

DESIGN COMPARISON OF ORDINARY CONCENTRIC BRACE FRAMES AND SPECIAL
CONCENTRIC BRACE FRAMES FOR SEISMIC LATERAL FORCE RESISTANCE FOR
LOW RISE BUILDINGS

by

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Abstract

Braced frames are a common seismic lateral force resisting system used in steel structures. Ordinary concentric braced frames (OCBFs) and special concentric braced frames (SCBFs) are two major types of frames. Brace layouts vary for both OCBFs and SCBFs. This report examines the inverted-V brace layout which is one common arrangement. OCBFs are designed to remain in the elastic range during the design extreme seismic event. As a result, OCBFs have relatively few special requirements for design. SCBFs are designed to enter the inelastic range during the design extreme seismic event while remaining elastic during minor earthquakes and in resisting wind loads. To achieve this, SCBFs must meet a variety of stringent design and detailing requirements to ensure robust seismic performance characterized by high levels of ductility.

The design of steel seismic force resisting systems must comply with the requirements of the American Institute of Steel Construction's (AISC) *Seismic Provisions for Structural Steel Buildings*. Seismic loads are determined in accordance with the American Society of Engineers *Minimum Design Loads for Buildings and Other Structures*. Seismic loads are very difficult to predict as is the behavior of structures during a large seismic event. However, a properly designed and detailed steel structure can safely withstand the effects of an earthquake.

This report examines a two-story office building in a region of moderately high seismic activity. The building is designed using OCBFs and SCBFs. This report presents the designs of both systems including the calculation of loads, the design of frame members, and the design and detailing of the connections. The purpose of this report is to examine the differences in design and detailing for the two braced frame systems.

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List of Notations

- A_g – Gross area
- C_a – Ratio of required strength to available strength
- C_d – Deflection amplification factor
- C_s – Seismic response coefficient
- C_t – Approximate period parameter
- C_{vx} – Vertical distribution factor
- D – Dead load
- D – Diameter of brace
- D_s – Elastic story drift at elastic seismic force, V_e
- D_u – Ultimate story drift
- H_{ub} – Required shear force on beam to gusset connection
- H_{uc} – Required axial force on column to gusset connection
- h_i, h_x – Height of level i, x
- h_n – Approximate period parameter
- E – Modulus of elasticity
- F_a – Short period site coefficient
- F_{cr} – Critical buckling stress
- F_{cre} – Expected critical buckling stress
- F_e – Elastic critical buckling stress
- F_v – Long period site coefficient
- F_y – Specified minimum yield stress
- F_x – Portion of seismic base shear acting at level x
- I_e – Importance factor for seismic design
- K – Effective length factor
- k – Distribution exponent
- L – Live load
- L – Member length
- M_p – Plastic bending moment
- P_c – Expected post-buckling strength

P_t – Expected tensile yield strength
 Q_E – Seismic load
 R – Response modification coefficient
 R_y – Material overstrength factor, ratio of expected yield stress to specified minimum yield stress
 r – Radius of gyration
 S – Snow load
 S_{DS} – Design spectral response acceleration parameter for short periods
 S_{D1} – Design spectral response acceleration parameter for periods of 1 second
 S_{MS} – Site class adjusted spectral response coefficient for short periods
 S_{M1} – Site class adjusted spectral response coefficient at a period of 1 second
 S_s – Mapped spectral response acceleration parameter for short periods
 S_1 – Mapped spectral response acceleration parameter for periods of 1 second
 T – Fundamental building period
 T_a – Approximate fundamental building period
 T_L – Long-period transition period
 t – Wall thickness of HSS
 V – Base Shear
 V_e – Elastic seismic force
 V_s – Design seismic force
 V_{ub} – Required axial force on beam to gusset connection
 V_{uc} – Required shear force on column to gusset connection
 V_y – Actual seismic force
 W – Effective seismic weight
 w_i, w_x – Portion of effective seismic weight at level i, x
 Z – Plastic section modulus about the axis of bending
 λ – Slenderness parameter
 λ_{md} – Limiting slenderness parameter for moderately ductile compactness
 λ_{hd} – Limiting slenderness parameter for highly ductile compactness
 ρ – Redundancy factor
 Ω_o – System overstrength factor

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Dedication

This report is dedicated to those that have made my education possible: my parents Ron and Mary Kay Grusenmeyer. Thank you for always supporting me. This report is also dedicated to my future wife, Emily Riley, who is my constant encourager.

Chapter 1 - Introduction

Brace frames are a common vertical lateral force resisting system (LFRS) used in steel structures with four main variations which provide varying levels of seismic performance. This report includes a design comparison of two of these systems - ordinary concentric brace frames and special concentric brace frames. Ordinary concentric brace frames are the least seismically robust brace frame system with the least stringent design and detailing requirements. Ordinary concentric brace frames are therefore common in low seismic risk regions. Special concentric brace frames are significantly more seismically robust with very stringent requirements for their design. This report begins with a discussion of the loading conditions, brace performance, and brace frame component design and detailing requirements. Finally, a comparison of the two different brace frame system designs for a two story office building is presented. Design calculations are presented for each of the two designs.

Chapter 2 - Scope of Research

This report discusses and compares the designs of ordinary concentric brace frames (OCBF) and special concentric brace frames (SCBF). The comparison is based on the frame design for a two story office building located in Henderson, NV. This location was chosen because of its moderately high seismicity. The building height is limited to 35 feet to comply with height limitations for OCBFs in seismic design category “D” according to the American Society of Civil Engineers’ *Minimum Design Loads for Building and Other Structures* (ASCE 7-10). The building is 120 feet long and 75 feet wide in plan consisting of four longitudinal bays and three transverse bays. The building has a symmetrical floor plan, thus stairs and elevators are assumed to be located outside the rectangular footprint. Refer to Figure 2-1 for plans.

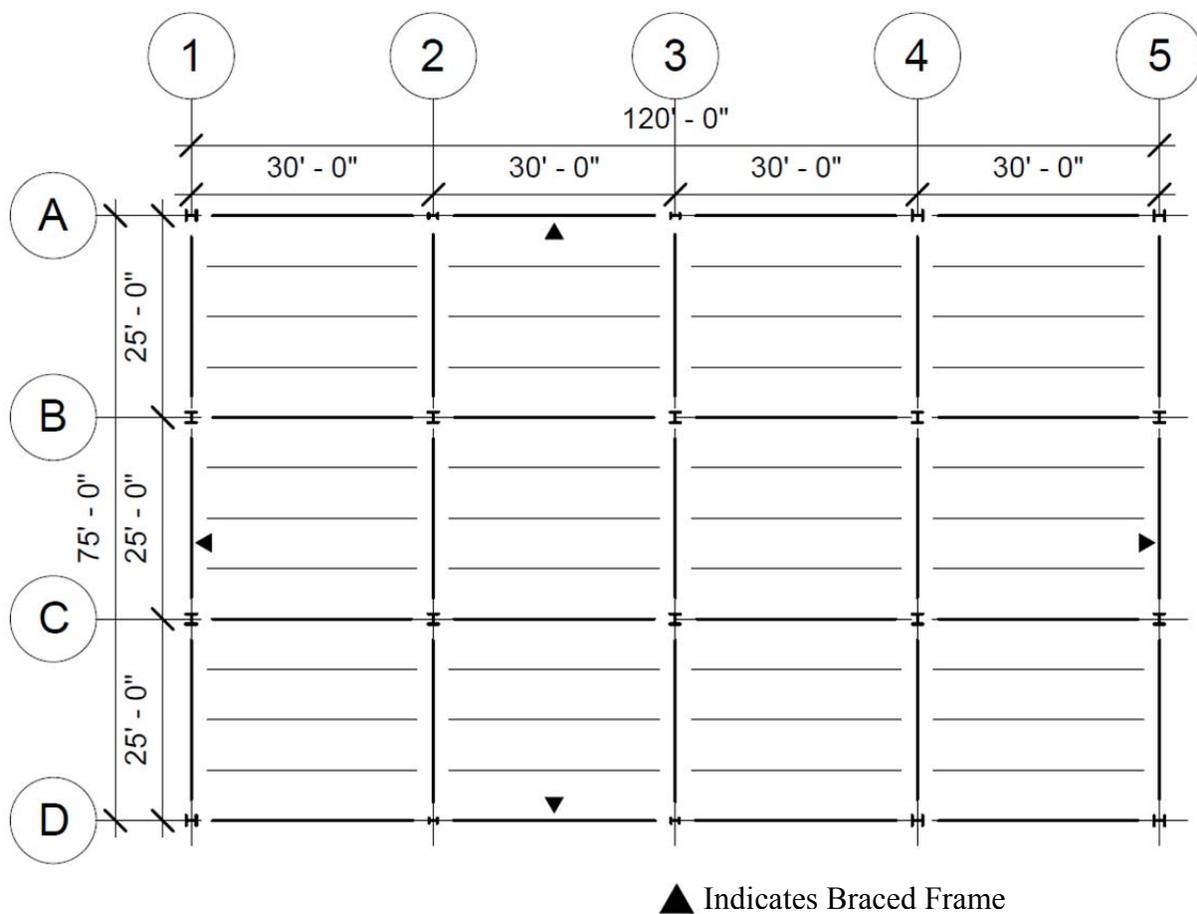


Figure 2-1: Building Framing Plan

The roof is assumed to be a flexible diaphragm consisting of metal deck. The second floor is a rigid diaphragm consisting of 1.0C 20 gauge metal deck with 3 1/2" of concrete topping; total floor thickness is 4 1/2". The floor-to-floor heights are 16 feet and the building has a parapet extending 3 feet above the roof level. Four brace frames are used: one on each side of the building. Refer to Figures 2-2 and 2-3 for elevations. The building envelope is a non-structural curtain wall system.

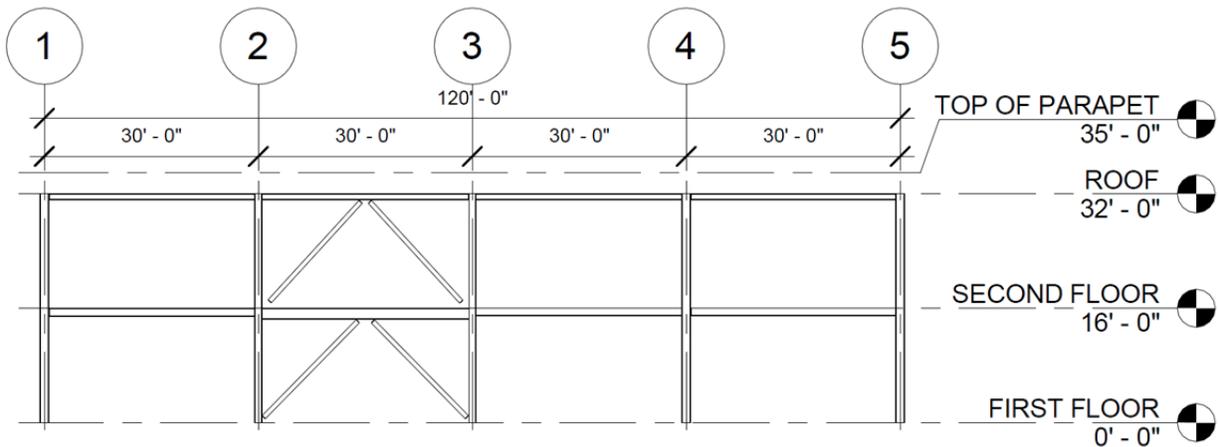


Figure 2-2: Longitudinal Building Elevation

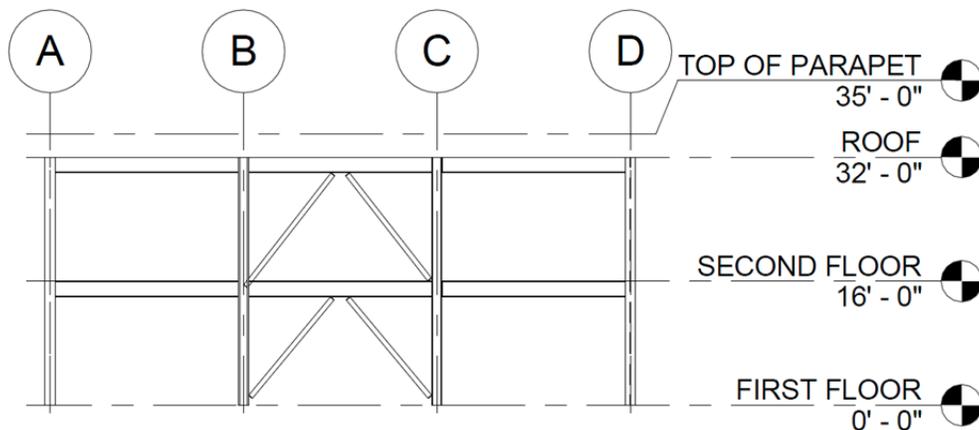


Figure 2-3: Transverse Building Elevation

Seismic ground motion parameters were determined using United States Geological Survey (USGS) *Seismic Hazard Curves and Uniform Hazard Response Spectra v5.1.0*. Geographic coordinates within the City of Henderson, NV were used. Soil conditions are not known because a specific location in Henderson, NV was not chosen and a geotechnical report was not used. Therefore a site class “D” is reasonably assumed for this building to determine the ground motion criteria.

The LFRS for the parametric study has been designed for a OCBF system and a SCBF system. The report details the design of braces, columns, and beams in addition to connections between the elements. The designs adhere to the requirements provided in the American Institute of Steel Construction 360-10: *Specification for Structural Steel Buildings (AISC Specification)* and the American Institute of Steel Construction 341-10: *Seismic Provisions for Structural Steel Buildings (AISC Seismic Provisions)* as well as loading conditions provided by ASCE 7-10.

Chapter 3 - Building Loads

The first step in any structural design is the determination and establishment of applied building loads. This report does not cover the design of all structural systems within a building, but rather focuses on the main LFRS. Therefore, seismic forces are the primary discussion in this report. Seismic lateral loads are the most critical loads considered in the design of concentric brace frames. However, members within a brace frame, specifically columns and beams, must also carry gravity loads and it is critical to consider all load effects on each individual member. This report discusses the gravity and lateral loads, and seismic loads in greater detail. See Appendix B for the load calculations for the parametric study.

Gravity Loads

Typical office construction is assumed for the parametric study to establish reasonable dead loads for the structure. Seismic loads are directly proportional to the building dead load. Therefore, light-weight buildings are advantageous in high seismic regions, and steel structures are light compared to reinforced concrete counterparts. Live loads and snow loads generally do not affect seismic loads. However, according to ASCE 7-10 a portion of live loads must be included in seismic weight for storage areas and office partitions. Similarly, a portion of the snow loads must be included in seismic weight if the flat roof snow load exceeds 30psf. The following is a summary of the loads considered for the example office building.

Dead Loads

Dead loads consist of the weight of all permanent components and materials of a building including structural members, architectural finishes, and fixed equipment. The roof dead load includes 2" rigid insulation, 1.5B 20 gauge metal deck, open web steel joist framing, suspended acoustic ceiling system, mechanical, electrical, plumbing, and miscellaneous loads. The elevated second floor dead load includes 1.0C 20 gauge metal deck with 3-1/2" of normal weight concrete topping, steel floor framing, suspended acoustic ceiling system, mechanical, electrical, plumbing, and miscellaneous loads. The dead loads for the parametric study are summarized in Table 3.1.

Table 3-1: Dead Loads

Roof Dead Load=	18 psf
2 nd Floor Dead Load=	50 psf
Curtain Wall Dead Load=	15 psf

Live Loads

Both roof live loads and floor live loads must be considered in design. Roof live loads for an office building are induced during maintenance by workers, equipment, and related materials. Floor live loads in office buildings account for loads due to building occupancy. These loads are not constant for the life of the structure. Office space and corridors have different live loads, as specified by the ASCE 7-10. However, the floor plan of an office is likely to change over time so the highest load is applied to the entire floor for this report. The ASCE 7-10 also has special provisions for partition wall loads in office buildings to account for rearrangement of partition walls, and a portion of this load is accounted for in the seismic weight. The partition load was added to the office space live load. Then, the total office live load was compared to the corridor live load and the larger load was used. The live loads for the parametric study are summarized in Table 3-2. Live load reductions are applicable for structural members carrying live loads. This reduction, in accordance with the ASCE 7-10 Section 4.7, can result in reduced member sizes. Because live load reduction depends on tributary area, members with large tributary areas are affected most significantly. Therefore, for this report, live loads were reduced only for columns.

Table 3-2: Live Loads

Roof Live Load, L_r =	20 psf
2 nd Floor Live Load, L =	80 psf

Snow Loads

High snow loads can affect the seismic weight of a building and consequently the seismic base shear. Of course, gravity load carrying members will be affected by the direct load applied by snow. Snow loads are dependent on location and are related to historical snowfall for the area. For the parametric study, a ground snow load of 5psf was used. Snow drifts must also be

determined according to the ASCE 7-10. The flat roof snow load and drift load for the parametric study are summarized in Table 3-3. To simplify calculations the maximum drift load was applied to the entire roof. Ultimate loads calculated using the load combinations in Section 2.3

Wind Loads

Even in high seismic areas wind loads must be calculated to ensure that seismic forces govern the design of the main lateral force resisting system. Wind loads are the result of wind pressures acting on the surfaces of a structure, either toward or away from the surface. Wind loads depend on several factors including design wind speed, terrain, topography, and building shape. For the parametric study wind load calculations followed the directional procedure. The MLFRS is typically designed to perform within the elastic range for the maximum expected wind event, unlike MLFRS design for seismic loads. The total wind base shear in each direction must be compared to the seismic base shear. Because the ASCE 7-10 now calculates wind loads at strength levels, the two base shears can be compared without factoring the forces. Even in higher seismic regions it is possible for wind to govern the lateral design depending on the building parameters.

Seismic Loads

Seismic loads must be evaluated with one of three analysis methods prescribed by the ASCE 7-10 in Table 12.6-1; these include the equivalent lateral force procedure (ELFP), modal response analysis, and seismic response history analysis. This report uses the ELFP, which is outlined specifically in Section 12.8 of the ASCE 7-10. The ELFP provides a simplified method of calculating the total seismic lateral force, base shear, and the vertical distribution of that force (ASCE 7-10, 2010). This procedure is a common analysis method for this type and size of building. The ELFP depends on criteria from two primary sources: seismic ground motion and structure specific design coefficients and factors.

Seismic Ground Motion

ASCE 7, as referenced by the International Building Code (IBC), provides two spectral accelerations to describe seismic ground motion for a given location. The parameters, S_s and S_1 ,

represent risk targeted maximum considered earthquake (MCE) spectral response ground acceleration parameters for 0.2 and 1 second periods, respectively. They are determined based on maps from the USGS. These values are characteristics of the maximum considered earthquake, which has an assumed recurrence period of 2,500 years. S_S and S_1 are modified based on the site classification by site coefficients, F_a and F_v , to calculate the site class adjusted spectral response coefficients S_{MS} and S_{M1} .

$$S_{MS} = F_a S_S \quad (\text{ASCE 7-10 Eqn 11.4-1})$$

$$S_{M1} = F_v S_1 \quad (\text{ASCE 7-10 Eqn 11.4-2})$$

The S_{MS} and S_{M1} parameters are further reduced by 2/3 to provide a safety factor for collapse of 1.5.

$$S_{DS} = \frac{2}{3} S_{MS} \quad (\text{ASCE 7-10 Eqn 11.4-3})$$

$$S_{D1} = \frac{2}{3} S_{M1} \quad (\text{ASCE 7-10 Eqn 11.4-4})$$

The final ground motions, S_{DS} and S_{D1} , are then used to calculate seismic force using the ELFP (Nikolaou, 2008).

Site Classification

Building sites are classified A through F, with A classifications representing hard rock and F classification representing weak soils (Nikolaou, 2008). Requirements for site classification are prescribed in detail in Chapter 20 of ASCE 7-10. For the parametric study, site class D is assumed, which is a reasonably conservative assumption for unknown soil conditions. Sites classified as F require a site specific analysis to determine ground motion criteria.

Seismic Design Category

ASCE 7-10 requires that a structure be classified based on the design seismic ground motion and the building risk category. Buildings, like site classifications, are categorized A to F with Seismic Design Category (SDC) A representing the least seismic impact on structural design. The SDC is used to determine the level of seismic resistance a structure must have. Structural analysis methods, allowable structural systems, height limitations, detailing requirements, and other related requirements are all determined based, in part, by a building's

SDC (Nikolaou, 2008). The SDC is based on the spectral response accelerations and the building risk category according to ASCE 7-10 Tables 11.6-1 and 11.6-2.

Design Coefficients and Factors

Seismic forces are complex and largely unknown, yet engineers must design structures to safely resist these forces. Building response to seismic forces is difficult to quantify and much of the data is generated from historical successes and failures (SEAOC Seismology Committee, 2009). This history, along with seismic building theory reinforced with research and testing, provides sound evidence that a safe structure can be designed using only a portion of the elastic seismic design forces. The basis of this design theory is that every structure has inherent ductility to some degree, and that ductility is a mechanism through which seismic energy is absorbed. This relationship is illustrated in Figure 3-1 by an inelastic force-deformation curve. When seismic forces act on a building the structure initially responds in an elastic manner. The structure enters the inelastic range as the force increases. Plastic hinges begin to form throughout the structure until the system yield strength, V_y , is reached (SEAOC Seismology Committee, 2008).

The ELFP uses a linear elastic analysis which is much simpler than an inelastic analysis. This design procedure assumes that the structure will experience permanent deformations during an extreme seismic event. This method also results in a more cost effective structure than one designed using the full elastic seismic design force, which would not account for the structure's inherent ductility. It is that deformation, or ductility, that reduces the seismic force. The ASCE 7-10 uses three coefficients and factors to describe a structure's response to seismic forces: the response modification coefficient, R , system overstrength factor, Ω_o , and deflection amplification factor, C_d (SEAOC Seismology Committee, 2008). These factors vary based on the type of LFRS selected. See Table 3-3 for a summary of values for OCBF and SCBF systems.

Table 3-3: Design Coefficients and Factors

	OCBF	SCBF
R	3.25	6.0
Ω_o	2.0	2.0
C_d	3.25	5.0

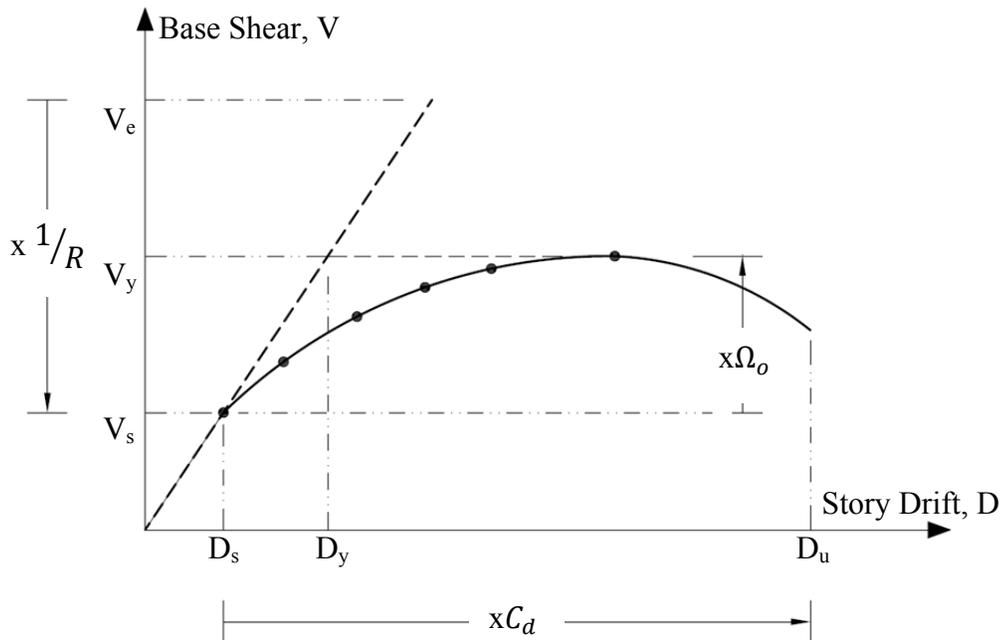


Figure 3-1: Inelastic Force-Deformation Curve

Response Modification Coefficient

The response modification factor, R , is a simplified numeric description of a seismic force resisting system's inherent ductility. This coefficient is alone responsible for reducing elastic seismic force, V_e , to the design seismic force level, V_s . The structural system must provide the specified ductility as expressed by its R value to ensure that the structure performs as predicted beyond the elastic limit, V_s (SEAOC Seismology Committee, 2008). In the case of a brace frame, this is accomplished by adhering to the requirements set forth in the AISC 341-10. As the R value of a structural system increases so does its propensity for ductile behavior and energy dissipation (SEAOC Seismology Committee, 2009).

Deflection Amplification Factor

The deflection amplification factor, C_d , is used to determine the predicted total deformation experienced beyond the elastic deformation experienced at V_e . For lateral force resisting systems, deformation is commonly expressed as story drift, or the difference in absolute deflections between floors. The ultimate story drift, D_u , is calculated by multiplying the story drift, D_u , by C_d (SEAOC Seismology Committee, 2008). D_u is the story drift corresponding to the design seismic force and is calculated in accordance with the ASCE 7-10 Sections 12.8.6, 12.9.2 or 16.1. Story drift limits are established to prevent large inelastic strains in the LFRS and to preserve structural stability. Drift limits also prevent non-structural damage from more frequent but smaller earthquakes (*AISC Seismic Manual*, 2005).

The deflection amplification factor is related to the response modification coefficient and it similarly describes system ductility. Lateral systems high C_d values have higher ductility (SEAOC Seismology Committee, 2008). Like response modification coefficients, the deflection amplification factor allows for simplified calculation of the drift of an inelastic structure using an elastic analysis.

Overstrength Factor

To ensure that a structural system performs as intended under seismic loading, some members must be designed to sustain the full, or actual, seismic force, V_y , as adjacent members yield. These members are referred to as force-controlled members. The design seismic force can be modified by the overstrength factor to calculate the V_y (SEAOC Seismology Committee, 2008). According to FEMA 450: *NEHRP Recommended Provisions* commentary, the overstrength factor, Ω_o , is described by three system specific criteria: design overstrength, material overstrength, and system overstrength. Design overstrength is directly related to a structural system's ductility and site-specific ground motion criteria. Material overstrength reflects that actual material strengths exceed nominal design material strengths. For example, A992 steel has a minimum yield strength of 50ksi. However, the actual yield strength typically exceeds the 50 ksi minimum. System overstrength describes structural system redundancy. For example, a LFRS consisted of a single brace frame and the brace yielded, the system would be fully yielded, assuming the system was perfectly optimized. However, if the LFRS consisted of multiple frames, even if all the frames were required to meet the design, it would have some

inherent overstrength. Additionally, system overstrength is affected by the optimization of the LFRS design. The closer the design is to matching the applied forces, the lower the system overstrength will be. All three of these overstrength criteria are accounted for in Ω_o as a single value which is used to modify the design elastic seismic force (*NEHRP Recommended Provisions*, 2003).

Seismic Design Force

Using the ELFP, seismic forces are determined by distributing the total seismic base shear force among each of the building's levels. The seismic base shear, V , is calculated by multiplying the effective seismic weight, W , by the seismic response coefficient, C_s .

$$V = C_s W \quad (\text{ASCE 7-10 Eqn 12.8-1})$$

The ELFP is outlined in ASCE 7-10 Section 12.8.

Effective Seismic Weight

The effective seismic weight is defined by ASCE 7-10 Section 12.7.2 as the total building dead load in addition to other specialized loading conditions that could be present. For storage areas, at least 25 percent of the floor live load must be included in the effective seismic weight. If partition floor loads are required by Section 4.2.2, a minimum weight of 10 psf must be included. The operating weight of permanent equipment must be added. If the flat roof snow load is greater than 30 psf, then 20 percent of that snow load must be added to the seismic weight.

Seismic Response Coefficient

According to ASCE 7-10 Section 12.8.1.1, the seismic response coefficient, C_s , depends on the response modification factor, the short period design spectral response acceleration parameter, and the building importance factor. Thus, C_s ultimately describes the acceleration applied to the building's mass.

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \quad (\text{ASCE 7-10 Eqn 12.8-2})$$

Section 12.8.1.1 also establishes maximum and minimum values for C_s . The maximum C_s values depend on the fundamental period, T , of the building compared with the site-specific long-period transition period, T_L . The value of C_s calculated using Equation 12.8-2 does not need to exceed the values calculated by Equations 12.8-3 or 12.8-4, whichever is applicable.

$$C_{s-max} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \text{ for } T \leq T_L \quad (12.8-3)$$

$$C_{s-max} = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} \text{ for } T > T_L \quad (12.8-4)$$

However, C_s must not be less than the value determined by Equation 12.8-5, unless the value of S_1 is greater than 0.6g, in which case C_s must exceed the value determined by Equation 12.8-6.

$$C_{s-min} = 0.044S_{DS}I_e \geq 0.01 \quad (\text{ASCE 7-10 Eqn 12.8-5})$$

$$C_{s-min} = \frac{0.5S_1}{\left(\frac{R}{I_e}\right)} I_e \geq 0.01 \quad (\text{ASCE 7-10 Eqn 12.8-6})$$

Building Period

The fundamental period, T , of a structure is determined according to ASCE 7-10 Section 12.8.2. This section requires an analysis to be performed to determine the period. Alternatively, Section 12.8.2.1 provides an approximate method for determining the building period by using Equation 12.8-7. The approximate period calculation depends on two parameters, C_t and x , that are related to the LFRS, and the height from ground level to the highest main diaphragm. The values of C_t and x are tabulated in Table 12.8-2 and h_n is the height of the highest level of the structure above the base.

$$T_a = C_t h_n^x \quad (\text{ASCE 7-10 Eqn 12.8-7})$$

For OCBFs and SCBFs: $C_t = 0.02$ and $x = 0.75$

In determining the maximum building response coefficient, C_{s-max} , the fundamental building period, T or T_a , is compared to the long-period transition period, T_L . T_L is determined from maps in ASCE 7-10 Chapter 22.

Distribution of Seismic Forces

In Section 12.8.3 the ASCE 7-10 specifies how the total seismic base shear is to be distributed among the levels of the structure. The distribution depends on the seismic weight and height of each level. Heavier levels attract higher forces than lighter levels. Higher levels generate higher forces than lower levels. The seismic base shear is multiplied by a given level's vertical distribution factor, C_{vx} , to determine the lateral seismic force, F_x , at a given level according to Equation 12.8-11. C_{vx} is the ratio of the product of the given level's seismic weight and height and the sum of the products of the seismic weights and heights of each level

according to Equation 12.8-12. The value, k , in Equation 12.8-12 is an exponent related to the building period.

$$F_x = C_{vx}V \quad (\text{ASCE 7-10 Eqn 12.8-11})$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} \quad (\text{ASCE 7-10 Eqn 12.8-12})$$

Torsion Forces

Diaphragms that are not flexible can induce a moment-couple when the diaphragm's center-of-mass and center-of-rigidity are not aligned. An eccentricity exists resulting in a moment equal to the seismic force multiplied by the distance between the two centers. A variety of irregularities detailed in ASCE 7-10 Section 12.3 can result in inherent torsion described in Section 12.8.4.2. However, even without obvious irregularities ASCE 7-10 Section 12.8.4.2 requires that accidental torsion be accounted for in design. The accidental torsion moment is calculated with an eccentricity equal to 5 percent of the building dimension perpendicular to the direction of the applied force. Accidental torsion must be examined for both the longitudinal and transverse building dimensions. It is important to account for accidental torsion because the actual center of mass of any building will likely differ from the assumed design center of mass, resulting in additional torsion loads that would not have otherwise been accounted for.

Chapter 4 - Ordinary Concentrically Braced Frame Member Design

Concentrically braced frames are LFRS that use axially loaded members to transfer lateral forces to the foundation. The brace members rely on axial strength, in both tension and compression, and stiffness to resist the applied axial loads. As illustrated in Figure 4-1, the braces within a frame are laid out such that the centerlines of the braces, columns, and beams intersect at their points of connection, unlike eccentrically braced frame. Lateral forces are transferred through floor and roof diaphragms at every level, and ideally, braced frames are stacked vertically between building levels to provide a clear load path to the foundation.

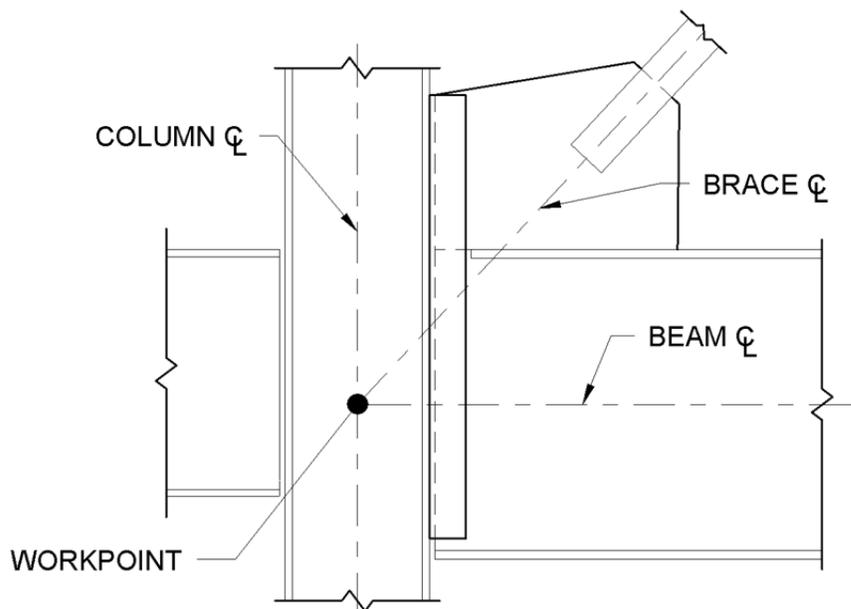


Figure 4-1: Intersection of Member Centerlines

Braced frames can be laid out in a variety of ways. The most commonly used concentrically braced frame layouts are V, Inverted V, X, and K as illustrated in Figure 4-2. Each layout has particular advantages and disadvantages for design, structural performance, fabrications, and construction. However, they all function similarly. This report focuses on the design of braced frames with inverted-V brace layouts. Inverted-V brace frames were selected because they can allow for openings such as doors at the first floor and they have similar

connections at each level. Inverted-V brace frames were also chosen for this report to explore the additional requirements imposed on such frame layouts.

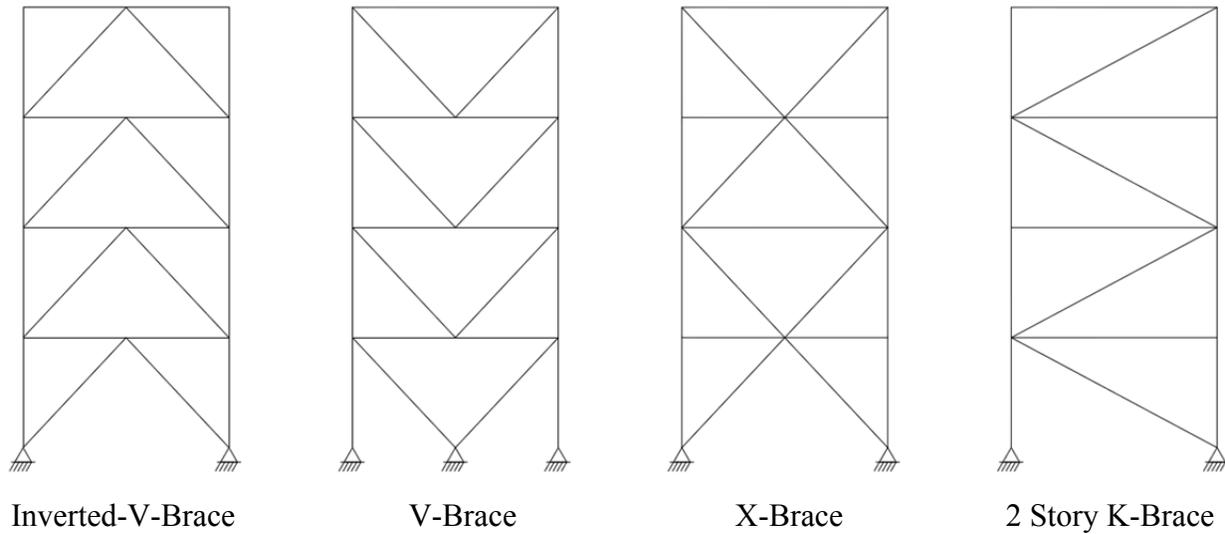


Figure 4-2: Brace Frame Layouts

Ordinary concentrically braced frames are very similar in layout to special concentrically braced frames. However, the *AISC Seismic Provisions* have much more restrictive requirements for SCBF. These requirements ensure that SCBFs have higher ductility, allowing more plastic hinges to form as the frame is pushed into the inelastic range, as illustrated in Figure 3-1. OCBFs have relatively few special seismic requirements and are thus simpler to design, detail, fabricate, and install. However, the inelastic behavior of OCBFs is unpredictable, and as a result the system has a low response modification coefficient of 3.5. Therefore, the OCBF system must be designed for higher seismic forces than a SCBF because it must perform elastically during the designed seismic event. Aside from a few limitations specified by the *AISC Seismic Provisions*, all of the components of an ordinary brace frame are designed using steel design procedures outlined in the *AISC Specification*. According to the ASCE 7-10, the use of OCBFs is limited to building heights less than 35 feet because of their lack of ductility and overall seismic robustness. Their use is ideal for smaller buildings or buildings with low seismic design requirements.

Analysis

Structural analysis of the parametric study was performed using RISA-2D computer analysis software. A simple two-dimensional frame consisting only of the bay in which the OCBF is located. Gravity loads were calculated and applied to the frame to represent the loads from adjacent framing. The seismic lateral forces were applied at each level as point loads representing the full seismic load for that side of the building. The base reactions of the frame were checked by hand calculation using simple statics considering overturning of the frame. Both seismic load combinations 5 and 7 were used as prescribed in Section 12.4.2.3 in the ASCE 7-10.

$$\text{Load Combination 5: } (1.2 + 0.2S_{DS})D + \rho Q_E + L + .2S$$

$$\text{Load Combination 7: } (0.9 - 0.2S_{DS})D + \rho Q_E$$

Member forces for the braces and columns were then used for member design. Beam forces, as discussed later, are specified in detail by the *AISC Seismic Provisions* due to the inverted-V layout of the braces. See Appendix C for all OCBF member design calculations.

Brace Design

Braces are the primary member within a braced frame for the transfer of lateral forces. They resist applied loads in pure axial tension or compression. Once the loads had been determined the member design commenced. For the parametric study, round HSS members were used for the braces. First member stability requirements, such as slenderness and local buckling, were established then the strengths of the selected member were calculated.

Slenderness

Typically, a brace must meet the slenderness criteria provided by the *AISC Specification* in Chapter E. However, because the inverted V brace layout is used for the parametric study, the braces must meet more stringent slenderness requirements specified by the Seismic Provisions. Section F1.5.5b requires that braces in inverted-V configurations meet the following slenderness limit:

$$\frac{KL}{r} \leq 4 \sqrt{\frac{E}{F_y}} \quad \text{Eqn 4-1}$$

Braces are assumed to be pinned at both ends, where member translation is fixed but member rotation is free. Thus K , the effective length factor, is equal to 1.0.

During the cyclic loading of a seismic event, a brace can buckle in compression resulting in a significant loss of strength. In the inverted-V layout this results in an unbalanced load on the beam because the tension brace will yield at an increased strength. Figure 4-3 illustrates the unbalanced load where P_1 is the buckled compression strength of the brace and P_2 is the expected tension strength. To help prevent the unbalanced load from occurring, the more stringent slenderness limit is used. This limit is not applied to X-braced frames because an unbalanced load is not applied to the beam (*AISC Seismic Provisions*).

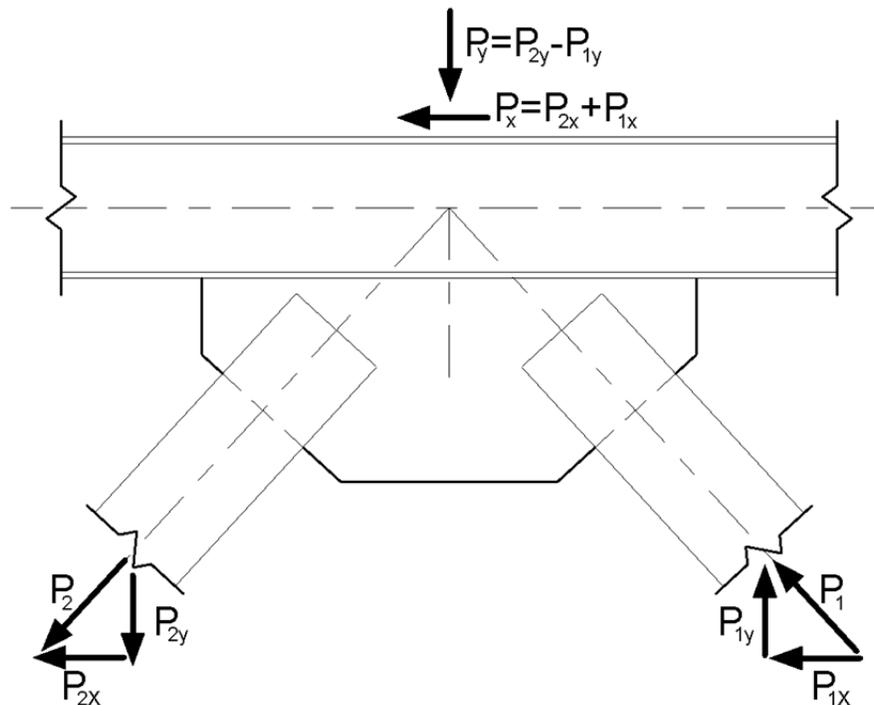


Figure 4-3: Unbalanced Load Due to Inverted-V Brace Layout

For the parametric study, the slenderness criterion was critical in selection of the brace member. The slenderness limit provided by the *AISC Seismic Provisions* is significantly less than that provided by the *AISC Specification*. The result, however, is a brace that performs better under compressive loads. For the brace length of approximately 22 feet, a radius of gyration value, r , of at least 2.51 was required to meet the stringent slenderness criteria.

Local Buckling

Like all compression members, braces must have cross-sections of sufficient dimensions to prevent local deformations detrimental to global member strength. For a seismic lateral force resisting system this is especially critical due to the cyclic nature of the load such that the brace experiences tension and compression. Furthermore, a brace that buckles locally under a compressive load experiences a significant degradation of strength. Therefore, the *AISC Seismic Provisions* exceed the compactness, or width-to-thickness ratio, requirements of the *AISC Specification*. The differences in limiting width-to-thickness ratios of the two codes depend on the shape used. For example, the *AISC Specification* requires compact round HSS members to meet the following compact limit:

$$\lambda_p = 0.07 \frac{E}{F_y} \quad (\text{Equation from AISC 360-10 Table B4.1})$$

This limit is 60 percent higher than that provided by the *AISC Seismic Provisions* for moderately ductile member, as shown in the equation below. The latest *AISC Seismic Provisions* (AISC 341-10) have two categories of seismic compactness limits: λ_{md} for moderately ductile members and λ_{hd} for highly ductile members. The moderately ductile member limits basically match those provided by *AISC Specification* Table B4.1 with a few exceptions. The *AISC Seismic Provisions* define member ductility designations for components within the lateral force resisting system. For an OCBF, the brace is required to be moderately ductile but is not required to meet the more strict requirements of highly ductile members. Thus, the round HSS brace in this report's parametric study must meet the following requirement from Table D1.1 in the *AISC Seismic Provisions*:

$$\lambda \leq \lambda_{md} \therefore \frac{D}{t} \leq .044 \frac{E}{F_y} \quad (\text{Equation from AISC 341-10 Tbl D1.1})$$

This limitation can severely affect the use of larger cross-sections. Fewer of the larger cross-section HSS members meet the seismic compactness criteria because the range of manufactured thicknesses, per the ASTM A500 standard, is limited, although work has been done to expand the standard to meet the more stringent Seismic Provision criteria. This issue can greatly affect long braces because the larger brace cross-sections are needed to meet slenderness requirements due to larger radii of gyration. However, the larger cross-sections are also more apt to fail the compactness limits, which prevent undesirable plastic hinges from forming (Lawson, 2010).

The width-to-thickness requirements in the *AISC Seismic Provisions* are a result of observations of the Northridge Earthquake in 1994 and further research. From these results, it has been determined that ductility is dramatically affected by local buckling. Therefore, seismic compactness is more stringent than for general members because local buckling needs to be prevented as the brace is pushed into its inelastic range (Lawson, 2010).

Brace Strength

Once member stability is verified in accordance with the *AISC Seismic Provisions*, the calculation of member strength follows the *AISC Specification* without any additional requirements exceeding the *AISC Specification*. Because the brace, in the inverted-V configuration, already meets the more stringent requirements, it will also meet the criteria for Equation E3-2 in the *AISC Specification* to determine the critical buckling stress of the member:

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}} \right] F_y \text{ where } F_e \text{ is the elastic critical buckling stress (AISC 360-10 E3-2)}$$

For the parametric study, the brace compression capacity exceeded the applied load because a larger brace was used to meet the slenderness requirements. For taller buildings or buildings in areas with higher seismic activity a brace frame would be required to take larger forces. In this situation the compression strength would likely become the critical factor because the brace length would not change.

The tensile strength of the brace, like its compression strength, follows the procedure prescribed by the *AISC Specification*. Tensile yielding must be calculated and, depending on the layout of the connection, tensile rupture must also be computed to determine the critical tensile strength. For the parametric study the tensile strength greatly exceeded the applied load. However, if a large portion of the gross cross-sectional area were removed the tensile strength could govern the brace design.

Column Design

The design of the columns in an OCBF does not have any special constraints prescribed by the *AISC Seismic Provisions* because they will perform elastically during the MCE. Therefore, the column design, follows the design requirements of the *AISC Specification*. The major difference in design for a column that is part of a brace frame is that additional forces can be imparted to the column depending on the brace layout. For the inverted-V layout of the

parametric study, the columns had to resist additional load to resolve the vertical component of the brace force. In other brace layouts, X-braces for example, the vertical component of one brace is resolved by that of the other brace at the connection. Other than the additional force, the column was not drastically affected by being part of the ordinary braced frame system.

Beam Design

Beams in a OCBF must resist the combined effects of bending, compression and shear. For inverted-V brace layouts the *AISC Seismic Provisions* require that unbalanced forces in the braces also be examined because this imbalance must be resolved by an additional load imparted on the beam. The *AISC Seismic Provisions* require that the assumed force in the tension brace is equal to its expected yield strength:

$$P_t = R_y F_y A_g \quad [\text{Equation from AISC 341-10 Section F1.4a(1)(i)(a)}]$$

$$\text{For A500 Steel: } R_y = 1.4$$

The expected yield strength accounts for material overstrength with the factor R_y as specified in Table A3.1. The yield strength is a minimum specified strength and actual material strengths will be higher. Because the brace will buckle in compression, losing its strength, the force in the compression brace must be assumed to be 30 percent of the compression strength, which represents the residual post-buckling strength of the brace:

$$P_c = 0.3 F_{cr} A_g \quad [\text{Equation from AISC 341-10 Section F1.4a(1)(ii)}]$$

Thus, the axial force imparted on the beam is the sum of the horizontal components of the two specified brace strengths. The most critical load, however, is the load due to the imbalance of the vertical components because only a fraction of the compressive strength can resist the tensile yield strength, which is likely much larger than even the full compressive strength. This vertical load on the beam significantly increases the applied bending moment and shear in addition to the combined effects of compression. The beam is a force-controlled member and must be designed to resist the full elastic seismic load, V_e , as illustrated in Figure 3-1.

The unbalanced load is very large because of high tensile yield strengths. However, the *AISC Seismic Provisions* allow for the smallest of the expected yield strength, the load effect based on the amplified seismic load, or the maximum force that can be developed by the system to be the assumed force in the tension brace. Thus, the tension brace force can be reduced from the expected yield strength if one of the other situations generated a smaller load.

For the parametric study, the floor beam was preliminarily designed using the expected yield strength of the tension brace; the selected beam was a W27x235. The design was refined using the amplified seismic load and the assumed tension force in the brace was reduced significantly; the selected beam was reduced to a W18x86. The large size difference illustrates the effects of designing the frame using the brace capacity instead of the maximum force that is expected to be developed by the system. For the parametric study, the braces were governed by slenderness, and as a result the tensile capacity of the brace was very large. To resist the unbalanced load, the beam would have been very large. X-braced frame layouts avoid the complexity of unbalanced brace forces because tension and compression braces frame into the beam on both the top and the bottom, preventing any force imbalance.

Once the forces in the beams are determined based on analysis and the *AISC Seismic Provisions*, the beam can be designed in accordance with the *AISC Specification*. The beam does not need to meet the compactness requirements of Table B4.1 in the *AISC Specification* so that Section F2 can be used to determine the flexural strength of the beam. The beam does not need to be seismically compact, because, similar to the columns, the beam will remain elastic during the MCE. Because the beam is subject to combined bending and compression forces, the combined interaction must be checked. For the parametric study the moment portion of the interaction controlled over the compression; this is due to the very large bending force induced by the unbalanced brace load.

The unbalanced brace force on the beam is so large that the same size beam was required for both the floor and roof levels. In other words, the bending loads imparted from gravity loads were negligible compared to the very large unbalanced forces from the braces.

Chapter 5 - Ordinary Concentric Brace Frame Connections

The connections within an OCBF, like the design of the brace, beam, and column members, have relatively few special requirements for seismic design. A large variety of connection layouts are used in brace frames depending on a range of factors. Some of the important factors affecting connection designs include seismic performance requirements, brace frame geometry and member types, fabrication and installation constraints, and material and labor costs. This report focuses on the connections used in the parametric study design for an inverted-V brace frame with round HSS braces and wide flange columns and beams. The scope of this report includes welded connections to allow for easier comparison of the two systems and simplicity of connection layout, although other connection layouts are available. The following is a discussion of general design considerations for OCBF connections and specific applications and results for the parametric study design. See Appendix D for OCBF connection design calculations and detailing.

Design Forces

The only seismic requirement for the design of OCBF connections is the determination of design forces. The *AISC Seismic Provisions* specifies that diagonal brace connections must be designed for the amplified seismic load effect using the overstrength factor, Ω_o . However, the maximum tension force may be the expected yield strength of the brace. The maximum compression force is the smaller of $R_y F_y A_g$ and $1.14 F_{cre} A_g$. F_{cre} is the expected critical buckling stress and is taken as F_{cr} in the *AISC Specification* Chapter E with F_y replaced by the expected yield stress, $R_y F_y$.

Brace-to Gusset Design

At the point of connection of the brace to the gusset plate, an adequate weld is first selected, the gusset plate thickness is determined, and then tensile rupture in the brace is checked. Shear rupture of the brace must also be also checked, but this check was included in the design of the weld.

Brace-to-Gusset Connection

The wall of the HSS brace can only develop a certain amount of load per linear inch before it will fail in shear. Therefore, the capacity of the weld connecting the brace to the gusset plate must not exceed the HSS wall shear capacity of the gusset capacity. The maximum weld size can be determined by setting the shear capacity to the weld strength and solving for the weld size. The number of welds must also be determined and it is determined based on the geometry of the connection. For example, HSS braces are slotted through their center to accept the gusset plate, thus providing four locations for welds, as shown in Figure 5-1. The length of weld required for each of the welds can then be determined based on the ultimate axial load in the brace. For round HSS braces weld lengths must exceed the distance between welds, according to Table D3.1. This may govern the weld length for braces with smaller forces.

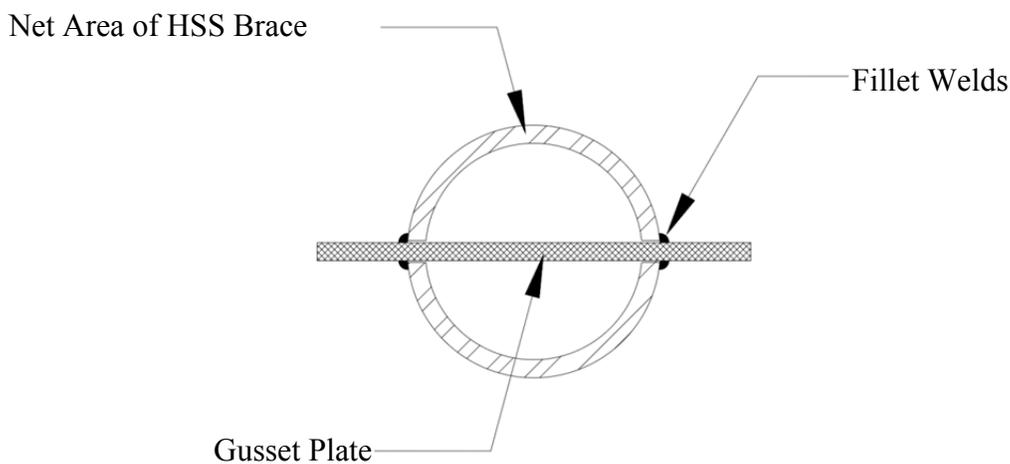


Figure 5-1: Net Area of HSS Brace at Gusset Connection

Tensile Rupture of Brace

Shear rupture of the brace has already been checked, however the slot cut in the brace induces shear lag effects. Tensile rupture strength depends on the net cross section, thus the gross cross-sectional area must be reduced by the area of the slot on both sides of the brace to determine the net area, as shown in Figure 5-1. The slot area must also account for erection tolerances, so 1/16" must be added to the gusset plate width on each side. The net area is then multiplied by a shear lag factor, U , as determined by Table D3.1 in the *AISC Specification*. The factor depends on the diameter of the brace and the length of the slot. The tensile rupture strength

can then be calculated. If the strength is not larger than the applied brace force, then cover plates may need to be added or the brace size increased.

Gusset Plate Design

The gusset plates are critical members for resisting the applied forces from the braces and transmitting them to the adjacent members. Gusset plates must meet certain geometric restraints to ensure stable structural behavior. They must also resist shear rupture due to tension and compression buckling.

Shear Rupture

An initial gusset plate thickness is determined by checking shear rupture of the plate along the two intersections with the brace. The applied load is the maximum brace force and the length of the shear plane equal to the weld length. The minimum plate thickness can then be determined. The plate must be thick enough to develop the fillet welds on both sides of the plate, or twice the min weld thickness specified by the *AISC Specifications*.

Compression Buckling

The strength of gusset plates subject to compression loads can be determined using *AISC Specification* Equation J4-6 for $\frac{KL}{r} \leq 25$. This serves as a practical limit for gusset plate design because more complicated design procedures must be followed per *AISC Specification* Chapter E for $\frac{KL}{r} > 25$. A K value is determined based on the fixity at each end of the gusset. The parametric study gusset plate is welded to the brace and the column and the gusset is assumed to be fixed at both ends. Furthermore, because the gusset plate fixities are approximated K was taken to be 0.65. The unbraced length, L, of the gusset plate is also approximated based on the geometry of the connection. The radius of gyration, r, can be calculated based on the plate thickness.

Once the conditions of J4-6 are met, the required area can be determined to resist the applied compression from the brace. The plate thickness is known from previous calculations, so the width is the only unknown. The geometry of the gusset plate must ensure that the Whitmore width is larger than the calculated minimum width.

Whitmore Section

The full width of the gusset plate perpendicular to the load is not necessarily effective in resisting tension or compression due to the force distribution in the steel. Therefore, only the area that is effective in resisting the force should be used in calculations regarding the cross-sectional area of the gusset plate. This area is determined by calculating the Whitmore Section to determine the effective width. The length of the section is equal to the length of the joint. In the case of a welded joint this is the length of the weld. For bolts the length is equal to the center to center distance between the first and last bolts. The width of the Whitmore Section is defined by a two lines projected from the either side of the start of the joint at a 30° angle. The width is measured between the points of intersection of the end of the joint and the projected line. Examples of the Whitmore section for both a bolted and a welded connection are shown in Figure 5-2, where the dashed lines indicate the limits of the Whitmore section. The Whitmore section is allowed to extend into adjacent connected members, but the section may not extend beyond an unconnected edge. To simplify connection design for the parametric study, the geometry of the Whitmore Section was chosen such that the section was wholly contained by the gusset plate alone.

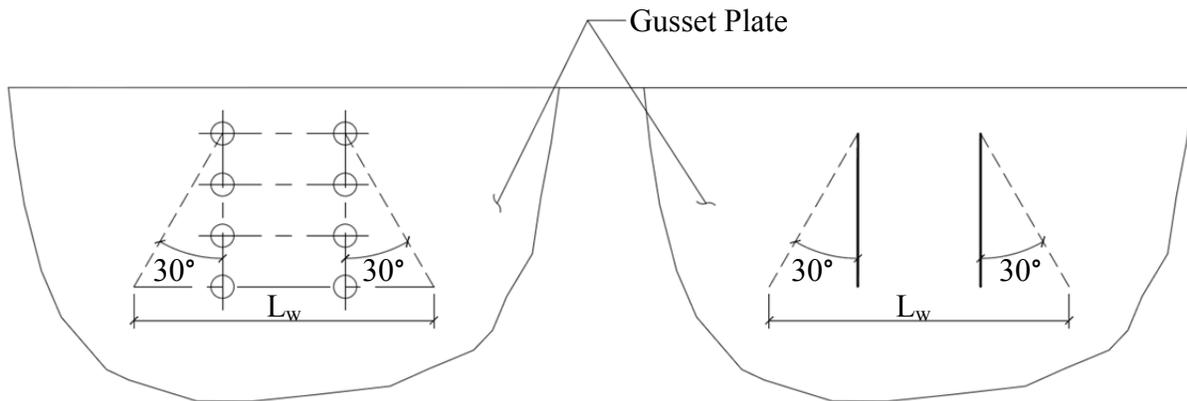


Figure 5-2: Whitmore Section

Gusset-to-Beam Connection

Two types of gusset-to-beam connections are used in an inverted-V brace frame. The first occurs at the beam connection to the column, and the second occurs at the mid-span of the beam

at the intersection of both braces. The forces at the interaction plane between the gusset and beam are determined using the uniform force method outlined in Chapter 13 of the *AISC Steel Construction Manual*. Both vertical and horizontal forces are applied to the beam by the gusset and the weld must be designed for that interaction. Additionally, the following limit states must be checked at the gusset-to-beam weld: gusset plate rupture, gusset plate yielding, beam web local yielding, and beam web crippling.

Gusset-to-Beam Weld

The gusset-to-beam weld must be designed for the interaction of both the horizontal and vertical components of the applied force, V_{ub} and H_{ub} respectively. First, the load angle must be determined, which is the angle, θ , formed by the result force relative to the long axis of the weld.

$$\theta = \tan^{-1} \left(\frac{V_{ub}}{H_{ub}} \right) \quad \text{Eqn 5-1}$$

The strength of the fillet weld, ϕr_n , in resisting the applied load at angle θ is determined by the *AISC Specification* Equations J2-4 and J2-5. The weld is sized to resist the resultant of the maximum shear and tensile load applied by the gusset plate.

Gusset Shear Rupture and Yielding

The gusset plate has already been checked for the limit states of shear rupture, shear yielding, and tension yielding for the connection to the brace. However, because the welds are different at the beam, these limits state must be checked again. These checks are simplified because the net area is equal to the gross area of the plate. Therefore, shear rupture is the governing limit state.

Web Local Yielding and Beam Web Crippling

The forces to the beam from the uniform force method are shear and axial forces only. The beam must be checked to ensure that web stability is maintained. For tension loads applied to the beam web, local yielding is a critical failure mode. For web local yielding, the most critical location is near the end of the beam, as the beams resistance to web yielding is lowest nearest the beam ends. However, web local yielding must be checked regardless where the tensile load is applied. *AISC Specification* Section J10.2 provides equations to determine the yielding strength.

When the brace force applied is compression, web crippling is a critical limit state. *AISC Specification* Section J10.3 provides equations to determine the crippling strength. If the beam does not have enough capacity in either web local yielding or web crippling, transverse stiffener plates can be added to the beam web.

Gusset-to-Column Connection

The gusset-to-column connection follows the same procedure as the gusset-to-beam connection. First the forces must be determined from the uniform forces method. Although the design procedure is the same for both the gusset connections to the beam and column, the applied forces at the two locations likely differ in magnitude. However, both connection locations must handle vertical and horizontal components of force. From those forces the gusset-to-column weld can be sized. Then, shear rupture, shear yielding, and tension yielding must be checked. The column web, since the column experiences a horizontal load, must be checked for web local yielding and web crippling. See the gusset-to-beam connection discussion for more information regarding the aforementioned limit states.

Beam-to-Column Connection

The beam-to-column connection must be designed for both the gravity loads applied to the beam and the vertical and horizontal components of the brace force. According to the *AISC Seismic Provisions*, the beams in an inverted-V brace frame must be continuous between columns. Therefore, connections using a beam stub shop welded to the column are not permitted. Beams in this type of frame layout are critical because they must resolve the force imbalance cause by brace buckling.

Chapter 6 - Special Concentric Brace Frame Member Design

Special Concentric Brace Frame components must adhere to stringent criteria set forth by the *AISC Seismic Provisions*. For member design, the goals of the Provisions are to ensure robust seismic performance, provide design flexibility by providing standards that cover a variety of member types and configurations, ensure stable post-elastic behavior (Roeder & Lehman 2008). First, the *AISC Seismic Provisions* specifically define the analysis procedures that must be performed. SCBF's are designed based on elastic analysis, but the frame's post-elastic behavior is of primary importance. The analysis requirements account for the inelastic behavior of the frame.

SCBF's differ from OCBF's in their unique and strict requirements that result in stable ductile behavior of the seismic lateral system. During a large seismic event, braces experience cyclic loading in compression and tension. For braces experiencing compression, the desired failure mode is buckling with the formation of plastic hinges, similar to beam plastic hinges in a moment frame system. For braces in tension, tension yielding is the desired ductile failure mode. To ensure that these two failure modes are the governing limit state, other frame components must be designed and detailed properly. In a severe earthquake, the braces may fail but the integrity of the columns and beams must be maintained.

SCBF Analysis

For the design of the braces themselves, an analysis must be performed in accordance to ASCE 7-10. This simple analysis assumes elastic behavior of the frame. However, for the design of other members in the frame, an analysis must account for the actual strengths of the braces and the vastly different performance of braces acting in tension and braces acting in compression. Thus, the *AISC Seismic Provisions* specify the following two analyses to examine this differing behavior:

- i) All braces in the frame are assumed to resist forces equal to their expected yield strengths in tension or compression.
- ii) All braces acting in tension are assumed to resist forces equal to their expected strength and all braces acting in compression are assumed to resist forces equal to their expected post-buckling strength.

Performing both analyses is critical in determining the maximum possible forces a particular member in the SCBF must resist. For example, in the inverted-V brace configuration of the parametric study, the beam design is governed by unbalanced forces induced from brace buckling. See Appendix E for all SCBF member design calculations.

Brace Design

Braces are the primary means of lateral force resistance and energy dissipation within a SCBF. As such, their ductility is critical to the performance of the whole system. A major cause of brittle failures in braces is buckling of the member. When a brace buckles, a plastic hinge is formed at the midpoint of the member. At this location, local buckling occurs resulting in increased material strain and fracture. This undesirable behavior is prevented by using a more compact member; increased compactness results in increased ductility of the brace. Due to the importance of preventing local buckling, the *AISC Seismic Provisions* require that braces meet the strictest local slenderness criteria as “highly ductile” members, as prescribed by Chapter D. For the round HSS braces in the parametric study, the limiting width-to-thickness ratio was as follows:

$$\lambda_{hd} = 0.038 \frac{E}{F_y} \quad \text{qn 6-1}$$

Comparatively, the limiting width-to-thickness ratio for moderately ductile members, as in the OCBF is:

$$\lambda_{md} = 0.044 \frac{E}{F_y} \quad \text{Eqn 6-2}$$

So, because the SCBF braces are more compact they exhibit improved ductility.

Global buckling is a favorable failure mode because the plastic hinges that form help dissipate seismic energy. Furthermore, research has shown that increasing the slenderness ratio of the brace increases its post-buckling strength when subjected to cyclic loading. The upper limit of 200 is provided to prevent detrimental effects of dynamic brace behavior for members with very high slenderness ratios. Because OCBFs are assumed to perform within the elastic range, brace buckling is undesirably and the brace slenderness ratios are decreased accordingly.

Net section rupture of the brace is also a critical limit state. For brace members with reduced cross-sections, this failure mode can govern resulting in reduced member ductility. To prevent this, the *AISC Seismic Provisions* require that a brace’s net area at least equal the gross

area. Many connections rely on slots being cut into the brace. These braces must be reinforced to increase the net area effective in resisting rupture at the brace end. Requirements for the reinforcement are also provided by the *AISC Seismic Provisions*.

Column Design

Columns are especially critical members because their failure can preempt progressive collapse of the surrounding structure. Preservation of columns is, therefore, an important goal of the *AISC Seismic Provisions*. It follows that columns must be designed based on amplified seismic loads. Additionally, the load effects of brace overstrength (the two analyses discussed previously) and brace buckling must also be considered in accordance with the Provisions. In some cases it is not possible for the brace frame system to develop the load according to the analysis. In these cases, the columns could be unnecessarily oversized for forces they will never actually experience. For these cases, the *AISC Seismic Provisions* supply the following three exceptions which establish maximum required strengths of columns:

- 1) Column forces determined from an analysis with load combinations using amplified seismic forces on a frame without using any compression braces
- 2) Column forces at the point of foundation failure due to overturning
- 3) Column forces from a nonlinear analysis per Section C3 of the *AISC Seismic Provisions*

These exceptions allow for more economical design while ensuring adequate strength.

Another added protection for columns is in local slenderness limits. Brace frame columns must meet the same criteria as braces, meeting the strictest standards for highly ductile members. The reason for this is to ensure that the column can withstand significant rotations due to story drift and are also capable of maintaining significant flexural strength. Research has shown that these two factors significantly affect the global stability and performance of the brace frame. The limiting width-to-thickness ratio for column flanges as a highly ductile member is the following:

$$\lambda_{hd} = 0.30 \sqrt{\frac{E}{F_y}} \quad \text{Eqn 6-3}$$

For W-shaped members, web compactness is also dependent on C_a , the ratio of the ultimate axial beam load to the axial beam strength. This provision allows for variance in required compactness

for member with small axial loads compared to their strength. The limiting width-to-thickness ratio for column webs as a highly ductile member is the following:

$$\text{For } C_a \leq 0.125, \lambda_{hd} = 2.45 \sqrt{\frac{E}{F_y} (1 - 0.93C_a)} \quad \text{Eqn 6-4}$$

$$\text{For } C_a > 0.125, \lambda_{hd} = 0.77 \sqrt{\frac{E}{F_y} (2.93 - C_a)} \geq 1.49 \sqrt{\frac{E}{F_y}} \quad \text{Eqn 6-5}$$

Because of the importance of column strength and stability in a brace frame, column splices must also be carefully designed and detailed. Although column splices are beyond the scope of this report, for taller buildings they are necessary. The detailing requirements as well as strength requirements are provided by *AISC Seismic Provisions* in Section F3.6d.

Beam Design

For inverted-V braced SCBFs, beam design is of particular concern for the same reasons as in an OCBF. The unbalanced force occurring on the beam due to brace buckling is an important consideration because of the assumed behavior of a SCBF. As the force on the frame increases, the brace in compression buckles and its strength decreases dramatically. As the compression brace strength decreases, the tension brace must resist a higher load, which increases until yielding of the brace. This is a favorable combined failure mode, buckling and tension yielding, in which the SCBF achieves its ductility and subsequent energy dissipation. To ensure that this behavior occurs before failure of other part of the frame, the beam must be designed with sufficient strength to resist the unbalanced brace load. It follows that the beam connections must also be capable of resisting the unbalanced load.

Lateral-torsional buckling can significantly reduce the capacity of the beam, especially due to the large unbalanced load occurring at the beam midpoint. Therefore, the *AISC Seismic Provisions* prevent lateral-torsional buckling from becoming a governing limit state by requiring intermediate lateral braces at the brace intersection location. However, an exception to this requirement is provided if the beam has “sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points” according to the *AISC Seismic Provisions*. *AISC Specification* Appendix 6 provides a method for determining lateral bracing required strengths. If intermediate beam frame into the SCBF beam they will likely provide sufficient brace strength. In the absence of secondary framing, the SCBF beam must be provided with other bracing.

Alternatively, Appendix 6 can be used to verify that the beam itself has enough inherent strength and stiffness to be stable without bracing per the Provision exception.

Ductility of the beam, like columns and braces, is important to the overall performance of the frame. However, beam ductility is not quite as critical as that of braces and columns, in part because the axial load in beams is relatively small. This is reflected in the local buckling requirements for beams. Beams, in all configurations of concentric brace frames, must meet the width-to-thickness limits for moderately ductile members according to Chapter D of the *AISC Seismic Provisions*. According to Table D1.1 the flanges must meet the following limit:

$$\lambda_{md} = 0.38 \sqrt{\frac{E}{F_y}} \quad \text{Eqn 6-6}$$

For beam webs, the limiting width-to-thickness ratio depends on C_a , the ratio of the ultimate axial beam load to the axial beam strength. The two possible limits for web width-to-thickness ratios are as follows:

$$\text{For } C_a \leq 0.125, \lambda_{md} = 3.76 \sqrt{\frac{E}{F_y}} (1 - 2.75C_a) \geq \quad \text{Eqn 6-7}$$

$$\text{For } C_a > 0.125, \lambda_{md} = 1.12 \sqrt{\frac{E}{F_y}} (2.33 - C_a) \geq 1.49 \sqrt{\frac{E}{F_y}} \quad \text{Eqn 6-8}$$

These limits are obviously less stringent than those for columns and braces if W-shapes are used. The moderate ductility limit for beam flanges is the same as the limiting width-to-thickness ratio for “compact” shapes according to Chapter B of the *AISC Specification* in Table B4.1. The limiting ratio for beam webs, however, differs from Table B4.1 in that the limit depends on C_a with a convergence on the “non-compact” limit in the table.

Chapter 7 - Special Concentric Brace Frame Connection Design

Connection design is extremely critical to the behavior of the entire brace frame. Numerous research has been done to study frame behavior relative to a variety of connection design differences or issues, and all have drawn a similar conclusion: the brace frame is not nearly as robust under cyclic loading if the connection is not capable of fully developing the brace capacity. Therefore, the *AISC Seismic Provisions* have very strict analysis and design criteria for SCBF connections. The process of design is much the same as that of OCBF connections, and the following discussion will focus on the differences in analysis, design, and behavior in SCBF connections. See Appendix F for all SCBF connection design calculations and detailing.

Brace Connection Design Forces

Design forces for connections differ from those used to design braces, beams, and columns. Connection failure is detrimental to the entire brace frame, especially due to the effects of cyclic seismic loading are not accounted for. Connections must be able to withstand the effects of extreme deformations including brace buckling. To achieve this, the *AISC Seismic Provisions* have set forth specific criteria for determining these loads for brace connections. Tension, compression, and flexural loads are all covered by the *AISC Seismic Provisions*. Each of these loads may be analyzed separately and interaction of the forces does not necessarily need to be considered.

Tension

Brace connections must be designed for the “required tensile strength” as defined by the *AISC Seismic Provisions* in Section F2.6c(1). Two options are available to determine this force and the required force is the lesser of the two. The first force is equal to the expected yield strength of the brace, $R_y F_y A_g$. The second force is equal to the maximum load effect that can be developed by the frame and transmitted to the connection, based upon analysis.

To achieve ductile behavior of the frame, the brace design must be governed by tensile yielding of the gross cross-section when subject to tension loads. To ensure that this occurs, the brace connection must be designed to at least match the expected brace strength. However, a

detailed analysis can determine that other parts of the components limit the system capacity, which would allow the connection to be designed for smaller forces.

Calculating the expected yield strength of the brace is by far the simplest method of determining the required design force. Calculation of the maximum load effect is very complex and may not accurately reflect the actual behavior of the frame during an earthquake. Additionally, many unknowns regarding ground motion and the distribution of seismic forces exist. Even after performing a detailed analysis it may still be necessary to design the connection for the expected yield strength to ensure adequate ductility. Therefore, it is most common, and perhaps the best practice, to design the connection for the expected yield strength (AISC 341-10).

Compression

Due to the critical nature of brace buckling behavior, it is required that brace connections be capable of delivering the compressive strength of the brace. Therefore, the *AISC Seismic Provisions* require that brace connections be designed to resist a force at least equal to expected brace strength in compression multiplied by a factor of 1.1. The expected brace compressive strength is the same as that used in the frame analysis as defined in Section F2.3. The factor of 1.1 is used, in part, to account for brace overstrength, a result of the conservative design approach for compression members.

Flexure

SCBF's are designed with the expectation that they will experience cyclic loading during a severe earthquake. As discussed previously, it is very important that the brace be allowed to buckle, dissipating energy. When a brace buckles, plastic hinges are formed at the middle and ends of the brace inducing flexural forces in the gusset plate. The plastic hinges at the brace ends can cause failure of the end connection due to the rotation of the brace plastic hinge. To prevent this, the end connection must have sufficient strength to force the brace end plastic hinge to occur in the brace outside of the connection. Alternately, the connection can be designed such that it has enough ductility to allow the brace to rotate within the connection. The *AISC Seismic Provisions* require that brace end connections conform to one of the two options in Section F2.6c(3).

For typical gusset plate connection it is very difficult to provide sufficient ductility for brace buckling in the plane of the gusset plate. Therefore, for in-plane brace buckling the connection will most likely need to be designed to withstand the maximum flexural strength of the brace multiplied by a factor of 1.1 per Section F2.6c(3)(a). The connection must also be designed for the maximum compression strength of the brace.

For brace buckling out-of- plane of the gusset plate, the gusset plate is much more flexible as it is subject to bending about its weak axis. This allows the brace end plastic hinges to form in the gusset plate. For this to occur, the gusset plate connection must be detailed so that enough clearance is provided to allow the plastic hinge to form unimpeded. The distance between the brace end and the gusset restraint must be large enough to allow the gusset to buckle. However, it is necessary for the brace to buckle prior to buckling of the plate. Thus, the clear distance of the brace end must not be too large (*AISC Seismic Provisions*).

Through their research, Astaneh-Asl et al. determined a minimum clear distance of two times the gusset plate thickness. This clear, or free distance is measured from the brace end to a fold line perpendicular to the brace intersecting the nearest gusset plate restraint as shown in Figure 7-2. This simple detail provides significant ductility for the connection when subjected to cyclic loading. The *AISC Seismic Provisions Commentary* recommends adding an inch to the minimum clear distance to provide for erection tolerances.

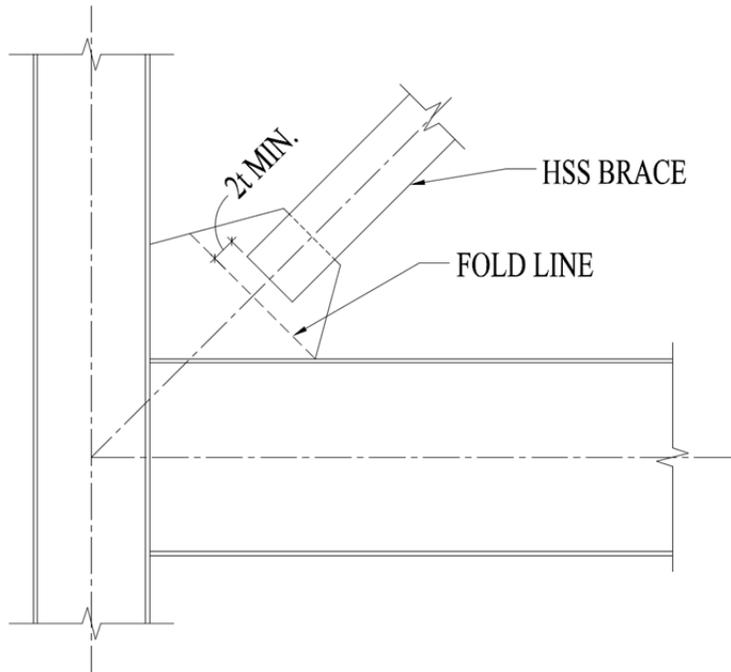


Figure 7-1: Brace End Clear Distance

Shear Lag and Brace Section Rupture at Gusset

One very common method for connecting an HSS brace to a gusset plate is slotting the tube to receive the gusset plate. This is a relatively simple and inexpensive method. However, the slot in the HSS obviously reduces the area of the brace that is effective in resisting the applied load. The Seismic Provision requirement in section F2.5b(3) states that “the brace effective net area shall not be less than the brace gross area.” This provision prevents net section rupture from becoming the limiting condition (*AISC Seismic Provisions*). Such fracture would be a sudden brittle failure with a small amount of deformation, an undesirable failure mode for seismic conditions (Cheng et al., 1998).

Shear lag must also be accounted for in the design of knife plate brace connections subject to tensile loads. When a tensile member is connected such that only a portion of its cross-section is fastened the effects of shear lag are introduced. At the connection location the full cross-section of the member is not effective in resisting the load and the member is not stressed uniformly. The effects of shear lag are dependent on the geometry of the connection. For a round brace welded to a knife plate, the welds parallel to the brace must be at least as long as the

diameter of the brace according to Chapter D of the *AISC Specification*. If the welds are longer than 1.3 times the brace diameter than shear lag effects are essentially eliminated because the member net cross-section (the full cross-section minus the area of the slot) is assumed to be fully developed.

Research performed by Cheng et al. examined the effects of shear lag on shear rupture of welded knife plate connections. In this research gusset plates were inserted into slotted round HSS members. They were welded with four fillet welds along the length of the slot. Most of the specimens were welded across the width of the gusset plate at the end of the slot allowing the full gross cross-section of the tube to be developed. A control specimen was not welded across the gusset thickness; only the net cross-sectional area was therefore assumed to be effective in resisting the load. The specimens were loaded to failure in tension. The results indicated that the specimens had greater ductility when the gusset width was welded. The control specimen fractured at the connection. However, all of the specimens experienced higher deformations at the end of the slot due to stress concentrations caused by shear lag effects (Cheng et al., 1998).

The method described in Cheng's research is an effective way to meet the requirements of the *AISC Seismic Provisions* to develop the full brace cross-section. However, it is not the most practical method for braces installed in the field and it can cause stress concentrations at the slot end (Cheng et al.). Slots must be cut longer to provide erection clearances making the slot-end weld impossible. Instead, reinforcement in the form of cover plates can be used to increase the effective area at the connection and develop the gross brace cross-section, as shown in Figure 7-1. The *AISC Seismic Provisions* have two requirements for brace reinforcement. First, the reinforcement yield strength must match or exceed that of the brace. Second, the full capacity of the reinforcement must be developed on each side of the connection. These requirements ensure that brace can be fully developed and ductility is provided at the connection between the brace and gusset (*AISC Seismic Provisions*).

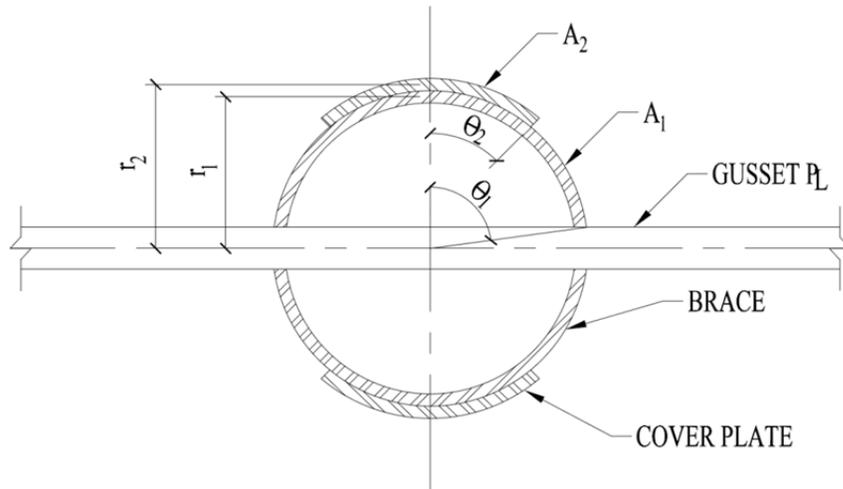


Figure 7-2: Reinforced Brace Connection

Protected Zones

Because of the high performance objectives of SCBFs special precautions must be made for critical parts of the frame. In areas expected to yield significantly in plastic hinges a stress concentration can cause fracture, eliminating the desired ductile behavior. Stress concentrations can occur in a member where welds, holes, changes in cross-section, or flaws are present, often results of construction processes or defects. These irregularities in a critical member can cause undesirable and unpredictable behavior when subjected to the extreme loading conditions present during a large earthquake. Therefore, the *AISC Seismic Provisions* specify protected zones within seismic lateral force resisting systems that must be free of such irregularities. As shown in Figure 7-3 by hatching the area, the following protected zones are defined for SCBFs:

- The middle of the brace equal to a quarter of the total brace length
- The ends of the brace at the connection equal to the brace diameter
- Connecting elements for braces, beams, and columns

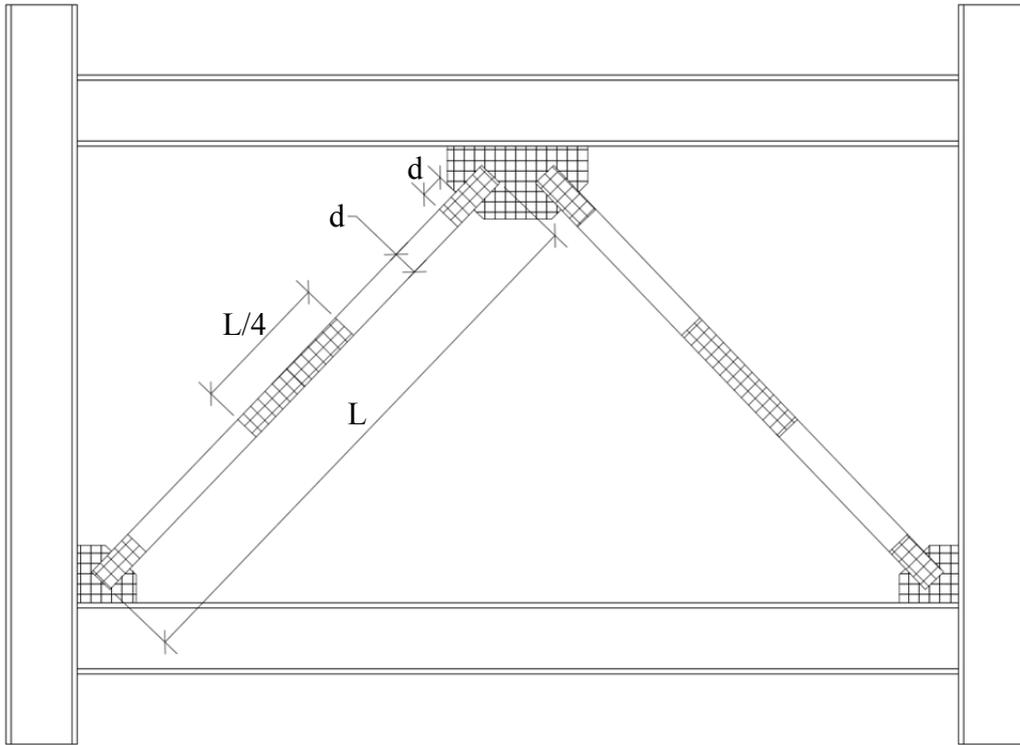


Figure 7-3: Protected Zone

Free-Edge Buckling

The 2005 Seismic Design Manual recommended checking free-edge buckling of the gusset plate at the beam-brace connection, although the check was not a requirement of the 2005 *AISC Seismic Provisions*. However, the 2010 Seismic Design Provisions Commentary briefly discusses the free-edge buckling. The reviewing committee determined that gusset plate edge stiffeners do not improve the behavior of the gusset plate. As a result, no limits to gusset plate edge dimensions have been established.

Demand Critical Welds

The *AISC Seismic Provisions* identify locations within the brace frame that can experience high stresses. These locations, occurring at joints, are critical in maintaining the performance of the braced frame. Therefore, the welds at such joints must be capable of withstanding increased levels of yielding in addition to higher stresses. These welds are specified as demand critical welds and must meet the requirements of the *AISC Seismic Provisions*.

All welds in a seismic force resisting system must meet the specifications of the Structural Welding Code-Seismic Supplement (AWS) Section D1.8. Demand critical weld requirements are more stringent. In addition to meeting D1.8 requirements, such welds must also conform to increased Charpy V-notch (CVN) toughness criteria. The filler material must undergo Heat Input Envelope Testing at two different temperatures to determine the CVN toughness. The required test temperatures can vary depending on the use of the structure, and are based on the expected temperatures the structure will experience.

The locations within a concentrically braced frame that require demand critical welds are defined by the *AISC Seismic Provisions* as follows:

- 1) Groove welds at column splices
- 2) Welds at column-to-base plate connections
- 3) Welds at beam-to-column connections that meet Section F2.6b(b)

Seismic forces during an extreme event are difficult to predict and the forces at these critical locations are unknown even with a detailed analysis. Demand critical welds provide additional safety by providing increased ductility to prevent brittle failures (*AISC Seismic Provisions*).

Beam-to-Column Connections

The *AISC Seismic Provisions* offer some design flexibility for beam connections to columns within brace frames. The connection may be designed in accordance with *AISC Specification* Section B3.6a as a simple connection. Alternatively, the beam-to-column connection must be designed to resist a moment equal to the beam flexural strength, $R_y M_p$, or a moment equal to the sum of expected column flexural strengths, $\sum R_y F_y Z$. The expected flexural strengths must be multiplied by a factor of 1.1. The design moment must be examined with other connection forces (*AISC Seismic Provisions*).

The use of a simple connection allows the beam to rotate relative to the column. This is important because brace frames are expected to have large story drifts which results in large rotations between beams and columns. Rotation of this joint can lead to poor frame performance if the connection is not designed to rotate or withstand the rotation. At joints with a gusset plate this is especially critical. The *AISC Seismic Provisions* require that beam-to-column connections designed in accordance with *AISC Specification* Section B3.6a allow a rotation of 0.025 radians.

AISC Specification Section B3.6a states that “a simple connection transmits a negligible moment across the connection.” In analysis, simple connections can be assumed to rotate uninhibited and the connection must be designed to provide a rotation capacity required by the analysis, or in this case, required by the *AISC Seismic Provisions*. Recent research has been performed to verify the rotation capacities of some simple connections. The alternative beam-to-column connection must resist the specified moment. In this case, a fully restrained moment connection, meeting the same requirements as ordinary moment frames, is required (*AISC Seismic Provisions*). While this could increase system strength, it would also increase costs.

Chapter 8 - Conclusions

The parametric study was performed to illustrate the differences in the design and detailing of OCBFs and SCBFs. The following conclusion is a discussion of the results and findings of the study.

Governing Lateral Load

The parametric study illustrated that determining the governing lateral load is not as simple as comparing total building base shears for wind and seismic. Seismic base shear is the same for both the longitudinal and transverse directions because no plan irregularities causing inherent torsion exist. However, wind base shear differs for the longitudinal and transverse directions because the wind pressure acts on differing wall areas for the two directions. This is especially true for rectangular buildings with lengths significantly longer than their widths. For this reason, both directions must be checked.

It is also important to consider the effects of an increased response modification coefficient. Systems with higher coefficients have lower base shears. It is possible that a reduction due to a higher response modification can reduce base shears below wind base shears. In this case, the lateral force resisting system must be designed to remain elastic up to the governing wind base shear, but it must also meet the requirements of the LFRS to achieve the ductility described by the response modification coefficient.

For the parametric study the OCBF seismic base shear was much higher than the wind base shear in both directions, and therefore governed design. The seismic base shear for the SCBF design, however, only exceeded the wind base shear for the LFRS in the longitudinal direction. Therefore, seismic loads governed for the longitudinal direction, but wind loads governed for the transverse direction. The wind and seismic base shears are summarized in Table 8-1.

Table 8-1: Comparison of Building Base Shears

Total Building Base Shear (kips)			
	Wind	OCBF Seismic (R=3.25)	SCBF Seismic (R=6)
Longitudinal	68	143	78
Transverse	118	143	78

Brace Slenderness Limits

The brace slenderness limits differ for OCBFs and SCBFs, which has a significant effect on the design of the entire frame. OCBFs, relying on the elastic strength of the frame, have a more stringent limit on brace slenderness to prevent brace buckling. For the OCBF parametric study, the design of all braces was governed by the brace slenderness limit. For OCBFs, this limit exists only for inverted-V brace layouts and prevents brace buckling that would result in an unbalanced load on the beam. The slenderness limit for SCBFs is in place for all configurations of braces. The limit is higher because research has shown that for SCBFs with slender braces still behave well due to their overstrength in tension, according to the *AISC Seismic Provisions*. The braces are governed by strength rather than slenderness which results in smaller brace sizes compared to those of the OCBF design. Because the SCBF was designed based on the expected strength of the braces, reducing the brace sizes allowed the beams and columns to be reduced as well.

Member Sizes

Aside from the braces, the SCBF beams and columns also differed significantly from their OCBF counterparts. This difference is primarily due to the analysis requirements of the two frame types. The beams and columns in the OCBF were designed for the maximum load effect base on amplified forces. The SCBF beams and columns were also designed based the load effect based on amplified forces. However, the analysis also accounted for the expected strengths of the braces and the resulting unbalanced load due to the inverted-V brace layout. The result was significantly larger beams and columns for the SCBF compared to the OCBF. A comparison of member sizes is shown in Table 8-2. The larger sizes for the SCBF are representative of the increased seismic robustness of SCBFs compared to OCBFs. This robustness exists because the SCBF design is centered on developing the full strength of the brace rather than just the maximum load effect of the system.

Member sizes can also provide a rough cost estimate base on their weight. Steel tonnage is often used to represent the total structure cost. A comparison of steel tonnage of a single brace frame for the OCBFs and SCBFs is shown in Table 8-3. While steel tonnage does reflect cost, it does not completely reflect the additional costs due to connection complexity of SCBFs.

Table 8-2: Comparison of Braced Frame Member Selection

Braced Frame Member Selection		
	OCBF	SCBF
Upper Braces	HSS 7.500x0.312	HSS 4.000x0.220
Lower Braces	HSS 7.500x0.312	HSS 5.500x0.258
2 nd Floor Beam	W18x76	W30x124
Roof Beam	W10x33	W27x94
Columns	W12x40	W14x68

Table 8-3: Comparison of Braced Frame Steel Tonnage

Braced Frame Member Weights (Tons)			
	OCBF	SCBF	Difference
Upper Braces	0.80	0.32	-0.48
Lower Braces	0.80	0.52	-0.28
2 nd Floor Beam	2.28	3.72	1.44
Roof Beam	0.99	2.82	1.82
Columns	2.56	4.35	1.79
Total	7.44	11.73	4.29

Inverted-V Brace Layout

The inverted-V brace layout used for the parametric study introduces several unique requirements to the design. The first special requirement specified by the *Seismic Provisions* is for beams. For both OCBFs and SCBFs the beam must be continuous between supporting columns. This limits the connection options, because a beam stub cannot be shop welded to the columns. This requirement can complicate erection because the gusset plate must then be at least partially field welded or bolted. The beam must also be braced at the brace intersection point unless the beam has sufficient out-of-plane strength. Additionally, the beam must also be designed to withstand the force imbalance caused by brace buckling. This requirement resulted

in significantly larger beams than beams for a similar X-braced frame would have been. Beams in SCBFs must meet the compactness criteria for moderately ductile members.

The braces themselves are only affected by the layout in OCBFs. As mentioned previously, OCBF braces have slenderness limits that are more stringent than those for SCBFs. This is, again, to prevent the adverse effects of brace buckling resulting in an unbalanced load on the beam.

Story Drift

For the parametric study the building story drifts were well within the limits established by ASCE 7-10 for both OCBFs and SCBFs. This is due in large part to the requirements of the *Seismic Provisions*. For the OCBF, the braces were significantly oversized to meet the slenderness limits for inverted-V brace layouts. Therefore, the building did not deflect significantly under the elastic lateral loads applied. Similarly, for the SCBF, the beams and columns had to be much larger to meet the *Seismic Provisions*. The building deflections are well below the allowable story drift limits as a result. The story drifts and story drift limits are summarized in Table 8-4.

Table 8-4: Summary of Building Story Drifts

Building Story Drift Summary		
	OCBF	SCBF
Allowable Story Drift (in.)	3.840	3.840
Design Story Drift (in.)	0.517	0.690

General Conclusions

Member design for SCBFs differs only slightly from the design of OCBF members. However, their predicted behavior differs dramatically; OCBFs behave elastically while SCBFs rely more heavily on system ductility in the post-elastic range. The major differences in design requirements are found in compactness criteria and slenderness limits. SCBF members must meet much stricter width-to-thickness limits for braces, columns, and beams than their OCBF counterparts. This provides additional ductility. OCBF braces are subject to more stringent limits

on member slenderness than SCBF braces. This is because SCBFs rely, in part, on brace buckling to develop plastic hinges providing energy dissipation and system ductility.

The cost of SCBFs will be significantly higher than that of the OCBF because of the detailing requirements of SCBFs. The addition of cover plates to the braces adds to the fabrication complexity. To provide clearance for the gusset plate fold lines, the gusset had to be larger than it would have otherwise. The HSS braces in the SCBF were smaller than those in the OCBF, but the difference in steel weight was minor compared to the additional weight for the SCBF beams and columns. SCBFs are certainly more expensive to construct, however, they have significantly better performance during extreme seismic events. Additionally, OCBFs are limited to buildings under 35 feet in height. Taller buildings must use SCBFs, eccentrically braced frames, buckling restrained brace frames, or another SLFRS. OCBFs provide a safe and inexpensive lateral force resisting system for low rise structures and SCBFs provide excellent seismic performance for extreme seismic loads for larger buildings.

References

- American Institute of Steel Construction Manual. 14th ED.* (2010). Chicago: American Institute of Steel Construction.
- American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures 2010.* (2010). American Society of Civil Engineers.
- Building Seismic Safety Council. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures: FEMA 450-1 2003 Edition.* (2004). Washington, D.C.: Federal Emergency Management Agency.
- Cheng, R.J.J., Kulak, G.L., and Khoo, H.A. *Strength of Slotted Tubular Tension Members.* (June 1999). Department of Civil and Environmental Engineering, University of Alberta, Edmonton.
- International Building Code 2012.* (2012). Country Club Hills: International Code Council, Inc.
- Lawson, J. W. (2010, February). A Solution to Seismic Bracing Restrictions: Expanding the Acceptance of New Large HSS Sections. *Structure Magazine* , pp. 14-15.
- American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures 2010.* (2010). American Society of Civil Engineers.
- Nikolaou, S. P. (2008, February). Site-Specific Seismic Studies for Optimal Structural Design: Part I-General. *Structure Magazine* , pp. 15-19.
- Roeder, C. P. and Lehman, D.E. (2008, February). Seismic Design and Behavior of Concentrically Braced Steel Frames. *Structure Magazine* , pp. 37-39.
- SEAOC Seismology Committee. (2008, September). A Brief Guide to Seismic Design Factors. *Structure Magazine* , pp. 30-32.
- SEAOC Seismology Committee. (2009, January). Seismic Force-Resisting Systems Part 1: Seismic Design Factors. *Structure Magazine* , pp. 27-29.
- Seismic Provisions for Structural Steel Buildings: AISC 341-10.* (2010). Chicago: American Institute of Steel Construction.
- Seismic Provisions for Structural Steel Buildings: AISC 341-05.* (2005). Chicago: American Institute of Steel Construction.

Appendix A - Building Plans and Elevations

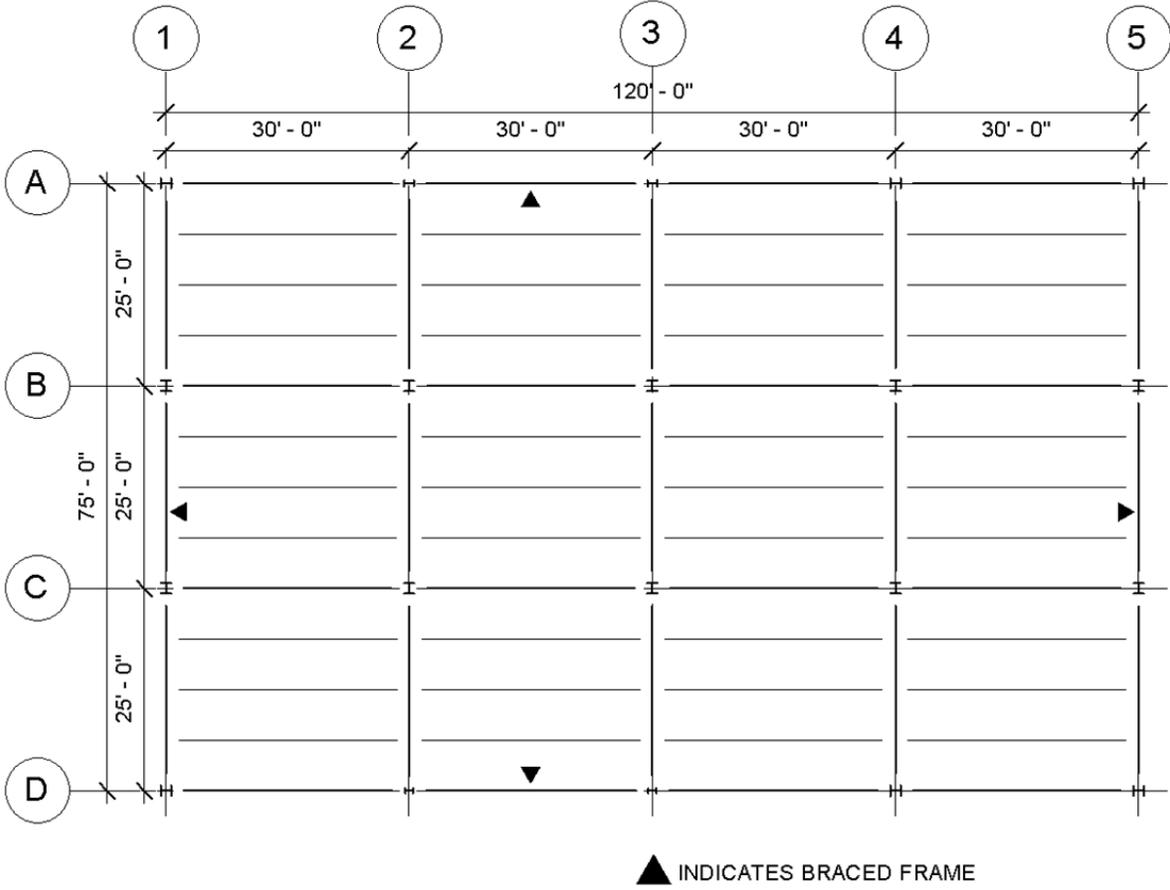


Figure A-1: Building Plan

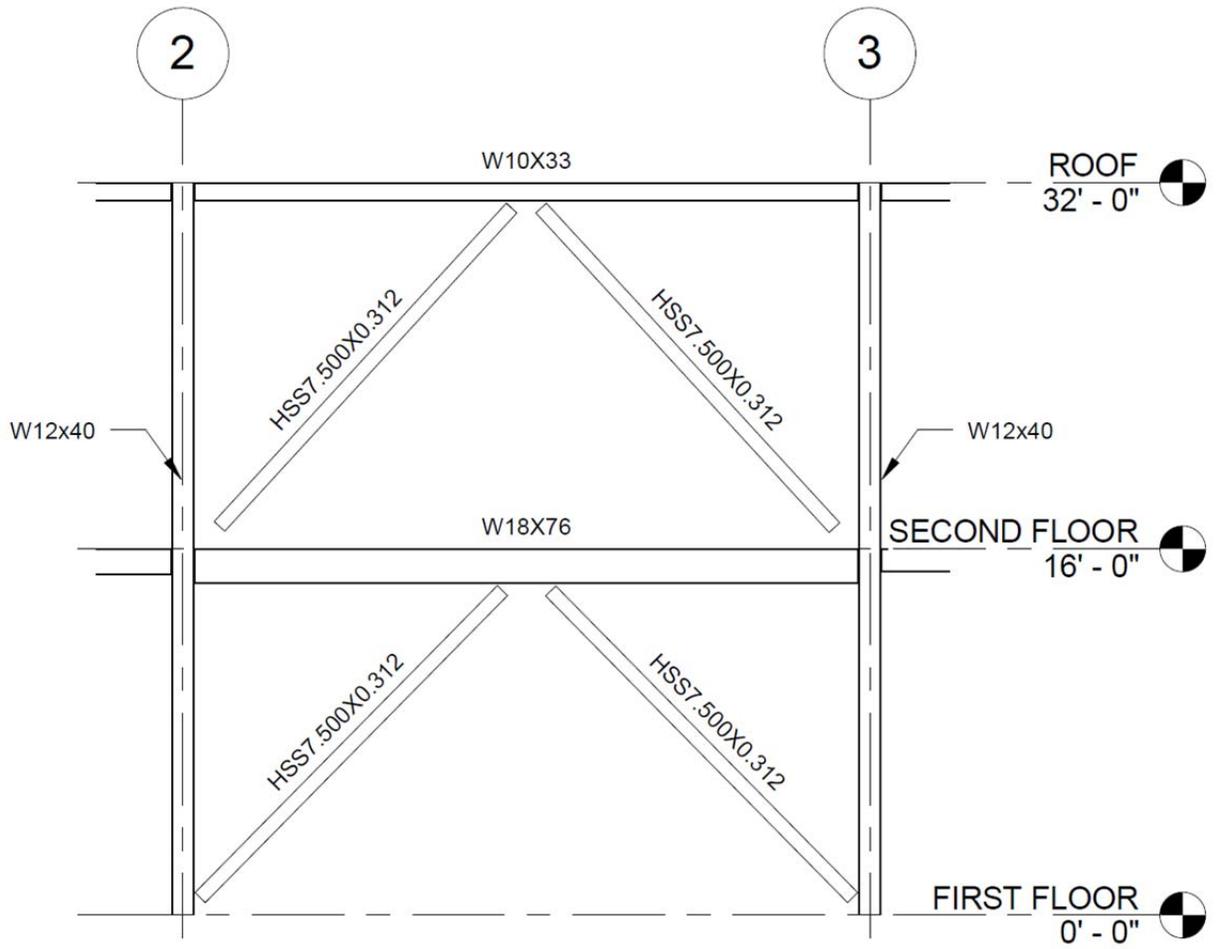


Figure A-2: OCBF Elevation

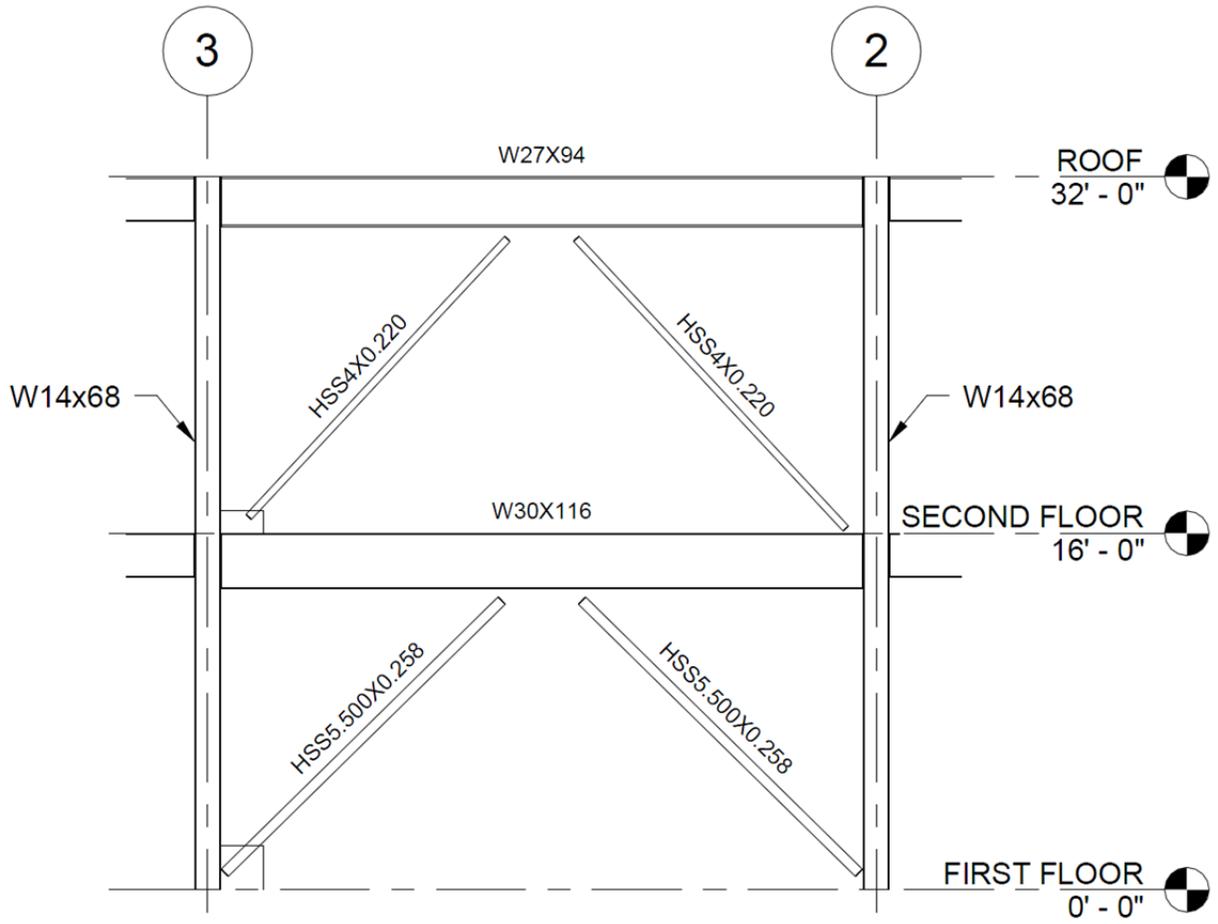


Figure A-3: SCBF Elevation

Appendix B - Load Calculations

Project Parameters

Location: Henderson, NV

Building Dimensions

h_n =	35	ft	(Max building height)
h =	32	ft	(Mean Roof Height)
2nd Floor h =	16	ft	above ground
Roof h =	32	ft	above ground
Parapet h =	35	ft	above ground
Transverse Length=	75	ft	
Longitudinal Length=	120	ft	
	II		[Tbl 1.5-1 ASCE 7-10]

Load Combinations

Reference: ASCE 7-10 UNO

General Load Combinations

2.3.2

- | | |
|----|------------------------------------|
| 1) | 1.4D |
| 2) | 1.2D+1.6L+0.5(Lr or S or R) |
| 3) | 1.2D+1.6(Lr or S or R)L+(L or .5W) |
| 4) | 1.2D+1.0W+L+0.5(Lr or S or R) |
| 6) | 0.9D+1.0W |

Seismic Load Combinations for Strength Design

12.4.2.3

- | | | | | |
|----|------------------|-----|--------|-----------------|
| 5) | $(1.2+(0.2SDS))$ | D + | ρ | $Q_E + L + .2S$ |
| 7) | $(0.9-(0.2SDS))$ | D + | ρ | $Q_E + 1.6H$ |

Seismic Load Combinations for Overstrength

12.4.3.2

- | | | | | |
|----|------------------|-----|------------|-----------------|
| 5) | $(1.2+(0.2SDS))$ | D + | Ω_o | $Q_E + L + .2S$ |
| 7) | $(0.9-(0.2SDS))$ | D + | Ω_o | $Q_E + 1.6H$ |

Seismic Load Combinations for Strength Design

12.4.2.3

- | | | | | |
|----|--------|-----|-----|-----------------|
| 5) | 1.3084 | D + | 1.3 | $Q_E + L + .2S$ |
| 7) | 0.7916 | D + | 1.3 | $Q_E + 1.6H$ |

Seismic Load Combinations for Overstrength

12.4.3.2

- | | | | | |
|----|--------|-----|---|-----------------|
| 5) | 1.3084 | D + | 2 | $Q_E + L + .2S$ |
| 7) | 0.7916 | D + | 2 | $Q_E + 1.6H$ |

Dead Load

ASCE 7-10 UNO

Roof

2" Rigid Insulation=	3	psf	TBL C3-1
1.5B Deck, 20GA=	2.5	psf	Vulcraft
Steel Framing=	3	psf	Assumed
Acoustic Ceiling=	3	psf	TBL C3-1
MEP=	5	psf	TBL C3-1
Misc.=	1.5	psf	Assumed

Total Dead Load= 18 psf

2nd Floor

1.0C Deck, 20GA, 3-1/2" Total Conc.=	38	psf	Vulcraft
Steel Framing=	5	psf	Assumed
Acoustic Ceiling=	3	psf	TBL C3-1
MEP=	4	psf	TBL C3-1

Total Dead Load= 50 psf

Walls

Curtain Wall	15	psf	Assumed
--------------	----	-----	---------

Total Wall Weight= 15 psf

Live Loads

Roof Live Load, L_r = 20 psf

2nd Floor Live Load= 80 psf

Partition Live Load= 15 psf

Total 2nd Floor Live Load, L = 95 psf

Snow Loads

Reference: ASCE 7-05

$$p_g = 5 \text{ psf}$$

Fig 7-1

$$I = 1.0$$

TBL 7-4

$$C_e = 0.9$$

TBL 7-2

$$C_T = 1.0$$

TBL 7-3

$$C_s = 1.0$$

Fig 7-2

$$p_g = 5 \text{ psf} < 20 \text{ psf} \therefore p_{fmin} = I * p_g = 5 \text{ psf}$$

$$p_f = .7 C_e C_T I p_g = 3.15 \text{ psf} < p_{fmin} = 5 \text{ psf}$$

Eqn 7-1
Sec. 7.3

$$\text{Use } p_f = 5 \text{ psf}$$

$$p_s = 5 \text{ psf} \quad p_s = C_s p_f$$

Eqn 7-2

$l_{u-Windward} = 120 \text{ ft}$
 $l_{u-Leeward} = 0 \text{ ft}$

$\gamma = 14.65 \text{ psf}$ $\gamma = .13p_g + 14 \text{ (Max 30psf)}$ Eqn 7-3 (snow density)

$h_b = 0.34 \text{ ft}$ $h_b = p_s / \gamma \text{ (End of drift at point where parapet height} = h_b)$ Sec 7.1

$h_{o-min} = 3 \text{ ft}$ (Smallest parapet height)

$h_{o-max} = 3 \text{ ft}$ (Largest parapet height)

$h_{o-mean} = 3 \text{ ft}$ (Average parapet height)

$h_c = 2.66 \text{ ft}$ (h_{o-max} -snow depth[h_b])

$h_{c-max} / h_b = 7.790 > 0.2$
 Drifts must be considered

$h_{d-Windward} = 1.63 \text{ ft}$ $h_{d-Windward} = .75 * [.43 * (l_u)^{1/3} * (p_g + 10)^{1/4}] - 1.5$ Fig 7-8

$h_{d-Leeward} = 0.00 \text{ ft}$ $h_{d-Leeward} = [.43 * (l_u)^{1/3} * (p_g + 10)^{1/4}] - 1.5$

$h_d = 1.63 \text{ ft} > h_c$

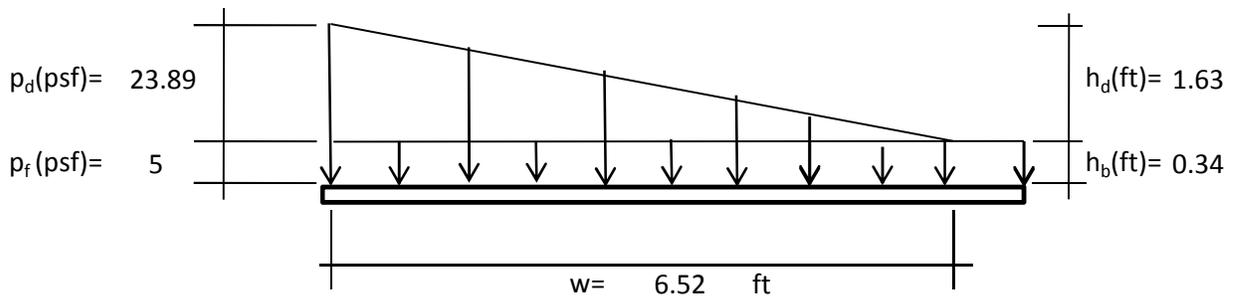
$h_{d-actual} = 1.63 \text{ ft}$ (min of h_c & h_d)

$w = 6.522 \text{ ft}$ $w = 4 * h_{d-actual}$ or $4h_d^2 / h_c$ (Drift width)

$w_{max} = 21.27 \text{ ft}$ (max drift width[$8h_c$])

$w_{actual} = 6.522 \text{ ft}$ (min of w & w_{max})

$p_d = 23.89 \text{ psf}$ (max drift load intensity [$\gamma * h_c$])



Note: No drifts for roof obstructions <15' (i.e. HVAC equipment)

Wind Loads

Reference: ASCE 7-10

User Input =
 Result =

Wind Velocity = 115 mph

Building Risk Category = II TBL 1.5-1

Structure Type= Buildings-MWFRS
 Load Combinations= yes
 K_d= 0.85 TBL 26.6-1

Exposure Category= C Sec 26.7.3
 Total Wall Height= 35 ft
 h (mean roof height)= 32.0 ft (low rise-cannot exceed 60ft in height)(If $\theta < 10$, h=eave height)

α = 9.5 TBL 26.9-1
 z_g= 900 ft TBL 26.9-1

K_z= 1.065 TBL 27.3-1
 K_h = 1.054
 K_z (0-15)= 0.850
 K_z (15-(x<20))= 0.900
 K_z (20-(x<25))= 0.940
 K_z (25-(x<30))= 0.980
 K_z (30-(x<40))= 1.040
 K_z (40-(x<50))= 1.054
 K_z (50-(x<60))= 1.090

Site Geometry Flat/Other
 Building Location From Peak Flat/Other

K_{zt} = 1.00 Fig 26.8-1 (Calculate K1, K2, and K3 for Kzt)

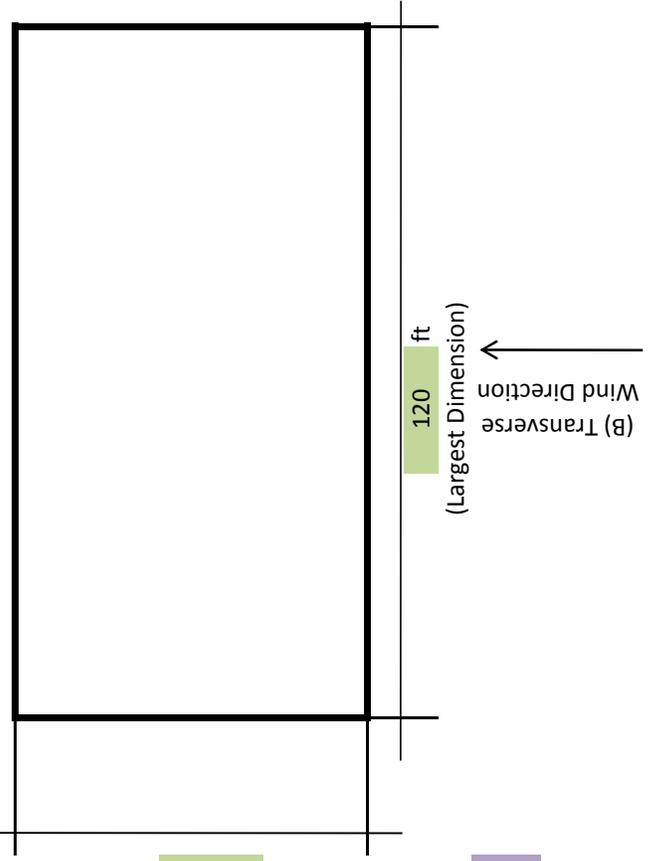
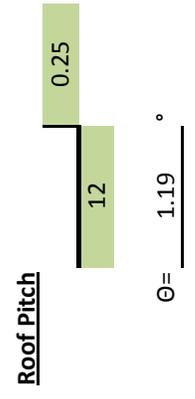
$q = 0.00256(K_{zt})(K_z)(K_d)(V)^2$ = 30.34 psf Eqn 27.3-1

q_p= 30.64 psf
 q_h= 30.34 psf
 q_z(0'-15')= 24.46 psf
 q_z(15'-20')= 25.90 psf
 q_z(20'-25')= 27.05 psf
 q_z(25'-30')= 28.20 psf
 q_z(30'-40')= 29.93 psf
 q_z(40'-50')= 30.34 psf
 q_z(50'-60')= 31.37 psf

Building Enclosure & Dimensions

A_0 =	1000.00	ft ²	Area of openings in a wall receiving (+) pressure
A_g =	1000.00	ft ²	Gross area of same wall receiving (+) pressure
A_{o1} =	4000	ft ²	Sum of areas of openings in building (walls and roof) excluding A_0
A_{g1} =	20000.00	ft ²	Sum of gross surface areas of building (walls and roof) excluding A_g

Enclosure Classification= **Enclosed**



Total Wall Height=	35	ft	
Mean Wall Height=	32.0	ft	
Mean Parapet Height=	3.0	ft	
Long. L/B=	1.600		h/L= 0.27
Trans. L/B=	0.625		h/L= 0.43

Main Wind Force Resisting System Wind Forces

Parapets

Pressure = $P_p = q_p GC_{pn}$ (psf) Eqn 27.4-4

$q_p = 30.64$ psf

Windward $GC_{pn} = 1.5$

Leeward $GC_{pn} = -1.0$

Windward $P_p = 45.96$ psf

Leeward $P_p = -30.64$ psf

Roof Main Wind Force Resisting System Forces

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Roof Zone	q	q _h	Fig 27.4-1		Fig 26.11-1		P = qGC _p -q _i (GC _{pi}) (psf) Eqn 27.4-1			
			C _{p1}	C _{p2}	GC _{pi} (+)	GC _{pi} (-)	P(1+)	P(1-)	P(2+)	P(2-)
0 to h/2	28.20	30.34	-0.9	-0.18	0.18	-0.18	-27.0	-16.1	-9.8	1.1
h/2 to h	28.20	30.34	-0.9	-0.18	0.18	-0.18	-27.0	-16.1	-9.8	1.1
h to 2h	28.20	30.34	-0.5	-0.18	0.18	-0.18	-17.4	-6.5	-9.8	1.1
>2h	28.20	30.34	-0.3	-0.18	0.18	-0.18	-12.7	-1.7	-9.8	1.1

h/2 = 16.00 ft

h = 32.00 ft

2h = 64.00 ft

Longitudinal Direction Wind Pressures

Sec 27.4.1

	Windward/Leeward Wall Length= 75 ft		Side Wall Length= 120.0 ft		Fig 27.4-1 Long. C _p	Fig 26.11-1 GC _{pi} (+)	GC _{pi} (-)	P=qGC _p -q _i (GC _{pi}) (psf)Eqn 27.4-1		Force (P) (kips)	
	Total Wall Area(-Parapet)= 2400.0 ft ²	.5*h= 16.0 ft	Total Parapet Area= 225 ft ²					(+)	(-)		(+)
	Area (ft ²)	q (psf)	q _i =q _h (psf)	Sec 26.9.4 G	Trans. C _p						
Windward Wall (0'-15')	1125.0	24.46	30.34	0.85	0.8	0.18	-0.18	11.172	22.095	12.57	24.86
Windward Wall (.5h-15')	525.0	25.90	30.34	0.85	0.8	0.18	-0.18	12.150	23.073	6.38	12.11
Windward Wall (15'-20')	375	25.90	30.34	0.85	0.8	0.18	-0.18	12.150	23.073	4.56	8.65
Windward Wall (20'-25')	375	27.05	30.34	0.85	0.8	0.18	-0.18	12.933	23.856	4.85	8.95
Windward Wall (25'-30')	375	28.20	30.34	0.85	0.8	0.18	-0.18	13.716	24.639	5.14	9.24
Windward Wall (30'-40')	150	29.93	30.34	0.85	0.8	0.18	-0.18	14.890	25.813	2.23	3.87
Windward Wall (40'-50')	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.00	0.00
Windward Wall (50'-60')	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.00	0.00
Leeward Wall (0'-15')	1125	24.46	30.34	0.85	-0.380	0.18	-0.18	-15.262	-4.339	-17.17	-4.88
Leeward Wall (.5h-15')	525.0	25.90	30.34	0.85	-0.380	0.18	-0.18	-15.262	-4.339	-8.01	-2.28
Leeward Wall (15'-20')	375	25.90	30.34	0.85	-0.380	0.18	-0.18	-15.262	-4.339	-5.72	-1.63
Leeward Wall (20'-25')	375	27.05	30.34	0.85	-0.380	0.18	-0.18	-15.262	-4.339	-5.72	-1.63
Leeward Wall (25'-30')	375	28.20	30.34	0.85	-0.380	0.18	-0.18	-15.262	-4.339	-5.72	-1.63
Leeward Wall (30'-40')	150	29.93	30.34	0.85	-0.380	0.18	-0.18	-15.262	-4.339	-2.29	-0.65
Leeward Wall (40'-50')	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.00	0.00
Leeward Wall (50'-60')	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.00	0.00
Side Wall (0'-15')	1125	24.46	30.34	0.85	-0.7	0.18	-0.18	-23.515	-12.592	-26.45	-14.17
Side Wall (15'-20')	375	25.90	30.34	0.85	-0.7	0.18	-0.18	-23.515	-12.592	-8.82	-4.72
Side Wall (20'-25')	375	27.05	30.34	0.85	-0.7	0.18	-0.18	-23.515	-12.592	-8.82	-4.72
Side Wall (25'-30')	375	28.20	30.34	0.85	-0.7	0.18	-0.18	-23.515	-12.592	-8.82	-4.72
Side Wall (30'-40')	150	29.93	30.34	0.85	-0.7	0.18	-0.18	-23.515	-12.592	-3.53	-1.89
Side Wall (40'-50')	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.00	0.00
Side Wall (50'-60')	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.00	0.00

Parapet Forces

Windward Parapet= 10.34115 kips
 Leeward Parapet= -6.8941 kips

Longitudinal Base Shear [Excluding Lower Part of Wall](kips)=
 Diaphragm Force (Base Shear/Span)(lb/ft)=

83.22
 67.87
 904.92

Seismic Loads (OCBF)

Reference: ASCE 7-10 UNO

Latitude= 36.0292 °
 Longitude= -115.025 °

Site Class= D

Seismic Ground Motion (From USGS Sesimic Hazard Program)

F_a= 1.299
 F_v= 2.066

S_s= 0.626
 S₁= 0.184

S_{DS}= 0.542
 S_{D1}= 0.253

Sesimic Design Category

I= 1.0

Tbl 1.5-2

SDC= D

Tbl 11.6-1

SDC= D

Tbl 11.6-2

SDC= D

Seismic Forces

R= 3.25

[Ordinary Concentric Brace Frame]

TBL 12.2-1

T_L= 6 s

Fig 22-12

C_t= 0.02

Tbl 12.8-2

x= 0.75

Tbl 12.8-2

T=T_a= 0.2878 s

$$T_a = C_t h_n^x$$

Eqn 12.8-7

T << T_L

C_s= 0.167

$$C_s = [S_{DS}/(R/I)]$$

Eqn 12.8-2

C_{s-Max}= 0.270

$$C_{s-Max} = [S_{D1}/(R/I)] \text{ for } T < T_L$$

Eqn 12.8-3

C_{s-Min}= 0.010

Eqn 12.8-5

Seismic Base Shear

Distributed Weights

Roof DL=	18	psf		
2nd Floor DL=	50	psf		
Partition Load=	10	psf	(2nd Floor)	12.7.2 (2)
Curtain Wall=	15	psf		

Building Dimensions

Trans. Length=	75	ft
Long. Length=	120	ft

Effective Wall Heights Contributing to Seismic Weight Per Diaphragm

Roof=	11.00	ft	(Half of Wall Height Below + Parapet Height)
2nd Floor=	16.00	ft	(Half of Wall Heights Above and Below)

Seismic Weight

W_{Roof} =	162.00	k	[Roof DL x Area]
$W_{\text{Wall to Roof}}$ =	64.35	k	[Wall DL x Effective Height x Perimeter]
$W_{\text{2nd Floor}}$ =	540.00	k	[2nd Floor DL x Area]
$W_{\text{Wall to 2nd Floor}}$ =	93.60	k	[Wall DL x Effective Height x Perimeter]
W_{Total} =	859.95		

Seismic Base Shear

$V_{\text{base Shear}}$ =	143.4	kips	$[V=C_s*W]$	Eqn 12.8-1
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$w_{x-Roof} =$	226.35	k	[Sum of Seismic Weights at Given Level]	
$w_{x-2nd Floor} =$	633.60	k		
$h_{x-Roof} =$	32	ft		
$h_{x-2nd Floor} =$	16	ft		
$k =$	1		[For $T < .5s$]	
$w_x h_x^k (Roof) =$	7243.2			
$w_x h_x^k (2nd Floor) =$	10137.6			
$\Sigma =$	17380.8			
$C_{vx-Roof} =$	0.42		$C_{vx} = (w_x h_x^k) / (\Sigma w_x h_x^k)$	Eqn 12.8-12
$C_{vx-2nd Floor} =$	0.58			
$F_{x-Roof} =$	59.7654	k	$F_x = C_{vx} V$	Eqn 12.8-11
$F_{x-2nd Floor} =$	83.6478	k		

Force Distribution to Brace Frames

Because there is a frame on each exterior wall (one pair in each direction) and they are located symmetrically, each frame carries half of the seismic base shear in its respective direction

Roof	Flexible Diaphragm		
	$F = 29.88$ k		$F = F_{x-Roof} * .5$
2nd Floor	Rigid Diaphragm, Building assumed to be symmetrical: no inherent Torsion		
	$F = 41.82$ k		$F = F_{x-Roof} * .5$
2nd Floor Accidental Torsion			12.8.4.2

$$x_{Trans} = 3.75 \text{ ft} \quad x = 5\% * \text{Transverse Length or Longitudinal Length}$$

$$x_{Long} = 6 \text{ ft} \quad [\text{Displacement of Center of Mass}]$$

$$M_{ta1} = 313.68 \text{ k-ft} \quad M_{ta} = x_{Trans} * F_{x-2nd Floor}$$

$$M_{ta2} = 501.89 \text{ k-ft} \quad M_{ta} = x_{Long} * F_{x-2nd Floor}$$

$$M_{ta-Total} = 815.57 \text{ k-ft}$$

All Brace Frames (4 Total, 1 on each exterior grid) resist torsion equally.

Therefore, Grid A and D brace frames resist moment, M_{ta1} . Grids 1 and 5 resist moment, M_{ta2} .

$$F = 4.18 \text{ k} \quad F = M_{ta1} / \text{Transverse Length (Grids A \& D)}$$

$$F = 4.18 \text{ k} \quad F = M_{ta2} / \text{Longitudinal Length (Grids 1 \& 5)}$$

Seismic Loads (SCBF)

Reference: ASCE 7-10 UNO

Latitude= 36.0292 °
 Longitude= -115.025 °

Site Class= D

Seismic Ground Motion (From USGS Sesimic Hazard Program)

F_a= 1.299
 F_v= 2.066

S_s= 0.626
 S₁= 0.184

S_{DS}= 0.542
 S_{D1}= 0.253

Sesimic Design Category

I= 1.0

Tbl 1.5-1

SDC= D

Tbl 11.6-1

SDC= D

Tbl 11.6-2

SDC= D

Seismic Forces

R= 6

[SCBF]

TBL 12.2-1

T_L= 6 s

Fig 22-15

C_t= 0.02

Tbl 12.8-2

x= 0.75

Tbl 12.8-2

T=T_a= 0.2878 s

$$T_a = C_t h_n^x$$

Eqn 12.8-7

T << T_L

C_s= 0.0903

$$C_s = [S_{DS}/(R/I)]$$

Eqn 12.8-2

C_{s-Max}= 0.147

$$C_{s-Max} = [S_{D1}/(R/I)] \text{ for } T < T_L$$

Eqn 12.8-3

C_{s-Min}= 0.010

Eqn 12.8-5

Seismic Base Shear

Distributed Weights

Roof DL=	18	psf	
2nd Floor DL=	50	psf	
Partition Load=	10	psf	12.7.2 (2)
Curtain Wall=	15	psf	

Building Dimensions

Trans. Length=	75	ft
Long. Length=	120	ft

Effective Wall Heights Contributing to Seismic Weight Per Diaphragm

Roof=	11.00	ft
2nd Floor=	16.00	ft

Seismic Weight

W_{Roof} =	162.00	k	[Roof DL x Area]
$W_{\text{Wall to Roof}}$ =	64.35	k	[Wall DL x Effective Height x Perimeter]
$W_{\text{2nd Floor}}$ =	540.00	k	[2nd Floor DL x Area]
$W_{\text{Wall to 2nd Floor}}$ =	93.60	k	[Wall DL x Effective Height x Perimeter]
W_{Total} =	859.95		

Seismic Base Shear

$V_{\text{base Shear}}$ =	77.7	kips	$[V=C_s * W]$	Eqn 12.8-1
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Vertical Distribution of Seismic Forces

12.8.3

$w_{x-Roof} =$	226.35	k	[Sum of Seismic Weights at Given Level]	
$w_{x-2nd Floor} =$	633.60	k		
$h_{x-Roof} =$	32	ft		
$h_{x-2nd Floor} =$	16	ft		
$k =$	1		[For $T < .5s$]	
$w_x h_x^k (Roof) =$	7243.2			
$w_x h_x^k (2nd Floor) =$	10137.6			
$\Sigma =$	17380.8			
$C_{vx-Roof} =$	0.42		$C_{vx} = (w_x h_x^k) / (\Sigma w_x h_x^k)$	Eqn 12.8-12
$C_{vx-2nd Floor} =$	0.58			
$F_{x-Roof} =$	32.37293	k	$F_x = C_{vx} V$	Eqn 12.8-11
$F_{x-2nd Floor} =$	45.30922	k		

Force Distribution to Brace Frames

Because there is a frame on each exterior wall (one pair in each direction) and they are located symmetrically, each frame carries half of the seismic base shear in its respective direction

Roof	Flexible Diaphragm
$F = 16.19$ k	$F = F_{x-Roof} * .5$
2nd Floor	Rigid Diaphragm, Building assumed to be symmetrical: no inherent Torsion
$F = 22.65$ k	$F = F_{x-Roof} * .5$
2nd Floor Accidental Torsion	12.8.4.2

$$x_{Trans} = 3.75 \text{ ft} \quad x = 5\% * \text{Transverse Length or Longitudinal Length}$$

$$x_{Long} = 6 \text{ ft} \quad [\text{Displacement of Center of Mass}]$$

$$M_{ta1} = 169.91 \text{ k-ft} \quad M_{ta} = x_{Trans} * F_{x-2nd Floor}$$

$$M_{ta2} = 271.86 \text{ k-ft} \quad M_{ta} = x_{Long} * F_{x-2nd Floor}$$

$$M_{ta-Total} = 441.76 \text{ k-ft}$$

All Brace Frames (4 Total, 1 on each exterior grid) resist torsion equally.

Therefore, Grid A and D brace frames resist moment, M_{ta1} . Grids 1 and 5 resist moment, M_{ta2} .

$$F = 2.27 \text{ k} \quad F = M_{ta1} / \text{Transverse Length (Grids A \& D)}$$

$$F = 2.27 \text{ k} \quad F = M_{ta2} / \text{Longitudinal Length (Grids 1 \& 5)}$$

Member Loads - Grids A & D

Ref. ASCE 7-10

Column D-2 (Loads excluding reactions from brace frame beams)

Trib Width=	12.5	ft		
Trib Length=	30	ft		
Vertical Trib Height=	35	ft		
Trib Area of BF Beams to Col=	46.875	ft ²		
Horizontal Trib Area=	328.125	ft ²	[Width x Length][Excluding BF Beam Trib]	
Total Trib Area, A _T =	375	ft ²	[Width x Length]	
Vertical Trib Area=	1050	ft ²	[Length x Height]	
Live Load Element Factor, K _{LL} =	4			Tbl 4-2
K _{LL} A _T =	1500	ft ²	≥ 400	ft ² 4.8.1
Live Load Reduction Applies				
Live Load Reduction Factor=	0.637		[0.25+(15/V(K _{LL} A _T))]	

Loads without Braced Frame Beam Reactions

Roof Dead Load=	5.91	kips	
Floor Dead Load=	16.41	kips	
Wall Dead Load=	15.75	kips	
Roof Live Load=	6.56	kips	
Floor Live Load=	19.87	kips	[Reduced]
Roof Snow Load=	7.84	kips	
P _u =	71.2	kips	[Seismic Combination 5]
P _u =	30.1	kips	[Seismic Combination 7]

Column D-2 (Loads excluding reactions from brace frame beams)

Trib Width=	12.5	ft	
Trib Length=	30	ft	
Vertical Trib Height=	35	ft	
Trib Area of BF Beams=	46.875	ft ²	
Horizontal Trib Area=	328.125	ft ²	[Width x Length][Excluding BF Beam Trib]
Total Trib Area, A _T =	375	ft ²	[Width x Length]
Vertical Trib Area=	1050	ft ²	[Length x Height]

Live Load Element Factor, K _{LL} =	4				Tbl 4-2	
K _{LL} A _T =	1500	ft ²	≥	400	ft ²	4.8.1
Live Load Reduction Applies						

Live Load Reduction Factor= 0.637 [0.25+(15/√(K_{LL}A_T))]

Loads without Braced Frame Beam Reactions

Roof Dead Load=	5.91	kips	
Floor Dead Load=	16.41	kips	
Wall Dead Load=	15.75	kips	
Roof Live Load=	6.56	kips	
Floor Live Load=	19.87	kips	[Reduced]
Roof Snow Load=	7.84	kips	
P _u =	71.2	kips	[Seismic Combination 5]
P _u =	30.1	kips	[Seismic Combination 7]

Roof Beam

Trib Width= 3.125 ft
Beam Length= 30 ft

Beam Self-Weight= 94 lb/ft

Roof Dead Load= 56.25 lb/ft
Roof Live Load= 62.5 lb/ft
Roof Snow Load= 75 lb/ft

$M_D = 16.90$ k-ft $[wl^2/8]$

$M_L = 7.03$ k-ft $[wl^2/8]$

$M_S = 8.40$ k-ft $[wl^2/8]$

$V_D = 2.25$ k $[wl/2]$

$V_L = 0.94$ k $[wl/2]$

$V_S = 1.12$ k $[wl/2]$

Floor Beam

Trib Width= 3.125 ft
Beam Length= 30 ft

Beam Self-Weight= 116 lb/ft

Floor Dead Load= 156.25 lb/ft
Floor Live Load= 296.875 lb/ft

$M_D = 30.63$ k-ft $[wl^2/8]$

$M_L = 33.40$ k-ft $[wl^2/8]$

$V_D = 4.08$ k $[wl/2]$

$V_L = 4.45$ k $[wl/2]$

SCBF Columns

Total column load including reaction from unbalanced brace forces

Sum of loads including brace frame beams acting at columns

Total Dead Load= 44.40 k

Total Live Load= 24.32 k

Total Snow Load= 8.96 k

P_u = 84.20 k

Distance to Opposite Col.= 30 ft

Unbalanced Brace Forces (Treated as seismic loads)

Roof Beam, Q_{bv} = 115 k

[ref. Q_{bv} , SCBF Upper Brace Design]

Floor Beam, Q_{bv} = 172 k

[ref. Q_{bv} , SCBF Lower Brace Design]

Load Factor= 1.3

[ref. Load Combinations (5)]

Roof Beam, P_u = 149.5 k

Floor Beam, P_u = 223.6 k

Distance from Column= 15 ft

Lateral Seismic Loads

Load Factor= 1.3

[ref. Load Combinations (5)]

Force @ Roof= 30 k

Factored Force= 39 k

Height Above Ground= 32 ft

Force @ Floor= 46 k

Factored Force= 59.8 k

Height Above Ground= 16 ft

Column Reaction

Column Design Force= 344.2 k

[Summing Moments about Column Base]

Summary of OCBF Member Loads from RISA

Column Left (M1)		Max Axial Load			
5 (+)		57.884	kips		
5 (-)		99.746	kips		
7 (+)		12.673	kips		
7 (-)		54.529	kips		
Column Right (M2)		Max Axial Load			
5 (+)		99.729	kips		
5 (-)		57.9	kips		
7 (+)		54.529	kips		
7 (-)		12.673	kips		
Column Design Forces					
Max Compression Force=	99.746	kips			
Lower Brace Left (M5)		Max Axial Load		Amplified Seismic	
5 (+)		-64.472	kips	-103.4	kips
5 (-)		80.411	kips	119.245	kips
7 (+)		-69.8	kips	-108.678	kips
7 (-)		74.714	kips	113.498	kips
Lower Brace Right (M6)		Max Axial Load		Amplified Seismic	
5 (+)		80.411	kips	119.243	kips
5 (-)		-64.471	kips	-103.399	kips
7 (+)		74.714	kips	113.498	kips
7 (-)		-69.8	kips	-108.678	kips
Lower Brace Design Forces					
Max Tension Force=	-69.8	kips	-108.678	kips	
Max Compression Force=	80.411	kips	119.245	kips	
Upper Brace Left (M7)		Max Axial Load		Amplified Seismic	
5 (+)		-26.629	kips	-42.094	kips
5 (-)		31.25	kips	46.697	kips
7 (+)		-27.565	kips	-43.027	kips
7 (-)		30.116	kips	45.56	kips
Upper Brace Right (M8)		Max Axial Load		Amplified Seismic	
5 (+)		31.249	kips	46.696	kips
5 (-)		-26.628	kips	-42.094	kips
7 (+)		30.116	kips	45.56	kips
7 (-)		-27.565	kips	-43.027	kips
Upper Brace Design Forces					
Max Tension Force=	-27.565	kips	-43.027	kips	
Max Compression Force=	31.25	kips	46.697	kips	

Summary of SCBF Member Loads from RISA

Column Left (M1)	Max Axial Load		Amplified Seismic	
5 (+)	66.7	kips	60.3	kips
5 (-)	90.3	kips	96.7	kips
7 (+)	21.7	kips	15.3	kips
7 (-)	45.3	kips	51.7	kips

Column Right (M2)	Max Axial Load		Amplified Seismic	
5 (+)	90.3	kips	96.7	kips
5 (-)	66.7	kips	60.3	kips
7 (+)	45.3	kips	51.7	kips
7 (-)	21.7	kips	15.3	kips

Column Design Forces				
Max Compression Force=	90.3	kips	96.7	kips

Lower Brace Left (M5)				
Load Combination	Max Axial Load		Amplified Seismic	
5 (+)	-30.7	kips	-52.3	kips
5 (-)	49.7	kips	71.2	kips
7 (+)	-36.8	kips	-58.3	kips
7 (-)	43.4	kips	64.8	kips

Lower Brace Right (M6)				
Load Combination	Max Axial Load		Amplified Seismic	
5 (+)	49.7	kips	71.2	kips
5 (-)	-30.7	kips	-52.3	kips
7 (+)	43.4	kips	64.8	kips
7 (-)	-36.8	kips	-58.3	kips

Lower Brace Design Forces				
Max Tension Force=	-36.8	kips	-58.3	kips
Max Compression Force=	49.7	kips	71.2	kips

Upper Brace Left (M7)				
Load Combination	Max Axial Load		Amplified Seismic	
5 (+)	-13.9	kips	-22.6	kips
5 (-)	19.1	kips	27.8	kips
7 (+)	-15	kips	-23.7	kips
7 (-)	17.8	kips	26.6	kips

Upper Brace Right (M8)				
Load Combination	Max Axial Load		Amplified Seismic	
5 (+)	19.1	kips	27.8	kips
5 (-)	-13.9	kips	-22.6	kips
7 (+)	17.8	kips	26.6	kips
7 (-)	-15	kips	-23.7	kips

Upper Brace Design Forces				
Max Tension Force=	-15	kips	-23.7	kips
Max Compression Force=	19.1	kips	27.8	kips

Appendix C - OCBF Member Designs

Upper Brace Design (OCBF)

Member Loads

Factored Compressive Forc, P_u = 31.3 k [From RISA]

Factored Tensile Force, T_u = 27.6 k [From RISA]

Member Properties

AISC Manual 14th ed.

Horizontal Component Length= 15 ft

Vertical Component Length= 16.17 ft

Brace Length, L = 22.05 ft

Brace Length, L = 264.64 in

Member Selection: HSS 7.50x0.312

Tbl 1-13

E = 29000 ksi

F_y = 42 ksi

Tbl 2-4

F_u = 58 ksi

Tbl 2-4

D = 7.5 in

t = 0.291 in

Tbl 1-13

A_g = 6.59 in²

Tbl 1-13

D/t = 25.8

Tbl 1-13

r = 2.55 in

Tbl 1-13

Check Slenderness

k = 1

AISC 360-10: Tbl C-A-7.4

Brace Slenderness Limit= 105.11 [$4\sqrt{E/F_y}$]

AISC 341-10: F1.5b

KL/r = 103.8 < 105.1 Member not Slender, OK

Check Local Buckling

λ = 25.800 [$\lambda=D/t$]

AISC Manual 14th ed. Tbl 1-13

λ_{md} = 30.38 [$\lambda_{md}=.044E/F_y$]

AISC 341-10 Tbl D1.1

λ < λ_{md} Member is Seismically Compact, OK

$$4.71\sqrt{E/F_y} = 123.76$$

$$KL/r = 103.8 < 123.8 \quad \text{Use Eqn E3-2 for } F_{cr} \quad \text{E3}$$

$$F_e = 26.57 \text{ ksi} \quad [F_e = \pi^2 E / (KL/r)^2] \quad \text{Eqn E3-4}$$

$$F_{cr} = 21.67 \text{ ksi} \quad [F_{cr} = (0.658^{F_y/F_e}) * F_y] \quad \text{Eqn 3-2}$$

$$\Phi = 0.9 \quad \text{E1(b)}$$

$$P_n = 142.8 \text{ k} \quad [P_n = F_{cr} A_g] \quad \text{Eqn E3-1}$$

$$\Phi P_n = 128.6 \text{ k} > 31.3 \text{ k} \quad \text{Brace is Adequate in Compression}$$

Tensile Strength of Brace

AISC 360-10

Tensile Yielding

D2(a)

$$\Phi = 0.9$$

$$P_n = 276.78 \text{ k} \quad [P_n = F_y A_g] \quad \text{Eqn D2-1}$$

$$\Phi P_n = 249.10 \text{ k}$$

Tensile Rupture

D2(b)

$$\Phi = 0.75$$

$$P_n = 382.22 \text{ k} \quad [P_n = F_u A_e; A_e = A_g] \quad \text{Eqn D2-2}$$

$$\Phi P_n = 286.67 \text{ k}$$

$$\text{Tensile Capacity, } \Phi P_n = 249.10 \text{ k} > 27.6 \text{ k} \quad \text{Brace is Adequate in Tension}$$

	$R_y = 1.4$			Tbl A3.1
Expected Yield Strength of Brace, $P_t =$	387.5 k	$[P_t = R_y F_y A_g]$		F1.4.4a(1)(i)(a)
Load Effect (Amplified Seismic), $T_u =$	43.1 k	[RISA Analysis]		F1.4.4a(1)(i)(b)
				ASCE 7-10 Seismic Load Combo 5 14.4.2.3
Assumed Tension Brace Force, $T_u = P_t =$	43.1 k	[Min. of Expected Yield Strength and Amp. Load Effect]		
Buckled Compressive Strength, $P_c =$	42.85 k	$[.3P_c]$		F1.4.4a(1)(ii)
Assumed Min. Compression Force, $P_u =$	42.85 k	(For Unbalanced Beam Loads)		
Max. Compression Load Effect =	46.7 k	[RISA Analysis, based on Amplified Seismic Loads]		
Horizontal Component, $P_{tx} =$	29.31 k			
Vertical Component, $P_{ty} =$	31.59 k			
Horizontal Component, $P_{cx} =$	29.15 k			
Vertical Component, $P_{cy} =$	31.41 k			
Unbalanced Vert. Load, $Q_{bv} =$	0 k	$[Q_{bv} = P_{ty} - P_{cy}]$ (On roof beam)		
Unbalanced Horiz. Load, $Q_{bp} =$	29 k	$[Q_{bp} = (P_{tx} + P_{cx})/2]$ (On roof beam)		

Lower Brace Design (OCBF)

Member Loads

Factored Compressive Force, $P_u = 80.5$ k [From RISA]
 Factored Tensile Force, $T_u = 69.8$ k [From RISA]

Member Properties

AISC Manual 14th ed.

Horizontal Component Length= 15 ft
 Vertical Component Length= 15.24 ft

Brace Length, $L = 21.38$ ft
 Brace Length, $L = 256.60$ in

Member Selection: HSS 7.50x0.312

Tbl 1-13

$E = 29000$ ksi
 $F_y = 42$ ksi
 $F_u = 58$ ksi

Tbl 2-3

Tbl 2-4

$D = 7.5$ in
 $t = 0.291$ in
 $A_g = 6.59$ in²
 $D/t = 25.8$
 $r = 2.55$ in

Tbl 1-13

Tbl 1-13

Tbl 1-13

Tbl 1-13

Tbl 1-13

Check Slenderness

$k = 1$

AISC 360-10: Tbl C-A-7.1

Brace Slenderness Limit= 105.11 [4√(E/F_y)]

F1.5b

$KL/r = 100.6 < 105.1$ Member not Slender, OK

Check Local Buckling

$\lambda = 25.800$ [λ=D/t]

AISC Manual 14th ed. Tbl 1-13

$\lambda_{md} = 30.38$ [λ_{md}=0.044E/F_y]

AISC 341-10 Tbl D1.1

$\lambda < \lambda_{md}$ Member is Seismically Compact, OK

$$4.71\sqrt{E/F_y} = 123.76$$

$$KL/r = 100.6 < 123.8 \quad \text{Use Eqn E3-2 for } F_{cr} \quad \text{E3}$$

$$F_e = 28.27 \text{ ksi} \quad [F_e = \pi^2 E / (KL/r)^2] \quad \text{Eqn E3-4}$$

$$F_{cr} = 22.55 \text{ ksi} \quad [F_{cr} = (0.658^{F_y/F_e}) * F_y] \quad \text{Eqn 3-2}$$

$$\Phi = 0.9 \quad \text{E1(b)}$$

$$P_n = 148.6 \text{ k} \quad [P_n = F_{cr} A_g] \quad \text{Eqn E3-1}$$

$$\Phi P_n = 133.7 \text{ k} > 80.5 \text{ k} \quad \text{Brace is Adequate in Compression}$$

Tensile Strength of Brace

AISC 360-10

Tensile Yielding

D2(a)

$$\Phi = 0.9$$

$$P_n = 276.78 \text{ k} \quad [P_n = F_y A_g] \quad \text{Eqn D2-1}$$

$$\Phi P_n = 249.10 \text{ k}$$

Tensile Rupture

D2(b)

$$\Phi = 0.75$$

$$P_n = 382.22 \text{ k} \quad [P_n = F_u A_e; A_e = A_g] \quad \text{Eqn D2-2}$$

$$\Phi P_n = 286.67 \text{ k}$$

$$\text{Tensile Capacity, } \Phi P_n = 249.10 \text{ k} > 69.8 \text{ k} \quad \text{Brace is Adequate in Tension}$$

	$R_y = 1.4$			Tbl A3.1
Expected Yield Strength, $T_u =$	387.5 k	$[P_t = R_y F_y A_g]$		F1.4.4a(1)(i)(a)
Load Effect (Amp. Seismic Load), $T_u =$ (For Tension Brace)	108.7 k	[RISA Analysis]	CE 7-10 Seismic Load Combo 5	14.4.2.3 F1.4.4a(1)(i)(b)
Assumed Tension Force, $T_u = P_t =$	108.7 k	[Min. of Expected Yield Strength and Amp. Load Effect]		
Buckled Compressive Strength, $P_c =$	44.58 k	$[.3P_c]$		F1.4.4a(1)(ii)
Assumed Min. Compression, $P_u =$	44.58 k			
Max. Compression Load Effect =	119.3 k	[RISA Analysis, based on Amp. Seismic Loads]		
Horizontal Component, $P_{tx} =$	76.25 k			
Vertical Component, $P_{ty} =$	77.47 k			
Horizontal Component, $P_{cx} =$	31.27 k			
Vertical Component, $P_{cy} =$	31.77 k			
Unbalanced Vertical Load, $Q_{bv} =$	46 k	$[P_{ty} - P_{cy}]$ (on floor beam)		
Unbalanced Horizontal Load, $Q_{bh} =$	54 k	$[(P_{tx} + P_{cx})/2]$ (on floor beam)		

Column Design Design (OCBF)

Reference: AISC 341-10 UNO

Member Loads

$P_u =$	100	k	[Factored Compression, From RISA]
$T_u =$	0	k	[Factored Tension, From RISA]

Member Properties

AISC Manual 14th ed.

$l_u =$	16.00	ft	(Column Unbraced Length)
$l_u =$	192.00	in	(Column Unbraced Length)

Member Selection: W12x40

$E =$	29000	ksi	
$F_y =$	50	ksi	Tbl 2-4
$F_u =$	65	ksi	Tbl 2-4
$A_g =$	11.7	in ²	Tbl 1-1
$d =$	11.9	in	Tbl 1-1
$t_w =$	0.295	in	Tbl 1-1
$b_f =$	8.01	in	Tbl 1-1
$t_f =$	0.515	in	Tbl 1-1
$k =$	1.020	in	Tbl 1-1
$h/t_w =$	33.6		Tbl 1-1
$r_x =$	5.13	in	Tbl 1-1
$r_y =$	1.94	in	Tbl 1-1

Check Local Buckling

AISC 360-10: B4/Tbl B4.1

$\lambda_f =$	7.78		$[\lambda = b_f / 2t_f]$	
$\lambda_r =$	13.49		$[\lambda_r = .56\sqrt{E/F_y}]$	Tbl B4.1a
λ	<	λ_r	Flanges are not Slender, OK	
$\lambda_w =$	33.60		$[\lambda = h/t_w]$	
$\lambda_r =$	35.88		$[\lambda_r = 1.49\sqrt{E/F_y}]$	Tbl B4.1a
λ	<	λ_r	Web is not Slender, OK	

Compressive Strength of Column

AISC 360-10

$K = 1$ Tbl C-A-7.1

$4.71\sqrt{E/F_y} = 113.43$

$KL/r = 99.0 < 113.43$ Use Eqn E3-2 E3

$F_e = 29.22$ ksi $[F_e = \pi^2 E / (KL/r)^2]$ Eqn E3-4

$F_{cr} = 24.43$ ksi $[F_{cr} = (0.658^{F_y/F_e}) * F_y]$ Eqn 3-2

$\Phi = 0.9$ E1(b)

$P_n = 285.8$ k $[P_n = F_{cr} A_g]$ Eqn E3-1

$\Phi P_n = 257.3$ k > 100 k Column is Adequate in Compression

Tensile Strength of Brace

AISC 360-10

Tensile Yielding

D2(b)

$\Phi = 0.9$

$P_n = 585$ k $[P_n = F_y A_g]$ Eqn D2-1

$\Phi P_n = 526.50$ k

Tensile Rupture

D2(b)

$\Phi = 0.75$

$P_n = 760.5$ k $[P_n = F_u A_e; A_e = A_g]$ Eqn D2-2

$\Phi P_n = 570.38$ k

Tensile Capacity, $\Phi P_n = 526.50$ k > 0 k Column is Adequate in Tension

Floor Beam Design Design (OCBF)

Reference: AISC 341-10 UNO

Load Combination Factors

Combination 5: ASCE 7-10 12.4.3.2

Dead Load= 1.3084
 Live Load= 1.0
 Seismic Load= 2.0

Member Loads

M_D =	27.25	k-ft		Member Loads Spreadsheet
M_L =	33.4	k-ft		Member Loads Spreadsheet
$M_{u_{Qbv}}$ =	343	k-ft	$[M_{Qb}=Q_b*L/4]$ [Load already factored]	Lower Brace Design
V_D =	3.7	k		Member Loads Spreadsheet
V_L =	4.5	k		Member Loads Spreadsheet
$V_{u_{Qbv}}$ =	23	k	[Load already factored]	Lower Brace Design
M_u =	411.8	k-ft	(Factored Moment)	
P_u =	53.8	k	$[P_u=Q_{bh}]$ (Factored Compression)	
V_u =	32.2	k	(Factored Shear)	
$V_{u(D+L)}$ =	9.3	k	(Shear without seismic contributions)	

Member Properties

AISC Manual 14th ed.

Beam Length, L = 30.00 ft = 360.00 in

Member Selection: W18x76

E =	29000	ksi	
F_y =	50	ksi	Tbl 2-4
F_u =	65	ksi	Tbl 2-4
A_g =	22.3	in ²	Tbl 1-1
d =	18.2	in	Tbl 1-1
t_w =	0.43	in	Tbl 1-1
k =	1.08	in	Tbl 1-1
T =	15.13	in	Tbl 1-1
$b_f/2t_f$ =	8.11		Tbl 1-1
h/t_w =	37.8		Tbl 1-1
I_x =	1330	in ⁴	Tbl 1-1
S_x =	146	in ³	Tbl 1-1
r_x =	7.73	in	Tbl 1-1
Z_x =	163.0	in ³	Tbl 1-1
r_y =	2.61	in	Tbl 1-1

Check Local Buckling

AISC 360-10

$$\lambda_f = 8.11 \quad [\lambda = b_f / 2t_f]$$

$$\lambda_p = 9.15 \quad [\lambda_r = .38\sqrt{E/F_y}]$$

Tbl B4.1b

$$\lambda < \lambda_r$$

Flanges are Compact, OK

$$\lambda_w = 37.80 \quad [\lambda = h/t_w]$$

$$\lambda_r = 90.55 \quad [\lambda_r = 3.76\sqrt{E/F_y}]$$

Tbl B4.1b

$$\lambda < \lambda_r$$

Web is Compact, OK

Flexural Strength of Beam

AISC 360-10

Compression flange is assumed to be adequately braced continuously. Therefore $L_b < L_p$ and lateral-torsional buckling does not apply.

$$\Phi = 0.9$$

F1(1)

$$M_n = 679.17 \text{ k-ft}$$

Eqn F2-1

$$\Phi M_n = 611.25 \text{ k-ft}$$

Compressive Strength of Beam

AISC 360-10

$$k = 1$$

Tbl C-A-7.1

$$4.71\sqrt{E/F_y} = 113.43$$

$$KL/r = 137.9 > 113.43$$

Use Equation E3-3

E3

$$F_e = 15.04 \text{ ksi} \quad [F_e = \pi^2 E / (KL/r)_2]$$

Eqn E3-4

$$F_{cr} = 13.19 \text{ ksi} \quad [F_{cr} = .877 F_e]$$

Eqn 3-3

$$\Phi = 0.9$$

E1(b)

$$P_n = 294.2 \text{ k} \quad [P_n = F_{cr} A_g]$$

Eqn E3-1

$$\Phi P_n = 264.8 \text{ k}$$

$\alpha =$	1.0		
$B_2 =$	1.0	[No translation of beam ends]	Eqn A-8-5
$P_{e1} =$	2937.3 k	$[P_{e1} = \pi^2 EI / (K_1 L)^2]$ $[K_1 = 1.0]$	Eqn A-8-5
$C_m =$	1.0	[For members subject to transverse Loading]	A8.2.1(b)
$P_{nt} =$	0.0 k		
$P_{lt} =$	53.8 k		
$P_r =$	53.8 k	$[P_r = P_{nt} + B_2 P_{lt}]$	A-8-2
$B_1 =$	1.02	$[B_1 = C_m / (1 - (\alpha P_r / P_{e1})) \geq 1]$	Eqn A-8-3
$M_{nt} =$	411.8 k-ft		
$M_{lt} =$	0.0 k-ft		
$M_{rx} =$	419.45 k-ft	$[M_r = B_1 M_{nt} + B_2 M_{lt}]$	Eqn A-8-1

Combined Loading

AISC 360-10: H1

$P_r / P_c =$	0.203	>	0.1	Use Eqn H1-1a
Beam Interaction Ratio =	0.813			$[(P_r / P_c) + \{(8/9) * ((M_{rx} / M_{cx}) + (M_{ry} / M_{cy}))\}]$
	0.813	<	1.0	Beam is Adequate for Combined Bending-Compression

Shear Strength of Beam

AISC 360-10: G2

h / t_w Limit =	53.95			$[2.24 V (E / F_y)]$
$h / t_w =$	37.8	<	53.95	
$\Phi_v =$	1			G2(a)
$C_v =$	1			G2(a)
$A_w =$	7.735			$[d * t_w]$
$V_n =$	232.1 k			$[V_n = 0.6 * F_y * A_w * C_v]$
$\Phi V_n =$	232.1 k	>	32.2 k	Beam is Adequate in Shear

Roof Beam Design Design (OCBF)

Reference: AISC 341-10 UNO

Load Combination Factors

Combination 5: ASCE 7-10 12.4.3.2

Dead Load=	1.3084
Live Load=	1.0
Snow Load=	0.2
Seismic Load=	2.0

Member Loads

M_D =	12.8	k-ft		
M_L =	7.1	k-ft		
M_S =	8.4	k-ft		
Mu_{Qbv} =	1	k-ft	$[M_{Qb}=Q_b*L/4]$ [Load already factored]	Lower Brace Design
V_D =	1.7	k		
V_L =	1.0	k		
V_S =	1.2	k		
Vu_{Qbv} =	0	k	[Load already factored]	Lower Brace Design
M_u =	26.9	k-ft	(Factored Moment)	
P_u =	29.2	k	$[P_u=Q_{bh}]$ (Factored Compression)	
V_u =	3.6	k	(Factored Shear)	
$V_{u(D+L)}$ =	3.2	k	(Shear without seismic contributions)	

Beam Length, L= 30.00 ft
 Brace Length, L= 360.00 in

Member Selection: W10x33

E=	29000	ksi	
F_y =	50	ksi	Tbl 2-4
F_u =	65	ksi	Tbl 2-4
A_g =	9.71	in ²	Tbl 1-1
d=	9.73	in	Tbl 1-1
t_w =	0.29	in	Tbl 1-1
k=	0.94	in	Tbl 1-1
T=	7.50	in	Tbl 1-1
$b_f/2t_f$ =	9.15		Tbl 1-1
h/t_w =	27.1		Tbl 1-1
I_x =	171	in ⁴	Tbl 1-1
S_x =	35	in ³	Tbl 1-1
r_x =	4.19	in	Tbl 1-1
Z_x =	38.8	in ³	Tbl 1-1
r_y =	1.94	in	Tbl 1-1

Check Local Buckling

AISC 360-10: B4/Tbl B4.1

λ_f =	9.15		$[\lambda=b_f/2t_f]$	
λ_p =	9.15		$[\lambda_r=.38v(E/F_y)]$	Tbl B4.1b
λ	<	λ_r	Flanges are Compact, OK	
λ_w =	27.10		$[\lambda=h/t_w]$	
λ_r =	90.55		$[\lambda_r=3.76v(E/F_y)]$	Tbl B4.1b
λ	<	λ_r	Web is Compact, OK	

Flexural Strength of Beam

AISC 360-10

Compression flange is assumed to be adequately braced continuously. Therefore $L_b < L_p$ and lateral-torsional buckling does not apply.

$$\Phi = 0.9 \quad \text{F1(1)}$$

$$M_n = 161.67 \text{ k-ft} \quad \text{Eqn F2-1}$$

$$\Phi M_n = 145.5 \text{ k-ft}$$

Compressive Strength of Beam

AISC 360-10

$$k = 1 \quad \text{Tbl C-A-7.1}$$

$$4.71\sqrt{E/F_y} = 113.43$$

$$KL/r = 185.6 > 113.43 \quad \text{Use Equation E3-3} \quad \text{E3}$$

$$F_e = 8.31 \text{ ksi} \quad [F_e = \pi^2 E / (KL/r)_2^2] \quad \text{Eqn E3-4}$$

$$F_{cr} = 7.29 \text{ ksi} \quad [F_{cr} = 0.877 F_e] \quad \text{Eqn 3-3}$$

$$\Phi = 0.9 \quad \text{E1(b)}$$

$$P_n = 70.8 \text{ k} \quad [P_n = F_{cr} A_g] \quad \text{Eqn E3-1}$$

$$\Phi P_n = 63.7 \text{ k}$$

Second Order Effects

AISC 350-10: Appendix 8

$$\alpha = 1.0$$

$$B_2 = 1.0$$

[No translation of beam ends]

$$P_{e1} = 377.6 \text{ k} \quad [P_{e1} = \pi^2 EI / (K_1 L)^2] \quad [K_1 = 1.0] \quad \text{Eqn A-8-5}$$

$$C_m = 1.0 \quad \text{[For members subject to transverse Loading]} \quad \text{A8.2.1(b)}$$

$$P_{nt} = 0.0 \text{ k}$$

$$P_{lt} = 29.2 \text{ k}$$

$$P_r = 29.2 \text{ k}$$

$$[P_r = P_{nt} + B_2 P_{lt}] \quad \text{A-8-2}$$

$$B_1 = 1.08$$

$$M_{nt} = 26.9 \text{ k-ft} \quad [B_1 = C_m / (1 - (\alpha P_r / P_{e1})) \geq 1] \quad \text{Eqn A-8-3}$$

$$M_{lt} = 0.0 \text{ k-ft}$$

$$M_{rx} = 29.15 \text{ k-ft} \quad [M_r = B_1 M_{nt} + B_2 M_{lt}] \quad \text{Eqn A-8-1}$$

$$P_r/P_c = 0.459 > 0.1 \quad \text{Use Eqn H1-1a}$$

$$\text{Beam Interaction Ratio} = 0.637 \quad \left[\left(\frac{P_r}{P_c} \right) + \left\{ \left(\frac{M_{rx}}{M_{cx}} \right)^2 + \left(\frac{M_{ry}}{M_{cy}} \right)^2 \right\} \right] \quad \text{Eqn H1-1a}$$

$$0.637 < 1.0 \quad \text{Beam is Adequate for Combined Bending-Compression}$$

Shear Strength of Beam

$$h/t_w \text{ Limit} = 53.95 \quad [2.24\sqrt{E/F_y}]$$

$$h/t_w = 27.1 < 53.95$$

$$\Phi_v = 1 \quad \text{G2(a)}$$

$$C_v = 1 \quad \text{G2(a)}$$

$$A_w = 2.8217 \text{ in}^2 \quad [d \cdot t_w]$$

$$V_n = 84.7 \text{ k} \quad [V_n = 0.6 \cdot F_y \cdot A_w \cdot C_v] \quad \text{Eqn G2-1}$$

$$\Phi V_n = 84.7 \text{ k} > 3.56 \text{ k} \quad \text{Beam is Adequate in Shear}$$

Story Drift (OCBF)

Story Drift Limit

ASCE 7-10

Building Risk Category: II

Story Height, h_{sx} = 16 ft

Allowable Story Drift, Δ_a = 0.020 * h_{sx} Tbl 12.12-1

Allowable Story Drift, Δ_a = 0.32 ft [$\Delta_a = 0.020 h_{sx}$] Tbl 12.12-1

Allowable Story Drift, Δ_a = 3.84 in

Calculated Story Drift

ASCE 7-10

Elastic Story Drift, δ_{xe} = 0.159 in (@ 2nd Floor) RISA Model

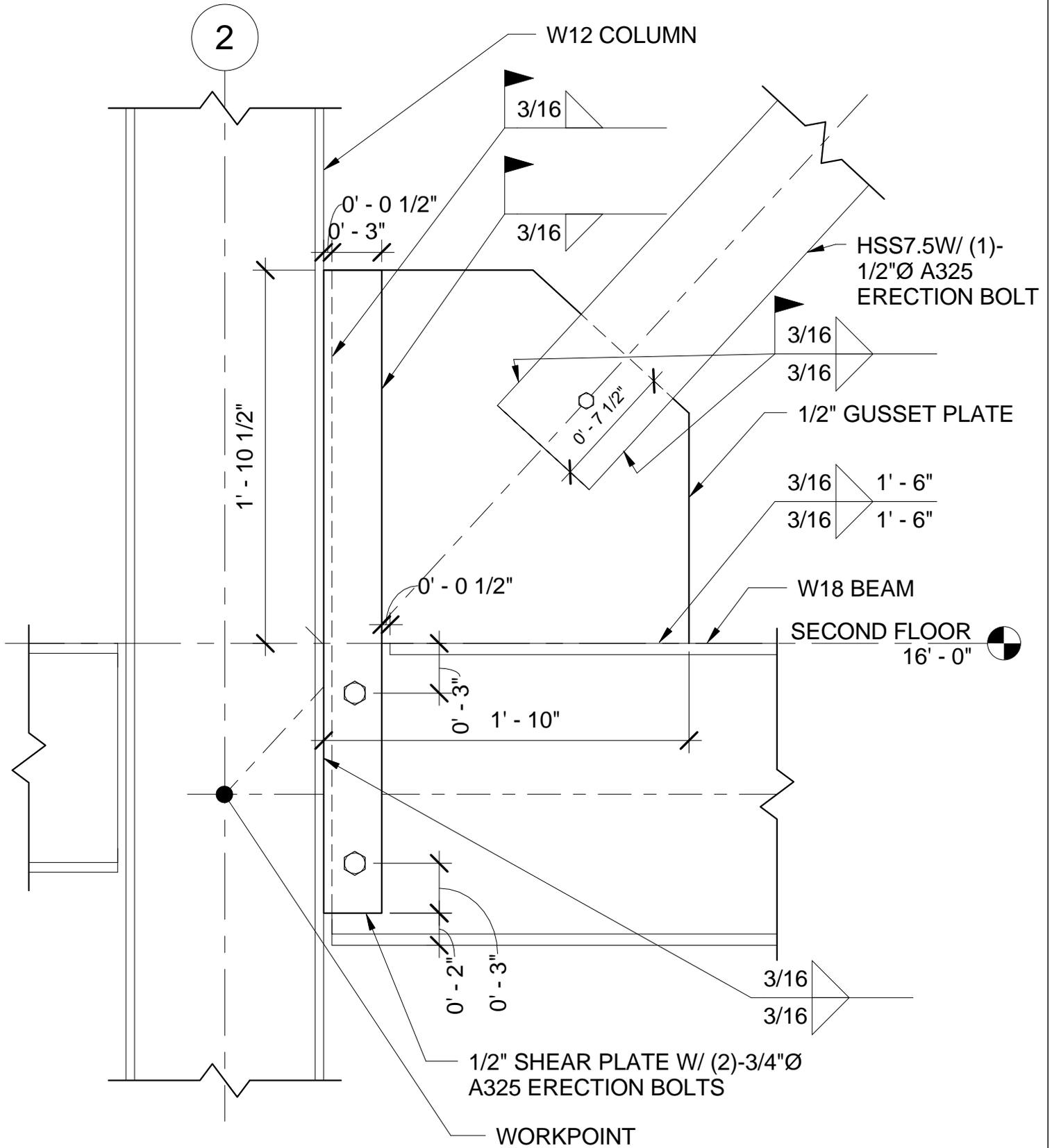
C_d = 3.25 Tbl 12.2-1

Importance Factor, I_e = 1.0 Tbl 1.5-2

Design Story Drift, δ_x = 0.517 in [$\delta_x = C_d \delta_{xe} / I_e$] Eqn 12.8-15

δ_x = 0.517 in < Δ_a = 3.8 in OK

Appendix D - OCBF Connection Designs



OCBF CONNECTION @ FLOOR BEAM & COLUMN

1 1/2" = 1'-0"

OCBF - 1

Scale 1 1/2" = 1'-0"

Gusset Plate Welded Design Design (OCBF) - At Floor Beam and Column

Loads to Plate

Required Tensile Strength, $T_u =$	43.1	k	(Max. Load Effect, $\Omega=2.0$)	Ref. Upper Brace Design
Req'd Compressive Strength, $P_u =$	46.7	k	(Max. Load Effect, $\Omega=2.0$)	Ref. Upper Brace Design

Gusset Plate Properties

AISC Manual 14th ed.

Gusset Plate, $F_{yg} =$	36	ksi		Tbl 2-4
Gusset Plate, $F_{ug} =$	58	ksi		Tbl 2-4
Gusset Length Along Beam, $l_b =$	22	in	[Measured from face of Col. to edge of gusset]	
Gusset Length Along Column, $l_c =$	22.5	in	[Measured from face of Beam to edge of gusset]	

Brace Properties

Ref. Brace Calculations, AISC Manual 14th ed.

$F_{y-HSS} =$	42	ksi		Tbl 2-4
$F_{u-HSS} =$	58	ksi		Tbl 2-4
$t_{HSS} =$	0.291	in		Tbl 1-13
$A_{g-HSS} =$	6.59	in ²		Tbl 1-13
$D_{HSS} =$	7.5	in		Tbl 1-13

Beam Properties

Ref. Beam Calculations, AISC Manual 14th ed.

$F_{y-WF} =$	50	ksi		Tbl 2-4
$F_{u-WF} =$	65	ksi		Tbl 2-4
$d_{Beam} =$	18.2	in		Tbl 1-1
$t_{w-beam} =$	0.43	in		Tbl 1-1
$k_{Beam} =$	1.08	in		Tbl 1-1
$T_{Beam} =$	15.13	in		Tbl 1-1

Column Properties

Ref. Column Calculations, AISC Manual 14th ed.

$F_{y-WF} =$	50	ksi		Tbl 2-4
$F_{u-WF} =$	65	ksi		Tbl 2-4
$d_{col} =$	11.9	in		Tbl 1-1
$k_{Col} =$	1.02	in		Tbl 1-1
$t_{w-Col} =$	0.30	in		Tbl 1-1
$t_{f-Col} =$	0.52	in		Tbl 1-1
$E =$	29000	ksi		

$$\text{Weld } F_{EXX} = 70 \text{ ksi}$$

$$\text{Weld Unit Strength, } \Phi R_n = 1.392 \text{ k/in} * D \quad [\Phi * (1/\sqrt{2}) * (1/16) * (0.6F_{EXX})] \quad \text{Eqn J2-3 \& Tbl J2.5}$$

Brace to Gusset Plate Weld

AISC 360-10

$$\text{Shear Rupture Strength, } \Phi R_{n-HSS} = 7.595 \text{ k/in} \quad [\Phi R_n = 0.75 * 0.6 F_{u-HSS} t] \quad \text{Eqn J4-4}$$

$$\text{Fillet Weld Strength, } \Phi R_{nw} = \Phi * (1/\sqrt{2}) * (D/16) * (0.6 F_{EXX}) \quad [\Phi = 0.75] \quad \text{Eqn J2-3 \& Tbl J2.5}$$

$$\text{Maximum Fillet Weld Size, } D = 5.46 \text{ /16"} \quad [D = \Phi R_{n-HSS} / \Phi R_{nw}]$$

$$\text{Use Weld Size, } D = 3 \text{ /16"} \quad \text{}$$

$$\text{Number of Fillet Welds} = 4 \quad \text{}$$

$$\text{Min. Fillet Weld Length, } l_w = 2.58 \text{ in} \quad [l_{w-min} = T_u / \Phi R_{nw}]$$

$$\text{Use Weld Length, } l_w = 7.5 \text{ in} = 7.5 \quad \text{}$$

Weld Length \geq D of Brace \therefore Weld Meets Requirements of Tbl D3.1 for Shear Lag Factor

$$\text{Weld Strength, } \Phi R_{nw} = 125.3 \text{ k} \quad [D = \text{Qty.} * \Phi * l_w * D * \text{Weld Unit Strength}]$$

$$\Phi R_{nw} = 125.3 \text{ k} > 43.1 \text{ k} \quad \text{Welds are Sufficient}$$

Gusset Plate Thickness

AISC 360-10

$$\text{Min. } t \text{ to Develop Force, } t_{min} = 0.110 \text{ in} \quad [T_u / (2\Phi(0.6F_{ug})l_w)] [\Phi = 0.75] \quad \text{Ref. Eqn J4-4}$$

$$\text{Use Gusset Plate, } t_g = 0.5 \text{ in} \quad \text{}$$

Shear Lag Rupture of Brace

AISC 360-10

$$\text{HSS Slot Width} = 0.0625 \text{ in} \quad (\text{Each Side of Gusset Plate})$$

$$\text{Net Area, } A_n = 6.23 \text{ in}_2 \quad [A_n = A_g - \text{Area of 2 Slots}]$$

$$l_w = 7.5 \text{ in} < 9.75 \text{ in [1.3D]} \quad \text{Tbl D3.1}$$

$$x = 2.39 \text{ in} \quad \text{Tbl D3.1}$$

$$U = 0.682 \quad [1 - (x/l_w)] \text{ for } l_w < 1.3D_{HSS} \quad \text{Tbl D3.1}$$

Effective Area of HSS, $A_e = 4.24 \text{ in}^2$ $[A_e = U \cdot A_n]$ Eqn D3-1

$P_n = 246.17 \text{ k}$ $[P_n = A_e F_u] \quad [\Phi = 0.75]$ Eqn D2-2

$\Phi P_n = 184.63 \text{ k} > 43.1 \text{ k}$ Brace is Adequate

Whitmore Section

AISC 360-10

$l_w = 7.5 \text{ in}$

Whitmore Width, $L_w = 16.16 \text{ in}$ $[2 \cdot l_w \tan(30^\circ) + D_{HSS}]$

Tension Yield of l_w , $\Phi R_n = 290.88 \text{ k}$ $[\Phi R_n = \Phi [F_{yg} \cdot L_w \cdot t_g]] [\Phi = 0.90]$ Eqn J4-1

$\Phi R_n = 290.88 \text{ k} > 43.1 \text{ k}$ l_w is Adequate

Connection Interface Forces (Uniform Force Method)

AISC Manual 14th ed. Part 13, pg. 13-3

Gusset to Beam Weld Setback = 4 in [Width & Height of Clip]

Angle from Column to Brace, $\Theta = 42.86 \text{ degrees}$

Beam Eccentricity, $e_b = 9.100 \text{ in}$ $[e_b = d_{beam}/2]$

Beam Eccentricity, $e_c = 5.950 \text{ in}$ $[e_c = d_{column}/2]$

$\alpha = 13 \text{ in}$

$\beta = 11.25 \text{ in}$

$e_b \tan \Theta - e_c = 2.493 \text{ in}$

$\alpha - \beta \tan \Theta = 2.562 \text{ in}$

Net Eccentricity = -0.069 in (Negligible)

$r = 27.8 \text{ in}$ $[\text{Sqrt}((\alpha + e_c)^2 + (\beta + e_b)^2)]$

$V_{uc} = 17.44 \text{ k}$ $[(\beta/r) \cdot T_u]$

$H_{uc} = 9.22 \text{ k}$ $[(e_c/r) \cdot T_u]$

$V_{ub} = 14.10 \text{ k}$ $[(\beta/r) \cdot T_u]$

$H_{ub} = 20.15 \text{ k}$ $[(e_c/r) \cdot T_u]$

Load Angle=	35.0	degrees	$[\arctan(V_{ub}/H_{ub})]$	
Thickness of Thinner Part=	0.29	in	[Beam Web or Gusset thickness]	
Min. Weld Size, D=	3	/16"		Tbl J2.4
Weld Size, D=	3	/16"	(One weld each side)	
Weld Length, l=	22.00	in	$[l_b - \text{Weld Setback}]$	
Eccentricity, e_x =	9.1	in	$[d_{beam}/2]$	
a=	0.414		$[e_x/l]$	(Round up for "a" in table)
"a" value used in table=	0.4			
C=	2.81			Tbl 8-4
Φ =	0.75			
Strength of Weld Group, ΦR_n =	139.1	k		
Eccentric Force, R_{ub} =	30.7	k	$[1.25V(V_{ub}^2 + H_{ub}^2)]$	
ΦR_n =	139.1	k	>	30.7 k
				Weld is Adequate

Gusset Plate Rupture at Beam Weld

AISC 360-10: Section J

Set gusset plate shear rupture strength equal to weld strength

t_{g-min} =	0.320	in	$[(2 * \Phi r_n * D_{req}) / (.75 * .6 F_{ug})]$	Ref. Eqn J4-4
t=	0.5	in	>	0.320 in
				Thickness is Adequate

Because the gusset plate is sufficient for shear rupture based on weld size, the plate is also sufficient for tension and shear yielding.

Beam Web Local Yielding

AISC 360-10

Distance to Force from Beam End= α =	13.00	in	\leq	d=	18.2	in	Use Eqn J-10-3
Beam Force Applied at α , ΦR_n =	524.88	k	>	V_{ub} =	14.10	k	OK
	$[\Phi R_n = \Phi(5k_{Beam} + l_{wb})F_y - W_{ftw-beam}]$ [$\Phi=1.00$]						Eqn J10-3

Set gusset plate shear rupture strength equal to weld strength

$$t_{g-min} = 0.320 \text{ in} \quad [(2 * \Phi_r * D_{req}) / (.75 * .6 F_{ug})] \quad \text{Ref. Eqn J4-4}$$

$$t = 0.5 \text{ in} > 0.320 \text{ in} \quad \text{Thickness is Adequate}$$

Because the gusset plate is sufficient for shear rupture based on weld size, the plate is also sufficient for tension

Shear Plate

AISC Manual 14th ed.

$$\text{Plate, } F_{y-pl} = 36 \text{ ksi} \quad \text{Tbl 2-4}$$

$$\text{Plate, } F_{u-pl} = 58 \text{ ksi} \quad \text{Tbl 2-4}$$

$$\text{Plate Thickness, } t_{pl} = 0.5 \text{ in}$$

$$\text{Plate Height, } h_{pl} = 43.75 \text{ in}$$

$$\text{Plate Width, } w_{pl} = 3.5 \text{ in}$$

Gusset to Shear Plate Weld

AISC 360-10

$$\text{Load Angle} = 62.1 \text{ degrees} \quad [\arctan(V_{uc}/H_{uc}) \text{ (Round down in table)}]$$

$$\text{Thickness of Thinner Part} = 0.50 \text{ in} \quad \text{[Shear Plate or Gusset thickness]}$$

$$\text{Min. Weld Size, } D = 3 / 16'' \quad \text{Tbl J2.4}$$

$$\text{Weld Size, } D = 3 / 16'' \quad \text{(One weld each side)}$$

$$\text{Weld Length, } l = 22.50 \text{ in} \quad [l_c]$$

$$kl = 3 \text{ in} \quad \begin{array}{l} \text{(Distance between fillet welds)} \\ \text{(Edge of shear plate to edge of gusset)} \end{array}$$

$$k = 0.13 \quad [kl/l] \quad \text{(Round down in table)}$$

$$\text{Eccentricity, } e_x = 2 \text{ in} \quad \text{[Col. Flange to Center of weld group]}$$

$$a = 0.089 \quad [e_x/l] \quad \text{(Round up for "a" in table)}$$

$$\text{a value used in table} = 0.2$$

$$\text{k value used in table} = 0.1$$

$$C = 3.92 \quad \text{Tbl 8-4}$$

$$\Phi = 0.75$$

$$\text{Strength of Weld Group, } \Phi R_n = 198.5 \text{ k}$$

$$\text{Eccentric Force, } R_{ub} = 24.7 \text{ k} \quad [1.25V(V_{uc}^2 + H_{uc}^2)]$$

$$\Phi R_n = 198.5 \text{ k} > 24.7 \text{ k}$$

Weld is Adequate

Shear Plate to Column Weld (at gusset)

AISC 360-10

$$\text{Load Angle} = 62.1 \text{ degrees} \quad [\arctan(V_{uc}/H_{uc})]$$

$$\text{Thickness of Thinner Part} = 0.43 \text{ in} \quad [\text{Shear Plate or Gusset thickness}]$$

$$\text{Min. Weld Size, } D = 3/16'' \quad \text{Tbl J2.4}$$

$$\text{Weld Size, } D = 3/16'' \quad (\text{One weld each side})$$

$$\text{Weld Length, } l = 22.50 \text{ in} \quad [l_c]$$

$$\text{Eccentricity, } e_x = 5.95 \text{ in} \quad [d_{col}/2]$$

$$a = 0.264 \quad [e_x/l] \quad (\text{Round up for "a" in table})$$

$$\text{a value used in table} = 0.5$$

$$C = 2.75$$

Tbl 8-4

$$\Phi = 0.75$$

$$\text{Strength of Weld Group, } \Phi R_n = 139.2 \text{ k} \quad [\Phi * D * l * C]$$

$$\text{Eccentric Force, } R_{ub} = 24.7 \text{ k} \quad [1.25V(V_{uc}^2 + H_{uc}^2)]$$

$$\Phi R_n = 139.2 \text{ k} > 24.7 \text{ k}$$

Weld is Adequate

Gusset Plate Rupture at Column Weld

AISC 360-10: Section J

Set gusset plate shear rupture strength equal to weld strength

$$t_{g-min} = 0.320 \text{ in} \quad [(2 * \Phi R_n * D_{req}) / (.75 * .6 F_{ug})] \quad \text{Ref. Eqn J4-4}$$

$$t = 0.5 \text{ in} > 0.320 \text{ in}$$

Thickness is Adequate

Because the gusset plate is sufficient for shear rupture based on weld size, the plate is also sufficient for tension and shear yielding.

Force applied to column is located at a distance greater than the depth of column from column end

$$\Phi R_n = 407.1 \text{ k} > H_{uc} = 9.22 \text{ k} \quad \text{OK}$$

$$[\Phi R_n = \Phi(5k_{Col} + l_{wc})F_{y-WF}t_{w-Col}] \quad [\Phi = 1.00] \quad \text{Eqn J10-2}$$

Forces at Beam to Column Connection

AISC 360-10

Shear, $V_u = 23.45 \text{ k}$ [$V_{ub} + V_u$ from gravity loads, ref. beam design]

Axial, $H_u = 20.15 \text{ k}$ [H_{ub}]

Beam to Shear Plate Weld

AISC 360-10

Load Angle = 40.7 degrees [$\arctan(V_{uc}/H_{uc})$] (Round down in table)

Thickness of Thinner Part = 0.43 in [Shear Plate or Gusset thickness]

Min. Weld Size, D = 3 /16" Tbl J2.4

Weld Size, D = 3 /16" (One weld each side)

Weld Length, $l_w = 17.12$ in [$d_{beam} - k_{beam}$] Tbl 1-1

$k_l = 3$ in (Distance between fillet welds)
(Edge of shear plate to edge of gusset)

$k = 0.175$ [k/l] (Round down in table)

Eccentricity, $e_x = 2$ in [Col. Flange to Center of weld group]

$a = 0.117$ [e_x/l] (Round up for "a" in table)

a value used in table = 0.1
k value used in table = 0.1
 $C = 4.49$ Tbl 8-4

$\Phi = 0.75$

Strength of Weld Group, $\Phi R_n = 173.0 \text{ k}$ [$\Phi * D * l * C$]

Eccentric Force, $R_{ub} = 38.6 \text{ k}$ [$1.25V(V_{uc}^2 + H_{uc}^2)$]

$\Phi R_n = 173.0 \text{ k} > 38.6 \text{ k}$ Weld is Adequate

Load Angle=	40.7	degrees		$[\arctan(V_{uc}/H_{uc})]$	
Thickness of Thinner Part=	0.50	in		[Shear Plate or Column Flange thickness]	
Min. Weld Size, D=	3	/16"			Tbl J2.4
Weld Size, D=	3	/16"		(One weld each side)	
Weld Length, l_w =	17.12	in		$[d_{beam}-k_{beam}]$	Tbl 1-1
Eccentricity, e_x =	5.95	in		$[d_{col}/2]$	
a=	0.348			$[e_x/l]$	(Round up for "a" in table)
a value used in table=	0.3				
C=	3.48				Tbl 8-4
Φ =	0.75				
Strength of Weld Group, ΦR_n =	134.0	k		$[\Phi * D * l * C]$	
Eccentric Force, R_{ub} =	38.6	k		$[1.25V(V_{uc}^2+H_{uc}^2)]$	
ΦR_n =	134.0	k	>	38.6 k	Weld is Adequate

Beam Web Shear Strength

ΦV_n =	192.84	k		$[\Phi * 0.6 F_y t_w T]$	Eqn J4-3
ΦV_n =	192.84	k	>	23.45 k	Beam Web is Adequate

Whitmore Area, $A_g = 8.08 \text{ in}^2$

$K = 1.2$

Tbl C-A-7.1

$L = 14.70 \text{ in}$

$r = 0.144 \text{ in}$

$[t/\sqrt{12}]$

$KL/r = 122.2 > 25$

Use Chapter E

$4.71\sqrt{E/F_y} = 133.68$

$KL/r = 122.2 < 133.68$

Use Eqn E3-2

E3

$F_e = 19.18 \text{ ksi}$

$[F_e = \pi^2 E / (KL/r)^2]$

Eqn E3-4

$F_{cr} = 16.41 \text{ ksi}$

$[F_{cr} = (0.658^{F_y/F_e}) * F_y]$

Eqn 3-2

$\Phi = 0.9$

E1(b)

$P_n = 132.6 \text{ k}$

$[P_n = F_{cr} A_g]$

Eqn E3-1

Compression Strength, $\Phi P_n = 119.3 \text{ k}$

$> 46.70 \text{ k}$

OK

Beam Web Crippling

AISC Manual 14th ed. Chapter 9

$N/d = 1.21 > 0.2$

For W18x76 Beam

$\Phi R_5 = 95 \text{ k}$

Tbl 9-4

$\Phi R_6 = 11.3 \text{ k/in}$

Tbl 9-4

$\Phi R_n = 343.6 \text{ k}$

$[\Phi R_5 + \Phi R_6 N]$

pg. 9-19

$\Phi R_n = 343.6 \text{ k}$

$> 46.7 \text{ k}$

OK

Column Web Crippling

AISC 360-10: J10.3

Force applied to column is located at a distance greater than the depth of column from column end

$R_n = 383.2 \text{ k}$

$[0.8 t_w^2 \{1 + 3(N/d)(t_w/t_f) 1.5\} \sqrt{E F_y w t_f / t_w}]$

Eqn J10-4

$\Phi = 0.75$

$\Phi R_n = 287.4 \text{ k}$

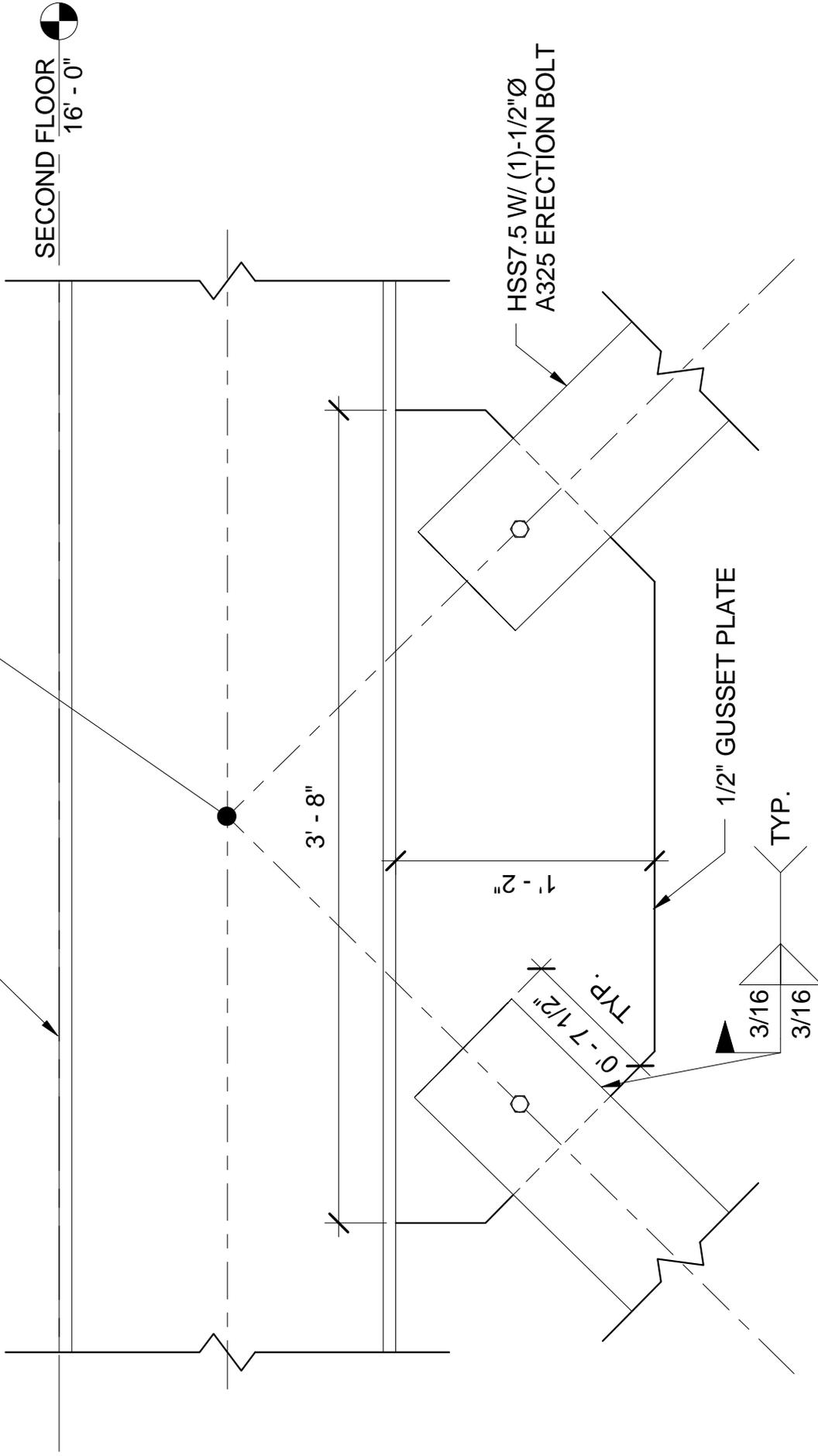
$> 9.22 \text{ k}$

OK

W18 BEAM

WORKPOINT

SECOND FLOOR
16' - 0"



HSS7.5 W/ (1)-1/2"Ø
A325 ERECTION BOLT

1/2" GUSSET PLATE

TYP.

3/16

3/16

3' - 8"

1' - 2"

0' - 7 1/2"
TYP.

Gusset Plate (OCBF) at Floor Beam - 2 Braces

Plate Material Properties

$F_y = 36$ ksi
 $F_u = 58$ ksi
 $E = 29000$ ksi

Brace Properties

$F_{y-HSS} = 42$ ksi
 $F_{u-HSS} = 58$ ksi
 $A_{g-HSS} = 6.59$ in²
 $t_{HSS} = 0.291$ in
 $D = 7.5$ in

Weld Properties

Weld $F_{EXX} = 70$ ksi
 Weld Unit Strength, $\Phi_r n = 1.392$ k/in * D [$\Phi * (1/\sqrt{2}) * (1/16) * (0.6F_{EXX})$] Eqn J2-3

Loads

Max Tension Force, $T_u = 108.7$ k Ref. Lower Brace
 Max Compression Force, $P_u = 119.3$ k Ref. Lower Brace
 Max Brace Force = 119.3 k

Vert. Brace Length = 16.73 ft [Distance Between Workpoints]
 Horiz. Brace Length = 15 ft

Brace Angle = 48.12 Degrees [arctan(Vert./Horiz.)]

Total Horizontal Force = 152.21 k [($T_u + P_u$) * Cos(Brace Angle)]
 Vertical Force = 88.82 k [Max of (T_u & P_u) * Sin(Brace Angle)]

$$\text{Weld Size, } D = 3 / 16''$$

$$\text{Number of Welds} = 4$$

$$\text{Weld Length, } l_w = 7.5 \text{ in} = 7.5 \text{ in}$$

Weld Length \geq D of Brace \therefore Weld Meets Requirements of Tbl D3.1 for Shear Lag Factor

$$\text{Weld Strength, } \Phi R_n = 125.3 \text{ k} \quad [\Phi r_n * D * l_w * \# \text{ of Welds}]$$

$$\Phi R_n = 125.3 \text{ k} > 119.3 \text{ k} \quad \text{OK}$$

HSS Wall Shear Rupture

AISC 360-10

$$A_{nv} = 4.365 \text{ in}^2 \quad [2 * l_w * t_{HSS}]$$

$$\text{Shear Rupture Strength, } \Phi R_n = 113.9 \text{ k} \quad [\Phi * 0.6 F_{u-HSS} * A_{nv}] \quad \text{Eqn J4-4}$$

$$\Phi R_n = 113.9 \text{ k} > 108.7 \text{ k} \quad \text{OK}$$

Gusset Shear Rupture at Brace

AISC 360-10

$$\text{Gusset Thickness, } t_g = 0.5 \text{ in}$$

$$A_{nv} = 7.5 \text{ in}^2 \quad [2 * l_w * t_{HSS}]$$

$$\text{Shear Rupture Strength, } \Phi R_n = 195.8 \text{ k} \quad [\Phi * 0.6 F_{u-HSS} * A_{nv}] [\Phi = 0.75] \quad \text{Eqn J4-4}$$

$$\Phi R_n = 195.8 \text{ k} > 108.7 \text{ k} \quad \text{OK}$$

HSS Slot=	0.0625 in	(Each Side of Gusset Plate for Clearance)	
Net Area, A_n =	6.23 in ²	$[A_n=A_g-\text{Area of 2 Slots}]$	
l_w =	7.50 in	<	9.75 in $[1.3D]$ Tbl D3.1
x =	2.39 in		Tbl D3.1
U =	0.682		$[1-(x/l_w)]$ for $l_w < 1.3D_{HSS}$ Tbl D3.1
Effective Area of HSS, A_e =	4.24 in ²	$[A_e=U*A_n]$	Eqn D3-1
P_n =	246.17 k	$[P_n=A_eF_u]$ $[\Phi=0.75]$	Eqn D2-2
ΦP_n =	184.63 k	>	108.7 k OK

Gusset Shear Yielding at Brace

AISC 360-10

Shear Yield Strength, ΦR_n =	349.06 k	$[2*\Phi*0.6F_yA_g]$ $[\Phi=1.0]$	Eqn J4-3
ΦR_n =	349.06 k	>	108.70 k OK

Whitmore Section

AISC 360-10

Whitmore Width, L_w =	16.16 in	$[2*l_w\tan(30^\circ)+D_{HSS}]$
A_g =	8.08 in ²	$[L_w*t_g]$

Gusset Tensile Yielding at Brace

AISC 360-10

Whitmore Area, A_g =	8.08 in ²		
Tensile Yield Strength, ΦR_n =	261.80 in ²	$[\Phi F_y A_g]$ $[\Phi=0.9]$	Eqn J4-1
ΦP_n =	261.80 k	>	108.70 k OK

Whitmore Area, $A_{\text{effective}}=A_g= 8.08 \text{ in}^2$

Tensile Rupture Strength, $\Phi P_n= 351.49 \text{ k}$ $[\Phi F_u A_e]$ $[\Phi=0.75]$ Eqn J4-1

$\Phi P_n= 351.49 \text{ k}$ $>$ 108.70 k OK

Compression in Gusset

Whitmore Area, $A_g= 8.08 \text{ in}^2$

$K= 1.2$ Tbl C-A-7.1

$L= 5.15 \text{ in}$

$r= 0.144 \text{ in}$ $[t/\sqrt{12}]$

$KL/r= 42.84$ $>$ 25 Use Chapter E

$4.71\sqrt{E/F_y}= 133.68$

$KL/r= 42.8$ $<$ 133.68 Use Eqn E3-2 E3

$F_e= 155.99 \text{ ksi}$ $[F_e=\pi^2 E/(KL/r)^2]$ Eqn E3-4

$F_{cr}= 32.69 \text{ ksi}$ $[F_{cr}=(.658^{F_y/F_e}) \cdot F_y]$ Eqn 3-2

$\Phi= 0.9$ E1(b)

$P_n= 264.1 \text{ k}$ $[P_n=F_{cr} A_g]$ Eqn E3-1

Compression Strength, $\Phi P_n= 237.7 \text{ k}$ $>$ 119.30 k OK

Gusset Length, $l_b = 44$ in

[Length of Gusset Along Beam]

 $A_g = 22$ in²[$l_b t_g$]Shear Rupture Strength, $\Phi R_n = 574.2$ k[$\Phi * 0.6 F_{u-HSS} * A_{nv}$] [$\Phi=0.75$]

Eqn J4-4

Shear Yield Strength, $\Phi R_n = 475.2$ k[$\Phi * 0.6 F_y A_g$] [$\Phi=1.0$]

Eqn J4-3

Governing $\Phi R_n = 475.2$ k

> 152.21 k

OK

Vertical Shear in Gusset

Gusset Length, $l_v = 14$ in $A_g = 7$ in²[$l_b t_g$]Shear Rupture Strength, $\Phi R_n = 182.7$ k[$\Phi * 0.6 F_{u-HSS} * A_{nv}$] [$\Phi=0.75$]

Eqn J4-4

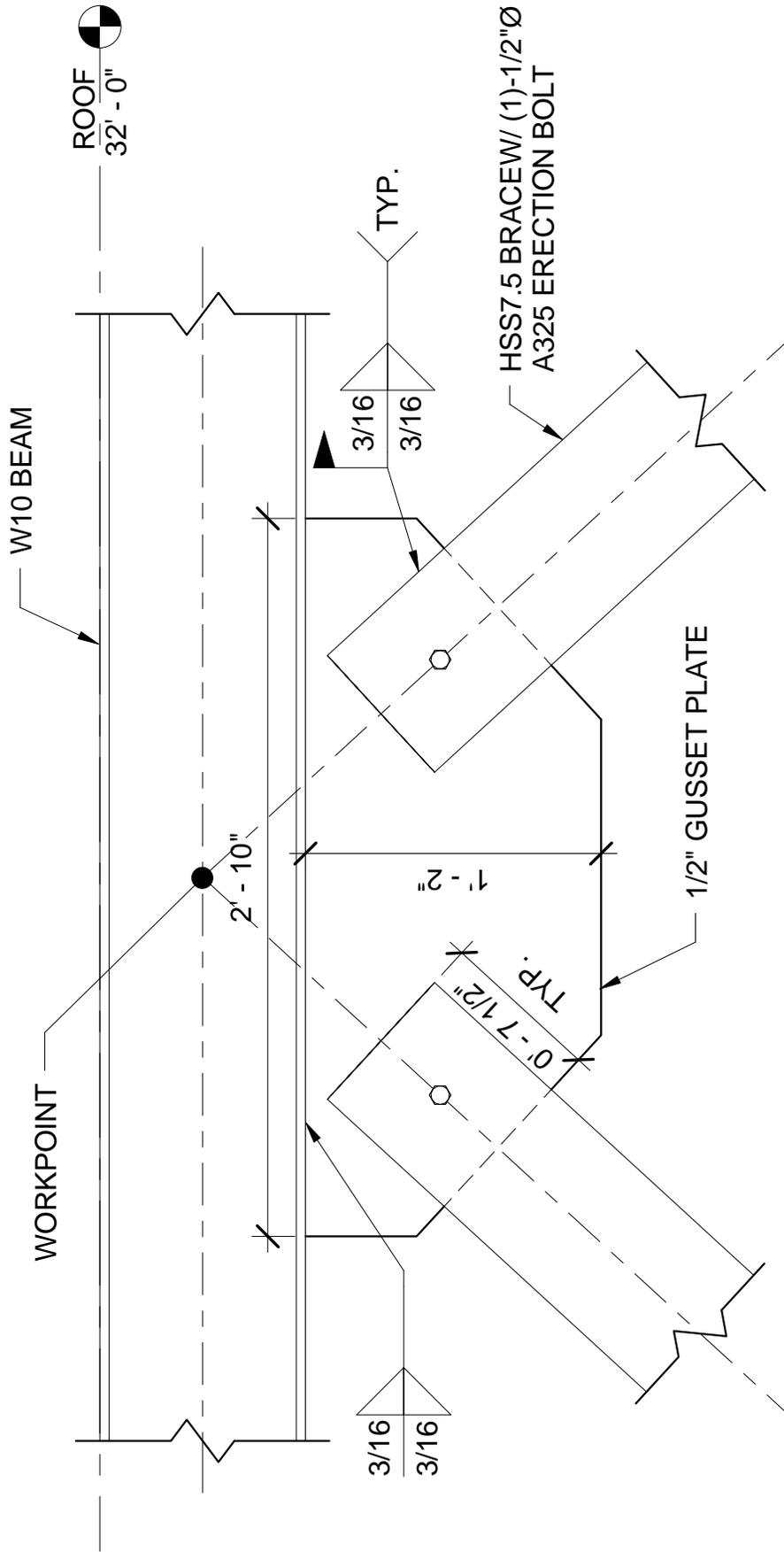
Shear Yield Strength, $\Phi R_n = 151.2$ k[$\Phi * 0.6 F_y A_g$] [$\Phi=1.0$]

Eqn J4-3

Governing $\Phi R_n = 151.2$ k

> 88.82 k

OK



Gusset Plate (OCBF) at Roof Beam - 2 Braces

Plate Material Properties

$F_y = 36$ ksi
 $F_u = 58$ ksi
 $E = 29000$ ksi

Brace Properties

$F_{y-HSS} = 42$ ksi
 $F_{u-HSS} = 58$ ksi
 $A_{g-HSS} = 6.59$ in²
 $t_{HSS} = 0.291$ in
 $D = 7.5$ in

Weld Properties

Weld $F_{EXX} = 70$ ksi
 Weld Unit Strength, $\Phi_r = 1.392$ k/in * D [$\Phi^*(1/\sqrt{2})*(1/16)*(0.6F_{EXX})$] Eqn J2-3

Loads

Max Tension Force, $T_u = 43.1$ k Ref. Lower Brace
 Max Compression Force, $P_u = 46.7$ k Ref. Lower Brace
 Max Brace Force = 46.7 k

Vert. Brace Length = 16.08 ft [Distance Between Workpoints]
 Horiz. Brace Length = 15 ft

Brace Angle = 47.00 Degrees [arctan(Vert./Horiz.)]

Total Horizontal Force = 61.25 k [($T_u + P_u$)*Cos(Brace Angle)]
 Vertical Force = 34.15 k [Max of (T_u & P_u)*Sin(Brace Angle)]

$$\text{Weld Size, } D = 3 / 16''$$

$$\text{Number of Welds} = 4$$

$$\text{Weld Length, } l_w = 7.5 \text{ in} = 7.5 \text{ in}$$

Weld Length \geq D of Brace \therefore Weld Meets Requirements of Tbl D3.1 for Shear Lag Factor

$$\text{Weld Strength, } \Phi R_n = 125.3 \text{ k} \quad [\Phi R_n * D * l_w * \# \text{ of Welds}]$$

$$\Phi R_n = 125.3 \text{ k} > 46.7 \text{ k} \quad \text{OK}$$

HSS Wall Shear Rupture

AISC 360-10

$$A_{nv} = 4.365 \text{ in}^2 \quad [2 * l_w * t_{HSS}]$$

$$\text{Shear Rupture Strength, } \Phi R_n = 113.9 \text{ k} \quad [\Phi * 0.6 F_{u-HSS} * A_{nv}] \quad \text{Eqn J4-4}$$

$$\Phi R_n = 113.9 \text{ k} > 43.1 \text{ k} \quad \text{OK}$$

Gusset Shear Rupture at Brace

AISC 360-10

$$\text{Gusset Thickness, } t_g = 0.5 \text{ in}$$

$$A_{nv} = 7.5 \text{ in}^2 \quad [2 * l_w * t_{HSS}]$$

$$\text{Shear Rupture Strength, } \Phi R_n = 195.8 \text{ k} \quad [\Phi * 0.6 F_{u-HSS} * A_{nv}] \quad [\Phi = 0.75] \quad \text{Eqn J4-4}$$

$$\Phi R_n = 195.8 \text{ k} > 43.1 \text{ k} \quad \text{OK}$$

HSS Slot=	0.0625 in	(Each Side of Gusset Plate for Clearance)	
Net Area, A_n =	6.23 in ²	$[A_n=A_g-\text{Area of 2 Slots}]$	D3.2
l_w =	7.50 in	< 9.75 in $[1.3D]$	Tbl D3.1
x =	2.39 in		Tbl D3.1
U =	0.682	$[1-(x/l_w)]$ for $l_w < 1.3D_{HSS}$	Tbl D3.1
Effective Area of HSS, A_e =	4.24 in ²	$[A_e=U*A_n]$	Eqn D3-1
P_n =	246.17 k	$[P_n=A_eF_u]$ $[\Phi=0.75]$	Eqn D2-2
ΦP_n =	184.63 k	> 43.1 k	OK

Gusset Shear Yielding at Brace

AISC 360-10

Shear Yield Strength, ΦR_n =	349.06 k	$[2*\Phi*0.6F_yA_g]$ $[\Phi=1.0]$	Eqn J4-3
ΦR_n =	349.06 k	> 43.10 k	OK

Whitmore Section

AISC 360-10

Whitmore Width, L_w =	16.16 in	$[2*l_w\tan(30^\circ)+D_{HSS}]$
A_g =	8.08 in ²	$[L_w*t_g]$

Gusset Tensile Yielding at Brace

AISC 360-10

Whitmore Area, A_g =	8.08 in ²		
Tensile Yield Strength, ΦR_n =	261.80 in ²	$[\Phi F_y A_g]$ $[\Phi=0.9]$	Eqn J4-1
ΦP_n =	261.80 k	> 43.10 k	OK

$$\text{Whitmore Area, } A_{\text{effective}}=A_g= 8.08 \text{ in}^2$$

$$\text{Tensile Rupture Strength, } \Phi P_n= 351.49 \text{ k} \quad [\Phi F_u A_e] \quad [\Phi=0.75] \quad \text{Eqn J4-1}$$

$$\Phi P_n= 351.49 \text{ k} > 43.10 \text{ k} \quad \text{OK}$$

Compression in Gusset

AISC 360-10

$$\text{Whitmore Area, } A_g= 8.08 \text{ in}^2$$

$$K= 1.2 \quad \text{Tbl C-A-7.1}$$

$$L= 4.85 \text{ in}$$

$$r= 0.144 \text{ in} \quad [t/\sqrt{12}]$$

$$KL/r= 40.36 > 25 \quad \text{Use Chapter E}$$

$$4.71\sqrt{E/F_y}= 133.68$$

$$KL/r= 40.4 < 133.68 \quad \text{Use Eqn E3-2} \quad \text{E3}$$

$$F_e= 175.74 \text{ ksi} \quad [F_e=\pi^2 E/(KL/r)^2] \quad \text{Eqn E3-4}$$

$$F_{cr}= 33.04 \text{ ksi} \quad [F_{cr}=(.658^{F_y/F_e}) * F_y] \quad \text{Eqn 3-2}$$

$$\Phi= 0.9 \quad \text{E1(b)}$$

$$P_n= 267.0 \text{ k} \quad [P_n=F_{cr} L_w t_g] \quad \text{Eqn E3-1}$$

$$\text{Compression Strength, } \Phi P_n= 240.3 \text{ k} > 46.70 \text{ k} \quad \text{OK}$$

Gusset Length, $l_b = 34$ in

[Length of Gusset Along Beam]

 $A_g = 17$ in²[$l_b t_g$]Shear Rupture Strength, $\Phi R_n = 443.7$ k[$\Phi * 0.6 F_{u-HSS} * A_{nv}$] [$\Phi=0.75$]

Eqn J4-4

Shear Yield Strength, $\Phi R_n = 367.2$ k[$\Phi * 0.6 F_y A_g$] [$\Phi=1.0$]

Eqn J4-3

Governing $\Phi R_n = 367.2$ k

> 61.25 k

OK

Vertical Shear in Gusset

Gusset Length, $l_v = 14$ in $A_g = 7$ in²[$l_b t_g$]Shear Rupture Strength, $\Phi R_n = 182.7$ k[$\Phi * 0.6 F_{u-HSS} * A_{nv}$] [$\Phi=0.75$]

Eqn J4-4

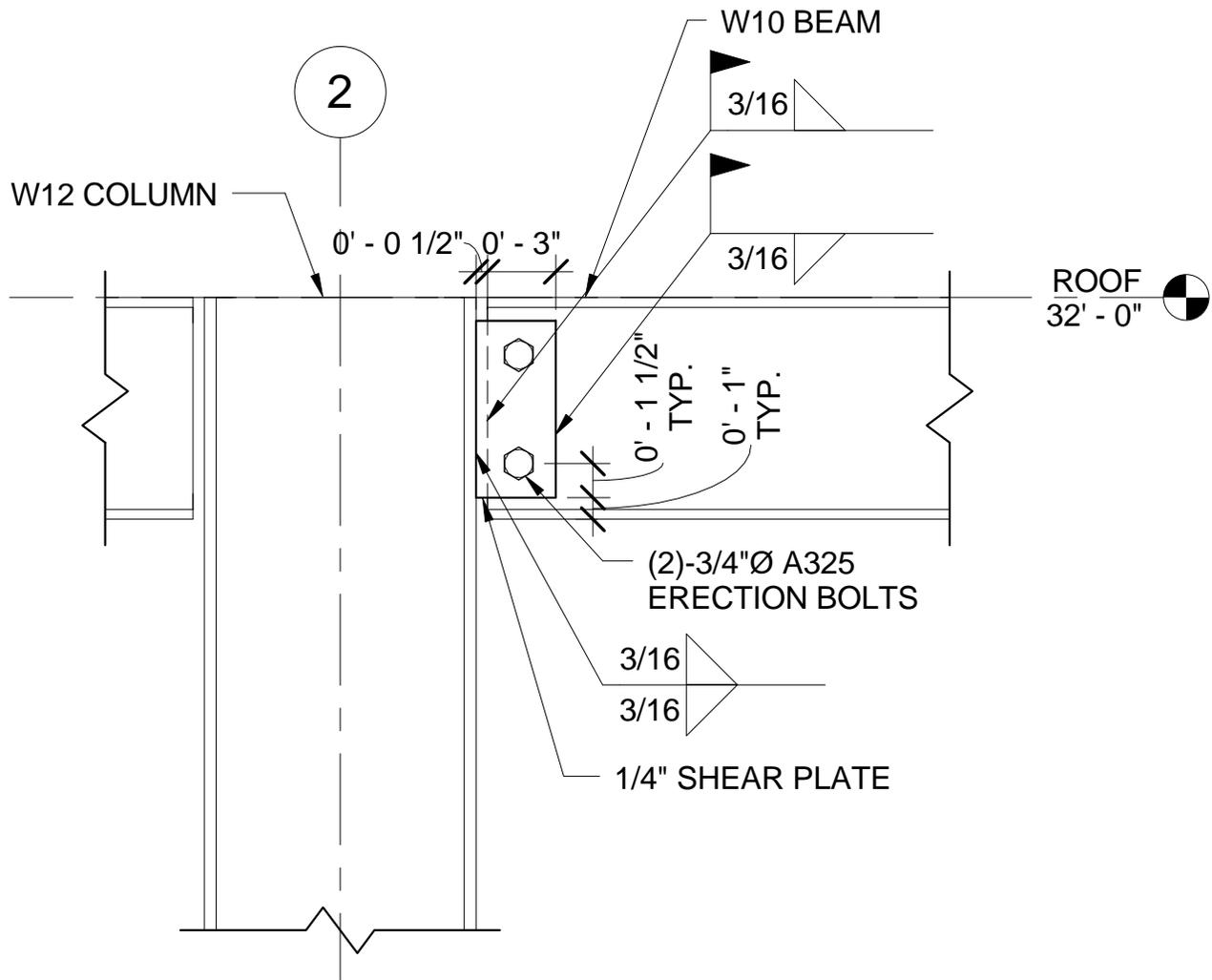
Shear Yield Strength, $\Phi R_n = 151.2$ k[$\Phi * 0.6 F_y A_g$] [$\Phi=1.0$]

Eqn J4-3

Governing $\Phi R_n = 151.2$ k

> 34.15 k

OK



OCBF CONNECTION @ ROOF BEAM & COLUMN

1 1/2" = 1'-0"

OCBF - 4

Scale 1 1/2" = 1'-0"

Roof Beam Connection to Column (OCBF)

Beam Properties

Ref. Beam Calculations, AISC Manual 14th ed.

Beam: W10x33

$F_{y-WF} = 50$ ksi

Tbl 2-4

$F_{u-WF} = 65$ ksi

Tbl 2-4

$d_{Beam} = 9.7$ in

Tbl 1-1

$t_{w-beam} = 0.29$ in

Tbl 1-1

$t_{f-beam} = 9.71$

$k_{Beam} = 0.94$ in

Tbl 1-1

$T_{Beam} = 7.50$ in

Tbl 1-1

Column Properties

Ref. Column Calculations, AISC Manual 14th ed.

Column: W12x40

$F_{y-WF} = 50$ ksi

Tbl 2-4

$F_{u-WF} = 65$ ksi

Tbl 2-4

$d_{col} = 11.9$ in

Tbl 1-1

$k_{Col} = 1.02$ in

Tbl 1-1

$t_{w-Col} = 0.30$ in

Tbl 1-1

$t_{f-Col} = 0.52$ in

Tbl 1-1

$E = 29000$ ksi

Weld Properties

AISC Manual 14th ed.

Weld $F_{EXX} = 70$ ksi

Weld Unit Strength, $\Phi_r = 1.392$ k/in * D

$[\Phi * (1/\sqrt{2}) * (1/16) * (0.6F_{EXX})]$

Eqn J2-3

Shear Plate

AISC Manual 14th ed.

Plate, $F_{y-pl} = 36$ ksi

Tbl 2-4

Plate, $F_{u-pl} = 58$ ksi

Tbl 2-4

Plate Thickness, $t_{pl} = 0.25$ in

Plate Height, $h_{pl} = 7.73$ in

(T_{beam})

Plate Width, $w_{pl} = 3.5$ in

Shear, V_u =	3.56	k	$[V_u \text{ ref. roof beam design}]$
Axial, H_u =	29.23	k	$[P_u \text{ ref. roof beam design}]$

Shear in Plate

A_g =	1.93	in ²	$[h_{pl}t_{pl}]$
Shear Rupture Strength, ΦR_n =	50.4	k	$[\Phi * 0.6F_{u-pl} * A_{nv}] [\Phi=0.75]$ Eqn J4-4
Shear Yield Strength, ΦR_n =	41.7	k	$[\Phi * 0.6F_{y-pl}A_g] [\Phi=1.0]$ Eqn J4-3
Governing ΦR_n =	41.7	k	> 3.56 k OK

Beam to Shear Plate Weld

Load Angle=	83.1	degrees	$[\arctan(V_{uc}/H_{uc})]$ (Round down in table)
Thick. of Thinner Part=	0.25	in	
Min. Weld Size, D=	2	/16"	Tbl J2.4
Weld Size, D=	3	/16"	(One weld each side)
Weld Length, l_w =	7.73	in	$[h_{pl}]$
kl =	3	in	(Distance between fillet welds) (Edge of shear plate to edge of gusset)
k =	0.388098		$[kl/l]$ (Round down in table)
Eccentricity, e_x =	2	in	[Col. Flange to Center of weld group]
a =	0.259		$[e_x/l]$ (Round up for "a" in table)
a value used in table=	0.1		
k value used in table=	0.1		
C=	4.49		Tbl 8-4
Φ =	0.75		
Strength of Weld Group, ΦR_n =	78.1	k	
Eccentric Force, R_{ub} =	36.8	k	$[1.25\sqrt{(V_{uc}^2 + H_{uc}^2)}]$
ΦR_n =	78.1	k	> 36.8 k OK

Load Angle=	83.1	degrees	$[\arctan(V_{uc}/H_{uc})]$	
Thick. of Thinner Part=	0.25	in		
Min. Weld Size, D=	2	/16"		Tbl J2.4
Weld Size, D=	3	/16"	(One weld each side)	
Weld Length, l_w =	7.73	in	$[h_{pl}]$	
Eccentricity, e_x =	5.95	in	$[d_{col}/2]$	
a=	0.770		$[e_x/l]$	(Round up for "a" in table)
a value used in table=	0.3			
C=	3.48			Tbl 8-4
Φ =	0.75			
Strength of Weld Group, ΦR_n =	60.5	k		
Eccentric Force, R_{ub} =	36.8	k	$[1.25\sqrt{(V_{uc}^2+H_{uc}^2)}]$	
ΦR_n =	60.5	k	>	36.8 k Weld is Adequate

Appendix E - SCBF Member Designs

Upper Brace Design (SCBF)

Member Loads

Factored Compressive Force, $P_u = 19.1$ k [From Example]

Factored Tensile Force, $T_u = 15.0$ k [From Example]

Member Properties

AISC Manual 14th ed.

Brace Horizontal Component Length = 15 ft

Brace Vertical Component Length = 16.13 ft

Brace Length, $L = 22.02$ ft [Workpoint to Workpoint]

Actual Brace Length, $L = 18.00$ ft

Actual Brace Length, $L = 216.00$ in

Member Selection: HSS4X0.220

Tbl 1-13

$E = 29000$ ksi

$F_y = 42$ ksi

Tbl 2-4

$F_u = 50$ ksi

Tbl 2-4

$D = 4$ in

Tbl 1-13

$t = 0.205$ in

Tbl 1-13

$A_g = 2.44$ in²

Tbl 1-13

$D/t = 19.5$

Tbl 1-13

$r = 1.34$ in

Tbl 1-13

Check Slenderness

$k = 1$

AISC 360-10: Tbl C-A-7.1

Brace Slenderness Limit = 200.00

AISC 341-10: F2.5b

$KL/r = 161.2 < 200.00$

Member not Slender, OK

Check Local Buckling

Brace must be "Highly Ductile" per AISC 341: F2.5a

$\lambda = 19.50$

$[\lambda = D/t]$

AISC Manual 14th ed. Tbl 1-13

$\lambda_{nd} = 26.24$

$[\lambda_{nd} = .038E/F_y]$

AISC 341-10 Tbl D1.1

$\lambda < \lambda_{nd}$

Member is Seismically Compact, OK

$$4.71\sqrt{E/F_y} = 123.76$$

$$KL/r = 161.2 > 123.76 \quad \text{Use Equation E3-3 for } F_{cr}$$

$$F_e = 11.02 \text{ ksi} \quad [F_e = \pi^2 E / (KL/r)^2] \quad \text{Eqn E3-4}$$

$$F_{cr} = 9.66 \text{ ksi} \quad [F_{cr} = 0.877 F_e] \quad \text{Eqn 3-3}$$

$$\Phi = 0.9 \quad \text{E1(b)}$$

$$P_n = 23.6 \text{ k} \quad [P_n = F_{cr} A_g] \quad \text{Eqn E3-1}$$

$$\Phi P_n = 21.2 \text{ k} > 19.1 \text{ k} \quad \text{Brace is Adequate}$$

Tensile Strength of Brace

AISC 360-10

Tensile Yielding

D2(a)

$$\Phi = 0.9$$

$$P_n = 102.48 \text{ k} \quad [P_n = F_y A_g] \quad \text{Eqn D2-1}$$

$$\Phi P_n = 92.23 \text{ k}$$

Tensile Rupture

D2(b)

$$\Phi = 0.75$$

$$P_n = 122 \text{ k} \quad [P_n = F_u A_e; A_e = A_g] \quad \text{Eqn D2-2}$$

$$\Phi P_n = 91.50 \text{ k}$$

$$\text{Tensile Capacity, } \Phi P_n = 91.50 \text{ k} > 15.0 \text{ k} \quad \text{Brace is Adequate}$$

Assumed Brace Forces

AISC 341-10

$$R_y = 1.6$$

Tbl A3.1

$$R_t = 1.2$$

Tbl A3.1

Assumed Brace Forces: Tension

AISC 341-10

Expected Yield Strength of Brace, $P_t = 164.0$ k

$$[P_t = R_y F_y A_g]$$

F1.4.4a(1)(i)(a)

Assumed Brace Forces: Compression

AISC 341-10

Buckled Compressive Strength, $P_{c-min} = 7.07$ k

$$[.3P_c]$$

F1.4.4a(1)(ii)

$$4.71\sqrt{E/F_y} = 123.76$$

AISC 350-10: E3

$$KL/r = 161.2 > 123.76$$

Use Equation E3-3 for F_{cr}

AISC 350-10: E3

$$F_e = 11.02 \text{ ksi } [F_e = \pi^2 E / (KL/r)^2]$$

AISC 350-10: Eqn E3-4

$$F_{cr} = 9.66 \text{ ksi } [F_{cr} = .877 F_e]$$

AISC 350-10: Eqn 3-3

AISC 341-10: F2.3

Expected Max Compression Strength, $P_c = 26.9$ k

$$[P_n = 1.14 F_{cr} A_g]$$

AISC 341-10: F2.3

Unbalanced Forces on Beam

AISC 341-10

Horizontal Component, $P_{tx} = 111.68$ k

Vertical Component, $P_{ty} = 120.06$ k

Horizontal Component, $P_{c-min x} = 4.82$ k

Vertical Component, $P_{c-min y} = 5.18$ k

Components of Buckled Compressive Strength

Horizontal Component, $P_{c-max x} = 18.30$ k

Vertical Component, $P_{c-max y} = 19.68$ k

Unbalanced Vertical Load, $Q_{bv} = 115$ k

$$[Q_{bv} = P_{ty} - P_{cy}] \text{ (on roof beam)}$$

Unbalanced Horizontal Load, $Q_{bp} = 65$ k

$$[Q_{bp} = (P_{tx} + P_{cx}) / 2] \text{ (on roof beam)}$$

Lower Brace Design (SCBF)

Member Loads

Compressive Force in Brace, P_u =	49.7	k	[From RISA]	(Factored)
Tensile Force in Brace, T_u =	36.8	k	[From RISA]	(Factored)

Member Properties

AISC Manual 14th ed.

Brace Horizontal Component Length=	15	ft
Brace Vertical Component Length=	14.75	ft

Brace Length, L =	21.04	ft
Brace Length, L =	252.45	in

Actual Brace Length, L =	18.00	ft
Actual Brace Length, L =	216.00	in

Member Selection: HSS5.500X0.258 Tbl 1-13

E =	29000	ksi	
F_y =	42	ksi	Tbl 2-4
F_u =	58	ksi	Tbl 2-4
D =	5.5	in	Tbl 1-13
t =	0.24	in	Tbl 1-13
A_g =	3.97	in ²	Tbl 1-13
D/t =	22.9		Tbl 1-13
r =	1.86	in	Tbl 1-13

Check Slenderness

k = 1 AISC 350-10: Tbl C-A-7.1

Brace Slenderness Limit= 200.00 AISC 341-10: F2.5b

KL/r = 116.1 < 200.00 Member not Slender, OK

Check Local Buckling

Brace must be "Highly Ductile" per AISC 341: F2.5a

λ = 22.900 [$\lambda=D/t$] AISC Manual 14th ed. Tbl 1-13

λ_{hd} = 26.24 [$\lambda_{hd}=.038E/F_y$] AISC 341-10 Tbl D1.1

λ < λ_{md} Member is Seismically Compact, OK

$$4.71\sqrt{E/F_y} = 123.76$$

$$KL/r = 116.1 < 123.76 \quad \text{Use Eqn E3-2 for } F_{cr}$$

$$F_e = 21.22 \text{ ksi} \quad [F_e = \pi^2 E / (KL/r)^2] \quad \text{Eqn E3-4}$$

$$F_{cr} = 18.35 \text{ ksi} \quad [F_{cr} = (0.658^{F_y/F_e}) F_y] \quad \text{Eqn 3-2}$$

$$\Phi = 0.9 \quad \text{E1(b)}$$

$$P_n = 72.8 \text{ k} \quad [P_n = F_{cr} A_g] \quad \text{Eqn E3-1}$$

$$\Phi P_n = 65.5 \text{ k} > 49.7 \text{ k} \quad \text{Brace is Adequate in Compression}$$

Tensile Strength of Brace

AISC 360-10

Tensile Yielding

D2(a)

$$\Phi = 0.9$$

$$P_n = 166.7 \text{ k} \quad [P_n = F_y A_g] \quad \text{Eqn D2-1}$$

$$\Phi P_n = 150.07 \text{ k}$$

Tensile Rupture

D2(a)

$$\Phi = 0.75$$

$$P_n = 230.3 \text{ k} \quad [P_n = F_u A_e; A_e = A_g] \quad \text{Eqn D2-1}$$

$$\Phi P_n = 172.70 \text{ k}$$

$$\text{Tensile Capacity, } \Phi P_n = 150.07 \text{ k} > 36.8 \text{ k} \quad \text{Brace is Adequate in Tension}$$

Assumed Brace Forces

AISC 341-10

$R_y = 1.6$

Tbl A3.1

$R_t = 1.2$

Tbl A3.1

Assumed Brace Forces: Tension

AISC 341-10

Expected Yield Strength of Brace, $P_t = 266.8$ k

$[P_t = R_y F_y A_g]$

F1.4.4a(1)(i)(a)

Assumed Brace Forces: Compression

AISC 341-10

Buckled Compressive Strength, $P_c = 21.85$ k

$[.3P_c]$

F1.4.4a(1)(ii)

$4.71\sqrt{E/F_y} = 123.76$

AISC 350-10: E3

$KL/r = 116.1 < 123.76$

Use Eqn E3-2 for F_{cr}

AISC 350-10: E3

$F_e = 21.22$ ksi

$[F_e = \pi^2 E / (KL/r)^2]$

AISC 350-10: Eqn E3-4

$F_{cre} = 17.86$ ksi

$[F_{cr} = (.658^{R_y F_y / F_e}) * R_y F_y]$

AISC 350-10: Eqn 3-2

AISC 341-10: F2.3

Expected Compression Strength, $P_c = 80.8$ k

$[P_n = 1.14 F_{cre} A_g]$

AISC 341-10: F2.3

Unbalanced Forces on Floor Beam

AISC 341-10

Horizontal Component, $P_{tx} = 190.22$ k

Vertical Component, $P_{ty} = 187.05$ k

Horizontal Component, $P_{c-min x} = 15.58$ k

Vertical Component, $P_{c-min y} = 15.32$ k

Components of Buckled Compressive Strength

Horizontal Component, $P_{c-max x} = 57.63$ k

Vertical Component, $P_{c-max y} = 56.67$ k

Unbalanced Vert. Load, $Q_{bv} = 172$ k

$[Q_{bv} = P_{ty} - P_{c-min y}]$

Unbalanced Horiz. Load, $Q_{bp} = 124$ k

$[Q_{bp} = (P_{tx} + P_{c-max x}) / 2]$

Column Design Design (SCBF)

Reference: AISC 341-10 UNO

Member Loads

Factored Compression, $P_u = 345.00$ k [ref.Member Loads (SCBF Columns)]
Factored Tension, $T_u = 0$ k [ref.Member Loads (SCBF Columns)]

Member Properties

AISC Manual 14th ed.

Column Unbraced Length, $l_u = 16.00$ ft
Column Unbraced Length, $l_u = 192.00$ in

Member Selection: **W14X68**

$E = 29000$	ksi	
$F_y = 50$	ksi	Tbl 2-4
$F_u = 65$	ksi	Tbl 2-4
$A_g = 20$	in ²	Tbl 1-1
$d = 14$	in	Tbl 1-1
$t_w = 0.415$	in	Tbl 1-1
$b_f = 10$	in	Tbl 1-1
$t_f = 0.720$	in	Tbl 1-1
$k = 1.310$	in	Tbl 1-1
$h/t_w = 27.5$		Tbl 1-1
$r_x = 6.01$	in	Tbl 1-1
$r_y = 2.46$	in	Tbl 1-1

Column must meet criteria for "Highly Ductile" members per AISC 341-10: F2.5a

$\lambda_f =$	6.94		$[\lambda = b_f / 2t_f]$	
$\lambda_{hd} =$	7.22		$[\lambda_{hd} = .30\sqrt{E/F_y}]$	Tbl D1.1
λ	<	λ_r	Flanges are not Slender, OK	
$\Phi P_y =$	900	k	$[\Phi F_y A_g]$ $[\Phi = 0.9]$	
$C_a =$	0.383	>	0.125 $[P_u / \Phi P_y]$	Tbl D1.1
$\lambda_w =$	27.50		$[\lambda = h / t_w]$	
$\lambda_{hd} =$	47.23		$[\lambda_r = 0.77\sqrt{E/F_y}(2.93 - C_a) > 1.49\sqrt{E/I}]$	Tbl D1.1
λ	<	λ_{hd}	Web is not Slender, OK	

Compressive Strength of Column

AISC 360-10

$K =$	1			Tbl C-A-7.1	
$4.71\sqrt{E/F_y} =$	113.43				
$KL/r =$	78.0	<	113.43	Use Eqn E3-2	
$F_e =$	46.99	ksi	$[F_e = \pi^2 E / (KL/r)_2]$	Eqn E3-4	
$F_{cr} =$	32.03	ksi	$[F_{cr} = (.658^{F_y/F_e}) * F_y]$	Eqn 3-2	
$\Phi =$	0.9			E1(b)	
$P_n =$	640.6	k	$[P_n = F_{cr} A_g]$	Eqn E3-1	
$\Phi P_n =$	576.5	k	>	345 k	Column is Adequate in Compression

Tensile Yielding

$$\Phi = 0.9$$

$$P_n = 1000 \text{ k}$$

$$\Phi P_n = 900.00 \text{ k}$$

$$[P_n = F_y A_g]$$

Eqn D2-1

Tensile Rupture

$$\Phi = 0.75$$

$$P_n = 1300 \text{ k}$$

$$\Phi P_n = 975.00 \text{ k}$$

$$[P_n = F_u A_e; A_e = A_g]$$

Eqn D2-1

D2(a)

$$\text{Tensile Capacity, } \Phi P_n = 900.00 \text{ k}$$

$$> 0 \text{ k}$$

Column is Adequate in Tension

Floor Beam Design Design (SCBF)

Reference: AISC 341-10 UNO

Load Combination Factors

Combination 5: ASCE 7-10 12.4.3.2

Dead Load= 1.3084
 Live Load= 1.0
 Seismic Load= 2.0

Member Loads

M_D =	30.63	k-ft		(ref. member loads)
M_L =	33.4	k-ft		(ref. member loads)
$M_{u_{Qbv}}$ =	1288	k-ft	$[M_{Qb} = Q_{bv} * L/4][\text{Factored}]$	Lower Brace Design
V_D =	4.08	k		(ref. member loads)
V_L =	4.45	k		(ref. member loads)
$V_{u_{Qbv}}$ =	86	k	[Factored]	Lower Brace Design
Factored Moment, M_u =	1361.5	k-ft		
Factored Compression, P_u =	123.9	k	$[P_u = Q_{bh}]$	
Factored Shear, V_u =	95.7	k		
Factored Shear, $V_{u(D+L)}$ =	9.8	k	(Shear without seismic contributions)	

Member Properties

AISC Manual 14th ed.

Beam Length, L= 30.00 ft
 Brace Length, L= 360.00 in

Member Selection: W30X124

E=	29000	ksi		
F_y =	50	ksi		Tbl 2-4
F_u =	65	ksi		Tbl 2-4
A_g =	36.5	in ²		Tbl 1-1
d=	30.2	in		Tbl 1-1
t_w =	0.585	in		Tbl 1-1
t_f =	0.93	in		
k=	1.58	in		Tbl 1-1
T=	27.04	in	$[d-2k]$	Tbl 1-2
$b_f/2t_f$ =	5.65			Tbl 1-1
h/t_w =	46.2			Tbl 1-1
I_x =	5360	in ⁴		Tbl 1-1
S_x =	355	in ³		Tbl 1-1

$r_x =$	12.1	in		Tbl 1-1
$Z_x =$	408	in ³		Tbl 1-1
$r_y =$	2.23	in		Tbl 1-1
$h_o =$	29.3	in		Tbl 1-1

Check Local Buckling

AISC 341-10

Beams must meet criteria for "Moderately Ductile" members per AISC 341-10: F2.5a

$\lambda_f =$	5.65			$[\lambda = b_f / 2t_f]$	
$\lambda_{md} =$	9.15			$[\lambda_r = .38v(E/F_y)]$	Tbl D1.1
λ	<	λ_r	Flanges are Compact, OK		
$\Phi P_y =$	1642.5	k		$[\Phi F_y A_g] [\Phi = 0.9]$	
$C_a =$	0.075	<	0.125	$[P_u / \Phi P_y]$	Tbl D1.1
$\lambda_w =$	46.20			$[\lambda = h / t_w]$	
$\lambda_{md} =$	71.76			$[\lambda_r = 3.76v(E/F_y)(1 - 2.75C_a)]$	Tbl D1.1
λ	<	λ_{md}	Web is not Slender, OK		

Flexural Strength of Beam

AISC 360-10

Limiting Unbraced Length, $L_p =$	12.2	ft		Tbl 3-2
Limiting Unbraced Length, $L_r =$	56.9	ft		Tbl 3-2
Unbraced Length, $L_b =$	14.4	ft		
$L_r >$	$L_b >$	L_p	Use Eqn F2-2	
$\Phi =$	0.9			F1(1)
$C_b =$	1.0			F1(2)
$M_p =$	1700.0	k-ft		Eqn F2-1
$M_n =$	2231.851	k-ft	>	$M_p \therefore$ Use M_p
$\Phi M_n =$	1530	k-ft		

Compressive Strength of Beam

AISC 360-10

$$K = 1$$

Tbl C-A-7.1

Unbraced Length for x Axis, $L_x = 30.00$ ft [Beam Length, Assume No Support From Braces]

Unbraced Length for y Axis, $L_y = 14.4$ ft [L_b]

$$KL_x/r_x = 29.75$$

$$KL_y/r_y = 77.58$$

Governing $KL_y/r_y = 77.58$ [Max KL/r]

$$4.71\sqrt{E/F_y} = 113.43$$

$$KL/r = 77.6 < 113.43 \quad \text{Use Eqn E3-2}$$

$$F_e = 47.56 \text{ ksi} \quad [F_e = \pi^2 E / (KL/r)_2] \quad \text{Eqn E3-4}$$

$$F_{cr} = 32.20 \text{ ksi} \quad [F_{cr} = (.658^{F_y/F_e}) * F_y] \quad \text{Eqn 3-2}$$

$$\Phi = 0.9 \quad \text{E1(b)}$$

$$P_n = 1175.3 \text{ k} \quad [P_n = F_{cr} A_g] \quad \text{Eqn E3-1}$$

$$\Phi P_n = 1057.8 \text{ k}$$

Second Order Effects

AISC 360-10: C2.1b

$$\alpha = 1.0$$

$$B_2 = 1.0 \quad \text{[No translation of beam ends]}$$

$$P_{e1} = 11837.4 \text{ k} \quad [\pi^2 EI / (K_1 L)^2] \quad [K_1 = 1.0] \quad \text{Eqn C-A-8-5}$$

$$C_m = 1.0 \quad \text{[For members subject to transverse Loading]} \quad \text{A8.2.1(b)}$$

$$P_{nt} = 0.0 \text{ k}$$

$$P_{lt} = 123.9 \text{ k} \quad [P_u]$$

$$P_r = 123.9 \text{ k} \quad [P_{nt} + B_2 P_{lt}] \quad \text{Eqn A-8-2}$$

$$B_1 = 1.01 \quad \{ [C_m / (1 - (\alpha P_r / P_{e1}))] \} \quad \text{Eqn A-8-3}$$

$$M_{nt} = 1361.5 \text{ k-ft} \quad [M_u]$$

$$M_{lt} = 0.0 \text{ k-ft}$$

$$M_{rx} = 1375.88 \text{ k-ft} \quad [B_1 M_{nt} + B_2 M_{lt}] \quad \text{Eqn A-8-1}$$

Combined Loading

AISC 360-10: H1

$$P_r/P_c = 0.117 < 0.2 \quad \text{Use Eqn H1-1b}$$

$$\text{Beam Interaction Ratio} = 0.958 \quad [(P_r/2P_c) + \{(M_{rx}/M_{cx}) + (M_{ry}/M_{cy})\}] \quad \text{Eqn H1-1b}$$

$$0.958 < 1.0 \quad \text{Beam is Adequate for Combined Loading}$$

Shear Strength of Beam

AISC 350-10

$$h/t_w \text{ Limit} = 53.95 \quad [2.24V_e/F_y] \quad \text{G2.1a}$$

$$h/t_w = 46.2 < 53.95$$

$$\Phi_v = 1$$

$$C_v = 1$$

$$A_w = 17.667 \text{ in}^2$$

$$V_n = 530.0 \text{ k} \quad [V_n = 0.6 * F_y * A_w * C_v] \quad \text{Eqn G2-1}$$

$$\Phi V_n = 530.0 \text{ k} > 95.65 \text{ k} \quad \text{Beam is Adequate in Shear}$$

Lateral Bracing

AISC 360-10: Appendix 6.3

$$\text{Distance Between Flanges, } h_o = 29.3 \text{ in} \quad \text{Tbl 1-1}$$

$$C_d = 1.0 \quad (\text{Single Curvature}) \quad \text{A-6.3.1a}$$

$$\text{Required Brace Strength, } P_{br} = 11.27 \text{ k} \quad [0.02 M_{rx} C_d / h_o] \quad \text{Eqn A-6-7}$$

(Bracing Provided for Top and Bottom Flanges)

Roof Beam Design Design (SCBF)

Reference: AISC 341-10 UNO

Load Combination Factors

Combination 5: ASCE 7-10 12.4.3.2

Dead Load= 1.3084
Snow Load= 0.2
Seismic Load= 2.0

Member Loads

$M_D =$	16.9	k-ft		
$M_S =$	8.4	k-ft		
$M_{u_{Qbv}} =$	862	k-ft	$[M_{Qb} = Q_b * L/4][\text{Factored}]$	Upper Brace Design
$V_D =$	2.25	k		
$V_S =$	1.12	k		
$V_{u_{Qbv}} =$	57	k		Upper Brace Design
Factored Moment, $M_u =$	885.4	k-ft	[Factored]	
Factored Compression, $P_u =$	65.0	k	$[P_u = Q_{bh}]$	
Factored Shear, $V_u =$	60.6	k		
Factored Shear, $V_{u(D+L)} =$	3.2	k	(Shear without seismic contributions)	

Beam Length, $L = 30.00$ ft
 Brace Length, $L = 360.00$ in

Member Selection: **W27X94**

$E =$	29000	ksi	
$F_y =$	50	ksi	Tbl 2-4
$F_u =$	65	ksi	Tbl 2-4
$A_g =$	27.6	in ²	Tbl 1-1
$d =$	26.9	in	Tbl 1-1
$t_w =$	0.49	in	Tbl 1-1
$t_f =$	0.745	in	Tbl 1-1
$k =$	1.34	in	Tbl 1-1
$T =$	24.22	in	Tbl 1-1
$b_f/2t_f =$	6.7		Tbl 1-1
$h/t_w =$	49.5		Tbl 1-1
$I_x =$	3270	in ⁴	Tbl 1-1
$S_x =$	243	in ³	Tbl 1-1
$r_x =$	10.9	in	Tbl 1-1
$Z_x =$	278	in ³	Tbl 1-1
$r_y =$	2.12	in	Tbl 1-1
$h_o =$	26.2	in	Tbl 1-1

Beams must meet criteria for "Moderately Ductile" members per AISC 341-10: F2.5a

$\lambda_f =$	6.70		$[\lambda = b_f / 2t_f]$	
$\lambda_{md} =$	9.15		$[\lambda_r = .38v(E/F_y)]$	Tbl D1.1
λ	<	λ_r	Flanges are Compact, OK	
$\Phi P_y =$	1242	k	$[\Phi F_y A_g]$ [$\Phi = 0.9$]	
$C_a =$	0.052	<	0.125	$[P_u / \Phi P_y]$ Tbl D1.1
$\lambda_w =$	49.50		$[\lambda = h / t_w]$	
$\lambda_{md} =$	77.52		$[\lambda_r = 3.76v(E/F_y)(1 - 2.75C_a)]$	Tbl D1.1
λ	<	λ_{md}	Web is not Slender, OK	

Flexural Strength of Beam

Limiting Unbraced Length, $L_p =$	12.2	ft	Tbl 3-2
Limiting Unbraced Length, $L_r =$	56.9	ft	Tbl 3-2

Unbraced Length, $L_b =$ 14.4 ft

$L_r > L_b > L_p$ Use Eqn F2-2

$\Phi =$ 0.9 F1(1)

$C_b =$ 1.0 F1(2)

$M_p =$ 1158.3 k-ft Eqn F2-1

$M_n =$ 1522.65 k-ft > M_p ∴ Use M_p

$\Phi M_n =$ 1042.5 k-ft

Tbl C-A-7.1

$$K = 1$$

Unbraced Length for x Axis, $L_x = 30.00$ ft [Beam Length, Assume No Support From Braces]

Unbraced Length for y Axis, $L_y = 14.4$ ft [L_b]

$$KL_x/r_x = 33.03$$

$$KL_y/r_y = 81.60$$

Governing $KL_y/r_y = 81.60$ [Max KL/r]

$$4.71\sqrt{E/F_y} = 113.43$$

$$KL/r = 81.6 < 113.43 \quad \text{Use Eqn E3-2}$$

$$F_e = 42.98 \text{ ksi} \quad [F_e = \pi^2 E / (KL/r)_2] \quad \text{Eqn E3-4}$$

$$F_{cr} = 30.73 \text{ ksi} \quad [F_{cr} = (.658^{F_y/F_e}) * F_y] \quad \text{Eqn 3-2}$$

$$\Phi = 0.9 \quad \text{E1(b)}$$

$$P_n = 848.0 \text{ k} \quad [P_n = F_{cr} A_g] \quad \text{Eqn E3-1}$$

$$\Phi P_n = 763.2 \text{ k}$$

$\alpha =$	1.0		
$B_2 =$	1.0	[No translation of beam ends]	Eqn A-8-5
$P_{e1} =$	7221.7 k	$[\pi^2 EI / (K_1 L)^2]$ [$K_1 = 1.0$]	Eqn C2-5
$C_m =$	1.0	[For members subject to transverse loading]	A-8.2.1(b)
$P_{nt} =$	0.0 k		
$P_{lt} =$	65.0 k	$[P_u]$	
$P_r =$	65.0 k	$[P_{nt} + B_2 P_{lt}]$	Eqn A-8-2
$B_1 =$	1.01	$[\{C_m / (1 - (\alpha P_r / P_{e1}))\} \geq 1.0]$	Eqn A-8-3
$M_{nt} =$	885.4 k-ft	$[M_u]$	
$M_{lt} =$	0.0 k-ft		
$M_{rx} =$	893.41 k-ft	$[B_1 M_{nt} + B_2 M_{lt}]$	Eqn A-8-1

Combined Loading

$P_r / P_c =$	0.085	<	0.2	Use Eqn H1-1b
Beam Interaction Ratio =	0.900			$[(P_r / 2P_c) + \{(M_{rx} / M_{cx}) + (M_{ry} / M_{cy})\}]$ Eqn H1-1b
	0.900	<	1.0	Beam is Adequate for Combined Loading

$$h/t_w \text{ Limit} = 53.95 \quad [2.24V_E/F_y] \quad \text{G2.1a}$$

$$h/t_w = 49.5 < 53.95$$

$$\Phi_v = 1$$

$$C_v = 1$$

$$A_w = 13.181 \text{ in}^2$$

$$V_n = 395.4 \text{ k} \quad [V_n = 0.6 * F_y * A_w * C_v] \quad \text{Eqn G2-1}$$

$$\Phi V_n = 395.4 \text{ k} > 60.6 \text{ k} \quad \text{Beam is Adequate in Shear}$$

Lateral Bracing

AISC 360-10: Appendix 6.3

$$\text{Distance Between Flanges, } h_o = 26.2 \text{ in} \quad \text{Tbl 1-1}$$

$$C_d = 1.0 \quad (\text{Single Curvature}) \quad \text{A-6.3.1a}$$

$$\text{Required Brace Strength, } P_{br} = 8.18 \text{ k} \quad [0.02M_{rx}C_d/h_o] \quad \text{Eqn A-6-7}$$

(Bracing Provided for Top and Bottom Flanges)

Story Drift (SCBF)

Story Drift Limit

ASCE 7-10

Building Risk Category: II

Story Height, h_{sx} = 16 ft

Allowable Story Drift, Δ_a = 0.020 * h_{sx} Tbl 12.12-1

Allowable Story Drift, Δ_a = 0.32 ft [$\Delta_a = 0.020 h_{sx}$] Tbl 12.12-1

Allowable Story Drift, Δ_a = 3.84 in

Calculated Story Drift

ASCE 7-10

Elastic Story Drift, δ_{xe} = 0.138 in (@ Roof Level) RISA Model

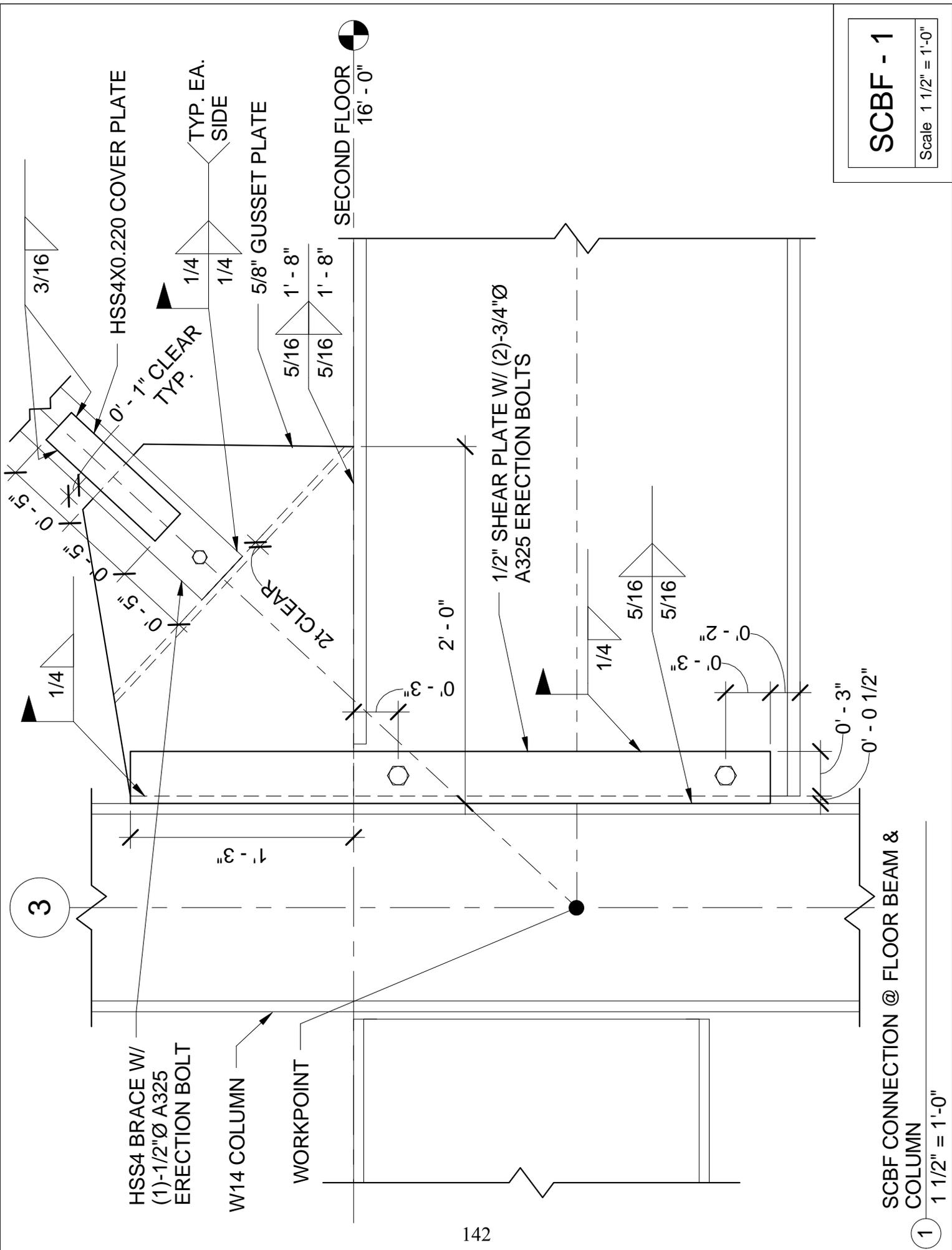
C_d = 5 Tbl 12.2-1

Importance Factor, I_e = 1.0 Tbl 1.5-2

Design Story Drift, δ_x = 0.690 in [$\delta_x = C_d \delta_{xe} / I_e$] Eqn 12.8-15

δ_x = 0.690 in < Δ_a = 3.8 in OK

Appendix F - SCBF Connection Designs



3

HSS4 BRACE W/
(1)-1/2"Ø A325
ERECTION BOLT

W14 COLUMN

WORKPOINT

HSS4X0.220 COVER PLATE

0'-1" CLEAR
TYP.

TYP. EA.
SIDE

5/8" GUSSET PLATE

1/4
1/4

5/16 1'-8"
5/16 1'-8"

SECOND FLOOR
16'-0"

2" CLEAR

2'-0"

1/2" SHEAR PLATE W/ (2)-3/4"Ø
A325 ERECTION BOLTS

1/4

5/16
5/16

0'-2"
0'-3"

0'-0 1/2"

SCBF CONNECTION @ FLOOR BEAM &
COLUMN

1 1 1/2" = 1'-0"

SCBF - 1

Scale 1 1/2" = 1'-0"

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Gusset Plate Welded Design Design (SCBF) - At Floor Beam and Column

Gusset Plate Properties

AISC Manual 14th ed.

Gusset Plate, $F_{yg} =$	36	ksi	Tbl 2-4
Gusset Plate, $F_{ug} =$	58	ksi	Tbl 2-4
Gusset Length Along Beam, $l_b =$	24	in	[Measured from face of Col. to edge of gusset]
Gusset Length Along Column, $l_c =$	15	in	[Measured from face of Beam to edge of gusset]

Brace Properties

Ref. Brace Calculations, AISC Manual 14th ed.

$F_{y-HSS} =$	42	ksi	Tbl 2-4
$F_{u-HSS} =$	50	ksi	Tbl 2-4
$t_{HSS} =$	0.205	in	Tbl 1-13
$A_{g-HSS} =$	2.44	in ²	Tbl 1-13
$D_{HSS} =$	4	in	Tbl 1-13
$r_{HSS} =$	1.34	in	

Beam Properties

Ref. Beam Calculations, AISC Manual 14th ed.

$F_{y-WF} =$	50	ksi	Tbl 2-4
$F_{u-WF} =$	65	ksi	Tbl 2-4
$d_{Beam} =$	30.2	in	Tbl 1-1
$t_{w-beam} =$	0.59	in	Tbl 1-1
$t_{f-beam} =$	0.93		
$k_{Beam} =$	1.58	in	Tbl 1-1
$T_{Beam} =$	27.04	in	Tbl 1-1

$F_{y-WF} =$	50	ksi		Tbl 2-4
$F_{u-WF} =$	65	ksi		Tbl 2-4
$d_{col} =$	14	in		Tbl 1-1
$k_{Col} =$	1.31	in		Tbl 1-1
$t_{w-Col} =$	0.42	in		Tbl 1-1
$t_{f-Col} =$	0.72	in		Tbl 1-1
$E =$	29000	ksi		

Weld Properties

AISC Manual 14th ed.

Weld $F_{EXX} =$	70	ksi		
Weld Unit Strength, $\Phi r_n =$	1.392	k/in * D	$[\Phi * (1/\sqrt{2}) * (1/16) * (0.6F_{EXX})]$	Eqn J2-3

Loads

Max Tension Brace Force, $T_u =$	164.0	k		Ref. Lower Brace
Buckled Brace Force, $P_{ub} =$	7.1	k		Ref. Lower Brace
Max Compr. Brace Force, $P_{um} =$	29.6	k	$[1.1 * \text{Expected Comp. Strength}]$	Ref. Lower Brace
Max Brace Force =	164.0	k		
Vert. Brace Length =	16.13	ft		[Distance Between Workpoints]
Horiz. Brace Length =	15.00	ft		
Brace Angle =	47.07	Degrees		$[\arctan(\text{Vert./Horiz.})]$ [Beam to Brace]
Shear at Beam Flange, $V_{ug} =$	131.81	k		$[(T_u + P_{um}) * \cos(\text{Brace Angle})]$
Tension at Bm Flange, $T_{ug-Max} =$	106.86	k		$[(T_u - P_{ub}) * \cos(\text{Brace Angle})]$
Tension at Bm Flange, $T_{ug-Min} =$	91.55	k		$[(T_u - P_{um}) * \cos(\text{Brace Angle})]$
Load Angle, $\Theta =$	39.03			$[\arctan(T_{ug}/V_{ug})]$
Moment: Gusset to Beam, $M_{ug} =$	1990.4	k-in		$[V_{ug}(d_{beam}/2)]$
Beam Reaction, $R_{u-Beam} =$	95.65	k		(Gravity Loads)

Thickness of Thinner Part= 0.21 in [Brace or Gusset thickness]
 Min. Weld Size, D= 2 /16" Tbl J2.4

Weld Size, D= 3 /16"

Number of Welds= 4

Weld Length, l_w = 10 in > 4 in

Weld Length \geq D of Brace \therefore Weld Meets Requirements of Tbl D3.1 for Shear Lag Factor

Weld Strength, ΦR_n = 167.0 k [$\Phi R_n * D * l_w * \#$ of Welds]

ΦR_n = 167.0 k > 163.968 k OK

HSS Wall Shear Rupture

AISC 360-10

A_{nv} = 8.2 in² [$4 * l_w t_{HSS}$]

Shear Rupture Strength, ΦR_n = 184.5 k [$\Phi * 0.6 F_{u-HSS} * A_{nv}$] [$\Phi=0.75$] Eqn J4-4

ΦR_n = 184.5 k > 163.968 k OK

Gusset Shear Rupture at Brace

AISC 360-10

Gusset Thickness, t_g = 0.625 in

A_{nv} = 12.5 in² [$2 * l_w t_{HSS}$]

Shear Rupture Strength, ΦR_n = 281.3 k [$\Phi * 0.6 F_{u-HSS} * A_{nv}$] [$\Phi=0.75$] Eqn J4-4

ΦR_n = 281.3 k > 163.968 k OK

$$A_e \geq A_g \quad \text{AISC 347-10: F2.5b(3)}$$

$$A_{e-Req'd} \geq 2.44 \text{ in}^2$$

HSS Slot at Gusset Plate Gap= 0.0625 in (Each Side)

Net Area, $A_n = 2.13 \text{ in}^2$ $[A_n = A_g - \text{Area of 2 Slots}]$ D3.2

$l_w = 10.00 \text{ in}$ $> 5.2 \text{ in } [1.3D]$ Tbl D3.1

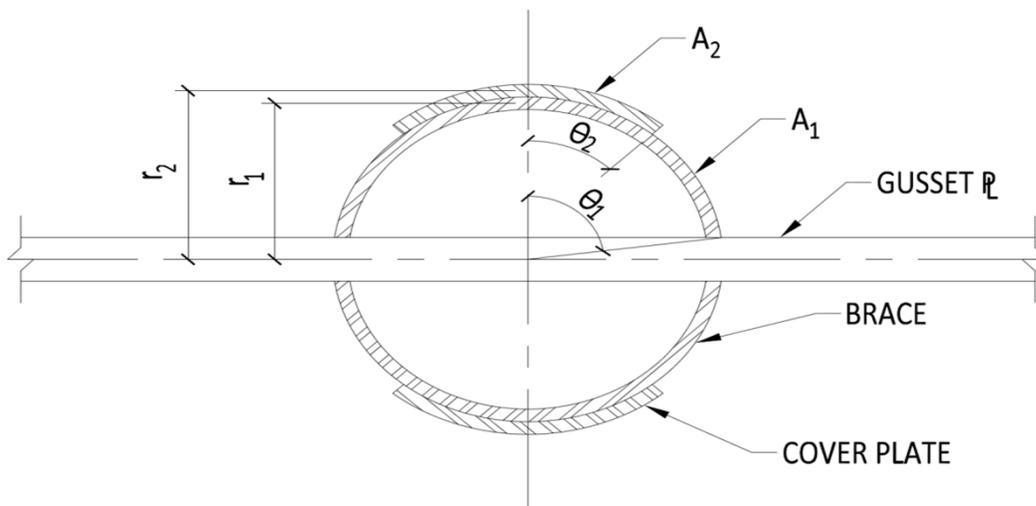
$x = 0.00 \text{ in}$ Tbl D3.1

$U = 1.000$ For $l_w > 1.3D$ Tbl D3.1

Effective Area of HSS, $A_e = 2.13 \text{ in}^2$ $[A_e = U * A_n]$ Eqn D3-1

Req'd Reinforcement, $A_{ecp} = 0.31 \text{ in}^2$

Reinforcement from: HSS4X0.220 (Using quarter sections)



$A_1 = 1.07 \text{ in}^2$ $[A_n/2]$

$A_2 = 0.61 \text{ in}^2$ $[A_{g-Cover Plate}/4]$

Radius of Brace, $r_1 = 1.90 \text{ in}$ $[(D/2) - (t/2)]$

Radius of Reinf, $r_2 = 2.10 \text{ in}$ $[(D/2) + (t/2)]$

$\theta_1 = 1.37 \text{ rad.}$ $[\text{radians}(A_n/A_g * 90)]$ (1/4 of brace net area)

$\theta_2 = 0.785 \text{ rad.}$ $[\text{radians}(45^\circ)]$

Partial Circle C.O.G., $x_{\text{Brace}} = 1.36$ in $[r_1(\sin(\theta_1)/\theta_1)]$

Center of Gravity, $x_{\text{Cover Plate}} = 1.89$ in $[r_2(\sin(\theta_2)/\theta_2)]$

$\Sigma A = 1.68$ in²

$\Sigma xA = 2.60$ in³

$x_{\text{Composite}} = 1.24$ in $[(\Sigma xA/\Sigma A) - (t_g/2)]$

$U = 0.876$ $[1 - (x_{\text{Comp.}}/l_w)]$

$A_e = 2.94$ in² $[2 * U * \Sigma A]$

$A_e = 2.94$ in² > $A_g = 2.44$ in² **OK**

$R_t = 1.6$

$R_t = 1.2$

Ref. Lower Brace Design

Ref. Lower Brace Design

Expected Rupture Strength of Cover Plate

$\Phi P_n = 27.45$ k $[P_n = R_t A_2 F_u]$ $[\Phi = 0.75]$ Eqn D2-2

Expected Yield Strength

$\Phi P_n = 36.89$ k $[P_n = R_y A_2 F_y]$ $[\Phi = 0.90]$ Eqn D2-2

$\Phi P_n = 36.89$ k $[\text{Max of Expected Strengths}]$

Cover Plate to Brace Weld

AISC 360-10

Thickness of Thinner Part = **0.21** in $[t_{\text{HSS}}]$

Min. Weld Size, $D = 2$ /16" Tbl J2.4

Weld Size, $D = 3$ /16"

Number of Welds = **2**

Weld Length, $l_w = 5$ in

Weld Strength, $\Phi R_n = 41.8$ k $[\Phi r_n * D * l_w * \# \text{ of Welds}]$

Expected Strength, $\Phi P_n = 36.9$ k $[\text{Cover Plate Expected Strength}]$

$\Phi R_n = 41.8$ k > 36.9 k **OK**

Gusset Shear Yielding at Brace

AISC 360-10

Shear Yield Strength, $\Phi R_n = 419.8 \text{ k}$ $[2*\Phi*0.6F_yA_g]$ $[\Phi=1.0]$ Eqn J4-3

$\Phi R_n = 419.8 \text{ k}$ $>$ 163.97 k OK

Whitmore Section

AISC 360-10

$l_w = 15.55 \text{ in}$ $[2*l_w \tan(30^\circ) + D_{HSS}]$

$A_{gw} = 9.72 \text{ in}^2$ (Gross Area of Whitmore Section)

Gusset Tensile Yielding at Brace

AISC 360-10

Tension Yield, $\Phi R_n = 349.81 \text{ k}$ $[\Phi R_n = \Phi[F_{yg} * L_w * t_g]]$ $[\Phi=0.90]$ Eqn J4-1

$\Phi R_n = 349.81 \text{ k}$ $>$ 163.968 k OK

Gusset Tensile Rupture at Brace

AISC 360-10

Whitmore Area, $A_{effective} = A_g = 9.72 \text{ in}^2$

Tensile Rupture Strength, $\Phi P_n = 422.68 \text{ k}$ $[\Phi F_u A_e]$ $[\Phi=0.75]$ Eqn J4-1

$\Phi P_n = 422.68 \text{ k}$ $>$ 163.97 k OK

Whitmore Area, $A_g = 9.72 \text{ in}^2$

$K = 1.2$

Tbl C-A-7.1

$L = 12.05 \text{ in}$

$r = 0.180 \text{ in}$

$[t/\sqrt{12}]$

$KL/r = 80.2 > 25$

Use Chapter E

$4.71\sqrt{E/F_y} = 133.68$

$KL/r = 80.2 < 133.68$

Use Eqn E3-2

$F_e = 44.54 \text{ ksi}$

$[F_e = \pi^2 E / (KL/r)^2]$

Eqn E3-4

$F_{cr} = 25.67 \text{ ksi}$

$[F_{cr} = (.658^{F_y/F_e}) * F_y]$

Eqn 3-2

$\Phi = 0.9$

E1(b)

$P_n = 249.4 \text{ k}$

$[P_n = F_{cr} A_g]$

Eqn E3-1

Compression Strength, $\Phi P_n = 224.5 \text{ k}$

$> 29.56 \text{ k}$

OK

Gusset Plate Dimensions

Thickness, $t_g = 0.625 \text{ in}$

Length, $L_g = 24.0 \text{ in}$

(Along beam)

Width, $W_g = 15.0 \text{ in}$

(Along Column)

Gusset to Beam Weld Setback= 4 in (Distance from column flange to start of weld)

Angle from Column to Brace, Θ = 42.93 degrees

Beam Eccentricity, e_b = 15.100 in [$e_b=d_{beam}/2$]

Beam Eccentricity, e_c = 7.000 in [$e_c=d_{column}/2$]

α = 14.00 in [$(l_b-Clip)/2+Clip$]

β = 7.50 in [$(l_c-Clip)/2+Clip$]

$e_b \tan \Theta - e_c$ = 7.047 in

$\alpha - \beta \tan \Theta$ 7.023 in

Net Eccentricity= 0.023 in (Negligible)

r = 30.9 in [$\text{Sqrt}((\alpha+e_c)^2+(\beta+e_b)^2)$]

Brace in Tension

V_{uc} = 39.86 k [$(\beta/r)*T_u$]

H_{uc} = 37.20 k [$(e_c/r)*T_u$]

V_{ub} = 80.26 k [$(\beta/r)*T_u$]

H_{ub} = 74.41 k [$(e_c/r)*T_u$]

Brace in Compression

V_{uc} = 7.19 k [$(\beta/r)*P_{um}$]

H_{uc} = 6.71 k [$(e_c/r)*P_{um}$]

V_{ub} = 14.47 k [$(\beta/r)*P_{um}$]

H_{ub} = 13.41 k [$(e_c/r)*P_{um}$]

Load Angle=	47.2	degrees	$[\arctan(V_{ub}/H_{ub})]$	
Thickness of Thinner Part=	0.59	in	[Beam Web or Gusset thickness]	
Min. Weld Size, D=	4	/16"		Tbl J2.4
Weld Size, D=	5	/16"	(One weld each side)	
Weld Length, l=	20.00	in	$[l_b - \text{Weld Setback}]$	
Eccentricity, e_x =	15.1	in	$[d_{beam}/2]$	
a=	0.755		$[e_x/l]$	(Round up for "a" in table)
"a" value used in table=	0.8			
C=	2			Tbl 8-4
Φ =	0.75			
Strength of Weld Group, ΦR_n =	150.0	k		
Eccentric Force, R_{ub} =	136.8	k	$[1.25\sqrt{(V_{ub}^2 + H_{ub}^2)}]$	
ΦR_n =	150.0	k	>	136.8 k
				Weld is Adequate

Gusset Plate Rupture at Beam Weld

AISC 360-10: Section J

Set gusset plate shear rupture strength equal to weld strength

t_{g-min} =	0.533	in	$[(2*\Phi R_n * D_{req}) / (.75 * .6F_{ug})]$	Ref. Eqn J4-4
t=	0.625	in	>	0.533 in
				OK

Because the gusset plate is sufficient for shear rupture based on weld size, the plate is also sufficient for tension and shear yielding.

Beam Web Local Yielding

AISC 350-10

Force Location, α =	14.00	in	$\leq d$ =	30.2	in	Use Eqn J-10-3
Beam Force Applied at α , ΦR_n =	700.54	k	>	V_{ub} =	80.26	k
	$[\Phi R_n = \Phi(2.5k_{Beam} + l_{wb})F_y - W_{Ftw-beam}]$			$[\Phi = 1.00]$		OK
						Eqn J10-3

Plate, F_{y-pl} =	36	ksi	Tbl 2-4
Plate, F_{u-pl} =	58	ksi	Tbl 2-4
Plate Thickness, t_{pl} =	0.5	in	
Plate Height, h_{pl} =	43.75	in	
Plate Width, w_{pl} =	3.5	in	

Gusset to Shear Plate Weld

AISC 360-10

Load Angle=	47.0	degrees	$[\arctan(V_{uc}/H_{uc})]$ (Round down in table)	
Thickness of Thinner Part=	0.50	in	[Shear Plate or Gusset thickness]	
Min. Weld Size, D=	3	/16"		Tbl J2.4
Weld Size, D=	3	/16"	(One weld each side)	
Weld Length, l=	15.00	in	$[l_c - \text{Clip}]$	
k l=	3	in	(Distance between fillet welds) (Edge of shear plate to edge of gusset)	
k=	0.2		$[kl/l]$ (Round down in table)	
Eccentricity, e_x =	2	in	[Col. Flange to Center of weld group]	
a=	0.133		$[e_x/l]$ (Round up for "a" in table)	
a value used in table=	0.2			
k value used in table=	0.1			
C=	3.92			Tbl 8-4
Φ =	0.75			
Strength of Weld Group, ΦR_n =	132.3	k		
Eccentric Force, R_{ub} =	68.2	k	$[1.25\sqrt{V_{uc}^2 + H_{uc}^2}]$	
ΦR_n =	132.3	k	>	68.2 k
Weld is Adequate				

Load Angle=	47.0 degrees	$[\arctan(V_{uc}/H_{uc})]$	
Thickness of Thinner Part=	0.50 in	[Shear Plate or Col. Flange thickness]	
Min. Weld Size, D=	3 /16"		Tbl J2.4
Weld Size, D=	3 /16"	(One weld each side)	
Weld Length, l=	14.25 in	$[l_c\text{-Clip}]$	
Eccentricity, e_x =	7 in	$[d_{col}/2]$	
a=	0.491	$[e_x/l]$	(Round up for "a" in table)
a value used in table=	0.5		
C=	2.75		Tbl 8-4
Φ =	0.75		
Strength of Weld Group, ΦR_n =	88.2 k		
Eccentric Force, R_{ub} =	68.2 k	$[1.25\sqrt{(V_{uc}^2 + H_{uc}^2)}]$	
ΦR_n =	88.2 k	>	68.2 k Weld is Adequate

Gusset Plate Rupture at Column Weld

AISC 360-10: Section J

Set gusset plate shear rupture strength equal to weld strength

$t_{g\text{-min}}$ =	0.320 in	$[(2*\Phi R_n * D_{req}) / (.75 * .6F_{ug})]$	Ref. Eqn J4-4
t=	0.625 in	>	0.320 in OK

Because the gusset plate is sufficient for shear rupture based on weld size, the plate is also sufficient for tension and shear yielding.

Column Web Local Yielding

AISC 360-10

Force applied to column is located at a distance greater than the depth of column from column end

ΦR_n =	431.6 k	>	H_{uc} =	37.20 k	OK
$[\Phi R_n = \Phi(5k_{col} + l_{wc})F_{y-WF}t_{w-col}]$	$[\Phi=1.00]$				Eqn J10-2

Shear, $V_u = 175.91$ k [$V_{ub} + V_u$ ref. beam design]

Axial, $H_u = 198.33$ k [$H_{ub} + P_u$ ref. beam design]

Beam to Shear Plate Weld

Load Angle = 48.4 degrees [$\arctan(V_{uc}/H_{uc})$ (Round down in table)]

Thickness of Thinner Part = 0.50 in [Shear Plate or Gusset thickness]
 Min. Weld Size, D = 3 /16" Tbl J2.4

Weld Size, D = 4 /16" (One weld each side)

Weld Length, $l_w = 28.2$ in [$d_{beam} - k_{beam}$] Tbl 1-1

$kl = 3$ in (Distance between fillet welds)
 (Edge of shear plate to edge of gusset)

$k = 0.106$ [kl/l] (Round down in table)

Eccentricity, $e_x = 2$ in [Col. Flange to Center of weld group]

$a = 0.071$ [e_x/l] (Round up for "a" in table)

a value used in table = 0.1
 k value used in table = 0.1
 $C = 4.49$ Tbl 8-4

$\Phi = 0.75$

Strength of Weld Group, $\Phi R_n = 379.9$ k

Eccentric Force, $R_{ub} = 331.4$ k [$1.25\sqrt{V_{uc}^2 + H_{uc}^2}$]

$\Phi R_n = 379.9$ k > 331.4 k Weld is Adequate

Load Angle=	48.4	degrees	$[\arctan(V_{uc}/H_{uc})]$	
Thickness of Thinner Part=	0.50	in	[Shear Plate or Column Flange thickness]	
Min. Weld Size, D=	3	/16"		Tbl J2.4
Weld Size, D=	5	/16"	(One weld each side)	
Weld Length, l_w =	28.2	in	$[d_{beam}-k_{beam}]$	Tbl 1-1
Eccentricity, e_x =	7	in	$[d_{col}/2]$	
a =	0.248		$[e_x/l]$	(Round up for "a" in table)
a value used in table=	0.3			
C=	3.48			Tbl 8-4
Φ =	0.75			
Strength of Weld Group, ΦR_n =	368.0	k		
Eccentric Force, R_{ub} =	331.4	k	$[1.25\sqrt{(V_{uc}^2+H_{uc}^2)}]$	
ΦR_n =	368.0	k	>	331.4 k
				Weld is Adequate

Beam Web Shear Strength

AISC 360-10

ΦV_n =	474.55	k	$[\Phi \cdot 6F_y t_w T]$	Eqn J4-3
ΦV_n =	474.55	k	>	175.91 k
				Web is Adequate

Location of H_{ub} , $\beta = 7.5$ in $<$ $d/2 = 15.1$ in

$N/d = 0.66 > 0.2$

Use: Eqn J10-5b

$\Phi R_n = 727.2$ k Eqn J10-5b

$\Phi R_n = 727.2$ k $>$ 13.4 k OK

Column Web Crippling

AISC 360-10: J10.3

Force applied to column is located at a distance greater than the depth of column from column end

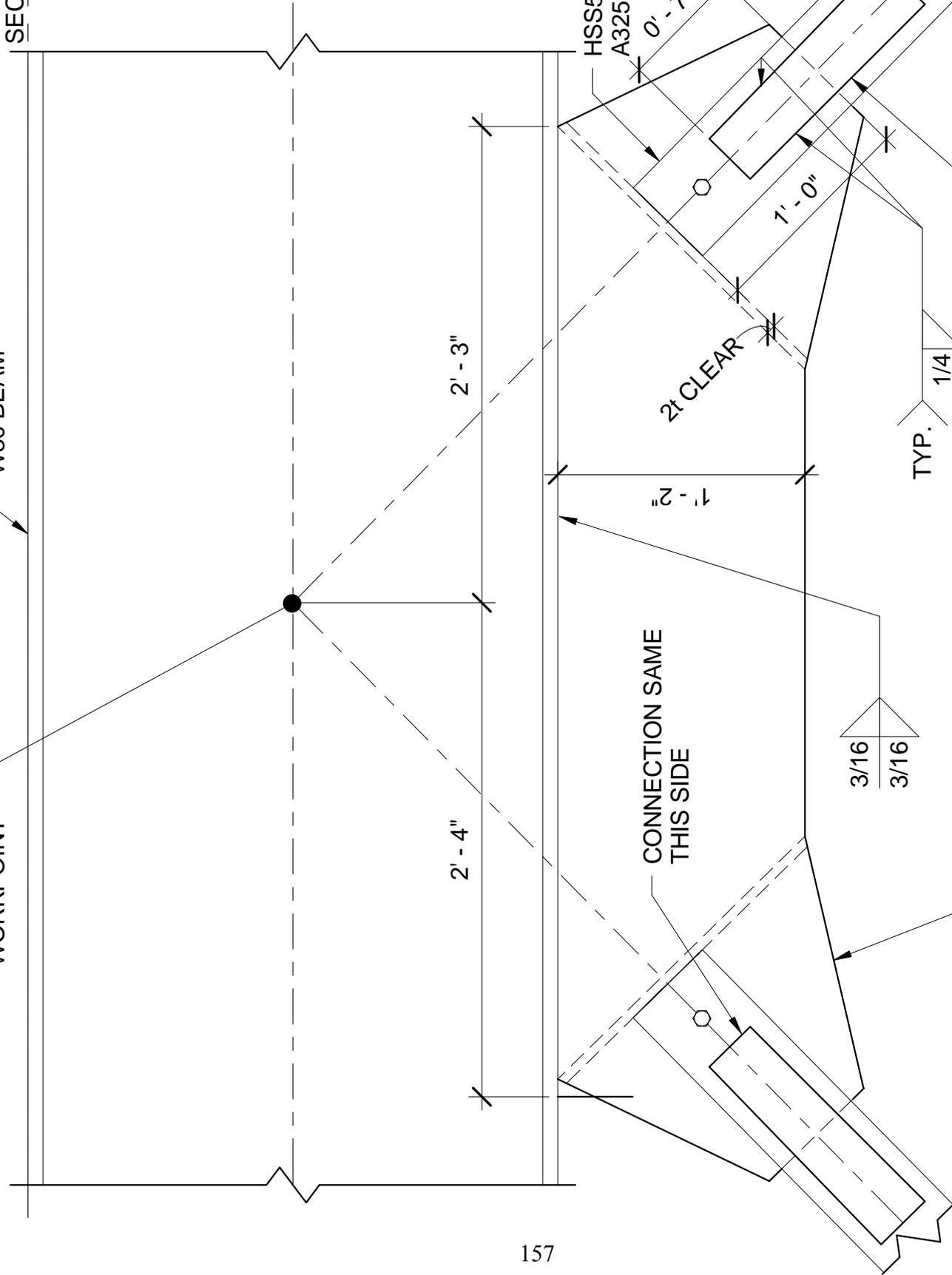
$R_n = 510.5$ k $[0.8t_w^2\{1+3(N/d)(t_w/t_f)1.5\}\text{Sqrt}(EF_ywt_f/t_w)]$ Eqn J10-4

$\Phi = 0.75$

$\Phi R_n = 382.9$ k $>$ 6.71 k OK

W30 BEAM

WORKPOINT



HSS5.5 W/ (1)-1/2"Ø
A325 ERECTION BOLT

CONNECTION SAME
THIS SIDE

HSS5.5 COVER PLATE

1/2" GUSSET PLATE

SCBF - 2

Scale 1 1/2" = 1'-0"

1 SCBF CONNECTION @ FLOOR BEAM

1 1/2" = 1'-0"

Gusset Plate (SCBF) at Floor Beam - 2 Braces

Plate Material Properties

$$\begin{aligned} F_y &= 36 \text{ ksi} \\ F_u &= 58 \text{ ksi} \\ E &= 29000 \text{ ksi} \end{aligned}$$

Brace Properties

$$\begin{aligned} F_{y-HSS} &= 42 \text{ ksi} \\ F_{u-HSS} &= 58 \text{ ksi} \\ A_{g-HSS} &= 3.97 \text{ in}^2 \\ t_{HSS} &= 0.24 \text{ in} \\ D &= 5.5 \text{ in} \end{aligned}$$

Beam Material

$$\begin{aligned} F_{y-WF} &= 50 \text{ ksi} \\ F_{u-WF} &= 65 \text{ ksi} \\ d_{\text{Beam}} &= 30.2 \text{ in} \\ t_{w-\text{Beam}} &= 0.585 \text{ in} \\ t_{f-\text{Beam}} &= 0.93 \text{ in} \\ k_{\text{Beam}} &= 1.58 \text{ in} \end{aligned}$$

Weld Properties

$$\begin{aligned} \text{Weld } F_{EXX} &= 70 \text{ ksi} \\ \text{Weld Unit Strength, } \Phi_r &= 1.392 \text{ k/in * D} \end{aligned} \quad [\Phi * (1/\sqrt{2}) * (1/16) * (0.6F_{EXX})] \quad \text{Eqn J2-3}$$

Loads

Max Tension Force, T_u =	266.8	k		Ref. Lower Brace
Buckled Brace Force, P_{ub} =	21.8	k		Ref. Lower Brace
Max Compression Force, P_{um} =	88.9	k	[1.1*Expected Comp. Strength]	Ref. Lower Brace
Max Brace Force=	266.78	k		
Vert. Brace Length=	14.75	ft	[Distance Between Workpoints]	
Horiz. Brace Length=	15	ft		
Brace Angle=	44.52	Degrees	[arctan(Vert./Horiz.)]	
Shear Force at Beam Flange, V_{ug} =	253.61	k	$[(T_u+P_{um})*\text{Cos}(\text{Brace Angle})]$	
Tension Force at Beam Flange, T_{ug-Max} =	174.64	k	$[(T_u-P_{ub})*\text{Cos}(\text{Brace Angle})]$	
Tension Force at Beam Flange, T_{ug-Min} =	126.84	k	$[(T_u-P_{um})*\text{Cos}(\text{Brace Angle})]$	
Load Angle, Θ =	34.55		[arctan(T_{ug}/V_{ug})]	
Moment: Gusset to Beam, M_{ug} =	3829.5	k-in	$[V_{ug}(d_{beam}/2)]$	

Brace-to-Gusset Weld

AISC 360-10

Thickness of Thinner Part=	0.24	in	[Brace or Gusset thickness]	
Min. Weld Size, D =	2	/16"		Tbl J2.4
Weld Size, D =	4	/16"		
Number of Welds=	4			
Weld Length, l_w =	12	in	> 5.5 in	
Weld Length $\geq D$ of Brace \therefore Weld Meets Requirements of Tbl D3.1 for Shear Lag Factor				
Weld Strength, ΦR_n =	267.3	k	$[\Phi r_n * D * l_w * \# \text{ of Welds}]$	
ΦR_n =	267.3	k	> 266.784 k	OK

HSS Wall Shear Rupture

AISC 360-10

A_{nv} =	11.52	in ²	$[4 * l_w t_{HSS}]$	
Shear Rupture Strength, ΦR_n =	300.7	k	$[\Phi * 0.6 F_{u-HSS} * A_{nv}]$	Eqn J4-4
ΦR_n =	300.7	k	> 266.784 k	OK

Gusset Thickness, $t_g = 0.625$ in

$$A_{nv} = 15 \text{ in}^2 \quad [2 * l_w t_{HSS}]$$

Shear Rupture Strength, $\Phi R_n = 391.5$ k Eqn J4-4

$$\Phi R_n = 391.5 \text{ k} > 266.784 \text{ k} \quad \text{OK}$$

Shear Lag Rupture of Brace

$$A_e \geq A_g \quad \text{AISC 347-10: F2.5b(3)}$$

$$A_{e-Req'd} \geq 3.97 \text{ in}^2$$

HSS Slot at Gusset Plate Gap = 0.0625 in (Each Side)

$$\text{Net Area, } A_n = 3.61 \text{ in}^2 \quad [A_n = A_g - \text{Area of 2 Slots}] \quad \text{D3.2}$$

$$l_w = 12.00 \text{ in} < 7.15 \text{ in } [1.3D] \quad \text{Tbl D3.1}$$

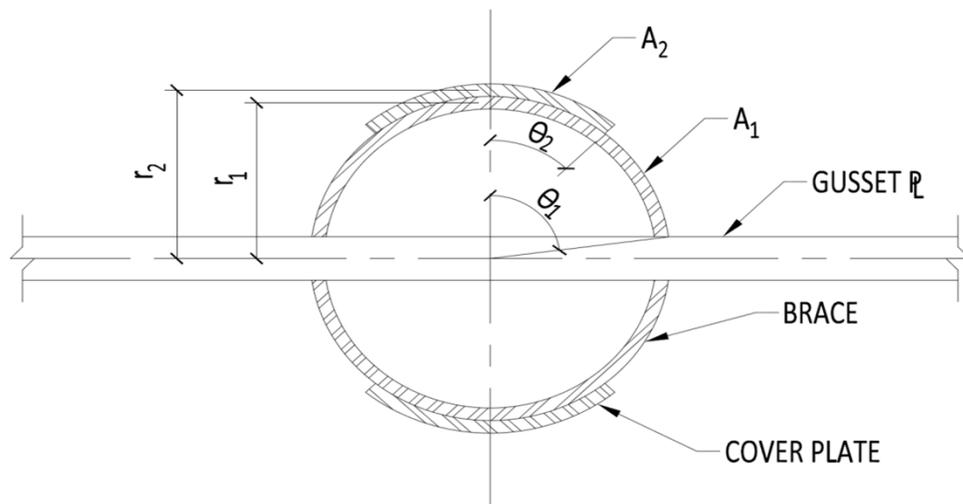
$$x = 0.00 \text{ in} \quad \text{Tbl D3.1}$$

$$U = 1.000 \quad \text{For } l_w > 1.3D \quad \text{Tbl D3.1}$$

$$\text{Effective Area of HSS, } A_e = 3.61 \text{ in}^2 \quad [A_e = U * A_n] \quad \text{Eqn D3-1}$$

$$\text{Req'd Reinforcement, } A_{ecp} = 0.36 \text{ in}^2$$

Reinforcement from: **HSS5.500X0.258** (Using 2 quarter sections)



$$A_1 = 1.81 \text{ in}^2 \quad [A_n/2]$$

$$A_2 = 0.99 \text{ in}^2 \quad [A_{g-Cover Plate}/4]$$

Radius of Brace, r_1 =	2.63	in		$[(D/2)-(t/2)]$	
Radius of Reinf, r_2 =	2.87	in		$[(D/2)+(t/2)]$	
Θ_1 =	1.43	rad.		$[\text{radians}(A_n/A_g*90)]$	(1/4 of brace net area)
Θ_2 =	0.785	rad.			
Partial Circle C.O.G., x_{Brace} =	1.82	in		$[r_1(\sin(\Theta_1)/\Theta_1)]$	
Center of Gravity, $x_{\text{Cover Plate}}$ =	2.58	in		$[r_2(\sin(\Theta_2)/\Theta_2)]$	
ΣA =	2.80	in ²			
ΣxA =	5.85	in ³			
$x_{\text{Composite}}$ =	1.78	in		$[(\Sigma xA/\Sigma A)-(t_g/2)]$	
U =	0.852			$[1-(x_{\text{Comp.}}/l_w)]$	
A_e =	4.76	in ²		$[2*U*\Sigma A]$	
A_e =	4.76	in ²	>	A_g =	3.97 in ² OK
R_y =	1.6				Ref. Lower Brace Design
R_t =	1.2				

Expected Rupture Strength of Cover Plate

ΦP_n =	69.08	k	$[P_n=R_t A_2 F_u]$	$[\Phi=0.75]$	Eqn D2-2
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Expected Yield Strength

ΦP_n =	60.03	k	$[P_n=R_y A_2 F_y]$	$[\Phi=0.90]$	Eqn D2-2
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ΦP_n =	69.08	k	$[\text{Max of Expected Strengths}]$		
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Thickness of Thinner Part=	0.24 in	[Brace thickness]	
Min. Weld Size, D=	2 /16"		Tbl J2.4
Weld Size, D=	4 /16"		
Number of Welds=	2		
Weld Length, l_w =	7 in		
Weld Strength, ΦR_n =	78.0 k	$[\Phi R_n * D * l_w * \# \text{ of Welds}]$	
Expected Strength, ΦP_n =	69.1 k	$[\Phi R_t F_u A_n] [\Phi=0.75]$	
ΦR_n =	78.0 k	>	69.1 k OK

Gusset Shear Yielding at Brace

AISC 360-10

Shear Yield Strength, ΦR_n =	522.62 k	$[2 * \Phi * 0.6 F_y A_g] [\Phi=1.0]$	Eqn J4-3
ΦR_n =	522.62 k	>	266.78 k OK

Whitmore Section

AISC 360-10

Whitmore Width, L_w =	19.36 in	$[2 * l_w \tan(30^\circ) + D_{HSS}]$
A_g =	12.10 in ²	$[L_w * t_g]$

Gusset Tensile Yielding at Brace

AISC 360-10

Whitmore Area, A_g =	12.10 in ²		
Tensile Yield Strength, ΦR_n =	391.97 in ²	$[\Phi F_y A_g] [\Phi=0.9]$	Eqn J4-1
ΦP_n =	391.97 k	>	266.78 k OK

$$\text{Whitmore Area, } A_{\text{effective}}=A_g= 12.10 \text{ in}^2$$

$$\text{Tensile Rupture Strength, } \Phi P_n= 526.25 \text{ k} \quad [\Phi F_u A_e] \quad [\Phi=0.75] \quad \text{Eqn J4-1}$$

$$\Phi P_n= 526.25 \text{ k} > 266.78 \text{ k} \quad \text{OK}$$

Compression in Gusset

AISC 360-10

$$\text{Whitmore Area, } A_g= 12.10 \text{ in}^2$$

$$K= 1.2 \quad \text{Tbl C-A-7.1}$$

$$L= 4.27 \text{ in}$$

$$r= 0.180 \text{ in}$$

$$[t/\sqrt{12}]$$

$$KL/r= 28.4 > 25 \quad \text{Use Chapter E}$$

$$4.71\sqrt{E/F_y}= 133.68$$

$$KL/r= 28.4 < 133.68 \quad \text{Use Eqn E3-2}$$

$$F_e= 354.29 \text{ ksi} \quad [F_e=\pi^2 E/(KL/r)^2] \quad \text{Eqn E3-4}$$

$$F_{cr}= 34.50 \text{ ksi} \quad [F_{cr}=(.658^{F_y/F_e}) * F_y] \quad \text{Eqn 3-2}$$

$$\Phi= 0.9 \quad \text{E1(b)}$$

$$P_n= 417.4 \text{ k} \quad [P_n=F_{cr} A_g] \quad \text{Eqn E3-1}$$

$$\text{Compression Strength, } \Phi P_n= 375.6 \text{ k} > 88.90 \text{ k} \quad \text{OK}$$

Gusset Plate Dimensions

AISC 360-10

$$\text{Thickness, } t_g= 0.625 \text{ in}$$

$$\text{Length, } L_g= 54 \text{ in}$$

(Along beam)

$$\text{Width, } W_g= 13 \text{ in}$$

(Perpendicular to beam)

Load Angle=	34.55	(Round angle down for tables)	
Thickness of Thinner Part=	0.59 in	[Beam or Gusset thickness]	
Min. Weld Size, D=	4 /16"		Tbl J2.4
Weld Size, D=	3 /16"	(One weld each side)	
Weld Length, l=	54 in	(Length of Gusset)	
Eccentricity, e_x =	15.1 in	$[d_{beam}/2]$	
a=	0.280	$[e_x/l]$ (Round up for "a" in table)	
a value used in table=	0.3		
C=	3.21		Tbl 8-4
Φ =	0.75		
Strength of Weld Group, ΦR_n =	390.0 k		
Eccentric Force, R_{ub} =	384.9 k	$[1.25\sqrt{(V_u^2+T^2)}]$	
ΦR_n =	390.0 k	> 384.9 k	Weld is Adequate

Horizontal Shear in Gusset at Beam

AISC 360-10

Gusset Length, l_g =	54 in		
A_g =	33.75 in ²	$[l_b t_g]$	
Shear Rupture Strength, ΦR_n =	880.9 k	$[\Phi * 0.6 F_{u-HSS} * A_{nv}]$ [$\Phi=0.75$]	Eqn J4-4
Shear Yield Strength, ΦR_n =	729.0 k	$[\Phi * 0.6 F_y A_g]$ [$\Phi=1.0$]	Eqn J4-3
Governing ΦR_n =	729.0 k	> 253.61 k	OK

Gusset Width, W_g =	13	in			
A_g =	8.13	in ²		$[I_b t_g]$	
Shear Rupture Strength, ΦR_n =	212.1	k		$[\Phi * 0.6 F_{u-HSS} * A_{nv}]$ $[\Phi=0.75]$	Eqn J4-4
Shear Yield Strength, ΦR_n =	175.5	k		$[\Phi * 0.6 F_y A_g]$ $[\Phi=1.0]$	Eqn J4-3
Governing ΦR_n =	175.5	k	>	174.64 k	OK

Unit Forces at Gusset-Beam Interaction

Gusset Section Modulus, S_g =	486.0	in ³ /in		$[L_g^2/6]$
Unit Shear, f_v =	4.70	k/in		$[V_{ug}/L_g]$
Max Unit Tension, f_a =	3.23	k/in		$[T_{ug-Max}/L_g]$
Min Unit Tension, f_a =	2.35	k/in		$[T_{ug-Min}/L_g]$
Contribution from Moment, f_b =	7.88	k/in		$[M_{ug}/S_g]$

Beam Web Local Yielding

Unit Force with Max Tension

Max Compressive Unit Force, f_{c-Max} =	11.11	k/in		$[f_b + f_{a-Max}]$
Max Tensile Unit Force, f_{t-Max} =	4.65	k/in		$[f_b - f_{a-Max}]$
Length of Gusset in Tension, L_t =	15.9	in		$[(f_t/(f_t+f_c)) * L_g]$
Resultant Tension Force, R_u =	37.0	k		$[.5 L_t f_t]$

Unit Force with MinTension

Max Compressive Unit Force, f_{c-Min} =	10.23	k/in		$[f_b + f_{a-Min}]$
Max Tensile Unit Force, f_{t-Min} =	5.53	k/in		$[f_b - f_{a-Min}]$
Length of Gusset in Tension, L_t =	19.0	in		$[(f_t/(f_t+f_c)) * L_g]$
Resultant Tension Force, R_u =	52.4	k		$[.5 L_t f_t]$

Web Local Yielding Strength, ΦR_n =	696.7	k	>	52.4 k	OK
				$[\Phi(5k_{Beam} + I_t)F_{y-WF}t_{w-beam}]$ $[\Phi=1.00]$	Eqn J10-2

Force with Max Tension

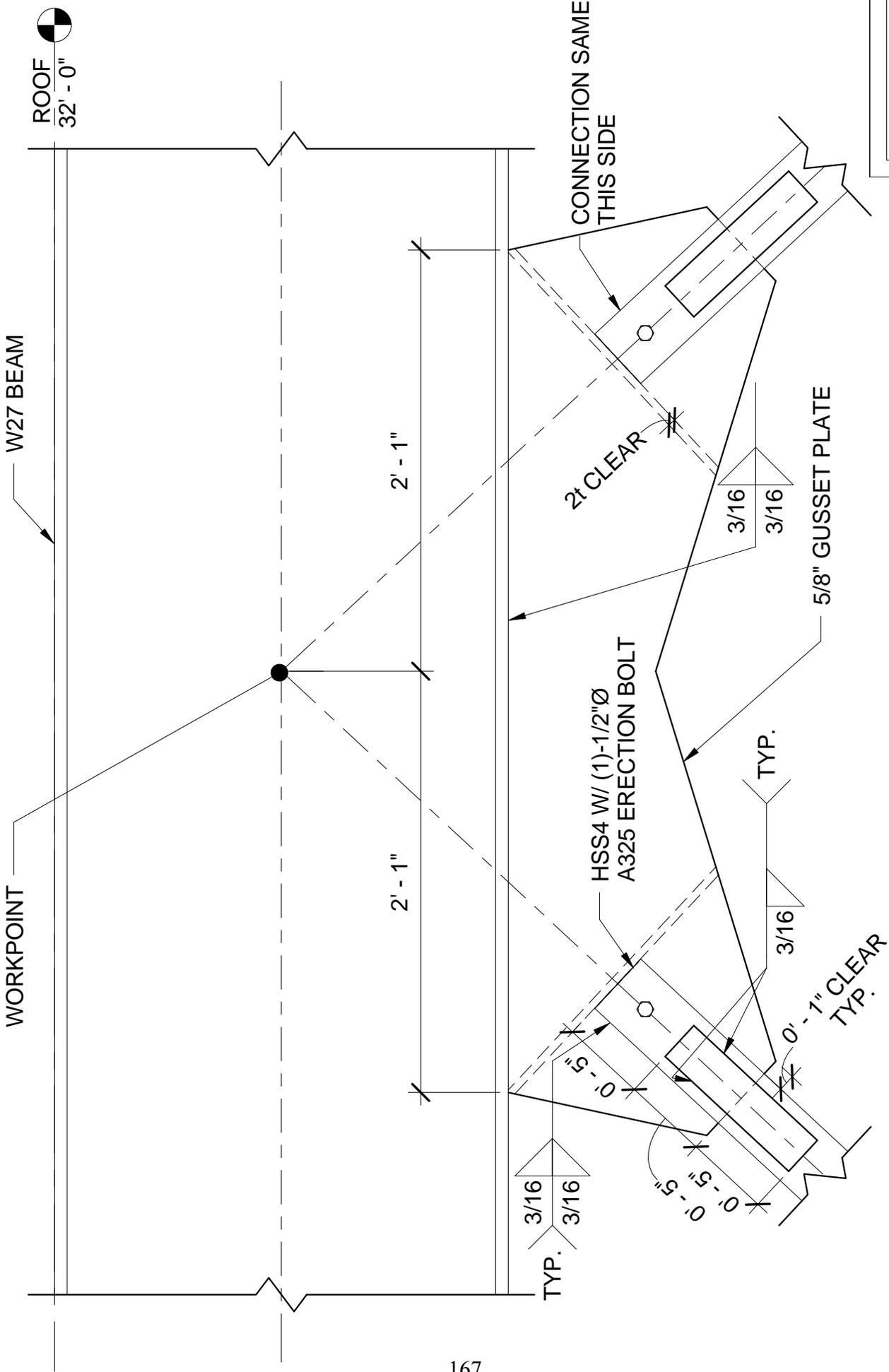
Resultant Compression Force, $R_u = 211.6 \text{ k}$ $[.5(L_g - L_t)f_c]$

Force with Min Tension

Resultant Compression Force, $R_u = 179.2 \text{ k}$ $[.5(L_g - L_t)f_c]$

Web Local Crippling Strength, $\Phi R_n = 900.1 \text{ k}$ > 211.6 k
 $[\Phi 0.8 t_w^2 \{1 + 3(N/d)(t_w/t_f) 1.5\} \sqrt{E F_{yw} t_f/t_w}] [\Phi = 0.75]$

OK



SCBF - 3

Scale 1 1/2" = 1'-0"

1 SCBF CONNECTION @ ROOF BEAM
1 1/2" = 1'-0"

Gusset Plate (SCBF) at Roof Beam - 2 Braces

Plate Material Properties

$$\begin{aligned}F_y &= 36 \text{ ksi} \\F_u &= 58 \text{ ksi} \\E &= 29000 \text{ ksi}\end{aligned}$$

Brace Properties

$$\begin{aligned}F_{y\text{-HSS}} &= 42 \text{ ksi} \\F_{u\text{-HSS}} &= 50 \text{ ksi} \\A_{g\text{-HSS}} &= 2.44 \text{ in}^2 \\t_{\text{HSS}} &= 0.205 \text{ in} \\D &= 4 \text{ in}\end{aligned}$$

Beam Material

$$\begin{aligned}F_{y\text{-WF}} &= 50 \text{ ksi} \\F_{u\text{-WF}} &= 65 \text{ ksi} \\d_{\text{Beam}} &= 26.9 \text{ in} \\t_{w\text{-Beam}} &= 0.49 \text{ in} \\t_{f\text{-Beam}} &= 0.745 \text{ in} \\k_{\text{Beam}} &= 1.34 \text{ in}\end{aligned}$$

Weld Properties

$$\begin{aligned}\text{Weld } F_{EXX} &= 70 \text{ ksi} \\ \text{Weld Unit Strength, } \Phi r_n &= 1.392 \text{ k/in * D} \quad [\Phi*(1/\sqrt{2})*(1/16)*(0.6FEXX)] \quad \text{Eqn J2-3}\end{aligned}$$

Loads

Max Tension Brace Force, T_u =	164.0	k		Ref. Upper Brace
Buckled Brace Force, P_{ub} =	7.1	k		Ref. Upper Brace
Max Compression Brace Force, P_{um} =	29.6	k	[1.1*Expected Comp. Strength]	Ref. Upper Brace
Max Brace Force=	163.97	k		
Vert. Brace Length=	16.13	ft		[Distance Between Workpoints]
Horiz. Brace Length=	15	ft		
Brace Angle=	47.07	Degrees		[arctan(Vert./Horiz.)]
Shear Force at Beam Flange, V_{ug} =	131.81	k		$[(T_u + P_{um}) * \cos(\text{Brace Angle})]$
Tension at Beam Flange, T_{ug-Max} =	106.86	k		$[(T_u - P_{ub}) * \cos(\text{Brace Angle})]$
Tension at Beam Flange, T_{ug-Min} =	91.55	k		$[(T_u - P_{um}) * \cos(\text{Brace Angle})]$
Load Angle, θ =	39.03			[arctan(T_{ug}/V_{ug})]
Moment: Gusset to Beam, M_{ug} =	1772.9	k-in		$[V_{ug}(d_{beam}/2)]$

Brace-to-Gusset Weld

AISC 360-10

Thickness of Thinner Part=	0.21	in		[Brace or Gusset thickness]	
Min. Weld Size, D =	2	/16"			Tbl J2.4
Weld Size, D =	3	/16"			
Number of Welds=	4				
Weld Length, l_w =	10	in	>	4	in
Weld Length $\geq D$ of Brace \therefore Weld Meets Requirements of Tbl D3.1 for Shear Lag Factor					
Weld Strength, ΦR_n =	167.0	k		$[\Phi R_n * D * l_w * \# \text{ of Welds}]$	
ΦR_n =	167.0	k	>	163.968	k
					OK

HSS Wall Shear Rupture

AISC 360-10

A_{nv} =	8.2	in ²		$[4 * l_w * t_{HSS}]$	
Shear Rupture Strength, ΦR_n =	184.5	k		$[\Phi * 0.6 F_{u-HSS} * A_{nv}]$	Eqn J4-4
ΦR_n =	184.5	k	>	163.968	k
					OK

Gusset Thickness, $t_g = 0.625$ in

$$A_{nv} = 12.5 \text{ in}^2 \quad [2 * l_w t_{HSS}]$$

Shear Rupture Strength, $\Phi R_n = 281.3$ k [$\Phi * 0.6 F_{u-HSS} * A_{nv}$] [$\Phi = 0.75$] Eqn J4-4

$$\Phi R_n = 281.3 \text{ k} > 163.968 \text{ k} \quad \text{OK}$$

Shear Lag Rupture of Brace

$$A_e \geq A_g \quad \text{AISC 347-10: F2.5b(3)}$$

$$A_{e-Req'd} \geq 2.44 \text{ in}^2$$

HSS Slot at Gusset Plate Gap = 0.0625 in (Each Side)

$$\text{Net Area, } A_n = 2.13 \text{ in}^2 \quad [A_n = A_g - \text{Area of 2 Slots}] \quad \text{D3.2}$$

$$l_w = 10.00 \text{ in} < 5.2 \text{ in [1.3D]} \quad \text{Tbl D3.1}$$

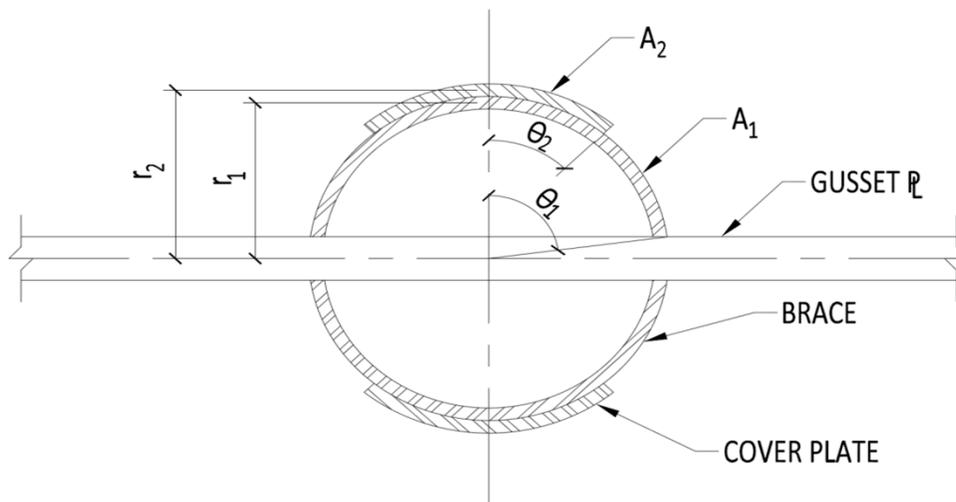
$$x = 0.00 \text{ in} \quad \text{Tbl D3.1}$$

$$U = 1.000 \quad \text{For } l_w > 1.3D \quad \text{Tbl D3.1}$$

$$\text{Effective Area of HSS, } A_e = 2.13 \text{ in}^2 \quad [A_e = U * A_n] \quad \text{Eqn D3-1}$$

$$\text{Req'd Reinforcement, } A_{ecp} = 0.31 \text{ in}^2$$

Reinforcement from: **HSS4X0.220** (Using 2 quarter sections)



$$A_1 = 1.07 \text{ in}^2 \quad [A_n/2]$$

$$A_2 = 0.61 \text{ in}^2 \quad [A_{g-Cover Plate}/4]$$

Radius of Brace, $r_1 = 1.8975$ in

$$[(D/2)-(t/2)]$$

Radius of Reinf, $r_2 = 2.1025$ in

$$[(D/2)+(t/2)]$$

$\Theta_1 = 1.37$ rad.

$$[\text{radians}(A_n/A_g * 90)] \text{ (1/4 of brace net area)}$$

$\Theta_2 = 0.785$ rad.

Partial Circle C.O.G., $x_{\text{Brace}} = 1.36$ in

$$[r_1(\sin(\Theta_1)/\Theta_1)]$$

Center of Gravity, $x_{\text{Cover Plate}} = 1.89$ in

$$[r_2(\sin(\Theta_2)/\Theta_2)]$$

$\Sigma A = 1.68$ in²

$\Sigma xA = 2.60$ in³

$x_{\text{Composite}} = 1.24$ in

$$[(\Sigma xA/\Sigma A)-(t_g/2)]$$

$U = 0.876$

$$[1-(x_{\text{Comp.}}/l_w)]$$

$A_e = 2.94$ in²

$$[2 * U * \Sigma A]$$

$A_e = 2.94$ in²

>

$A_g = 2.44$ in²

OK

$R_y = 1.6$

Ref. Lower Brace Design

$R_t = 1.2$

Expected Rupture Strength of Cover Plate

$\Phi P_n = 36.60$ k

$$[P_n = R_t A_2 F_u] [\Phi = 0.75]$$

Eqn D2-2

Expected Yield Strength

$\Phi P_n = 36.89$ k

$$[P_n = R_y A_2 F_y] [\Phi = 0.90]$$

Eqn D2-2

$\Phi P_n = 36.89$ k

$$[\text{Max of Expected Strengths}]$$

Thickness of Thinner Part=	0.21	in		[Brace thickness]	
Min. Weld Size, D=	2	/16"			Tbl J2.4
Weld Size, D=	3	/16"			
Number of Welds=	2				
Weld Length, l_w =	5	in			
Weld Strength, ΦR_n =	41.8	k		$[\Phi R_n * D * l_w * \# \text{ of Welds}]$	
Expected Strength, ΦP_n =	36.9	k		$[\Phi R_t F_u A_n]$ [$\Phi=0.75$]	
ΦR_n =	41.8	k	>	36.9 k	OK

Gusset Shear Yielding at Brace

AISC 360-10

Shear Yield Strength, ΦR_n =	419.77	k		$[2 * \Phi * 0.6 F_y A_g]$ [$\Phi=1.0$]	Eqn J4-3
ΦR_n =	419.77	k	>	163.97 k	OK

Whitmore Section

AISC 360-10

Whitmore Width, L_w =	15.55	in		$[2 * l_w \tan(30^\circ) + D_{HSS}]$	
A_g =	9.72	in ²		$[L_w * t_g]$	

Gusset Tensile Yielding at Brace

AISC 360-10

Whitmore Area, A_g =	9.72	in ²			
Tensile Yield Strength, ΦR_n =	314.83	in ²		$[\Phi F_y A_g]$ [$\Phi=0.9$]	Eqn J4-1
ΦP_n =	314.83	k	>	163.97 k	OK

$$\text{Whitmore Area, } A_{\text{effective}}=A_g= 9.72 \text{ in}^2$$

$$\text{Tensile Rupture Strength, } \Phi P_n= 422.68 \text{ k}$$

$$[\Phi F_u A_e] [\Phi=0.75]$$

Eqn J4-1

$$\Phi P_n= 422.68 \text{ k}$$

>

$$163.97 \text{ k}$$

OK

Compression in Gusset

AISC 360-10

$$\text{Whitmore Area, } A_g= 9.72 \text{ in}^2$$

$$K= 1.2$$

Tbl C-A-7.1

$$L= 3.21 \text{ in}$$

$$r= 0.180 \text{ in}$$

$$[t/\sqrt{12}]$$

$$KL/r= 21.4 <$$

25

Use Chapter E

$$4.71\sqrt{E/F_y}= 133.68$$

$$KL/r= 21.4 <$$

133.68

Use Eqn E3-2

$$F_e= 626.03 \text{ ksi}$$

$$[F_e=\pi^2 E/(KL/r)^2]$$

Eqn E3-4

$$F_{cr}= 35.14 \text{ ksi}$$

$$[F_{cr}=(.658^{F_y/F_e}) * F_y]$$

Eqn 3-2

$$\Phi= 0.9$$

E1(b)

$$P_n= 341.5 \text{ k}$$

$$[P_n=F_{cr} A_g]$$

Eqn E3-1

$$\text{Compression Strength, } \Phi P_n= 307.3 \text{ k}$$

>

$$29.56 \text{ k}$$

OK

Gusset Plate Dimensions

AISC 360-10

$$\text{Thickness, } t_g= 0.625 \text{ in}$$

$$\text{Length, } L_g= 50 \text{ in}$$

(Along beam)

$$\text{Width, } W_g= 8 \text{ in}$$

(Perpendicular to beam)

Load Angle=	39.03	(Round angle down for tables)	
Thickness of Thinner Part=	0.49 in	[Beam or Gusset thickness]	
Min. Weld Size, D=	3 /16"		Tbl J2.4
Weld Size, D=	3 /16"	(One weld each side)	
Weld Length, l=	50 in	(Length of Gusset)	
Eccentricity, e_x =	13.45 in	$[d_{beam}/2]$	
a=	0.269	$[e_x/l]$ (Round up for "a" in table)	
a value used in table=	0.3		
C=	3.21		Tbl 8-4
Φ =	0.75		
Strength of Weld Group, ΦR_n =	361.1 k		
Eccentric Force, R_{ub} =	212.1 k	$[1.25v(Vu^2+T^2)]$	
ΦR_n =	361.1 k	>	212.1 k Weld is Adequate

Horizontal Shear in Gusset at Beam

AISC 360-10

Gusset Length, l_g =	50 in		
A_g =	31.25 in ²	$[l_b t_g]$	
Shear Rupture Strength, ΦR_n =	703.1 k	$[\Phi * 0.6 F_{u-HSS} * A_{nv}]$ [$\Phi=0.75$]	Eqn J4-4
Shear Yield Strength, ΦR_n =	675.0 k	$[\Phi * 0.6 F_y A_g]$ [$\Phi=1.0$]	Eqn J4-3
Governing ΦR_n =	675.0 k	>	131.81 k OK

Gusset Width, W_g =	8	in		
	A_g =	5.00	in ²	$[l_b t_g]$
Shear Rupture Strength, ΦR_n =	112.5	k		$[\Phi * 0.6 F_{u-HSS} * A_{nv}]$ $[\Phi=0.75]$ Eqn J4-4
Shear Yield Strength, ΦR_n =	108.0	k		$[\Phi * 0.6 F_y A_g]$ $[\Phi=1.0]$ Eqn J4-3
Governing ΦR_n =	108.0	k	>	106.86 k OK

Unit Forces at Gusset-Beam Interaction

Gusset Section Modulus, S_g =	416.7	in ³ /in		$[L_g^2/6]$
Unit Shear, f_v =	2.64	k/in		$[V_{ug}/L_g]$
Max Unit Tension, f_a =	2.14	k/in		$[T_{ug-Max}/L_g]$
Min Unit Tension, f_a =	1.83	k/in		$[T_{ug-Min}/L_g]$
Contribution from Moment, f_b =	4.25	k/in		$[M_{ug}/S_g]$

Beam Web Local Yielding

Unit Force with Max Tension

Max Compressive Unit Force, f_{c-Max} =	6.39	k/in		$[f_b + f_{a-Max}]$
Max Tensile Unit Force, f_{t-Max} =	2.12	k/in		$[f_b - f_{a-Max}]$
Length of Gusset in Tension, L_t =	12.4	in		$[(f_t / (f_t + f_c)) * L_g]$
Resultant Tension Force, R_u =	13.2	k		$[.5 L_t f_t]$

Unit Force with MinTension

Max Compressive Unit Force, f_{c-Min} =	6.09	k/in		$[f_b + f_{a-Min}]$
Max Tensile Unit Force, f_{t-Min} =	2.42	k/in		$[f_b - f_{a-Min}]$
Length of Gusset in Tension, L_t =	14.2	in		$[(f_t / (f_t + f_c)) * L_g]$
Resultant Tension Force, R_u =	17.3	k		$[.5 L_t f_t]$

Web Local Yielding Strength, ΦR_n =	469.0	k	>	17.3 k	OK
				$[\Phi(5k_{Beam} + l_t)F_{y-WF}t_{w-beam}]$ $[\Phi=1.00]$	Eqn J10-2

Force with Max Tension

Resultant Compression Force, $R_u = 120.0$ k $[.5(L_g - L_t)f_c]$

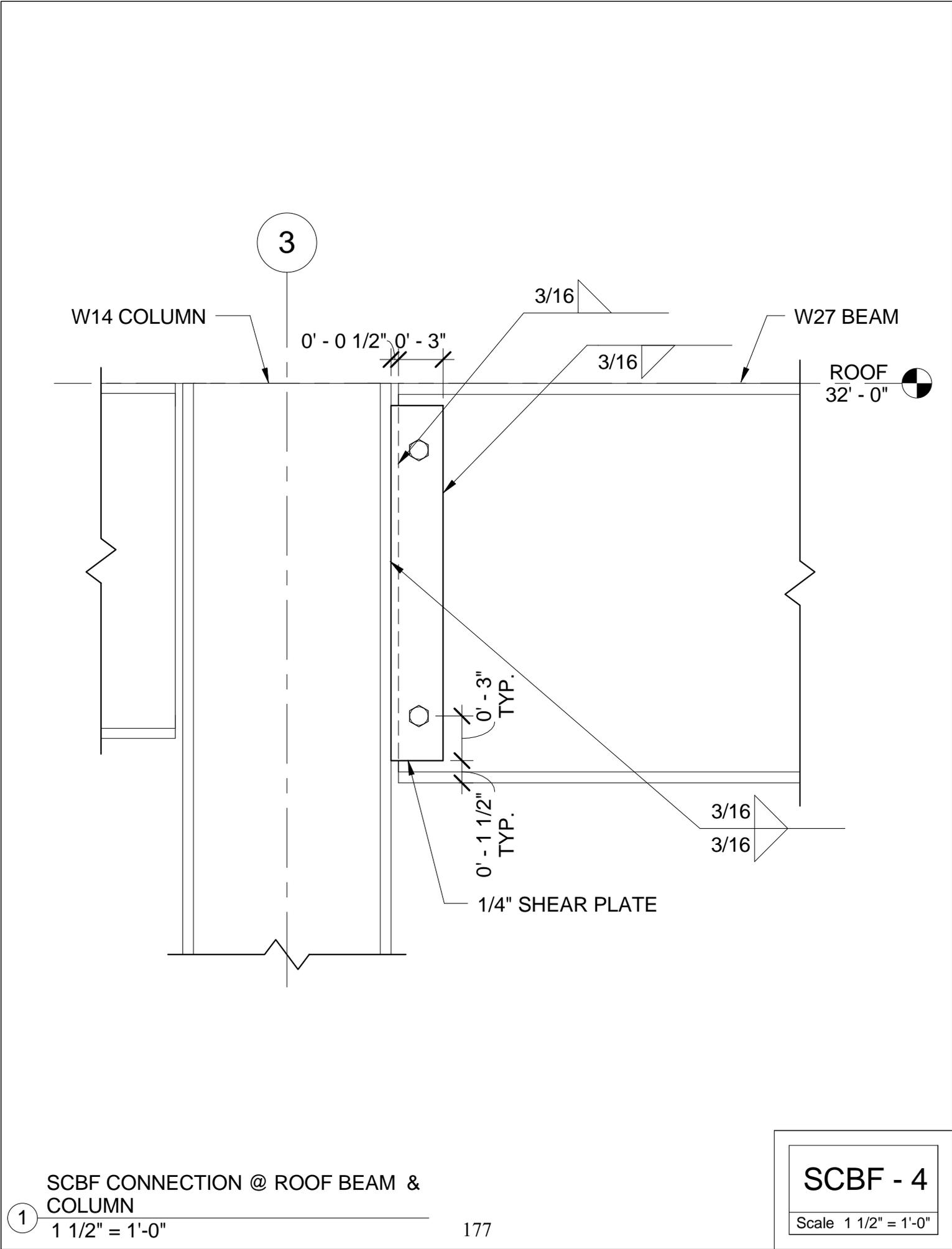
Force with Min Tension

Resultant Compression Force, $R_u = 108.8$ k $[.5(L_g - L_t)f_c]$

Web Local Crippling Strength, $\Phi R_n = 691.8$ k > 120.0 k

$$[\Phi 0.8 t_w^2 \{1 + 3(N/d)(t_w/t_f) 1.5\} \sqrt{E F_{yw} t_f/t_w}] [\Phi = 0.75]$$

OK



3

W14 COLUMN

W27 BEAM

0' - 0 1/2" 0' - 3"

ROOF
32' - 0"

0' - 3" TYP.
0' - 1 1/2" TYP.

1/4" SHEAR PLATE

3/16
3/16

SCBF CONNECTION @ ROOF BEAM & COLUMN

1

1 1/2" = 1'-0"

SCBF - 4

Scale 1 1/2" = 1'-0"

Roof Beam Connection to Column (SCBF)

Beam Properties

Ref. Beam Calculations, AISC Manual 14th ed.

Beam:	W27X94		
F_{y-WF}	50	ksi	Tbl 2-4
F_{u-WF}	65	ksi	Tbl 2-4
d_{Beam}	26.9	in	Tbl 1-1
t_{w-beam}	0.49	in	Tbl 1-1
t_{f-beam}	0.75		
k_{Beam}	1.34	in	Tbl 1-1
T_{Beam}	24.22	in	Tbl 1-1

Column Properties

Ref. Column Calculations, AISC Manual 14th ed.

Column:	W14X68		
F_{y-WF}	50	ksi	Tbl 2-4
F_{u-WF}	65	ksi	Tbl 2-4
d_{col}	14	in	Tbl 1-1
k_{Col}	1.31	in	Tbl 1-1
t_{w-col}	0.42	in	Tbl 1-1
t_{f-col}	0.72	in	Tbl 1-1
E	29000	ksi	

Weld Properties

AISC Manual 14th ed.

Weld F_{EXX}	70	ksi	
Weld Unit Strength, $\Phi_r n$	1.392	k/in * D	$[\Phi^*(1/\sqrt{2})^*(1/16)^*(0.6F_{EXX})]$ Eqn J2-3

Shear Plate

AISC Manual 14th ed.

Plate, F_{y-pl}	36	ksi	Tbl 2-3
Plate, F_{u-pl}	58	ksi	Tbl 2-3
Plate Thickness, t_{pl}	0.25	in	
Plate Height, h_{pl}	24.22	in	(T_{beam})
Plate Width, w_{pl}	3.5	in	

Shear, V_u =	60.61	k	$[V_u \text{ ref. roof beam design}]$
Axial, H_u =	64.99	k	$[P_u \text{ ref. roof beam design}]$

Shear in Plate

A_g =	6.06	in ²	$[h_{pl}t_{pl}]$
Shear Rupture Strength, ΦR_n =	158.0	k	$[\Phi * 0.6F_{u-pl} * A_{nv}] [\Phi=0.75]$ Eqn J4-4
Shear Yield Strength, ΦR_n =	130.8	k	$[\Phi * 0.6F_{y-pl}A_g] [\Phi=1.0]$ Eqn J4-3
Governing ΦR_n =	130.8	k	> 60.61 k OK

Beam to Shear Plate Weld

Load Angle=	47.0	degrees	$[\arctan(V_{uc}/H_{uc})]$ (Round down in table)
Thick. of Thinner Part=	0.25	in	
Min. Weld Size, D=	2	/16"	Tbl J2.4
Weld Size, D=	3	/16"	(One weld each side)
Weld Length, l_w =	24.22	in	$[h_{pl}]$
kl =	3	in	(Distance between fillet welds) (Edge of shear plate to edge of gusset)
k =	0.123865		$[kl/l]$ (Round down in table)
Eccentricity, e_x =	2	in	[Col. Flange to Center of weld group]
a =	0.083		$[e_x/l]$ (Round up for "a" in table)
a value used in table=	0.1		
k value used in table=	0.1		
C=	4.49		Tbl 8-4
Φ =	0.75		
Strength of Weld Group, ΦR_n =	244.7	k	
Eccentric Force, R_{ub} =	111.1	k	$[1.25\sqrt{(V_{uc}^2 + H_{uc}^2)}]$
ΦR_n =	244.7	k	> 111.1 k Weld is Adequate

Load Angle=	47.0	degrees	$[\arctan(V_{uc}/H_{uc})]$	
Thick. of Thinner Part=	0.25	in		
Min. Weld Size, D=	2	/16"		Tbl J2.4
Weld Size, D=	3	/16"	(One weld each side)	
Weld Length, l_w =	24.22	in	$[h_p]$	
Eccentricity, e_x =	5.95	in	$[d_{col}/2]$	
a=	0.246		$[e_x/l]$	(Round up for "a" in table)
a value used in table=	0.3			
C=	3.48			Tbl 8-4
Φ =	0.75			
Strength of Weld Group, ΦR_n =	189.6	k		
Eccentric Force, R_{ub} =	111.1	k	$[1.25\sqrt{(V_{uc}^2+H_{uc}^2)}]$	
ΦR_n =	189.6	k	>	111.1 k Weld is Adequate