \cdot \text{ft} \)

Number of Interior Columns: \( NIC = 5 \)

Elasticity: \( E = 29000 \cdot \text{ksi} \)

Yield Strength of Steel: \( F_y = 50 \cdot \text{ksi} \)

\( F_u = 65 \cdot \text{ksi} \)
2 - Lateral Loads:

Place: Los Angeles, CA

Response modification factor: \( R = 8 \)

Risk Category II

Importance parameter: \( I = 1 \)

Site Class C

Accidental torsion: \( AT = 5\% \)

Risk-Target MCE:
\[ S_1 = 0.705 \quad g \quad \text{USGS Web Site} \]

Risk-Target MCE:
\[ S_S = 1.978 \quad g \quad \text{USGS Web Site} \]

Site Coefficient:
\[ F_a = 1.2 \quad \text{USGS Web Site} \]

Site Coefficient:
\[ F_v = 1.40 \quad \text{USGS Web Site} \]

\[ S_{MS} = F_a \cdot S_S = 2.374 \quad \text{https://earthquake.usgs.gov/designmaps/beta/us/} \]

\[ S_{M1} = F_v \cdot S_1 = 0.987 \]

\[ S_{DS} = \left( \frac{2}{3} \right) \cdot S_{MS} = 1.5824 \quad g \]

\[ S_{DI} = \left( \frac{2}{3} \right) \cdot S_{M1} = 0.658 \quad g \]

3 - Seismic Weight Calculation:

Multiply weights from SAC Project by 1.5. the design will have the same approach that Dr. Prinz' Thesis

Add 42 inches of wall at the top because of the parapet for roof weight calculations

\[ SW = 1.5 \cdot (D_1 \cdot (NF - 1) \cdot SA + WL \cdot (h_1 \cdot ft + (NF - 1) \cdot h_2 \cdot ft + 42 \cdot in) \cdot P + D_2 \cdot SA) = 28070.454 \text{ kip} \]
4 - Estimation of Period:

Coeff. for upper limit on calculated period: \( C_u := 1.4 \) 
If \( S_{01} > 0.4 \)

Approximate Period Parameters:

\( C_t := 0.028 \)
\( x := 0.8 \)

\( T := C_u \cdot C_t \cdot (h_1 + (N\Gamma - 1) \cdot h_2)^x = 1.279 \quad < T_L = 8 \text{ s} \)

5 - Base Shear Calculation:

\( C_{s1} := \frac{S_{DS}}{R} \cdot \frac{R}{I} = 0.1978 \)

\( C_{s2} := \frac{S_{D1}}{T \cdot \frac{R}{I}} = 0.064 \)

\[ CS := \begin{cases} 
\text{if } C_{s2} > 0.01 \vee C_{s2} < C_{s1} \\
C_{s2} \\
\text{if } C_{s1} < C_{s2} \\
C_{s1} 
\end{cases} \]

\( CS = 0.0643 \)

\( C_{s3} := 0.044 \cdot S_{DS} \cdot I = 0.07 \quad > 0.01 \quad \text{OK} \)

\[ C_s := \begin{cases} 
\text{if } CS < C_{s3} \\
C_{s3} \\
\text{if } CS > C_{s3} \\
CS 
\end{cases} \]

\( C_s = 0.07 \)

\( V := C_s \cdot SW = 1954.422 \text{ kip} \)
5.1 - Vertical Distribution of Base Shear to SMF:

\[
k = \begin{cases} 
  1 & \text{if } T < 0.5 \\
  2 & \text{if } T \geq 2.5 \\
  0.5 \cdot T + 0.75 & \text{if } T > 0.5 \land T < 2.5 
\end{cases}
\]

\[k = 1.39\]

\[w_1 = (SA \cdot D_1) + WL \cdot \frac{(h_1 \cdot ft + h_2 \cdot ft)}{2} \cdot P = 3052.9476 \text{ kip} \quad h_1^k = 35.316\]

\[w_2 = (SA \cdot D_1) + WL \cdot h_2 \cdot ft \cdot P = 3052.9476 \text{ kip} \quad (h_1 + h_2)^k = 92.533\]

\[w_3 = (SA \cdot D_1) + WL \cdot h_2 \cdot ft \cdot P = 3052.9476 \text{ kip} \quad (h_1 + 2 \cdot h_2)^k = 162.554\]

\[w_4 = (SA \cdot D_1) + WL \cdot h_2 \cdot ft \cdot P = 3052.9476 \text{ kip} \quad (h_1 + 3 \cdot h_2)^k = 242.447\]

\[w_5 = (SA \cdot D_2) + WL \cdot (0.5 \cdot h_2 \cdot ft + 42 \cdot \text{in}) \cdot P = 3327.5787 \text{ kip} \quad (h_1 + 4 \cdot h_2)^k = 330.588\]

\[w_6 = (SA \cdot D_2) + WL \cdot (0.5 \cdot h_2 \cdot ft + 42 \cdot \text{in}) \cdot P = 3327.5787 \text{ kip} \quad (h_1 + 5 \cdot h_2)^k = 425.912\]

\[M_1 = w_1 \cdot h_1^k \cdot ft = 107818.493 \text{ kip} \cdot ft\]

\[M_2 = w_2 \cdot (h_1 + h_2)^k \cdot ft = 282497.668 \text{ kip} \cdot ft\]

\[M_3 = w_3 \cdot (h_1 + 2 \cdot h_2)^k \cdot ft = 496269.201 \text{ kip} \cdot ft\]

\[M_4 = w_4 \cdot (h_1 + 3 \cdot h_2)^k \cdot ft = 740178.521 \text{ kip} \cdot ft\]

\[M_5 = w_5 \cdot (h_1 + 4 \cdot h_2)^k \cdot ft = 1009266.731 \text{ kip} \cdot ft\]

\[M_6 = w_6 \cdot (h_1 + 5 \cdot h_2)^k \cdot ft = 1417254.745 \text{ kip} \cdot ft\]
\[ C_{v1} := \frac{M_1}{M_1 + M_2 + M_3 + M_4 + M_5 + M_6} = 0.0266 \]

\[ C_{v2} := \frac{M_2}{M_1 + M_2 + M_3 + M_4 + M_5 + M_6} = 0.0697 \]

\[ C_{v3} := \frac{M_3}{M_1 + M_2 + M_3 + M_4 + M_5 + M_6} = 0.1224 \]

\[ C_{v4} := \frac{M_4}{M_1 + M_2 + M_3 + M_4 + M_5 + M_6} = 0.1826 \]

\[ C_{v5} := \frac{M_5}{M_1 + M_2 + M_3 + M_4 + M_5 + M_6} = 0.249 \]

\[ C_{v6} := \frac{M_6}{M_1 + M_2 + M_3 + M_4 + M_5 + M_6} = 0.3497 \]

**Accidental Torsion:**

\[ \text{Exc} := AT \cdot L = 9 \text{ ft} \quad \text{Eccentricity} \]

\[ V \cdot \left( \frac{L}{2} + \text{Exc} \right) \]

\[ R_1 := \frac{V \cdot \left( \frac{L}{2} + \text{Exc} \right)}{L} = 1074.932 \text{ kip} \]

\[ L = 180 \text{ ft} \]

\[ F_1 := C_{v1} \cdot R_1 = 28.593 \text{ kip} \]

\[ F_2 := C_{v2} \cdot R_1 = 74.918 \text{ kip} \]

\[ F_3 := C_{v3} \cdot R_1 = 131.611 \text{ kip} \]

\[ F_4 := C_{v4} \cdot R_1 = 196.296 \text{ kip} \]

\[ F_5 := C_{v5} \cdot R_1 = 267.658 \text{ kip} \]

\[ F_6 := C_{v6} \cdot R_1 = 375.856 \text{ kip} \]
6 - SMRF Design:

6.1 - Floors 1 and 2 (Sized based on Drift Allowables):

\[ V_T := F_3 + F_4 + F_5 + F_6 = (1.046 \times 10^3 \text{ kip}) \]

\[ V_{TB} := F_1 + F_2 + F_4 + F_5 + F_6 = 1074.932 \text{ kip} \]

Assume \( I_c = I_d = 1.0 \)

\[ I_c = 1.0 \cdot \text{in}^4 \]

\[ K_e := \frac{6 \cdot E}{(0.5 \cdot h_1 \cdot ft + 0.5 \cdot h_2 \cdot ft)^2} \left( \frac{1}{(0.5 \cdot h_1 \cdot ft + 0.5 \cdot h_2 \cdot ft) + \frac{L_b}{2 \cdot I_c}} \right) \approx 0.0163 \text{ kip/in} \]

\[ K_i := \frac{12 \cdot E}{(0.5 \cdot h_1 \cdot ft + 0.5 \cdot h_2 \cdot ft)^2} \left( \frac{1}{(0.5 \cdot h_1 \cdot ft + 0.5 \cdot h_2 \cdot ft) + \frac{L_b}{I_c}} \right) \approx 0.0277 \text{ kip/in} \]

\[ K_{TOT} := 2 \cdot K_e + NIC \cdot K_i = 0.1712 \text{ kip/in} \]

\[ V_{e1} := \frac{K_e}{K_{TOT}} \cdot V_T = 99.762 \text{ kip} \]

\[ V_{i1} := \frac{K_i}{K_{TOT}} \cdot V_T = 169.363 \text{ kip} \]

\[ V_{eb1} := \frac{K_e}{K_{TOT}} \cdot V_{TB} = 102.488 \text{ kip} \]

\[ V_{ib1} := \frac{K_i}{K_{TOT}} \cdot V_{TB} = 173.991 \text{ kip} \]

Calculation of Drift in terms of \( I_c \):

\[ \Delta_e = \frac{(V_{e1} + V_{eb1})}{2} \cdot \left( \frac{(0.5 \cdot h_1 \cdot ft + 0.5 \cdot h_2 \cdot ft)}{6 \cdot E} \right)^2 \]

\[ \Delta_e = \left( \frac{(V_{e1} + V_{eb1})}{2} \right) \cdot \left( \frac{(0.5 \cdot h_1 \cdot ft + 0.5 \cdot h_2 \cdot ft)}{2 \cdot I_c} + \frac{L_b}{I_c} \right) = 6194.869 \text{ in}^4 \cdot \text{in} \]
\[ \Delta_a = 0.02 \cdot (0.5 \cdot \text{ft} + 0.5 \cdot \text{ft}) = 3.12 \text{ in} \]

\[ C_d = 5.5 \]

\[ \Delta_{ac} = \frac{\Delta_a}{C_d} = 0.567 \text{ in} \]

\[ I_{rod} = \frac{\Delta_e}{\Delta_{ac}} = 10920.442 \text{ in}^4 \]

Choose column and beams from AISC:

**Beam W24 x 335**

**Column W33 x 387**

\[ I = 11900 \text{ in}^4 \]

\[ I = 24300 \text{ in}^4 \]

\[ d_b = 27.5 \text{ in} \quad k_{bdes} = 2.98 \text{ in} \]

\[ d_c = 36 \text{ in} \quad k_{cdes} = 3.07 \text{ in} \]

\[ b_{bf} = 13.5 \text{ in} \quad t_{wb} = 1.38 \text{ in} \]

\[ b_{cf} = 16.2 \text{ in} \quad t_{we} = 1.26 \text{ in} \]

\[ t_{bf} = 2.48 \text{ in} \quad h_b = d_b - 2 \cdot k_{bdes} = 21.54 \text{ in} \]

\[ t_{cf} = 2.28 \text{ in} \quad h_c = d_c - 2 \cdot k_{cdes} = 29.86 \text{ in} \]

Compactness Verification:

\[ \frac{h_{bf}}{2 \cdot t_{bf}} \leq 0.32 \cdot \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK} \]

\[ \frac{b_{cf}}{2 \cdot t_{cf}} \leq 0.32 \cdot \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK} \]

\[ \frac{h_b}{t_{wb}} \leq 1.57 \cdot \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK} \]

\[ \frac{h_c}{t_{we}} \leq 1.57 \cdot \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK} \]

The sections satisfy Local Buckling Requirements (Table 1-2; Seismic Design Manual)
6.2 - Floors 3 and 4 (Sized based on Drift Allowables):

\[ V_T := F_4 + F_5 + F_6 = 839.81 \text{ kip} \]

\[ V_{TB} := F_3 + F_4 + F_5 + F_6 = 971.42 \text{ kip} \]

Assume \( I_C = I_b = 1.0 \)

\[ I_C := 1.0 \cdot \text{in}^4 \]

\[ K_e := \frac{6 \cdot E}{(h_2 \cdot ft)^2} \left( \frac{1}{\frac{h_2 \cdot ft}{2 \cdot I_C} + \frac{I_b}{I_C}} \right) = 0.0163 \frac{\text{kip}}{\text{in}} \]

\[ K_i := \frac{12 \cdot E}{(h_2 \cdot ft)^2} \left( \frac{1}{\frac{h_2 \cdot ft}{I_C} + \frac{I_b}{I_C}} \right) = 0.0277 \frac{\text{kip}}{\text{in}} \]

\[ K_{TOT} := 2 \cdot K_e + NIC \cdot K_i = 0.1712 \frac{\text{kip}}{\text{in}} \]

NIC: Number of Interior Columns

\[ V_{e2} := \frac{K_e}{K_{TOT}} \cdot V_T = 80.071 \text{ kip} \]

\[ V_{i2} := \frac{K_i}{K_{TOT}} \cdot V_T = 135.934 \text{ kip} \]

\[ V_{e92} := \frac{K_e}{K_{TOT}} \cdot V_{TB} = 92.619 \text{ kip} \]

\[ V_{i92} := \frac{K_i}{K_{TOT}} \cdot V_{TB} = 157.237 \text{ kip} \]

Calc of Drift in terms of I:

\[ \Delta_e := \frac{V_{e2} + V_{e92}}{2}, \quad \Delta_i := \frac{V_{i2} + V_{i92}}{2} \]

\[ \Delta_e := \frac{(V_{e2} + V_{e92})}{2} \cdot \left( \frac{h_2 \cdot ft}{6 \cdot E} \right)^2 \left( \frac{h_2 \cdot ft}{2 \cdot I_C} + \frac{I_b}{I_C} \right) = 5289.438 \text{ in}^4 \cdot \text{in} \]

\[ \Delta_i := 0.02 \cdot (h_2 \cdot ft) = 3.12 \text{ in} \]

\[ C_d := 5.5 \]
\[
\Delta_{ae} = \frac{\Delta_a}{C_d} = 0.567 \text{ in}
\]

\[
I_{tgd} = \frac{\Delta_e}{\Delta_{ae}} = 9324.33 \text{ in}^4
\]

Choose column and beams from AISC:

**Beam W24 x 279**

\[
I = 9600 \text{ in}^4
\]

\[
d_s = 26.7 \text{ in} \quad k_{bdes} = 2.59 \text{ in}
\]

\[
b_{sf} = 13.3 \text{ in} \quad t_{wbf} = 1.16 \text{ in}
\]

\[
t_{sf} = 2.09 \text{ in} \quad h_b = d_b - 2 \cdot k_{bdes} = 21.52 \text{ in}
\]

Compactness Verification:

\[
\frac{b_{sf}}{2 \cdot t_{sf}} \leq 0.32 \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK}
\]

\[
\frac{h_b}{t_{wbf}} \leq 1.57 \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK}
\]

**Column W33 x 354**

\[
I = 22000 \text{ in}^4
\]

\[
d_c = 35.6 \text{ in} \quad k_{cdes} = 2.88 \text{ in}
\]

\[
b_{cf} = 16.1 \text{ in} \quad t_{wce} = 1.16 \text{ in}
\]

\[
t_{cf} = 2.09 \text{ in} \quad h_c = d_c - 2 \cdot k_{cdes} = 29.84 \text{ in}
\]

Compactness Verification:

\[
\frac{b_{cf}}{2 \cdot t_{cf}} \leq 0.32 \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK}
\]

\[
\frac{h_c}{t_{wce}} \leq 1.57 \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK}
\]

The sections satisfy Local Buckling Requirements (Table 1-2; Seismic Design Manual)
6.3 - Floors 5 and 6 (Sized based on Drift Allowables):

\[ V_T := F_6 = 375.856 \text{ kip} \]

\[ V_{TB} := F_5 + F_6 = 643.514 \text{ kip} \]

Assume \( I_c = I_b = 1.0 \):

\[ I_c := 1.0 \cdot \text{in}^4 \]

\[ K_e := \frac{6 \cdot E}{(h_2 \cdot ft)^2} \left( \frac{1}{\frac{h_2 \cdot ft}{2 \cdot I_c} + \frac{L_b}{I_c}} \right) = 0.0163 \text{ kip in} \]

\[ K_i := \frac{12 \cdot E}{(h_2 \cdot ft)^2} \left( \frac{1}{\frac{h_2 \cdot ft}{I_c} + \frac{L_b}{I_c}} \right) = 0.0277 \text{ kip in} \]

\[ K_{TOT} := 2 \cdot K_e + NIC \cdot K_i = 0.1712 \text{ kip in} \]

NIC: Number of Interior Columns

\[ V_{e3} := \frac{K_e}{K_{TOT}} \cdot V_T = 35.836 \text{ kip} \]

\[ V_{i3} := \frac{K_i}{K_{TOT}} \cdot V_T = 60.837 \text{ kip} \]

\[ V_{eb3} := \frac{K_e}{K_{TOT}} \cdot V_{TB} = 61.355 \text{ kip} \]

\[ V_{ib3} := \frac{K_i}{K_{TOT}} \cdot V_{TB} = 104.161 \text{ kip} \]

**Calc of Drift in terms of \( I_c \):**

\[ I_c := 1 \]

\[ \Delta_e := \Delta_c \]

\[ \Delta_e := \frac{(V_{e3} + V_{eb3})}{2} \cdot \left( \frac{h_2 \cdot ft}{6 \cdot E} \right)^2 \cdot \left( \frac{h_2 \cdot ft}{2 \cdot I_c} + \frac{L_b}{I_c} \right) = 2976.926 \text{ in}^4 \cdot \text{in} \]

\[ \Delta_d := 0.02 \cdot (h_2 \cdot ft) = 3.12 \text{ in} \]
$C_d := 5.5$

$\Delta_{ac} := \frac{\Delta_a}{C_d} = 0.567 \text{ in}$

$I_{req} := \frac{\Delta_e}{\Delta_{ae}} = 5247.785 \text{ in}^4$

Choose column and beams from AISC:

**Beam W21 x 201**

$I = 5310 \text{ in}^4$

$d_b := 23 \cdot \text{in} \quad k_{iads} := 2.13 \cdot \text{in}$  

$b_{bf} := 12.6 \cdot \text{in} \quad t_{wb} := 0.91 \cdot \text{in}$  

$t_{bf} := 1.63 \cdot \text{in} \quad h_b := d_b - 2 \cdot k_{iads} = 18.74 \text{ in}$

**Column W33 x 241**

$I = 14200 \text{ in}^4$

$d_c := 34.2 \cdot \text{in} \quad k_{cads} := 2.19 \cdot \text{in}$  

$b_{cf} := 15.9 \cdot \text{in} \quad t_{wc} := 0.83 \cdot \text{in}$  

$t_{cf} := 1.40 \cdot \text{in} \quad h_c := d_c - 2 \cdot k_{cads} = 29.82 \text{ in}$

**Compactness Verification:**

$$\frac{b_{bf}}{2 \cdot t_{bf}} \leq 0.32 \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK}$$

$$\frac{b_{cf}}{2 \cdot t_{cf}} \leq 0.32 \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK}$$

$$\frac{h_b}{t_{wb}} \leq 1.57 \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK}$$

$$\frac{h_c}{t_{wc}} \leq 1.57 \sqrt{\frac{E}{1.1 \cdot F_y}} = 1 \quad \text{OK}$$

The sections satisfy Local Buckling Requirements (Table 1-2; Seismic Design Manual)
7 - RBS Design:

7.1 - Floors 1 and 2:

Beam W24 x 335

\[ b_{yf} = 13.5 \text{ in} \quad d_b = 27.5 \text{ in} \quad t_{yf} = 2.48 \text{ in} \quad Z_x = 1020 \text{ in}^3 \]

Column W33 x 387

\[ d_{ct} = 36 \text{ in} \quad t_{w} = 1.26 \text{ in} \quad b_{cf} = 16.2 \text{ in} \quad t_{cf} = 2.28 \text{ in} \quad A_c = 114 \text{ in}^2 \quad Z_c = 1560 \text{ in}^3 \quad r_{pc} = 3.77 \text{ in} \]

\[ a = 0.5 \cdot b_{yf} = 6.75 \text{ in} \quad a = 0.75 \cdot b_{yf} = 10.125 \text{ in} \quad a = 6.75 \text{ in} \]

\[ b = 0.65 \cdot d_b = 17.875 \text{ in} \quad b = 0.85 \cdot d_b = 23.375 \text{ in} \quad b = 18.0 \text{ in} \]

\[ c = 0.1 \cdot b_{yf} = 1.35 \text{ in} \quad c = 0.25 \cdot b_{yf} = 3.375 \text{ in} \quad c = 3.375 \text{ in} \]

\[ Z_{RBS} = Z_x - 2 \cdot c \cdot t_{yf} \cdot (d_b - t_{yf}) = 601.165 \text{ in}^3 \]

\[ C_{pr} = \frac{F_g + F_u}{2 \cdot F_y} = 1.15 < 1.2 \]

\[ R = \frac{4 \cdot c^2 + b^2}{8 \cdot c} = 13.6875 \text{ in} \]

\[ R_y = 1.1 \quad \text{Table A3.1 - AISC Seismic Provisions} \]

\[ M_{pr} = C_{pr} \cdot R_y \cdot Z_{RBS} \cdot F_y = 38023.699 \text{ kip \cdot in} \]

\[ V_p = \frac{M_{pr}}{L_b - d_{ct} - 2 \cdot \left( a + \frac{b}{2} \right)} = 259.991 \text{ kip} \]

\[ M_{f1} = M_{pr} + V_p \cdot \left( a + \frac{b}{2} \right) = 42118.559 \text{ kip \cdot in} \]

**Capacity at Column face:**

\[ M_{pe} = R_y \cdot Z_x \cdot F_y = 56100 \text{ kip \cdot in} \]

Criteria Check:

\[ M_{pe} \geq M_{f1} = 1 \quad \text{OK} \]

\[ \frac{M_{pe}}{M_{f1}} = 1.332 \]
Strong Column Verification for an Interior Column W33x:

\[ Z_b = Z_x = 1020 \text{ } in^3 \quad KL = 156 \cdot in \]

\[ L := L_b = 360 \text{ } in \]

\[ F_e := \left( \frac{\pi \cdot \pi \cdot E}{KL} \right) = 167.16 \text{ } ksi \]

\[ \phi F_{cr} := 0.9 \cdot F_y \cdot 658 \left( \frac{F_y}{F_y} \right) = 39.705 \text{ } ksi \]

\[ \phi P_u := \phi F_{cr} \cdot A_c = 4526.33 \text{ kip} \]

\[ P_u := 0.5 \cdot \phi P_u = 2263.164 \text{ kip} \]

\[ Z_{RBS} := Z_b - 2 \cdot c \cdot t_{bf} \cdot (d_b - t_{bf}) = 601.165 \text{ } in^3 \]

\[ C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15 \]

\[ M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 38023.699 \text{ kip} \cdot in \]

\[ V_{RBS} := \frac{2 \cdot M_{pr}}{\left( L - d_{cl} - 2 \cdot \left( a + \frac{b}{2} \right) \right)} = 259.991 \text{ kip} \]

\[ M_v := 2 \cdot V_{RBS} \left( a + \frac{b}{2} + \frac{d_{cl}}{2} \right) = 17549.399 \text{ kip} \cdot in \]

\[ M_{pb} := 2 \cdot M_{pr} + M_v = 93596.797 \text{ kip} \cdot in \]

\[ M_{pc} := 2 \cdot Z_c \left( F_y - \frac{P_u}{A_c} \right) = 94060.787 \text{ kip} \cdot in \]

\[ \frac{M_{pc}}{M_{pb}} > 1.0 = 1 \]

\[ \frac{M_{pc}}{M_{pb}} = 1.005 \]
7.2 - Floors 3 and 4:

Beam W24 x 279

\[ b_{bf} = 13.3 \text{ in} \quad d_b = 26.7 \text{ in} \quad t_{bf} = 2.09 \text{ in} \quad Z_z = 835 \text{ in}^3 \]

Column W33 x 354

\[ d_{cl} = 35.6 \text{ in} \quad t_w = 1.16 \text{ in} \quad b_{cf} = 16.1 \text{ in} \quad t_{cf} = 2.09 \text{ in} \quad A_c = 104 \text{ in}^2 \quad Z_c = 1420 \text{ in}^3 \quad r_p = 3.74 \text{ in} \]

\[ a = 0.5 \cdot b_{bf} = 6.65 \text{ in} \quad a = 0.75 \cdot b_{bf} = 9.975 \text{ in} \quad a = 6.75 \text{ in} \]

\[ b = 0.65 \cdot d_b = 17.355 \text{ in} \quad b = 0.85 \cdot d_b = 22.695 \text{ in} \quad b = 18.0 \text{ in} \]

\[ c = 0.1 \cdot b_{bf} = 1.33 \text{ in} \quad c = 0.25 \cdot b_{bf} = 3.325 \text{ in} \quad c = 3.25 \text{ in} \]

\[ Z_{RBS} = Z_z - 2 \cdot c \cdot t_{bf} \cdot (d_b - t_{bf}) = 500.673 \text{ in}^3 \]

\[ R = \frac{4 \cdot c^2 + b^2}{8 \cdot c} = 14.0865 \text{ in} \]

\[ C_{pr} = \frac{F_y + F_u}{2 \cdot F_y} = 1.15 \quad < 1.2 \]

\[ R_y = 1.1 \quad \text{Table A3.1 - AISC Seismic Provisions} \]

\[ M_{pr} = C_{pr} \cdot R_y \cdot Z_{RBS} \cdot F_y = 31667.577 \text{ kip \cdot in} \]

\[ V_p = \frac{M_{pr}}{I_b - d_{cl} - 2 \cdot \left( \frac{a + b}{2} \right)} = 216.235 \text{ kip} \]

\[ M_{f2} = M_{pr} + V_p \cdot \left( \frac{a + b}{2} \right) = 35073.274 \text{ kip \cdot in} \]

**Capacity at Column face:**

\[ M_{pc} = R_y \cdot Z_z \cdot F_y = 45925 \text{ kip \cdot in} \]

Criteria check:

\[ M_{pr} \geq M_{f2} = 1 \quad \text{OK} \]

\[ \frac{M_{pr}}{M_{f2}} = 1.309 \]
Strong Column Verification for an Interior Column W33x:

\[ Z_b := Z_e = 835 \text{ in}^3 \quad KL := 156 \cdot \text{in} \]

\[ L := L_b = 360 \text{ in} \]

\[ F_e := \left( \frac{\pi \cdot E}{KL} \right)^2 \cdot \frac{F_y}{F_y} = 164.51 \text{ ksi} \]

\[ \phi F_{cr} := 0.9 \cdot F_y \cdot \frac{F_y}{F_y} = 39.625 \text{ ksi} \]

\[ \phi P_u := \phi F_{cr} \cdot A_c = 4120.96 \text{ kip} \]

\[ 50 \% \quad P_u := 0.5 \cdot \phi P_u = 2060.481 \text{ kip} \]

\[ Z_{RBS} := Z_b - 2 \cdot c \cdot t_{bf} \cdot (d_b - t_{bf}) = 500.673 \text{ in}^3 \]

\[ C_r := \frac{F_y + F_u}{2 \cdot F_y} = 1.15 \]

\[ M_{pr} := C_r \cdot R_y \cdot F_y \cdot Z_{RBS} = 31667.577 \text{ kip \cdot in} \]

\[ V_{RBS} := \frac{2 \cdot M_{pr}}{L - d_{cl} - 2 \cdot \left( a + \frac{b}{2} \right)} = 216.235 \text{ kip} \]

\[ M_v := 2 \cdot V_{RBS} \cdot \left( a + \frac{b}{2} + \frac{d_{cl}}{2} \right) = 14509.351 \text{ kip \cdot in} \]

\[ M_{pb} := 2 \cdot M_{pr} + M_v = 77844.504 \text{ kip \cdot in} \]

\[ M_{pc} := 2 \cdot Z_c \cdot \left( F_y - \frac{P_u}{A_c} \right) = 85733.013 \text{ kip \cdot in} \]

\[ \frac{M_{pc}}{M_{pb}} > 1.0 = 1 \]

\[ \frac{M_{pc}}{M_{pb}} = 1.101 \]
7.3 - Floors 5 and 6:

Beam W21 x 201

\[ b_{bf} = 12.6 \cdot \text{in} \quad d_b = 23.0 \cdot \text{in} \quad t_{bf} = 1.63 \cdot \text{in} \quad Z_x = 530 \cdot \text{in}^3 \]

Column W33x 241

\[ d_{c1} = 34.2 \cdot \text{in} \quad t_w = 0.83 \cdot \text{in} \quad b_{cf} = 15.9 \cdot \text{in} \quad t_{cf} = 1.4 \cdot \text{in} \quad A_c = 71.1 \cdot \text{in}^2 \quad Z_c = 940 \cdot \text{in}^3 \quad r_{yc} = 3.62 \cdot \text{in} \]

\[ a = 0.5 \cdot b_{bf} = 6.3 \cdot \text{in} \quad a = 0.75 \cdot b_{bf} = 9.45 \cdot \text{in} \quad a = 6.75 \cdot \text{in} \]

\[ b = 0.65 \cdot d_b = 14.95 \cdot \text{in} \quad b = 0.85 \cdot d_b = 19.55 \cdot \text{in} \quad b = 18.0 \cdot \text{in} \]

\[ c = 0.1 \cdot b_{bf} = 1.26 \cdot \text{in} \quad c = 0.25 \cdot b_{bf} = 3.15 \cdot \text{in} \quad c = 3.0 \cdot \text{in} \]

\[ Z_{RBS} = Z_x - 2 \cdot c \cdot t_{bf} \cdot (d_b - t_{bf}) = 321.001 \cdot \text{in}^3 \]

\[ C_{pr} = \frac{F_y + F_u}{2 \cdot F_y} = 1.15 < 1.2 \]

\[ R_y = 1.1 \]

\[ M_{pr} = C_{pr} \cdot R_y \cdot Z_{RBS} \cdot F_y = 20303.339 \text{ kip \cdot in} \]

\[ V_p = \frac{M_{pr}}{2 \left( d_b - d_{c1} - 2 \cdot \left( a + \frac{b}{2} \right) \right)} = 137.977 \text{ kip} \]

\[ M_{f3} = M_{pr} + V_p \left( a + \frac{b}{2} \right) = 22476.479 \text{ kip \cdot in} \]

Capacity at Column face:

\[ M_{pr} = R_y \cdot Z_x \cdot F_y = 29150 \text{ kip \cdot in} \]

Criteria Check:

\[ M_{pr} \geq M_{f3} = 1 \quad \text{OK} \]

\[ M_{pe} = 1.297 \]
Strong Column Verification for an Interior Column W33x:

\[ Z_b = Z_z = 530 \text{ in}^3 \quad KL = 156 \text{ in} \]

\[ L = L_b = 360 \text{ in} \]

\[ F_c = \frac{\pi \cdot r_y \cdot E}{KL} = 154.122 \text{ ksi} \quad \phi F_{cr} = 0.9 \cdot F_y \cdot \frac{F_y}{r_y} = 39.286 \text{ ksi} \]

\[ \phi P_u = \phi F_{cr} \cdot A_c = 2793.26 \text{ kip} \quad P_u = 0.5 \cdot \phi P_u = 1396.63 \text{ kip} \]

\[ Z_{RBS} = Z_b - 2 \cdot c \cdot t_{bf} \cdot (d_b - t_{bf}) = 321.001 \text{ in}^3 \]

\[ C_{pr} = \frac{F_y + F_u}{2 \cdot F_y} = 1.15 \]

\[ M_{pr} = C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 20303.339 \text{ kip} \cdot \text{in} \]

\[ V_{RBS} = \frac{2 \cdot M_{pr}}{L - d_{cl} - 2 \cdot \left(\frac{a + b}{2}\right)} = 137.977 \text{ kip} \]

\[ M_v = 2 \cdot V_{RBS} \cdot \left(\frac{a + b}{2} + \frac{d_{cl}}{2}\right) = 9065.099 \text{ kip} \cdot \text{in} \]

\[ M_{pb} = 2 \cdot M_{pr} + M_v = 49671.776 \text{ kip} \cdot \text{in} \]

\[ M_{pc} = 2 \cdot Z_c \cdot \left(1 - \frac{P_u}{A_c}\right) = 57070.82 \text{ kip} \cdot \text{in} \quad \frac{M_{pc}}{M_{pb}} > 1.0 = 1 \quad \frac{M_{pc}}{M_{pb}} = 1.149 \]
8 - Panel Zone Design

8.1 - Levels 1 and 2

8.1.1 - Design for Exterior Columns:

\[ M_{fl} = 42118.559 \text{ kip} \cdot \text{in} \]

Beam W24 x 335

\[ d_b = 27.5 \cdot \text{in} \quad b_{bf} = 13.5 \cdot \text{in} \quad t_{bf} = 2.48 \cdot \text{in} \]

Column W33 x 387

\[ d_{c1} = 36 \cdot \text{in} \quad t_w = 1.26 \cdot \text{in} \quad b_{cf} = 16.2 \cdot \text{in} \quad t_{cf} = 2.28 \cdot \text{in} \quad A_c = 114 \cdot \text{in}^2 \]

\[ \Sigma M_{0.5b} = 0 \]

\[ V_c = \frac{M_{fl}}{h_1 \cdot ft + h_2 \cdot ft} = 269.991 \text{ kip} \]

\[ \Sigma Fx = 0 \]

\[ R_u = \frac{M_{fl}}{d_b - t_{bf}} = 1683.396 \text{ kip} \]

8.1.1.1 - Capacity of Panel Zone:

\[ V_c = \text{Column Shear} \]

\[ \phi_v = 1.0 \]

\[ \phi R_n = \phi_v \cdot 0.6 \cdot F_y \cdot d_{c1} \cdot t_w \left( 1 + \frac{3 \cdot b_{cf} \cdot t_{cf}^2}{d_b \cdot d_{c1} \cdot t_w} \right) = 1636.41 \text{ kip} \]

\[ \phi R_n < 0.75 \cdot F_y \cdot A_c = 1 \quad \text{OK} \]
Criteria Check:

\[ \phi R_y \geq R_y = 0 \quad \text{NG} \]

**Need to design Doubler Plate**

**Design of Doubler Plates:**

\[ t_w \geq \frac{(d_z + w_z)}{90} \]

Beam W24 x 335

\[ \frac{d_z}{90} = 0.25 \text{ in} \quad d_z = 0.25 \cdot \text{in} \cdot 90 = 22.5 \text{ in} \]

Column W33 x 387

\[ t_w = 1.26 \cdot \text{in} \]

\[ \frac{w_z}{90} = 0.349 \text{ in} \quad w_z = 0.349 \cdot \text{in} \cdot 90 = 31.41 \text{ in} \]

\[ t_w \geq \frac{(d_z + w_z)}{90} = 1 \quad \text{The column web satisfies the minimum requirement.} \]

Now, \( t_w \) is replaced by \( t_w + t_p \):

\[ t_w + t_p \geq \left( R_y - \frac{0.6 \cdot F_y \cdot (3 \cdot b_{cf} \cdot t_{cf}^2)}{d_b} \right) \cdot \left( \frac{1}{0.6 \cdot F_y \cdot d_{cl}} \right) \]

\[ t_p = \left( R_y - \frac{0.6 \cdot F_y \cdot (3 \cdot b_{cf} \cdot t_{cf}^2)}{d_b} \right) \cdot \left( \frac{1}{0.6 \cdot F_y \cdot d_{cl}} \right) - t_w = 0.044 \text{ in} \]

\[ t_{cp} = \frac{d_z}{90} + \frac{w_z}{90} = 0.599 \text{ in} \]

Use \( t_{cp} = 0.625 \text{ in} \)

**8.1.1.2 - Design of Continuity Plates:**

\[ t_{cf} = 2.28 \text{ in} \quad R_{yc} = 1.1 \]

\[ R_{ybf} = R_{yc} \]

\[ t_{cf} \geq 0.4 \cdot \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{ybf} \cdot F_y}{R_{yc} \cdot F_y}} = 0 \quad \text{NG} \]

\[ t_{cfi} = 0.4 \cdot \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{ybf} \cdot F_y}{R_{yc} \cdot F_y}} = 3.105 \text{ in} \]
\[ t_{cf} \geq \frac{b_{bf}}{6} = 1 \quad \text{OK} \]

**Need to design continuity plates.**

One half of the thicker beam flange is used to address the range of demands.

\[ t_{cp} = \left( \frac{1}{2} \right) \cdot t_{bf} = 1.24 \text{ in} \quad t_{cp} = \frac{1}{4} \cdot t_{in} \quad t_{cp} = \text{Thickness of Continuity Plates} \]

### 8.1.2 - Panel Zone Design for Interior Columns:

\[ M_{f1} = 42118.559 \text{ kip in} \]

\[ \Sigma M_{0.5b} = 0 \]

\[ V_c = \frac{2 \cdot M_{f1}}{h_1 \cdot ft} + \frac{h_2 \cdot ft}{2} = 539.982 \text{ kip} \]

\[ \Sigma F_x = 0 \]

\[ R_u = \frac{2 \cdot M_{f1}}{d_b - t_{bf}} = 3366.791 \text{ kip} \]

#### 8.1.2.1 - Capacity of Panel Zone:

\[ \phi_v = 1.0 \]

\[ \phi R_n = \phi_v \cdot 0.6 \cdot F_y \cdot d_{c1} \cdot t_w \cdot \left( 1 + \frac{3 \cdot b_{cf} \cdot t_{cf}^2}{d_b \cdot d_{c1} \cdot t_w} \right) = 1636.41 \text{ kip} \]

\[ \phi R_n < 0.75 \cdot F_y \cdot A_c = 1 \quad \text{OK} \]
Criteria Check:

$$\phi R_u \geq R_u = 0$$  \hspace{1cm} \text{NG}$$

**Design of Doubler Plates:**

$$t_w \geq \frac{(d_z + w_z)}{90}$$

Beam W24 x 335

$$\frac{d_z}{90} = 0.25 \text{ in} \hspace{1cm} d_z = 0.25 \cdot \text{in} \cdot 90 = 22.5 \text{ in}$$

Column W33 x 387

$$\frac{w_z}{90} = 0.349 \text{ in} \hspace{1cm} w_z = 0.349 \cdot \text{in} \cdot 90 = 31.41 \text{ in}$$

$$t_w \geq \frac{(d_z + w_z)}{90} = 1$$  \hspace{1cm} The column web satisfies the minimum requirement.

Now, $t_w$ is replaced by $t_w + t_p$:

$$t_w + t_p \geq \left( R_u - \frac{0.6 \cdot F_y \cdot (3 \cdot b_{cf} \cdot t_{cf}^2)}{d_b} \right) \cdot \left( \frac{1}{0.6 \cdot F_y \cdot d_{c1}} \right)$$

$$t_p = \left( R_u - \frac{0.6 \cdot F_y \cdot (3 \cdot b_{cf} \cdot t_{cf}^2)}{d_b} \right) \cdot \left( \frac{1}{0.6 \cdot F_y \cdot d_{c1}} \right) - t_w = 1.602 \text{ in}$$

Use a 2.885-in-thick doubler plate (1.26 inches + 1 5/8 inches)

Use $t_{cp} = 0.625 \text{ in}$

**8.1.2.2 - Design of Continuity Plates:**

$$t_{cf} = 2.28 \text{ in} \hspace{1cm} R_{yc} = 1.1$$

$$R_{yb} = R_{yc}$$

$$t_{cf} \geq 0.4 \cdot \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_y}{R_{yc} \cdot F_y}} = 0$$  \hspace{1cm} \text{NG}$$

$$t_{cf} = 0.4 \cdot \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_y}{R_{yc} \cdot F_y}} = 3.105 \text{ in}$$

$$t_{cf} \geq \frac{b_{bf}}{6} = 1$$  \hspace{1cm} \text{OK}$$
Need to design continuity plates.

Three-quarters of the thicker beam flange is used to address the range of demands.

\[ t_{cp} = \left(\frac{3}{4}\right) \cdot t_{bf} = 1.86 \text{ in} \quad t_{cp} = 1 \frac{7}{8} \text{ in} \]

\[ t_{cp} = \text{Thickness of Continuity Plates} \]

8.2 - Levels 3 and 4

8.2.1 - Panel Zone Design for Exterior Columns:

Moment at the face of the Column: \( M_f \)

\[ M_{f2} = 35073.274 \text{ kip} \cdot \text{in} \]

Beam W24 x 279

\[ d_b = 26.7 \text{ in} \quad b_{bf} = 13.3 \text{ in} \quad t_{bf} = 2.09 \text{ in} \]

Column W33 x 354

\[ d_{c1} = 35.6 \text{ in} \quad t_w = 1.16 \text{ in} \quad b_{cf} = 16.1 \text{ in} \quad t_{cf} = 2.09 \text{ in} \quad A_c = 104 \text{ in}^2 \]

\[ \Sigma M_{0,sh} = 0 \]

\[ V_c = \frac{M_{f2}}{h_2 \cdot ft} = 224.829 \text{ kip} \]

\[ \Sigma F_X = 0 \]

\[ R_u = \frac{M_{f2}}{d_b - t_{bf}} = 1425.164 \text{ kip} \]
8.2.1.1 - Capacity of Panel Zone:

\[ \phi R_n = \phi_v \cdot 0.6 \cdot F_y \cdot d_{c1} \cdot t_{w1} \left( 1 + \frac{3 \cdot b_{cf} \cdot t_{cf}}{d_b \cdot d_{c1} \cdot t_w} \right) = 1475.935 \text{ kip} \]

\[ \phi R_n < 0.75 \cdot F_y \cdot A_e = 1 \quad \text{OK} \]

Criteria Check:

\[ \phi R_n \geq R_u = 1 \quad \text{OK} \]

**No need to design Doubler Plate**

8.2.1.2 - Design of Continuity Plates:

\[ t_{cf} = 2.09 \text{ in} \quad R_{yc} = 1.1 \]

\[ R_{yb} = R_{yc} \]

\[ t_{cf} \geq 0.4 \cdot \sqrt{1.8 \cdot b_{cf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_y}{R_{yc} \cdot F_y}} = 0 \quad \text{NG} \]

\[ t_{cfi} = 0.4 \cdot \sqrt{1.8 \cdot b_{cf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_y}{R_{yc} \cdot F_y}} = 2.829 \text{ in} \]

\[ t_{cf} \geq \frac{b_{bf}}{6} = 0 \quad \text{NG} \]

**Need to design continuity plates.**

One half of the thicker beam flange is used to address the range of demands.

\[ t_{cp} = \left( \frac{1}{2} \right) \cdot t_{bf} = 1.045 \text{ in} \quad t_{cp} = 1 \cdot \frac{1}{8} \cdot \text{in} \]

\[ t_{cp} = \text{Thickness of Continuity Plates} \]

8.2.2 - Panel Zone Design for Interior Columns:
Moment at the face of the Column: \( M_f \)

\[
M_f = 35073.274 \text{ kip} \cdot \text{in}
\]

\[
\Sigma M_{0.5h} = 0
\]

\[
V_c := \frac{2 \cdot M_f}{h_2 \cdot ft} = 449.657 \text{ kip}
\]

\[
\Sigma F_X = 0
\]

\[
R_u := \frac{2 \cdot M_f}{d_b - t_{bf}} = 2850.327 \text{ kip}
\]

8.2.2.1 - Capacity of Panel Zone:

\[
\phi_y := 1.0
\]

\[
\phi R_n := \phi_y \cdot 0.6 \cdot F_y \cdot d_{c1} \cdot t_w \cdot \left( 1 + \frac{3 \cdot b_{cf} \cdot t_{cf}^2}{d_b \cdot d_{c1} \cdot t_w} \right) = 1475.935 \text{ kip}
\]

\[
\phi R_n < 0.75 \cdot F_y \cdot A_c = 1 \quad \text{OK}
\]

Criteria Check:

\[
\phi R_n \geq R_u = 0 \quad \text{NG}
\]

**Design of Doubler Plates:**

\[
t_w \geq \frac{(d_z + w_z)}{90}
\]

Beam W24 x 279

\[
\frac{d_z}{90} = 0.25 \text{ in} \quad \text{Page 4-124, Table 4-2}
\]

\[
d_z = 0.25 \cdot \text{in} \cdot 90 = 22.5 \text{ in} \quad \text{AISC Seismic Desing Manual}
\]

Column W33 x 354

\[
t_w := 1.16 \cdot \text{in}
\]

\[
\frac{w_z}{90} = 0.349 \text{ in}
\]

\[
w_z = 0.349 \cdot \text{in} \cdot 90 = 31.41 \text{ in}
\]

\[
t_w \geq \frac{(d_z + w_z)}{90} = 1
\]

The column web satisfies the minimum requirement.

Now, \( t_w \) is replaced by \( t_w + t_p \):
\[ t_w + t_p \geq \left( \frac{R_u - 0.6 \cdot F_y \cdot (3 \cdot b_{cf} \cdot t_{cf}^2)}{d_b} \right) \cdot \left( \frac{1}{0.6 \cdot F_y \cdot d_{c1}} \right) \]

\[ t_p = \left( \frac{R_u - 0.6 \cdot F_y \cdot (3 \cdot b_{cf} \cdot t_{cf}^2)}{d_b} \right) \cdot \left( \frac{1}{0.6 \cdot F_y \cdot d_{c1}} \right) - t_w = 1.287 \text{ in} \]

Use a 2.535-in-thick doubler plate (1.16 inches + 1 3/8 inches)

8.2.2.2 - Design of Continuity Plates:

\[ t_{cf} = 2.09 \text{ in} \quad R_{yc} = 1.1 \]

\[ R_{gb} = R_{yc} \]

\[ t_{cf} \geq 0.4 \cdot \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{gb} \cdot F_y}{R_{yc} \cdot F_y}} = 0 \quad \text{NG} \]

\[ t_{cf_{fi}} = 0.4 \cdot \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{gb} \cdot F_y}{R_{yc} \cdot F_y}} = 2.829 \text{ in} \]

\[ t_{cf} \geq \frac{b_{bf}}{6} = 0 \quad \text{NG} \]

**Need to design continuity plates.**

Three-quarters of the thicker beam flange is used to address the range of demands. Page 9.1-50; 9.1-243, Seismic Provisions, 2016

\[ t_{cp} = \left( \frac{3}{4} \right) \cdot t_{bf} = 1.568 \text{ in} \quad t_{cp} = 1 \frac{5}{8} \cdot \text{in} \quad t_{cp} = \text{Thickness of Continuity Plates} \]

8.3 - Level 5 and 6

8.3.1 - Panel Zone Design for Exterior Columns:
Moment at the face of the Column: Mf
\[ M_f = 22476.479 \text{ kip} 
\]

Beam W21 x 201
\[ d_b = 23 \cdot \text{in} \quad b_{bf} = 12.6 \cdot \text{in} \quad t_{bf} = 1.63 \cdot \text{in} \]

Column W33x 241
\[ d_{c1} = 34.2 \cdot \text{in} \quad t_w = 0.83 \cdot \text{in} \quad b_{cf} = 15.9 \cdot \text{in} \quad t_{cf} = 1.4 \cdot \text{in} \quad A_c = 71.1 \cdot \text{in}^2 \]

\[ \Sigma M_{0.5h} = 0 \]
\[ V_c := \frac{M_f}{h \cdot f_t} = 144.08 \text{ kip} \]
\[ \Sigma F_x = 0 \]
\[ R_u := \frac{M_f}{d_b - t_{bf}} = 1051.777 \text{ kip} \]

**8.3.1.1 - Capacity of Panel Zone:**

\[ \phi_w = 1.0 \]

\[ \phi R_n := \phi_w \cdot 0.6 \cdot F_y \cdot d_{c1} \cdot t_w \cdot \left( 1 + \frac{3 \cdot b_{cf} \cdot t_{cf}^2}{d_b \cdot d_{c1} \cdot t_w} \right) = 973.526 \text{ kip} \]

\[ \phi R_n < 0.75 \cdot F_y \cdot A_c = 1 \quad \text{OK} \]

Criteria Check:
\[ \phi R_n \geq R_u = 0 \quad \text{NG} \]

**Need to design Doubler Plate**

**Design of Doubler Plates:**

\[ t_w \geq \frac{(d_z + w_z)}{90} \]

Beam W21 x 201
\[ \frac{d_z}{90} = 0.219 \text{ in} \quad \frac{d_z}{90} = 0.219 \cdot \text{in} = 19.71 \text{ in} \quad \text{Page 4-124, Table 4-2, AISC Seismic Desing Manual} \]

Column W33 x 241
\[ t_w = 0.83 \cdot \text{in} \]
\[ \frac{w_z}{90} = 0.349 \text{ in} \quad \frac{w_z}{90} = 0.349 \cdot \text{in} = 31.41 \text{ in} \]
\[ t_w \geq \frac{(d_z + w_z)}{90} = 1 \] The column web satisfies the minimum requirement.

Now, \( t_w \) is replaced by \( t_w + t_p \):

\[
t_w + t_p \geq \left( R_u - \frac{0.6 \cdot F_y \cdot (3 \cdot b_{cf} \cdot t_{cf}^2)}{d_b} \right) \cdot \left( \frac{1}{0.6 \cdot F_y \cdot d_{el}} \right)
\]

\[
t_p = \left( R_u - \frac{0.6 \cdot F_y \cdot (3 \cdot b_{cf} \cdot t_{cf}^2)}{d_b} \right) \cdot \left( \frac{1}{0.6 \cdot F_y \cdot d_{el}} \right) - t_w = 0.076 \text{ in}
\]

Use a 1.455-in-thick doubler plate (5/8 inches + 0.83 inches)

\[ t_{cp} = \frac{d_z}{90} + \frac{w_z}{90} = 0.568 \text{ in} \]

Use \( t_{cp} = 0.625 \text{ in} \)

**8.3.1.2 - Design of Continuity Plates:**

\[ t_{cf} = 1.4 \text{ in} \quad R_{yc} = 1.1 \]

\[ R_{yb} = R_{yc} \]

\[ t_{cf} \geq 0.4 \cdot \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_y}{R_{yc} \cdot F_y}} = 0 \quad \text{NG} \]

\[ t_{cf} \geq 0.4 \cdot \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_y}{R_{yc} \cdot F_y}} = 2.432 \text{ in} \]

\[ t_{cf} \geq \frac{b_{bf}}{6} = 0 \quad \text{NG} \]

**Need to design continuity plates.**

One half of the thicker beam flange is used to address the range of demands.  

\[ t_{cp} = \left( \frac{1}{2} \right) \cdot t_{bf} = 0.815 \text{ in} \]

\[ t_{cp} = \frac{7}{8} \text{ in} \]

\[ t_{cp} = \text{Thickness of Continuity Plates} \]

**8.3.2 - Panel Zone Design for Interior Columns:**
Moment at the face of the Column: $M_f$

$$M_f = 22476.479 \text{ kip} \cdot \text{in}$$

$$\Sigma M_{0.5h} = 0$$

$$V_c := \frac{2 \cdot M_f}{h_2 \cdot ft} = 288.16 \text{ kip}$$

$$\Sigma Fx = 0$$

$$R_u := \frac{2 \cdot M_f}{d_b - t_{bf}} = 2103.554 \text{ kip}$$

8.3.2.1 - Capacity of Panel Zone:

$$\phi_u := 1.0$$

$$\phi R_n := \phi v \cdot 0.6 \cdot F_y \cdot d_{c1} \cdot t_w \cdot \left( 1 + \frac{3 \cdot b_{cf} \cdot t_{cf}^2}{d_b \cdot d_{c1} \cdot t_w} \right) = 973.526 \text{ kip}$$

$$\phi R_n < 0.75 \cdot F_y \cdot A_c = 1$$

OK

Criteria Check:

$$\phi R_n \geq R_u = 0$$

NG

Design of Doubler Plates:

$$t_w \geq \frac{(d_z + w_z)}{90}$$

Beam W21 x 201

$$\frac{d_z}{90} = 0.219 \text{ in}$$

$$d_z = 0.219 \cdot \text{in} \cdot 90 = 19.71 \text{ in}$$

Page 4-124, Table 4-2
AISC Seismic Desing Manual

Column W33 x 241

$$\frac{w_z}{90} = 0.349 \text{ in}$$

$$w_z = 0.349 \cdot \text{in} \cdot 90 = 31.41 \text{ in}$$

$$t_w \geq \frac{(d_z + w_z)}{90} = 1$$

The column web satisfies the minimum requirement.

Now, $t_w$ is replaced by $t_w + t_p$: 
\[ t_w + t_p \geq \left( R_u - \frac{0.6 \cdot F_y \cdot (3 \cdot b_c f \cdot t_c f^2)}{d_b} \right) \cdot \left( \frac{1}{0.6 \cdot F_y \cdot d_c 1} \right) \]

\[ t_p := \left( R_u - \frac{0.6 \cdot F_y \cdot (3 \cdot b_c f \cdot t_c f^2)}{d_b} \right) \cdot \left( \frac{1}{0.6 \cdot F_y \cdot d_c 1} \right) = 1.101 \text{ in} \]

Use a 1.955-in-thick doubler plate (0.83 inches + 1 1/8 inches)

8.3.2.2 - Design of Continuity Plates:

\[ t_c f = 1.4 \text{ in} \quad R_{yc} := 1.1 \]

\[ R_{yb} := R_{yc} \]

\[ t_c f \geq 0.4 \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_y}{R_{yc} \cdot F_y}} = 0 \quad \text{NG} \]

\[ t_{cfl} := 0.4 \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_y}{R_{yc} \cdot F_y}} = 2.432 \text{ in} \]

\[ t_{cf} \geq \frac{b_{bf}}{6} = 0 \quad \text{NG} \]

Need to design continuity plates.

Three-quarters of the thicker beam flange is used to address the range of demands.


\[ t_{cp} := \left( \frac{3}{4} \right) \cdot t_{bf} = 1.223 \text{ in} \]

\[ t_{cp} := 1 \frac{1}{4} \cdot \text{in} \]

\[ t_{cp} = \text{Thickness of Continuity Plates} \]
DESIGN SUMMARY

Figure A-1. Beam and Column sizes for Plan B.
## Reduced Beam Section Dimensions

<table>
<thead>
<tr>
<th>Story</th>
<th>RBS Dimensions (in)</th>
<th>$M_{pe}/M_f$ Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 &amp; 2</td>
<td>6.75 18.0</td>
<td>3.25 1.306</td>
</tr>
<tr>
<td>3 &amp; 4</td>
<td>6.75 18.0</td>
<td>3.25 1.306</td>
</tr>
<tr>
<td>5 &amp; 6</td>
<td>6.75 18.0</td>
<td>3.00 1.297</td>
</tr>
</tbody>
</table>

### Doubler Plate Thickness

<table>
<thead>
<tr>
<th>Story</th>
<th>Interior Cols. Thickness (in)</th>
<th>Exterior Cols. Thickness (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 &amp; 2</td>
<td>1 5/8</td>
<td>5/8</td>
</tr>
<tr>
<td>3 &amp; 4</td>
<td>1 3/8</td>
<td>None</td>
</tr>
<tr>
<td>5 &amp; 6</td>
<td>1 1/8</td>
<td>5/8</td>
</tr>
</tbody>
</table>

### Continuity Plate Thickness

<table>
<thead>
<tr>
<th>Story</th>
<th>Interior Cols. Thickness (in)</th>
<th>Exterior Cols. Thickness (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 &amp; 2</td>
<td>1 7/8</td>
<td>1 1/4</td>
</tr>
<tr>
<td>3 &amp; 4</td>
<td>1 5/8</td>
<td>1 1/8</td>
</tr>
<tr>
<td>5 &amp; 6</td>
<td>1 1/4</td>
<td>7/8</td>
</tr>
</tbody>
</table>
APPENDIX B. PLANS CONSIDERED FOR THIS RESEARCH PROJECT

The next figures show all the plans used in this research project. All of them were created looking for different configurations of skew connections, and evaluate their behavior under seismic lateral loading.

Figure B-1. Floor Plan A configuration.
Figure B-2. Floor Plan B configuration.
Figure B-3. Floor Plan C configuration.
Figure B-4. Floor Plan D configuration.
Figure B-5. Traditional Floor Plan configuration.
APPENDIX C. CALCULATION OF SESIMIC ACCELERATION PARAMETERS FOR LOS ANGELES

1. Mapped Acceleration Parameters

*Risk-targeted Ground Motion (0.2 s):*

\[ C_{RS} := 0.897 \quad S_{SUH} := 2.205 \quad S_{SD} := 2.474 \text{g} \]

\[ S_{S1} := C_{RS} \cdot S_{SUH} = 1.978 \text{ g} \]

\[ S_S := \begin{cases} S_D & \text{if } S_{S1} > S_{SD} \\ S_{SD} & \text{if } S_{S1} < S_{SD} \end{cases} \]

\[ S_S = 1.978 \text{ g} \]

*Risk-targeted Ground Motion (1.0 s):*

\[ C_{RS} := 0.897 \quad S_{SUH} := 0.786 \quad S_{1D} := 0.784 \]

\[ S_{11} := C_{RS} \cdot S_{SUH} = 0.705 \]

\[ S_1 := \begin{cases} S_{1D} & \text{if } S_{11} > S_{1D} \\ S_{1D} & \text{if } S_{11} < S_{1D} \end{cases} \]

\[ S_1 = 0.705 \text{ g} \]

2. Site Coefficient and Risk-Targeted MCE \(_R\) Spectral Response Acceleration Parameters

*Site-adjusted MCEr (0.2 s):*

\[ F_a := 1.2 \]

\[ S_{MS} := F_a \cdot S_S = 2.373 \text{ g} \]
Site-adjusted MCEr (1.0 s):

\[ F_v := 1.4 \]

\[ S_{M1} := F_v \cdot S_1 = 0.987 \quad g \]

3. Design Spectral Acceleration Parameters

Design Ground Motion (0.2 s):

\[ S_{DS} := \left(\frac{2}{3}\right) \cdot S_{MS} = 1.582 \quad g \]

Design Ground Motion (1.0 s):

\[ S_{D1} := \left(\frac{2}{3}\right) \cdot S_{M1} = 0.658 \quad g \]
APPENDIX D. ULTRA-LOW CYCLE FATIGUE ANALYSIS RESULT PLOTS FOR RBS BARE FRAME AND COMPOSITE CONNECTIONS.

Figure D-1. Fatigue analysis for bare frame orthogonal shallow column (W14) connections with 0% $\phi_c$ at a) Bottom Flange b) Top Flange.

Figure D-2. Fatigue analysis for bare frame orthogonal shallow column (W14) connections with 10% $\phi_c$ at a) Bottom Flange b) Top Flange.

Figure D-3. Fatigue analysis for bare frame orthogonal shallow column (W14) connections with 25% $\phi_c$ at a) Bottom Flange b) Top Flange.
Figure D-4. Fatigue analysis for bare frame orthogonal shallow column (W14) connections with 50% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-5. Fatigue analysis for bare frame skewed (10°) shallow column (W14) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-6. Fatigue analysis for bare frame skewed (10°) shallow column (W14) connections with 10% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-7. Fatigue analysis for bare frame skewed (10°) shallow column (W14) connections with 25% $\phi_c$Pn at a) Bottom Flange b) Top Flange.

Figure D-8. Fatigue analysis for bare frame skewed (10°) shallow column (W14) connections with 50% $\phi_c$Pn at a) Bottom Flange b) Top Flange.

Figure D-9. Fatigue analysis for bare frame skewed (20°) shallow column (W14) connections with 0% $\phi_c$Pn at a) Bottom Flange b) Top Flange.
Figure D-10. Fatigue analysis for bare frame skewed (20°) shallow column (W14) connections with 10% $\phi_c$Pn at a) Bottom Flange b) Top Flange.

Figure D-11. Fatigue analysis for bare frame skewed (20°) shallow column (W14) connections with 25% $\phi_c$Pn at a) Bottom Flange b) Top Flange.

Figure D-12. Fatigue analysis for bare frame skewed (20°) shallow column (W14) connections with 50% $\phi_c$Pn at a) Bottom Flange b) Top Flange.
Figure D-13. Fatigue analysis for bare frame skewed (30°) shallow column (W14) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-14. Fatigue analysis for bare frame skewed (30°) shallow column (W14) connections with 10% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-15. Fatigue analysis for bare frame skewed (30°) shallow column (W14) connections with 25% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-16. Fatigue analysis for bare frame skewed (30°) shallow column (W14) connections with 50% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-17. Fatigue analysis for bare frame orthogonal medium column (W18) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-18. Fatigue analysis for bare frame orthogonal medium column (W18) connections with 10% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-19. Fatigue analysis for bare frame orthogonal medium column (W18) connections with 25% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-20. Fatigue analysis for bare frame orthogonal medium column (W18) connections with 50% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-21. Fatigue analysis for bare frame skewed (10°) medium column (W18) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-22. Fatigue analysis for bare frame skewed (10°) medium column (W18) connections with 10% $\phi_cP_n$ at a) Bottom Flange b) Top Flange.

Figure D-23. Fatigue analysis for bare frame skewed (10°) medium column (W18) connections with 25% $\phi_cP_n$ at a) Bottom Flange b) Top Flange.

Figure D-24. Fatigue analysis for bare frame skewed (10°) medium column (W18) connections with 50% $\phi_cP_n$ at a) Bottom Flange b) Top Flange.
Figure D-25. Fatigue analysis for bare frame skewed (20°) medium column (W18) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-26. Fatigue analysis for bare frame skewed (20°) medium column (W18) connections with 10% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-27. Fatigue analysis for bare frame skewed (20°) medium column (W18) connections with 25% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-28. Fatigue analysis for bare frame skewed (20°) medium column (W18) connections with 50% $\phi_c$Pn at a) Bottom Flange b) Top Flange.

Figure D-29. Fatigue analysis for bare frame skewed (30°) medium column (W18) connections with 0% $\phi_c$Pn at a) Bottom Flange b) Top Flange.

Figure D-30. Fatigue analysis for bare frame skewed (30°) medium column (W18) connections with 10% $\phi_c$Pn at a) Bottom Flange b) Top Flange.
Figure D-31. Fatigue analysis for bare frame skewed (30°) medium column (W18) connections with 25% $\phi_cP_n$ at a) Bottom Flange b) Top Flange.

Figure D-32. Fatigue analysis for bare frame skewed (30°) medium column (W18) connections with 50% $\phi_cP_n$ at a) Bottom Flange b) Top Flange.

Figure D-33. Fatigue analysis for bare frame orthogonal medium column (W24) connections with 0% $\phi_cP_n$ at a) Bottom Flange b) Top Flange.
Figure D-34. Fatigue analysis for bare frame orthogonal medium column (W24) connections with 10% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-35. Fatigue analysis for bare frame orthogonal medium column (W24) connections with 25% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-36. Fatigue analysis for bare frame orthogonal medium column (W24) connections with 50% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-37. Fatigue analysis for bare frame skewed (10°) medium column (W24) connections with 0% $\phi_cP_n$ at a) Bottom Flange b) Top Flange.

Figure D-38. Fatigue analysis for bare frame skewed (10°) medium column (W24) connections with 10% $\phi_cP_n$ at a) Bottom Flange b) Top Flange.

Figure D-39. Fatigue analysis for bare frame skewed (10°) medium column (W24) connections with 25% $\phi_cP_n$ at a) Bottom Flange b) Top Flange.
Figure D-40. Fatigue analysis for bare frame skewed (10°) medium column (W24) connections with 50% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-41. Fatigue analysis for bare frame skewed (20°) medium column (W24) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-42. Fatigue analysis for bare frame skewed (20°) medium column (W24) connections with 10% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-43. Fatigue analysis for bare frame skewed (20°) medium column (W24) connections with 25% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-44. Fatigue analysis for bare frame skewed (20°) medium column (W24) connections with 50% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-45. Fatigue analysis for bare frame skewed (30°) medium column (W24) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-46. Fatigue analysis for bare frame skewed (30°) medium column (W24) connections with 10% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-47. Fatigue analysis for bare frame skewed (30°) medium column (W24) connections with 25% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-48. Fatigue analysis for bare frame skewed (30°) medium column (W24) connections with 50% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-49. Fatigue analysis for bare frame orthogonal deep column (W33) connections with 0% $\phi_{cPn}$ at a) Bottom Flange b) Top Flange.

Figure D-50. Fatigue analysis for bare frame orthogonal deep column (W33) connections with 10% $\phi_{cPn}$ at a) Bottom Flange b) Top Flange.

Figure D-51. Fatigue analysis for bare frame orthogonal deep column (W33) connections with 25% $\phi_{cPn}$ at a) Bottom Flange b) Top Flange.
Figure D-52. Fatigue analysis for bare frame orthogonal deep column (W33) connections with 50% \( \phi_{cPn} \) at a) Bottom Flange b) Top Flange.

Figure D-53. Fatigue analysis for bare frame skewed (10°) deep column (W33) connections with 0% \( \phi_{cPn} \) at a) Bottom Flange b) Top Flange.

Figure D-54. Fatigue analysis for bare frame skewed (10°) deep column (W33) connections with 10% \( \phi_{cPn} \) at a) Bottom Flange b) Top Flange.
Figure D-55. Fatigue analysis for bare frame skewed (10°) deep column (W33) connections with 25% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-56. Fatigue analysis for bare frame skewed (10°) deep column (W33) connections with 50% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-57. Fatigue analysis for bare frame skewed (20°) deep column (W33) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-58. Fatigue analysis for bare frame skewed (20°) deep column (W33) connections with 10% $\phi_{nPn}$ at a) Bottom Flange b) Top Flange.

Figure D-59. Fatigue analysis for bare frame skewed (20°) deep column (W33) connections with 25% $\phi_{nPn}$ at a) Bottom Flange b) Top Flange.

Figure D-60. Fatigue analysis for bare frame skewed (20°) deep column (W33) connections with 50% $\phi_{nPn}$ at a) Bottom Flange b) Top Flange.
Figure D-61. Fatigue analysis for bare frame skewed (30°) deep column (W33) connections with 0% $\phi_Pn$ at a) Bottom Flange b) Top Flange.

Figure D-62. Fatigue analysis for bare frame skewed (30°) deep column (W33) connections with 10% $\phi_Pn$ at a) Bottom Flange b) Top Flange.

Figure D-63. Fatigue analysis for bare frame skewed (30°) deep column (W33) connections with 25% $\phi_Pn$ at a) Bottom Flange b) Top Flange.
Figure D-64. Fatigue analysis for bare frame skewed (30°) deep column (W33) connections with 50% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-65. Fatigue analysis for composite (4000 psi) frame skewed (10°) shallow column (W14) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-66. Fatigue analysis for composite (4000 psi) frame skewed (20°) shallow column (W14) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-67. Fatigue analysis for composite (4000 psi) frame skewed (30°) shallow column (W14) connections with 0% \( \phi_c \cdot P_n \) at a) Bottom Flange b) Top Flange.

Figure D-68. Fatigue analysis for composite (4000 psi) frame skewed (10°) medium column (W24) connections with 0% \( \phi_c \cdot P_n \) at a) Bottom Flange b) Top Flange.

Figure D-69. Fatigue analysis for composite (4000 psi) frame skewed (20°) medium column (W24) connections with 0% \( \phi_c \cdot P_n \) at a) Bottom Flange b) Top Flange.
Figure D-70. Fatigue analysis for composite (4000 psi) frame skewed (30°) medium column (W24) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-71. Fatigue analysis for composite (4000 psi) frame skewed (10°) deep column (W33) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.

Figure D-72. Fatigue analysis for composite (4000 psi) frame skewed (20°) deep column (W33) connections with 0% $\phi_c P_n$ at a) Bottom Flange b) Top Flange.
Figure D-73. Fatigue analysis for composite (4000 psi) frame skewed (30°) deep column (W33) connections with 0% $\phi_Pn$ at a) Bottom Flange b) Top Flange.

Figure D-74. Fatigue analysis for composite (6000 psi) frame skewed (10°) deep column (W14) connections with 0% $\phi_Pn$ at a) Bottom Flange b) Top Flange.

Figure D-75. Fatigue analysis for composite (6000 psi) frame skewed (20°) deep column (W14) connections with 0% $\phi_Pn$ at a) Bottom Flange b) Top Flange.
Figure D-76. Fatigue analysis for composite (6000 psi) frame skewed (30°) deep column (W14) connections with 0% $\phi \cdot P_n$ at a) Bottom Flange b) Top Flange.
APPENDIX E. SAMPLE FABRICATION DETAILS FOR EXPERIMENTAL TESTING.

The fabrication details for the skewed beam experimental testing are presented below. Complete details for the shallow column (W14x132) are shown in Figure 112 and Figure E-1 through Figure E-5. For the medium column (W24x131), Figure E-6 through Figure E-9 depict the necessary details for fabrication.

Figure E-1. Connection detail the three skew levels of the column W14x132.
Figure E-2. Continuity plate detail for column W14x132.

Figure E-3. Weld access holes for the beam W24x76.
Figure E-4. Beam plan view for the three skew levels for the column 14x132.

Figure E-5. Doubler plate detail for the column W 14x132.

Notes:
1. Cut Reduced Beam Section (RBS) at both Top and Bottom Flange.
Figure E-6. Welding connection details for sample fabrication (W24x131).
Figure E-7. Connection detail the three skew levels of the column W24x131.
Figure E-8. Continuity plate and shear tab detail for column W24x131.
Notes:
1. Cut Reduced Beam Section (RBS) at both Top and Bottom Flange.

Figure E-9. Beam plan view for the three skew levels for the column W24x131.
APPENDIX F. DISTRIBUTION OF PEEQ ALONG COLUMN FLANGE, TWIST AND BACKBONE COMPARISON FOR RBS MODELS.

Figure F-1. PEEQ distribution along the normalized column flange with varying skews and axial loads at 4% drift (W14x193 RBS models).

Figure F-2. PEEQ distribution along the normalized column flange with varying skews and axial loads at 5% drift (W14x193 RBS models).
Figure F-3. PEEQ distribution along the normalized column flange with varying skews and axial loads at 4% drift (W18x143 RBS models).

Figure F-4. PEEQ distribution along the normalized column flange with varying skews and axial loads at 5% drift (W18x143 RBS models).
Figure F-5. PEEQ distribution along the normalized column flange with varying skews and axial loads at 4% drift (W24x131 RBS models).

Figure F-6. PEEQ distribution along the normalized column flange with varying skews and axial loads at 5% drift (W24x131 RBS models).
Figure F-7. PEEQ distribution along the normalized column flange with varying skews and axial loads at 4% drift (W33x291 RBS models).

Figure F-8. PEEQ distribution along the normalized column flange with varying skews and axial loads at 5% drift (W33x291 RBS models).
Figure F-9. Column twist versus skew angle at 2% beam drift.

Figure F-10. Column twist versus skew angle at 3% beam drift.
Figure F-11. Column twist versus skew angle at 5% beam drift.

Figure F-12. Column twist comparison for W14x193 at 0% $\phi_{Pn}$ (RBS models).

Figure F-13. Column twist comparison for W14x193 at 10% $\phi_{Pn}$ (RBS models).
Figure F-14. Column twist comparison for W14x193 at 50% $\phi_c$Pn (RBS models).

Figure F-15. Column twist comparison for W18x143 at 0% $\phi_c$Pn (RBS models).

Figure F-16. Column twist comparison for W18x143 at 10% $\phi_c$Pn (RBS models).

Figure F-17. Column twist comparison for W18x143 at 25% $\phi_c$Pn (RBS models).
Figure F-18. Column twist comparison for \( W18 \times 143 \) at 50% \( \phi \)Pn (RBS models).

Figure F-19. Column twist comparison for \( W24 \times 131 \) at 0% \( \phi \)Pn (RBS models).

Figure F-20. Column twist comparison for \( W24 \times 131 \) at 10% \( \phi \)Pn (RBS models).

Figure F-21. Column twist comparison for \( W24 \times 131 \) at 50% \( \phi \)Pn (RBS models).
Figure F-22. Column twist comparison for W33x291 at 0% $\phi_Pn$ (RBS models).

Figure F-23. Column twist comparison for W33x291 at 10% $\phi_Pn$ (RBS models).

Figure F-24. Column twist comparison for W33x291 at 50% $\phi_Pn$ (RBS models).

Figure F-25. Backbone comparison for W14x193 column with different skew angles at 0% $\phi_Pn$ (RBS models).
Figure F-26. Backbone comparison for W14x193 column with different skew angles at 10% $\phi_P n$ (RBS models).

Figure F-27. Backbone comparison for W14x193 column with different skew angles at 25% $\phi_P n$ (RBS models).

Figure F-28. Backbone comparison for W14x193 column with different skew angles at 50% $\phi_P n$ (RBS models).
Figure F-29. Backbone comparison for W18x143 column with different skew angles at 0% $\phi_Pn$ (RBS models).

Figure F-30. Backbone comparison for W18x143 column with different skew angles at 10% $\phi_Pn$ (RBS models).

Figure F-31. Backbone comparison for W18x143 column with different skew angles at 25% $\phi_Pn$ (RBS models).
Figure F-32. Backbone comparison for W18x143 column with different skew angles at 50% $\phi_Pn$ (RBS models).

Figure F-33. Backbone comparison for W24x131 column with different skew angles at 0% $\phi_Pn$ (RBS models).

Figure F-34. Backbone comparison for W24x131 column with different skew angles at 10% $\phi_Pn$ (RBS models).

Figure F-35. Backbone comparison for W24x131 column with different skew angles at 25% $\phi_Pn$ (RBS models).
Figure F-36. Backbone comparison for W24x131 column with different skew angles at 50% $\phi_c P_n$ (RBS models).

Figure F-37. Backbone comparison for W33x291 column with different skew angles at 0% $\phi_c P_n$ (RBS models).

Figure F-38. Backbone comparison for W33x291 column with different skew angles at 10% $\phi_c P_n$ (RBS models).

Figure F-39. Backbone comparison for W33x291 column with different skew angles at 25% $\phi_c P_n$ (RBS models).
Figure F-40. Backbone comparison for W33x291 column with different skew angles at 50% $\phi_v P_n$ (RBS models).
APPENDIX G. DISTRIBUTION OF PEEQ ALONG COLUMN FLANGE, TWIST AND BACKBONE COMPARISON FOR WUF-W MODELS.

Figure G-1. PEEQ distribution along the normalized column flange with varying skews and axial loads at 4% drift (W14x257 WUF-W models).

Figure G-2. PEEQ distribution along the normalized column flange with varying skews and axial loads at 5% drift (W14x257 WUF-W models).
Figure G-3. PEEQ distribution along the normalized column flange with varying skews and axial loads at 4% drift (W24x162 WUF-W models).

Figure G-4. PEEQ distribution along the normalized column flange with varying skews and axial loads at 5% drift (W24x162 WUF-W models).
Figure G-5. PEEQ distribution along the normalized column flange with varying skews and axial loads at 4% drift (W33x354 WUF-W models).

Figure G-6. PEEQ distribution along the normalized column flange with varying skews and axial loads at 5% drift (W33x354 WUF-W models).
Figure G-7. Column twist comparison for W14x257 at 0% $\phi_n$ (WUF-W models).

Figure G-8. Column twist comparison for W14x257 at 10% $\phi_n$ (WUF-W models).

Figure G-9. Column twist comparison for W14x257 at 25% $\phi_n$ (WUF-W models).

Figure G-10. Column twist comparison for W14x257 at 50% $\phi_n$ (WUF-W models).
Figure G-11. Column twist comparison for W24x162 at 0% $\phi_{cPn}$ (WUF-W models).

Figure G-12. Column twist comparison for W24x162 at 10% $\phi_{cPn}$ (WUF-W models).

Figure G-13. Column twist comparison for W24x162 at 25% $\phi_{cPn}$ (WUF-W models).

Figure G-14. Column twist comparison for W24x162 at 50% $\phi_{cPn}$ (WUF-W models).
Figure G-15. Column twist comparison for W33x354 at 0% $\phi_c$Pn (WUF-W models).

Figure G-16. Column twist comparison for W33x354 at 10% $\phi_c$Pn (WUF-W models).

Figure G-17. Column twist comparison for W33x354 at 25% $\phi_c$Pn (WUF-W models).

Figure G-18. Column twist comparison for W33x354 at 50% $\phi_c$Pn (WUF-W models).
Figure G-19. Backbone comparison for W14x257 column with different skew angles at 0% $\phi_c P_n$ (WUF-W models).

Figure G-20. Backbone comparison for W14x257 column with different skew angles at 10% $\phi_c P_n$ (WUF-W models).

Figure G-21. Backbone comparison for W14x257 column with different skew angles at 25% $\phi_c P_n$ (WUF-W models).

Figure G-22. Backbone comparison for W14x257 column with different skew angles at 50% $\phi_c P_n$ (WUF-W models).
Figure G-23. Backbone comparison for W24x162 column with different skew angles at 0% $\phi_c P_n$ (WUF-W models).

Figure G-24. Backbone comparison for W24x162 column with different skew angles at 10% $\phi_c P_n$ (WUF-W models).

Figure G-25. Backbone comparison for W24x162 column with different skew angles at 25% $\phi_c P_n$ (WUF-W models).

Figure G-26. Backbone comparison for W24x162 column with different skew angles at 50% $\phi_c P_n$ (WUF-W models).
Figure G-27. Backbone comparison for W33x354 column with different skew angles at 0% $\phi_c P_n$ (WUF-W models).

Figure G-28. Backbone comparison for W33x354 column with different skew angles at 10% $\phi_c P_n$ (WUF-W models).

Figure G-29. Backbone comparison for W33x354 column with different skew angles at 25% $\phi_c P_n$ (WUF-W models).

Figure G-30. Backbone comparison for W33x354 column with different skew angles at 50% $\phi_c P_n$ (WUF-W models).
APPENDIX H. PLASTIC ROTATION COMPARISON GRAPHTHS FOR BARE AND COMPOSITE RBS CONNECTION FRAMES.

Figure H-1. Plastic rotation comparison for shallow column-bare and composite frame at 10-degree skew.

Figure H-2. Plastic rotation comparison for shallow column-bare and composite frame at 20-degree skew.

Figure H-3. Plastic rotation comparison for shallow column-bare and composite frame at 30-degree skew.

Figure H-4. Plastic rotation comparison for medium column-bare and composite frame at 10-degree skew.
Figure H-5. Plastic rotation comparison for medium column-bare and composite frame at 20-degree skew.

Figure H-6. Plastic rotation comparison for medium column-bare and composite frame at 30-degree skew.

Figure H-7. Plastic rotation comparison for deep column-bare and composite frame at 10-degree skew.

Figure H-8. Plastic rotation comparison for deep column-bare and composite frame at 20-degree skew.
Figure H-9. Plastic rotation comparison for deep column-bare and composite frame at 30-degree skew.

Figure H-10. Plastic rotation comparison for shallow column-composite (4000 vs 6000 psi) frame at 10-degree skew.

Figure H-11. Plastic rotation comparison for shallow column-composite (4000 vs 6000 psi) frame at 20-degree skew.

Figure H-12. Plastic rotation comparison for shallow column-composite (4000 vs 6000 psi) frame at 30-degree skew.
APPENDIX I. RBS AND WUF-W PANEL ZONE PLASTIC ROTATION COMPARISON GRAPHS FOR BARE FRAMES.

Figure I-1. Bare frame shallow column (W14_orthogonal) RBS vs WUF-W panel zone plastic rotation comparison.

Figure I-2. Bare frame shallow column (W14_10° skew) RBS vs WUF-W panel zone plastic rotation comparison.
Figure I-3. Bare frame shallow column (W14_20° skew) RBS vs WUF-W panel zone plastic rotation comparison.

Figure I-4. Bare frame shallow column (W14_30° skew) RBS vs WUF-W panel zone plastic rotation comparison.
Figure I-5. Bare frame medium column (W24_orthogonal) RBS vs WUF-W panel zone plastic rotation comparison.

Figure I-6. Bare frame medium column (W24_10° skew) RBS vs WUF-W panel zone plastic rotation comparison.
Figure I-7. Bare frame medium column (W24_20° skew) RBS vs WUF-W panel zone plastic rotation comparison.

Figure I-8. Bare frame medium column (W24_30° skew) RBS vs WUF-W panel zone plastic rotation comparison.
Figure I-9. Bare frame deep column (W33_orthogonal) RBS vs WUF-W panel zone plastic rotation comparison.

Figure I-10. Bare frame deep column (W33_10° skew) RBS vs WUF-W panel zone plastic rotation comparison.
Figure I-11. Bare frame deep column (W33_20° skew) RBS vs WUF-W panel zone plastic rotation comparison.

Figure I-12. Bare frame deep column (W33_30° skew) RBS vs WUF-W panel zone plastic rotation comparison.
APPENDIX J. RBS AND WUF-W PANEL ZONE SHEAR STRAIN COMPARISON GRAPHTHS FOR BARE FRAMES.

Figure J-1. Bare frame shear strain comparison for orthogonal medium columns (W24).

Figure J-2. Bare frame shear strain comparison for medium columns (W24) at 10° skew.
Figure J-3. Bare frame shear strain comparison for medium columns (W24) at 20° skew.

Figure J-4. Bare frame shear strain comparison for medium columns (W24) at 30° skew.
Figure J-5. Bare frame shear strain comparison for orthogonal deep columns (W33).

Figure J-6. Bare frame shear strain comparison for deep columns (W33) at 10° skew.
Figure J-7. Bare frame shear strain comparison for deep columns (W33) at 20° skew.

Figure J-8. Bare frame shear strain comparison for deep columns (W33) at 30° skew.