Evaluation of force distribution within a dual special moment-resisting and special concentric-brace frame system

by

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Abstract

Dual Lateral Force Resisting Systems are currently required by code to include a Moment Resisting Frame capable of resisting at least 25% of the lateral loads. This thesis evaluates the seismic performance of a specific type of dual system: a Special Moment Resisting Frame-Special Concentric Brace Frame System (SMRF-SCBF) under three different force distributions. The three distributions were 80% - 20%, 75% - 25%, and 70% - 30% with the lesser force being allotted to the Special Moment Resisting Frame (SMRF) portion of the system.

In order to evaluate the system, a parametric study was performed. The parametric study consisted of three SMRF-SCBF systems designed with different seismic force distributions. The aim of this study was to determine accuracy of the three different seismic force distributions. The accuracy was measured by comparing individual system models' data and combined system models' data. The data used for comparison included joint deflections (both horizontal and vertical), induced moments at moment connections, brace axial loads, column shears, and column base reactions.

Two-dimensional models using the structural software RISA 3D were used to assist in designing the independent Seismic Force Resisting Systems. The designs of the frames were not finely tuned (smallest member size for strength), but were designed for drift (horizontal deflection) requirements and constructability issues. Connection designs were outside the scope of the study, except for constructability considerations – the SMRF and the SCBF did not have a common column; the frames were a bay apart connected with a link beam.

The results indicated that a seismic force distribution of 75% to the SCBF and 25% to the SMRF most accurately predicts that frame's behavior. A force distribution of 80% to the SCBF and 20% to the SMRF resulted in moderately accurate results as well.

A vast opportunity for further research into this area of study exists. Alterations to the design process, consideration of wind loads, or additional force distributions are all recommended changes for further research into this topic.

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Dedication

I would like to dedicate this Thesis to my family, friends, and academic faculty who have supported me throughout my academic career. My parents, Ben and Betsy Wearing, have been particularly supportive and helpful during my time at Kansas State University.

Chapter 1 - Introduction

This thesis evaluates the seismic force distribution within a Special Moment-Resisting Frame – Special Concentric-Brace Frame dual system (SMRF-SCBF) designed using three different design seismic force distributions. Specifically, the accuracy of the three different seismic force distributions are presented. For this thesis, accuracy is defined as how closely the individual Seismic Force Resisting System (SFRS) results match the dual SFRS results. The range of force distributions analyzed is based around the American Society of Civil Engineers Structural Engineering Institute (ASCE/SEI) 7-10 *Minimum Design Loads for Buildings and Other Structures* mandated force distribution between the two individual systems of the dual Lateral Force Resisting Systems (LFRS) as it relates to the design of the dual system. An SMRF-SCBF is a dual LFRS that consists of a Special Moment Resisting Frame (SMRF) and a Special Concentric Brace Frame (SCBF). The most recent International Building Code (IBC), released in 2015, adopted the ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures*, making this document code. In the ASCE/SEI 7-10, a dual LFRS is any system that combines two LFRSs together with the stipulation that one of these systems is a special or intermediate Moment Resisting Frame (MRF) capable of resisting no less than 25% of the lateral forces (Engineers, A. S., 2013).

A parametric study of a five-story building's SFRS was conducted using three load distributions. The five-story building is 150ft by 75ft in plan (five 30ft bays in the east/west direction and three 25ft bays in the north/south direction) with floor-to-floor heights of fourteen (14) feet for the first story and twelve (12) feet for the second, third, fourth, and fifth stories. Two-dimensional models using the structural software RISA 3D were used to assist in designing the independent SFRSs. The total seismic load for the building was divided between the two independent SFRSs during the design. Following completion of the individual designs, combined analyses of the dual systems for the three load distributions were conducted. The three seismic-load distributions studied are:

- Case I with SCBF – 80% of seismic loads and SMRF – 20% of seismic loads
- Case II with SCBF – 75% of seismic loads and SMRF – 25% of seismic loads
- Case III with SCBF – 70% of seismic loads and SMRF – 30% of seismic loads.

These distributions were chosen around the ASCE/SEI 7-10 minimum for a dual system to have the SMRF capable of resisting a minimum of 25% or the lateral forces.

Chapter 2 - Literature Review

Research over SFRSs and their collapse mechanisms is presented in this chapter. It is crucial that design (idealized) models accurately predict the likely response of an SFRS during the Maximum Considered Earthquake (MCE). When the design of the structure does not accurately reflect the true behavior of that structure, it can lead to detrimental results.

Structural Robustness and Failure Mechanisms

In 2009, Ye Lieping authored *Failure Mechanism and its Control of Building Structures under Earthquakes Based on Structural System Concept.* This work focuses on failure mechanisms formed by tall steel moment-resisting frame buildings under seismic excitation (Ye L. P., 2008). Lieping's research has a strong emphasis on the structural robustness of a structural system. Within Lieping's work, structural robustness is defined as a structure's ability to, "…resist unexpected overloads induced by disasters, such as severe earthquakes and explosions," (Ye L. P., 2008). Hierarchies within the structural systems are a large component of discussion. Lieping defines a SFRS hierarchy of elements as (Ye L. P., 2008): vertical load-bearing; lateral load-bearing; potential plastic energy-dissipating; and special energy-dissipating elements. Design of SFRSs reflects this hierarchy via the overstrength factor, Ω_o. Only certain elements within a SFRS are designed to experience inelastic deformations. In order to keep other elements of the SFRS elastic, the overstrength factor amplifies the seismic loads applied to the elements which are intended to remain elastic.

Structural robustness is considered the, or one of the, top benefits to utilizing a dual LFRS in lieu of a singular LFRS. However, the robustness and integrity of any structural system can be compromised if it is not correctly designed and analyzed. "In order to achieve higher robustness, a structural system should be designed to experience the desirable failure procedure under the external actions" (Ye L. P., 2008). This excerpt from Lieping's work explains the direct link to a system's structural robustness and its failure mechanisms. It also explains that if the design of a structural system does not experience the desired yielding and/or failure mechanisms, the structural integrity of the system is compromised.

Undesirable failure mechanisms that structures experience are local failures. For example, when a soft story develops within an SMRF, it can prevent desired plastic hinging from occurring in the beams of the stories above or below it. This leads to exceptionally large localized stresses and inelastic deformations. These local failures can lead to the collapse of even the most ductile of systems.

Collapse Mechanisms under Seismic Excitation

Krishnan Swaminathan authored *Mechanism of Collapse of Tall Steel Moment-Frame Buildings under Earthquake Excitation*; this article was then published in 2012. The research presented in the article is over the collapse of two different tall, steel buildings from the 1985 Mexico City earthquake. Swaminathan conducted three-dimensional, non-linear analyses of the buildings with the aim of observing how these structures might have responded during the earthquake. From field observations after the earthquake, the collapse of these buildings was deemed to have been local failures, including column flange buckling and weld failure (Krishnan S., 2012).

One of the buildings Swaminathan analyzed was an 18-story building with an MRF. The building was subjected to the seismic motion from an earthquake that occurred in Northridge, CA in 1994. Plastic hinging occurred in the columns in the fourth and eighth stories leading to what Krishnan referred to as a quasi-shear band (QSB) mechanism forming (Krishnan S., 2012). This term is derived from the shear-like deformations which resemble, "plastic shear bands in ductile solids that are severely (shear) strained," (Krishnan S., 2012). This implies that the damages have been disproportionately distributed within the structure. The localization of the building's inelastic deformations prevents plastic hinges that were intended to develop in additional elements both above and below the fourth and eighth stories from developing to the desired extent. Seismic excitation of the building lead to an undesired mechanism forming. In this particular case, the overall building failure mechanism was a QSB. Figure 1 shows the building's deformed shape.

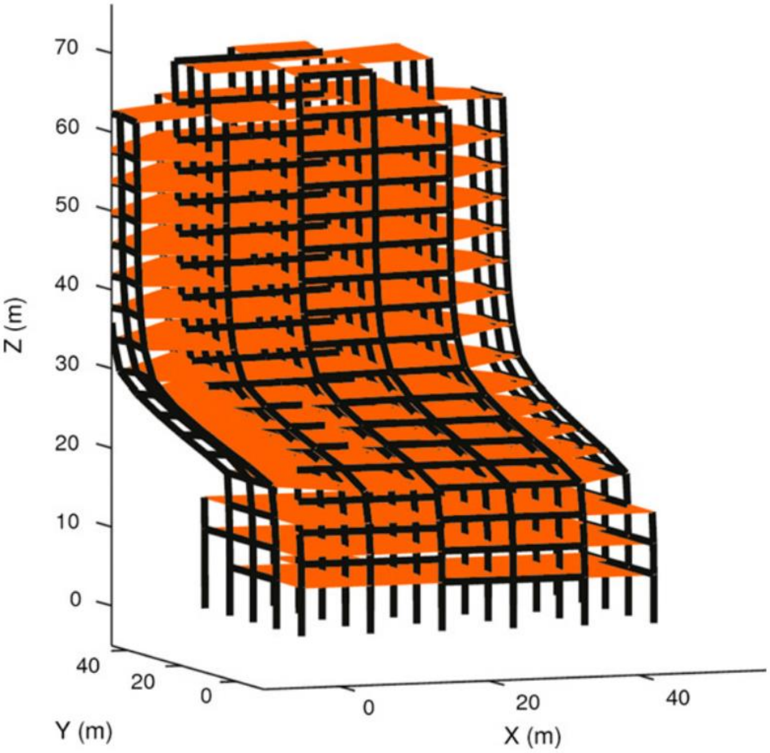


Figure 1: Resulting Deformed Shape, Adapted from Krishnan (Krishnan S., 2012) with permission from ASCE

The deformations in Figure 1 are amplified by a magnitude of five to clearly depict the building's response. The QSB resulted in a sidesway mechanism and ultimately failure due to the destabilization that occurred in the sidesway stories (Krishnan S., 2012). This reiterates the point that if a structure's design does not accurately reflect the true response of the structure, the consequences can be catastrophic. Thus, the interest in determining if the force distribution mandate for dual LFRSs that is in place accurately reflects the true force distribution within that structural system.

Design Methods Applied to a Dual Moment-Resisting – Concentrically Braced Frame Seismic Force Resisting System

Alessandra Longo authored articles over dual SMRF-CBF systems including *Moment frames – concentrically braced frames dual systems: Analysis of different design criteria* and *Failure Mode and Drift Control of MRF-CBF Dual Systems*. These articles were published from approximately 2010 to 2015. A large portion of Longo's work is aimed at design methods currently used for dual LFRSs, and specifically SMRF-SCBF systems subjected to seismic loads. This research focuses on the collapse mechanism a building is likely to form during an extreme seismic event. A collapse mechanism can be defined as the mechanism formed by a structural system as it initially experiences structural failure. Structural engineers accept that global failure is the desired collapse mechanism of a structure. This should not be confused with progressive collapse. A global failure implies that the structure is stable under dead loads and minimal live loads after the maximum considered earthquake has occurred. Failure in this context is not necessarily an unwanted result since certain elements within SFRSs are designed such that they will experience inelastic deformations (failure) when a large seismic event takes place. Rather the aim of a global failure type response is that these inelastic responses throughout the structural system occur simultaneously and to the desired extent. If inelastic deformations within the structural system either do not develop simultaneously or reach unaccounted for magnitudes, the structural system's integrity can be compromised. "The collapse mechanism typology strongly affects the energy dissipation capacity of structures subjected to destructive seismic events" (Longo A., 2012). This excerpt reiterates that the capacity and performance of a structure is heavily dependent on the collapse mechanism it forms.

The analyses methods used in Longo's studies included both push-over analyses and incremental dynamic analyses. These are common analysis methods used for design in areas of high seismicity or for Performance Based Design (PBD) projects. The modeling conducted for the parametric study presented in this thesis was a two-dimensional and linear-elastic analysis. This is a drastically different and less encompassing analysis method compared to Longo's work. However, for the aim of the study presented in this thesis, a two-dimensional and linear-elastic analysis is sufficient.

Longo's work focused on the Theory of Plastic Mechanism Control (TPMC) and presented significant data suggesting that TPMC is a superior design method than what is currently mandated in the European building codes. The European building codes mandates on dual systems is similar to American building codes in that there are not specific design rules pertaining to dual LFRSs. For this reason, dual LFRSs are typically designed independently with the requirements for the individual systems comprising the dual system being applied to the relevant individual LFRS. SMRF-SCBF systems were the SFRS of interest for Longo's work due to the appeal of exploiting the benefits of the two very common SFRSs. TPMC focuses on the energy induced into the SFRS during an earthquake. It requires rigorous energy balance calculations that account for post-buckling behavior and axial shortening/elongation of braces, the order in which plastic hinges will form within the MRF, distribution of column plastic moments, etc. (Longo A., 2012). In "Moment Frames – Concentrically Braced Frames Dual Systems: Analysis of Different Design Criteria," Longo provides the following design flowchart for SMRF-SCBF using TPMC as the design method:

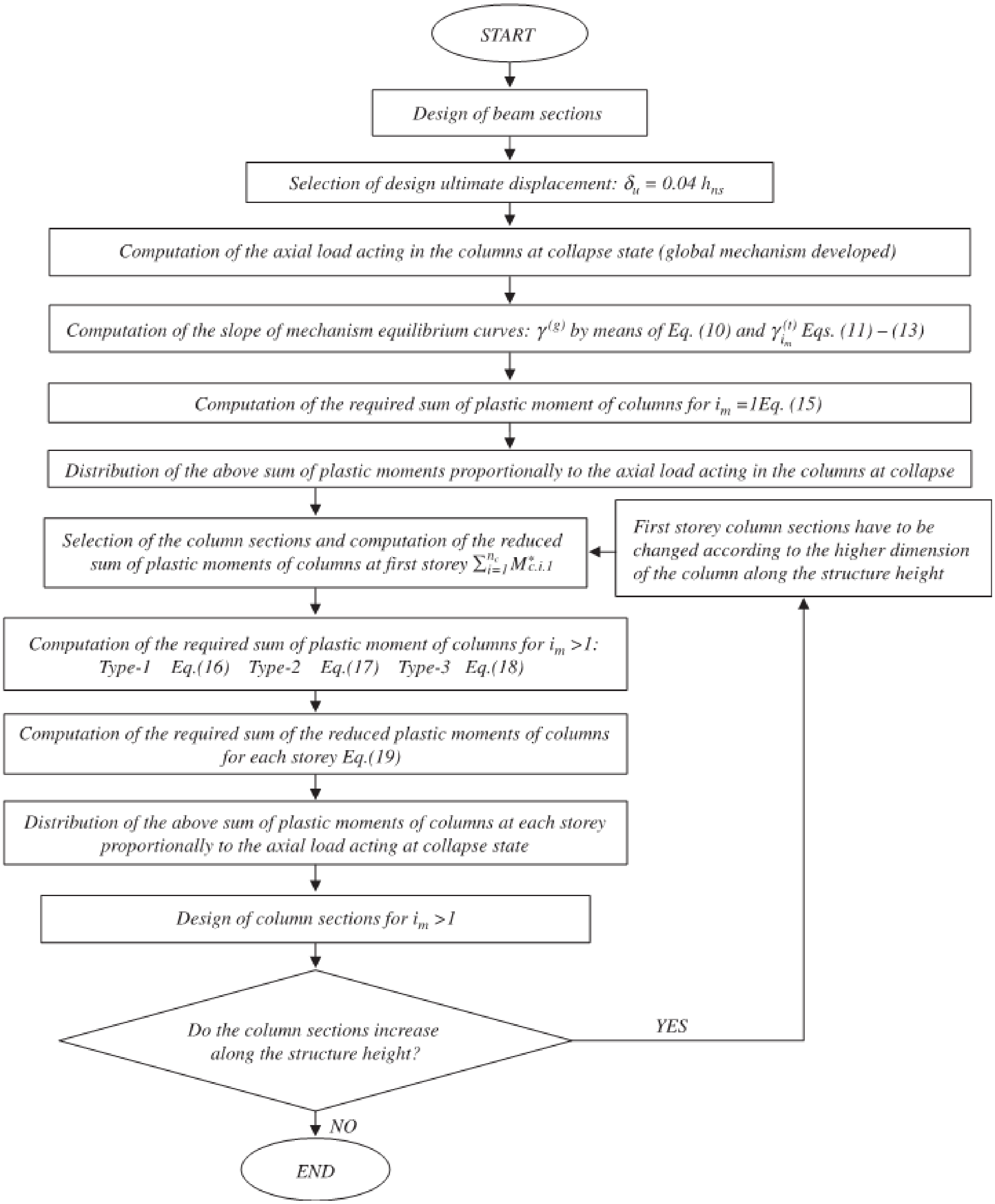


Figure 2: Theory of Plastic Mechanism Control Design Procedure Flowchart, Adapted from Longo (Longo A., 2012)

It is evident in Figure 2 that TPMC design is a detailed and tasking design process. It requires the development of curves/graphs, exceptionally complex equations, moment distribution within the LFRS, etc. By comparison, the Equivalent Lateral Force Procedure (ELFP) per the ASCE/SEI 7-10 requires simple algebraic calculations along with table and figure look-ups. The simplicity of the ELFP should not be seen as an invalidation of itself; rather it is an efficient and effective method for analyzing seismic forces on low and mid-rise structures.

Longo’s research indicates that the TPMC design process for the eight-story case study building was more successful in achieving a global failure mechanism than the current European building codes’ design procedures. A similar result could be expected if TPMC and ELFP designs were carried out on an identical eight-story structure.

The framing used in Longo’s study consisted of two bays of MRF and one bay of Concentric Brace Frame (CBF). The difference between the mechanisms formed by the two designs when subjected to seismic forces is reflected in Figure 3.

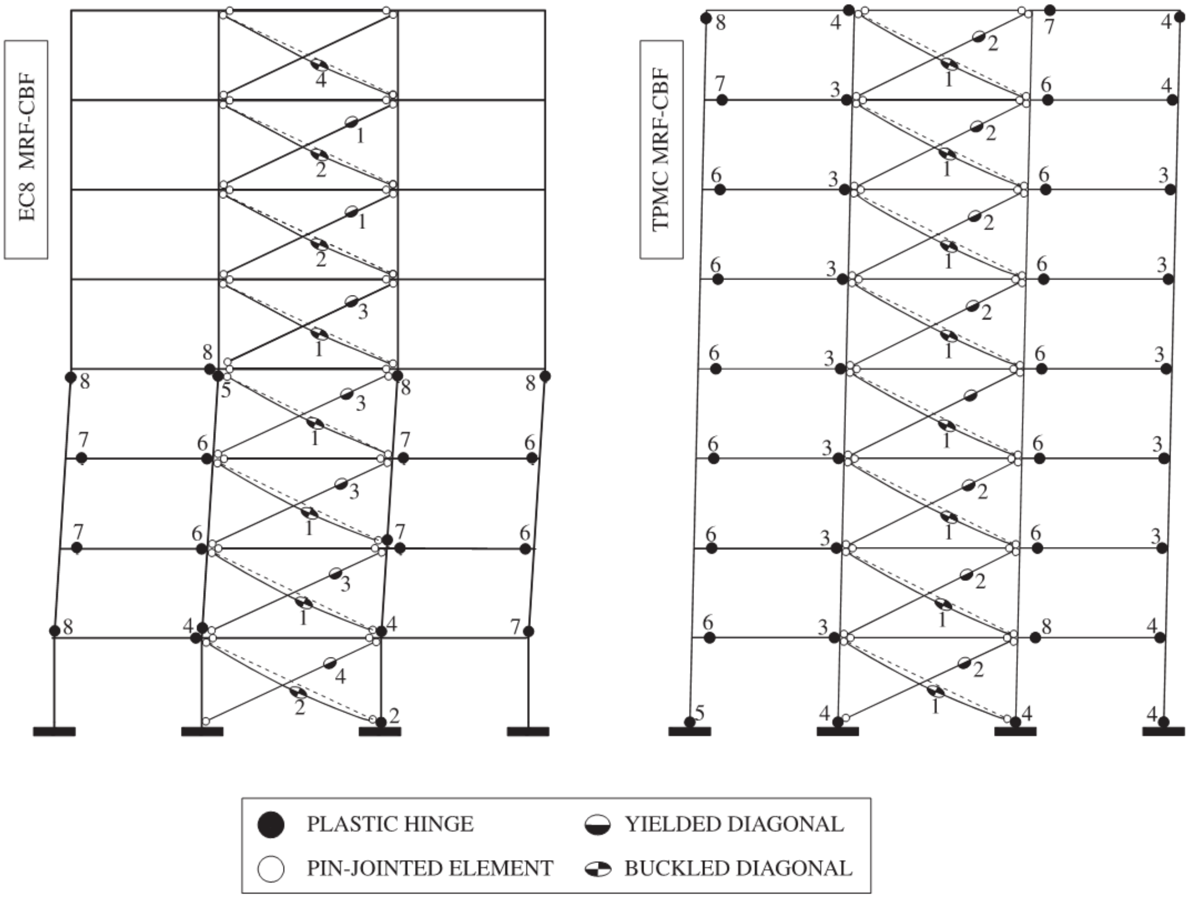


Figure 3: Developed Pattern of Yielding, Adapted from Longo, (Longo A., 2012)

When subjected to seismic forces, the TPMC design for the eight-story structure resulted in a perfectly global response. This is indicated on the right in Figure 3 where every desired plastic hinge within the frame has developed and every brace within the frame has experienced either tensile yielding or compression induced buckling. In contrast, the European Building Codes design shown on the left in Figure 3 resulted in a partially global response. This is primarily evident by the lack of plastic hinge development within the frame.

Longo found that the braced frame accounted for approximately 50% of the combined system's lateral stiffness for the TPMC design (Longo A., 2012). This makes sense given the virtually perfect global failure mechanism formed by the system when subjected to seismic loads.

Longo's work indicates that TPMC is a valid design alternative to the current European codes' standards and practices. Although, the volume of computations and analyses required for TPMC design are notably more tasking than currently accepted design processes. For this reason, the application of TPMC design seems most relevant and applicable for PBD projects. However, for the greater majority of structures being designed and built, a less tasking design procedure is desirable.

Chapter 3 - Parametric Study

The parametric study was conducted on a five-story building located in Reno, NV. The building floor plan and location are identical to the floor plan and location used by Kansas State University Master of Science graduates, Eric Grusenmeyer in 2012 and Samuel Hague in 2013, for their research. The building floor plan consisted of five 30 foot bays in the east-west direction and three 25 foot bays in the north-south direction (150’ – 0” by 75’ – 0” overall). The building’s elevation is nearly identical to the elevation of the building Samuel Hague performed his study on totally approximately 60 feet in height (one 14’ – 0” story and four 12’ – 0” stories). The building floor plan and an example frame elevation are shown in Figure 4 and Figure 5.

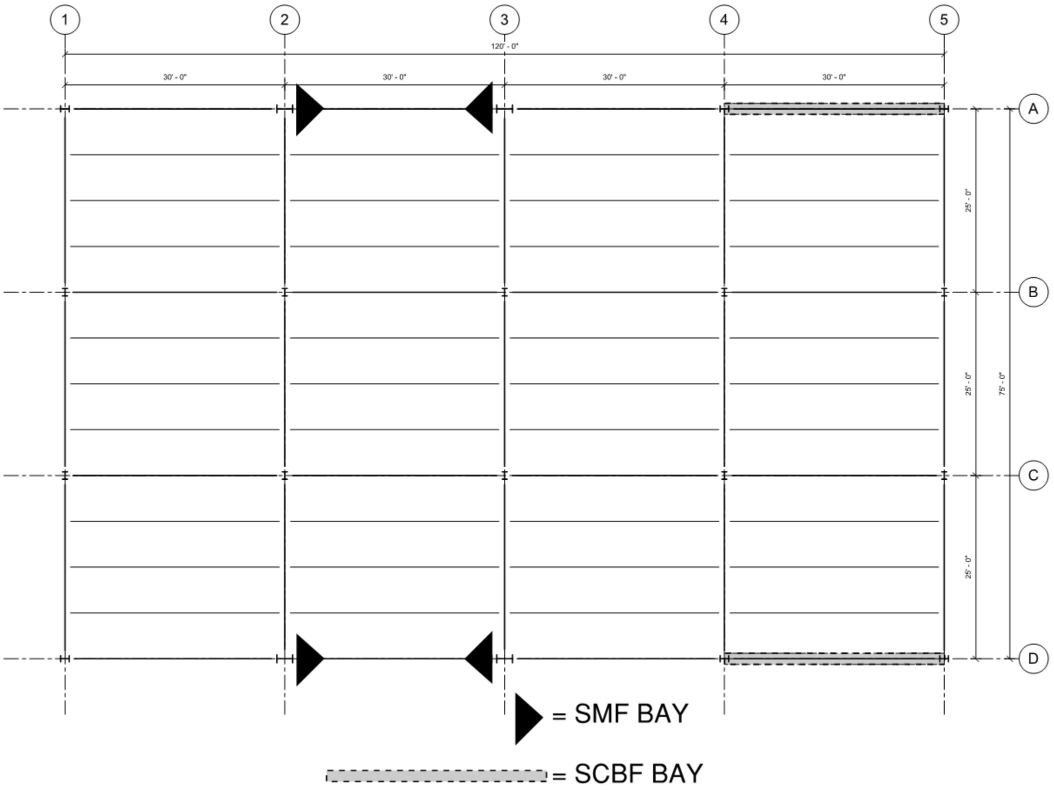


Figure 4: Building Framing Plan

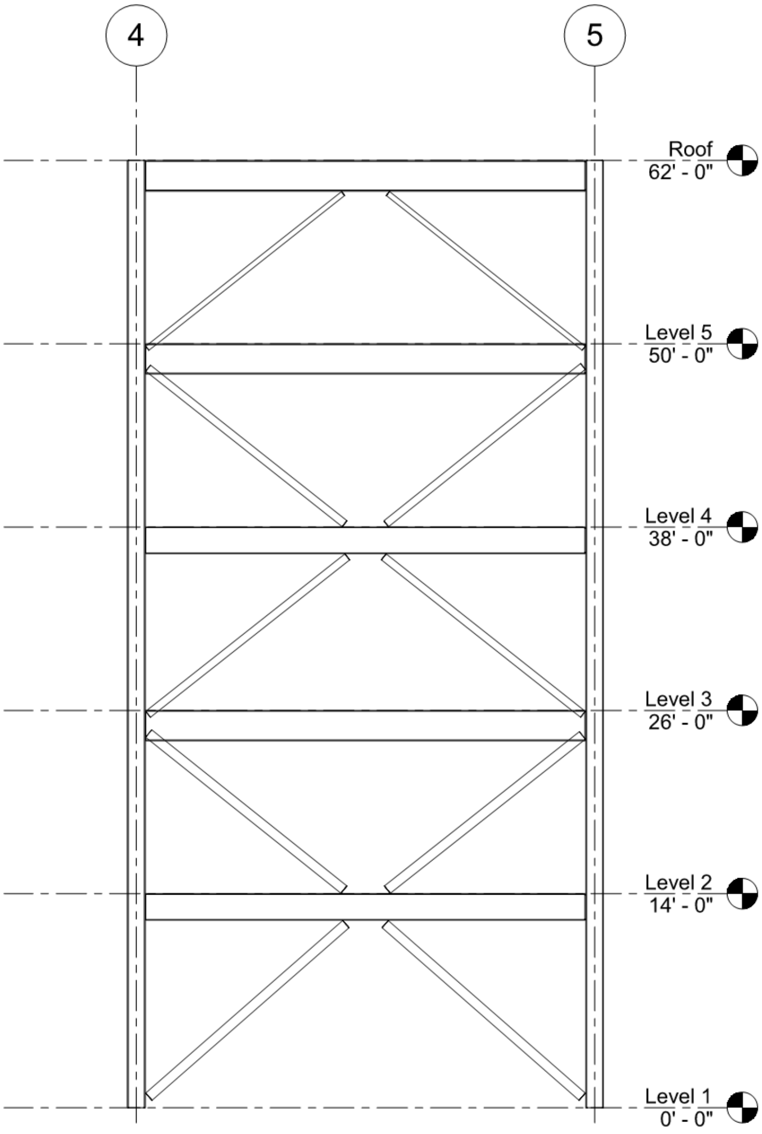


Figure 5: Brace Frame Elevation

Design

The design process for the parametric study was:

1. Determination of seismic loads
2. Allocation of seismic loads to SCBF and SMRF portions of the SFRS for each Case

3. Design of the SCBF individually for each case
4. Design of the SMRF individually for each case

Determining the loads a system and its individual elements must withstand is a crucial step in any structural engineering design. There are several accepted methods used to determine the seismic loads. The method most commonly used in practice for low and mid-rise structures is the ELFP; this is the method used for this parametric study. The ELFP idealizes seismic energies induced into a building as static loads applied externally. This idealization greatly simplifies the determination of the seismic loads caused by the building seismic inertia.

The ELFP accounts for damping equal to 5% of the critical damping for the mapped values of ground motions corresponding to the maximum considered earthquake (Engineers, A. S., 2013). This is done because all structures have inherent damping within the structural system.

The ASCE/SEI 7-10 lists load combinations that structures must be designed to withstand. These load combinations aim to account for the likelihood that different types of loads will occur at the same time over the life of the structure. The two load combinations of interest for this study are load combinations 5 and 7, these load combinations account for seismic loads. The largest discrepancy between these two load combinations is the magnitude of gravity loads that is accounted. The two load combinations aim to emulate the worst case conditions for seismic system failures such as sliding or overturning.

The allocation of the seismic loads between the two systems was of special interest for the parametric study. The three different load distributions utilized for the study are:

- Case I – SCBF = 80% of seismic loads (258.3 kips), SMRF = 20% of seismic loads (64.6 kips)

- Case II – SCBF = 75% of seismic loads (242.2 kips) and SMRF = 25% of seismic loads (80.7 kips)
- Case III – SCBF = 70% of seismic loads (226.0 kips) and SMRF = 30% of seismic loads (96.9 kips).

For each Case, the two SFRSs, SMRF and SCBF, were designed independent of one another before being analyzed together in the same two-dimensional model in RISA 3D. This permitted the SMRF to be designed to a quantifiable portion of the seismic loads. It is important that the SMRF's capacity has a quantified value because of the ASCE/SEI 7-10's requirement that the MRF portion of any dual LFRS has the capacity to withstand at least 25% of the lateral load.

Because these two systems were designed independently of each other, the drifts of the two systems were added together in order to check drift and stability limitations set by the ASCE/SEI 7-10 (i.e., the amplified drifts from the SCBF design and the SMRF design were added together and then compared to the serviceability limits set by the ASCE/SEI 7-10). The SCBF was designed first because the SMRF's design was likely to be governed by drift rather than capacity. After the SCBF was designed, the remaining allowable drift was used as the allowable drift limit for the SMRF design. Had the SMRF been designed first, it is likely that the remaining allowable deflection would have forced the SCBF's design to be governed by drift instead of strength. This would have resulted in a less economical design for the SFRS.

Two-dimensional models from RISA 3D were used to establish preliminary member sizes when designing both the SCBF and the SMRF. This next step in design was analysis of members and the frames as a whole. The analysis and design processes are inherently intertwined due to iterations or alterations to the design as design progresses. RISA 3D was used throughout the design process to calculate internal member forces as well as structural displacements of the frames.

Once a preliminary section was selected for the beams, columns, and braces, the design of these members began with hand calculations. Hand calculations were used to verify a section's capacity and ductility. Hand calculations are a great method for checking design since the opportunity for error is reduced relative to software calculations. In spreadsheets or analysis software, it is very easy to miss or incorrectly input data and/or improperly model a structure. These errors can be difficult to catch without an in-depth review of the software.

Hand calculations were performed for all members in the Case I load distribution. Microsoft Excel was then used to perform these same calculations. The hand calculations followed the example procedures laid out for SCBF and SMRF design per the AISC Seismic Design Manual. The hand calculations provided a convenient means of checking the spreadsheet calculations as the spreadsheet was developed. Once the spreadsheet was complete and verified, it was used to design the members for the other load Cases.

Seismic design is an inherently iterative process where a designer can spend virtually endless time trying to develop the absolute most economical design solution. For the parametric study conducted for this thesis, the design of both the SCBF and SMRF aimed toward a practical design in lieu of a more refined design. The results of the study could be less applicable towards industry practice if the designs were further honed. For this reason, maintaining a practical design was important to the study. Two key design decisions were made based off this criterion. Bracing sizes were changed every two stories instead of every story, totaling three different brace sizes per load Case (2nd & 3rd, 4th & 5th, roof). This method of design is common practice for a five-story structure such as the one used in this study. Similarly, the SMRF beams were varied in this same pattern; every two stories the beam size changed, totaling three different beam sizes per load Case (2^{nd} & 3^{rd}, 4^{th} & 5^{th}, roof).

Analysis

Seismic loads were modeled as static point loads pushing against the frame during design. Modeling the seismic loads as point loads should result in a conservative design since the loads are not as evenly distributed as they will actually be. Upon the completion of the frame designs, the seismic loads were modeled as distributed lateral loads along the beams of the frame. This more accurately represents the transfer of the lateral loads from the diaphragms to the beams. The results used for comparisons and conclusions were outputs from the final RISA 3D models with the seismic loads modeled as distributed loads.

Additional Design Notes

The serviceability limits for the structure were a typical interstory drift limit slightly less than three inches (2.88") for all stories except for the second story (first elevated level) where the interstory drift limit was slightly more than three inches (3.36"). This discrepancy is due to a difference in story heights; the second story is elevated fourteen (14) feet above the first story while the rest of the story heights are all twelve (12) feet (see Figure 5). In order to meet these serviceability limits set by the ASCE/SEI 7-10, the bases for the SMRF columns were fixed instead of pinned. This design decision was deemed necessary after preliminary design showed that member sizes would have to be significantly larger sections (75% to 200% increases in structural weight) if the bases were pinned. Again, this was a product of satisfying serviceability limits. Fixing the bases of a MRF introduces additional complexities and steps to the design process of the foundation system and the column to foundation connection. These additional design requirements are outside the scope of this study.

The SFRS was designed, analyzed, and evaluated in one of the building's orthogonal directions. In practice, this process must be performed for both orthogonal directions of the building. It was deemed unnecessary to carry out design in both orthogonal directions for the scope of this study.

During the design of the individual SFRSs, both the SMRFs and SCBFs were analyzed assuming they were located between gridlines 2 and 3 along gridline A or D. When the frames were analyzed together, the SCBF was relocated so it was between gridlines 4 and 5 along gridline A or D (see Figure 4). This relocation led to a change in the gravity loads that column CL2 was subjected to (see Figure 6). The SCBF was relocated in lieu of the SMRF because the axial load of the SMRF columns is likely to have more of an effect on the SFRS design relative to the axial load of the SCBF columns. Thus, it is more reasonable to allow a variation in the SCBF column axial loads opposed to the SMRF column axial loads. The small variation of gravity loads on the SCBF columns should have a negligible effect on the frames' displacements.

The SCBF could have been relocated between gridlines 3 and 4 along gridline A or D (see Figure 4). This placement would have negated the issue of the variable gravity loads on column CL2 (see Figure 6). However, as already stated, maintaining an applicable design was important to the study. Placing the SCBF and SMRF directly next to one another introduces numerous conflicts. Some of these conflicts are column size, connection design and constructability, column base connection design, etc. For these reasons, a so-called linking bay was maintained in order to separate the two frames and prevent these design conflicts from occurring.

Member Sizes

The Table 1, Table 2, and Table 3 present the member sizes for the various SFRSs designed for this study. The member labels in these tables correlate with the elevations shown in Figure 6.

Table 1: Case I Member Sizes

Case	System	Member	Section
Case I	SCBF	CL-1 & 2	W14x68
		BM-1	W21x44
		BM-2	W24x68
		BM-3	W21x44
		BM-4	W24x68
		BM-5	W24x84
		BR-1, 2, 3, & 4	HSS7.50 x 0.312
		BR-5, 6, 7, & 8	HSS6.00 x 0.312
		BR-9 & 10	HSS4.00 x 0.226
	SMRF	CL-1 & 2	W24x207
		BM-1 & 2	W24x84
		BM-3 & 4	W24x76
		BM-5	W24x62

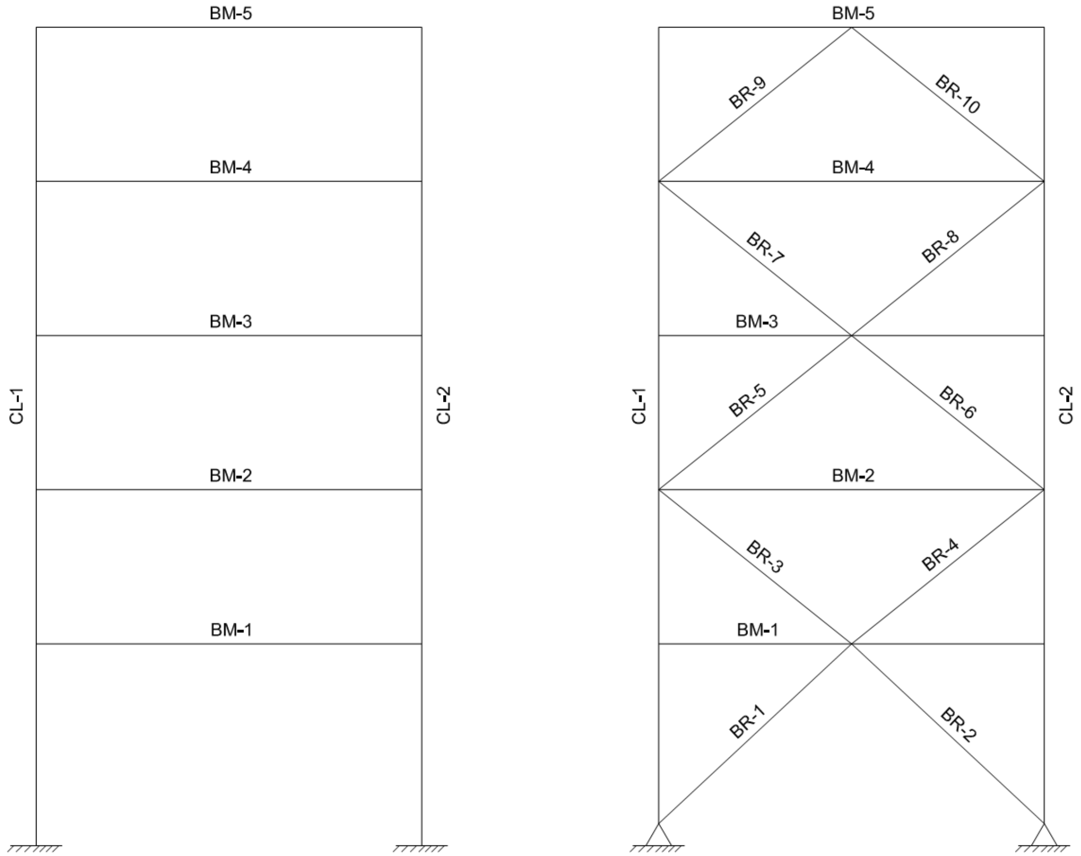
Table 2: Case II Member Sizes

Case	System	Member	Section
Case II	SCBF	CL-1 & 2	W14x68
		BM-1	W21x44
		BM-2	W24x55
		BM-3	W21x44
		BM-4	W24x55
		BM-5	W24x68
		BR-1, 2, 3, & 4	HSS7.50 x 0.312
		BR-5, 6, 7, & 8	HSS6.00 x 0.280
		BR-9 & 10	HSS4.00 x 0.220
	SMRF	CL-1 & 2	W27x258
		BM-1 & 2	W24x103
		BM-3 & 4	W24x84
		BM-5	W24x76

Table 3: Case III Member Sizes

Case	System	Member	Section
Case III	SCBF	CL-1 & 2	W14x68
		BM-1	W21x44
		BM-2	W21x44
		BM-3	W21x44
		BM-4	W21x44
		BM-5	W24x68
		BR-1, 2, 3, & 4	HSS7.00 x 0.312
		BR-5, 6, 7, & 8	HSS6.00 x 0.250
		BR-9 & 10	HSS4.00 x 0.220
	SMRF	CL-1 & 2	W27x258
		BM-1 & 2	W24x131
		BM-3 & 4	W24x103
		BM-5	W24x84

The member sizes presented in Table 1,Table 2,Table 3 reflect the different force distributions used for design. Case I's design resulted in the largest member sizes for the SCBF members and the smallest member sizes for the SMRF. Case III's design resulted in the smallest member sizes for the SCBF members and the largest member sizes for the SMRF.



SPECIAL MOMENT RESISTING FRAME ELEVATION WITH MEMBER LABELS

SPECIAL CONCENTRIC BRACE FRAME ELEVATION WITH MEMBER LABELS

Figure 6: Frame Elevations with Member Labels

Results

The results of the parametric study are presented in this section. The results of interest for this study include joint deflections, column-base reactions, moments at the joints, braces' axial forces, and story shears in the SMRF columns.

Throughout this section, the term accuracy often is used. In the context of this study, accuracy is referring to how closely the individual SFRS values match the dual SFRS values. A high accuracy would imply that an individual SFRS (SCBF or SMRF) value is close to the SMRF-SCBF value. A low accuracy would imply that an individual SFRS (SCBF or SMRF) value is not close to the SMRF-SCBF value. The meaning of accuracy within this study does not depend on the parameter or metric being discussed.

Deflections

The deflections presented are taken at each story along the columns. The deflections of each column are then graphed as a single data set input (i.e. the deflections of column CL-1 of the SCBF under Case I loading is one data set input). For reference of the location of joints and columns as they pertain to the data presented in this section, see Figure 7 below.

The drifts presented are the elastic drifts calculated from the computer models in RISA 3D. In order to account for the inelastic deformations of the SFRS and the additional drift that the inelastic deformations cause, the elastic drift must be multiplied by a drift amplification factor (C_d) given in the ASCE/SEI 7. Additionally, if reduced beam sections are used in the SMRF, an additional amplification of the elastic drift must be accounted for unless this reduced stiffness was accounted for in the computer model. The drifts were amplified as required for the design of the SFRSs. However, for comparing the deflections of the different SFRSs with each other, the drift amplification factor will do nothing but exaggerate any disparities that exist within the deflections. For that reason, the deflections presented in this section have not been amplified.

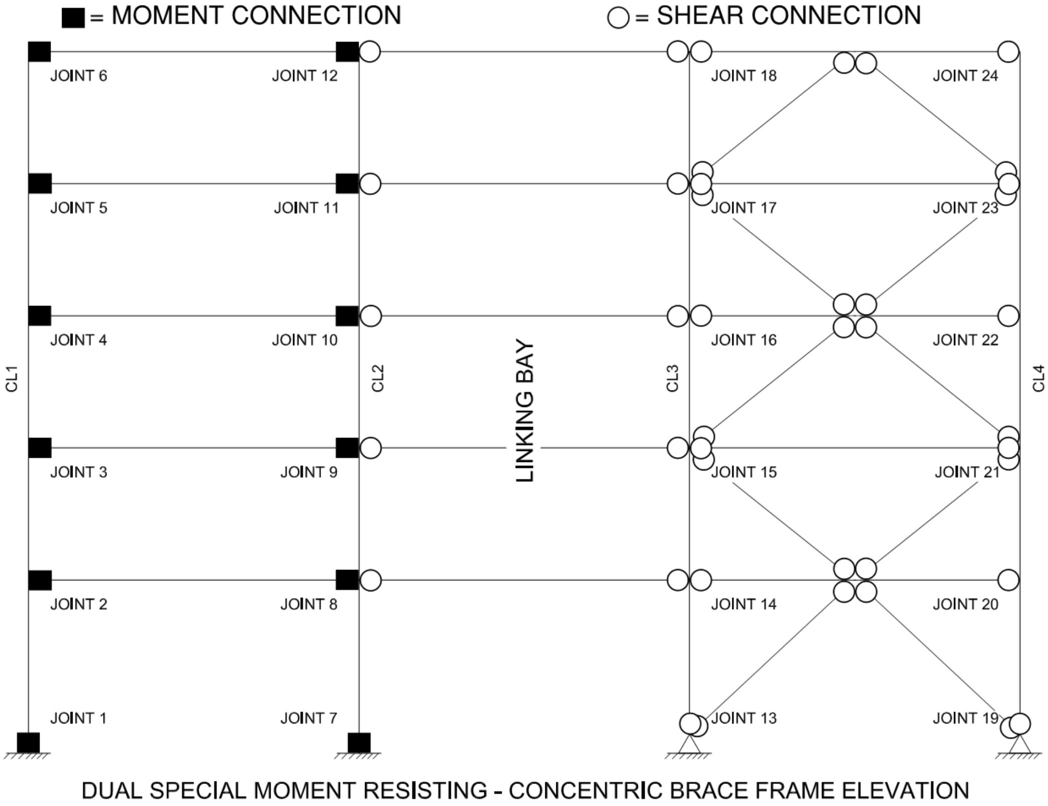
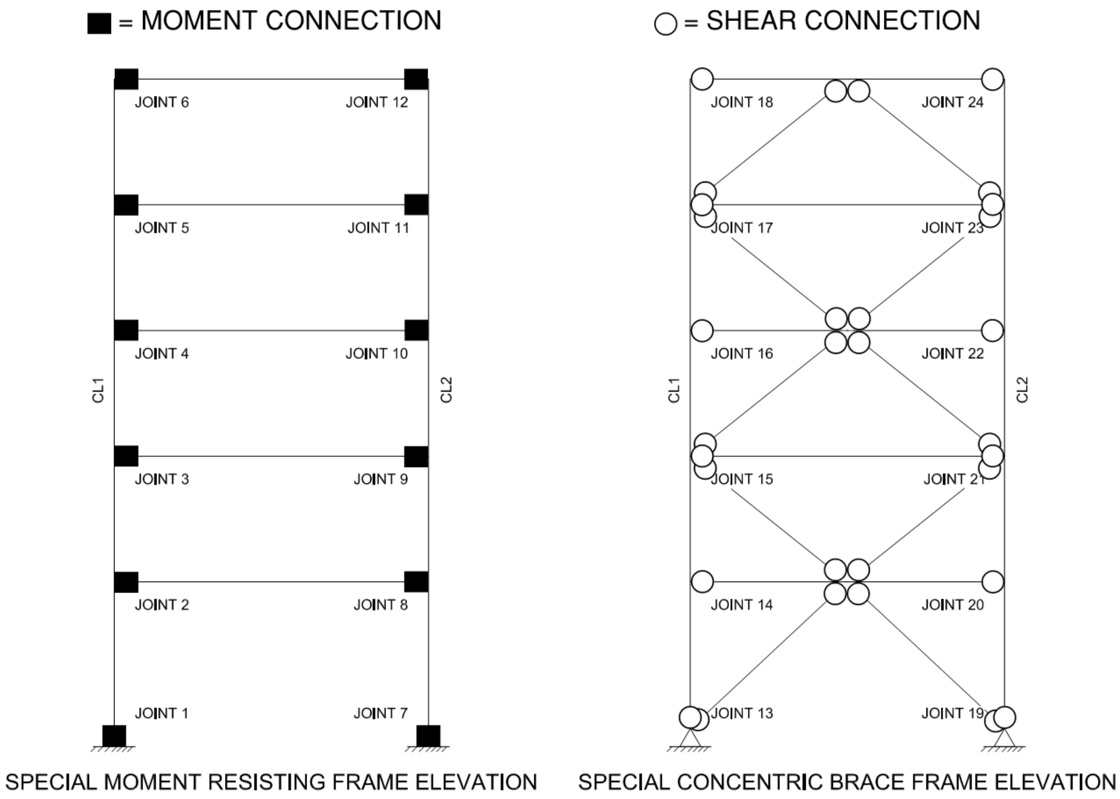


Figure 7: Frame Elevations with Joint and Column Labels (Dual System Frame)



SPECIAL MOMENT RESISTING FRAME ELEVATION SPECIAL CONCENTRIC BRACE FRAME ELEVATION

Figure 8: Frame Elevations with Joint and Column Labels (Individual System Frames)

It is important to note that columns CL-1 and CL-2 in the SCBF elevation correlate with columns CL-3 and CL-4 in the dual frame system.

In the discussion pertaining to Design, it was noted that the drifts of the two systems were added together and that this sum was used to check serviceability limitations. This was done because the designs of the two individual SFRSs were carried out independently of one another before being analyzed together. The results presented in this section indicate that this summation and check method was a valid and accurate means of determining the global drift of the system subjected to the full magnitude of the expected seismic loads.

Only the deflections along column line CL1 are presented in the graphs for this section. When both the column line deflections are presented, the graphs quickly become cluttered making it difficult to interpret the results. When the results are presented in a numerical table format, results for both column lines is presented.

Load Combination 5

The following data was tabulated using Load Combination 5 ($(1.2 + .2S_{DS})D + \rho E + 0.5L + 0.2S$) from the ASCE/SEI 7-10 where (1):

- S_{DS} = design, 5% damped, spectral response acceleration parameter at short periods
- D = dead load
- ρ = redundancy factor based on the extent of structural redundancy present in a building
- E = earthquake load
- L = live load
- S = snow load

The snow load did not govern for the building used in the parametric study, and thus snow load was not applied in the load combination.

In Figure 9, Figure 10, and Figure 11 lateral drifts of the independent systems and the dual system are presented. These three graphs provide a comparison of the different drifts along the column lines for the SFRSs studied.

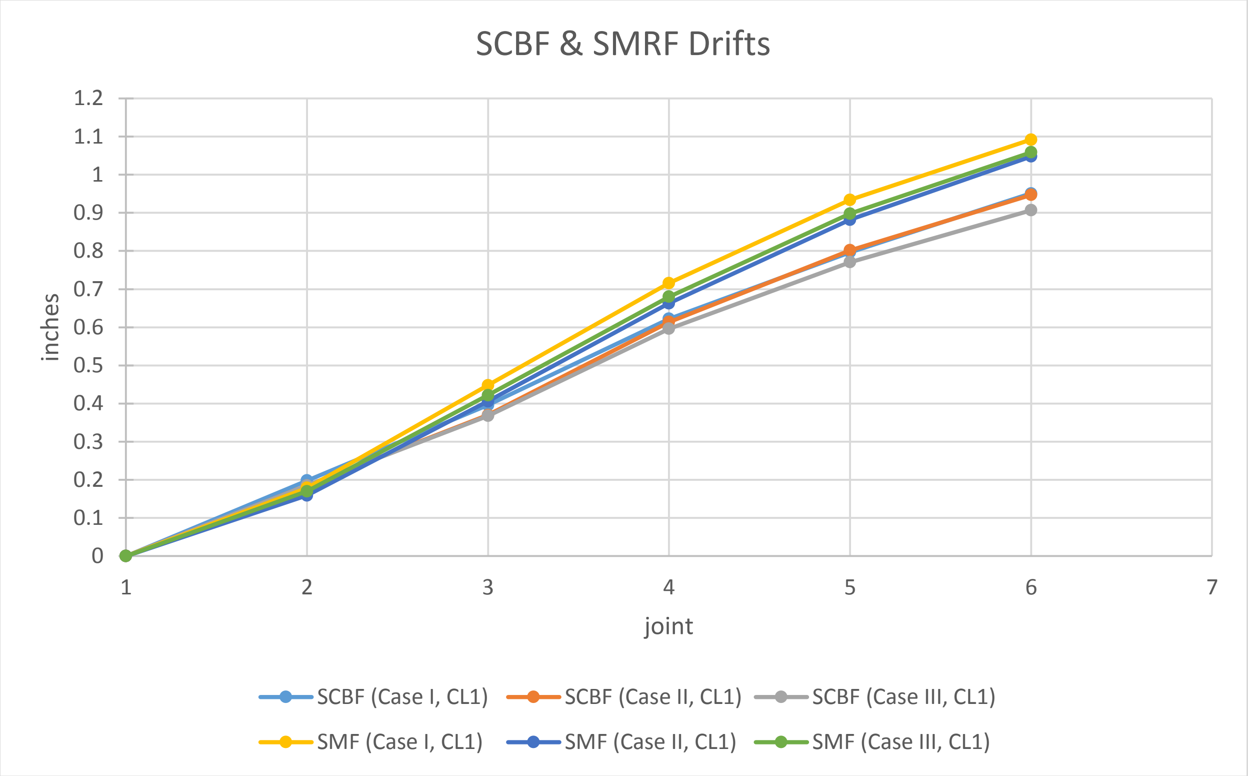


Figure 9: SCBF & SMRF Lateral Drifts (LC5)

The drifts pertaining to the designed SCBF and SMRF systems when subjected to load combination 5 are depicted in Figure 9. Joint 1 is located at the base of the column, joint 2 is located on the first elevated level (second floor), joint 3 is located on the second elevated level (third floor), joint 4 – fourth floor, joint five – fifth level, joint 6 is located at the roof level. The base of the columns for the SCBF are pinned and the base of the columns for the SMRF are fixed.

For the Case I designs, Figure 9 shows approximate deviations of: one fiftieth of an inch (0.02”) at the second story – one twentieth of an inch (0.05”) at the third story – one tenth of an inch (0.1”) at the fourth story – three twenty-fifths of an inch (0.12”) at the fifth story – seven fiftieths of an inch (0.14”) at the roof story.

For Case II designs, Figure 9 shows approximate deviations of: one fiftieth of an inch (0.02”) at the second story – three one-hundredths of an inch (0.03”) at the third story – one twentieth of an inch (0.05”) at the fourth story – two twenty-fifths of an inch (0.08”) at the fifth story – one tenth of an inch (0.1”) at the roof story.

For Case III designs, Figure 9 shows approximate deviations of: one one-hundredth of an inch (0.01”) at the second story – one twentieth of an inch (0.05”) at the third story – two twenty-fifths of an inch (0.08”) at the fourth story – thirteen one-hundredths of an inch (0.13”) at the fifth story – three twentieths of an inch (0.15”) at the roof story.

This data suggests that the designs pertaining to Case II behaved most similarly to one another. The designs pertaining to Case III behaved least similarly to one another here.

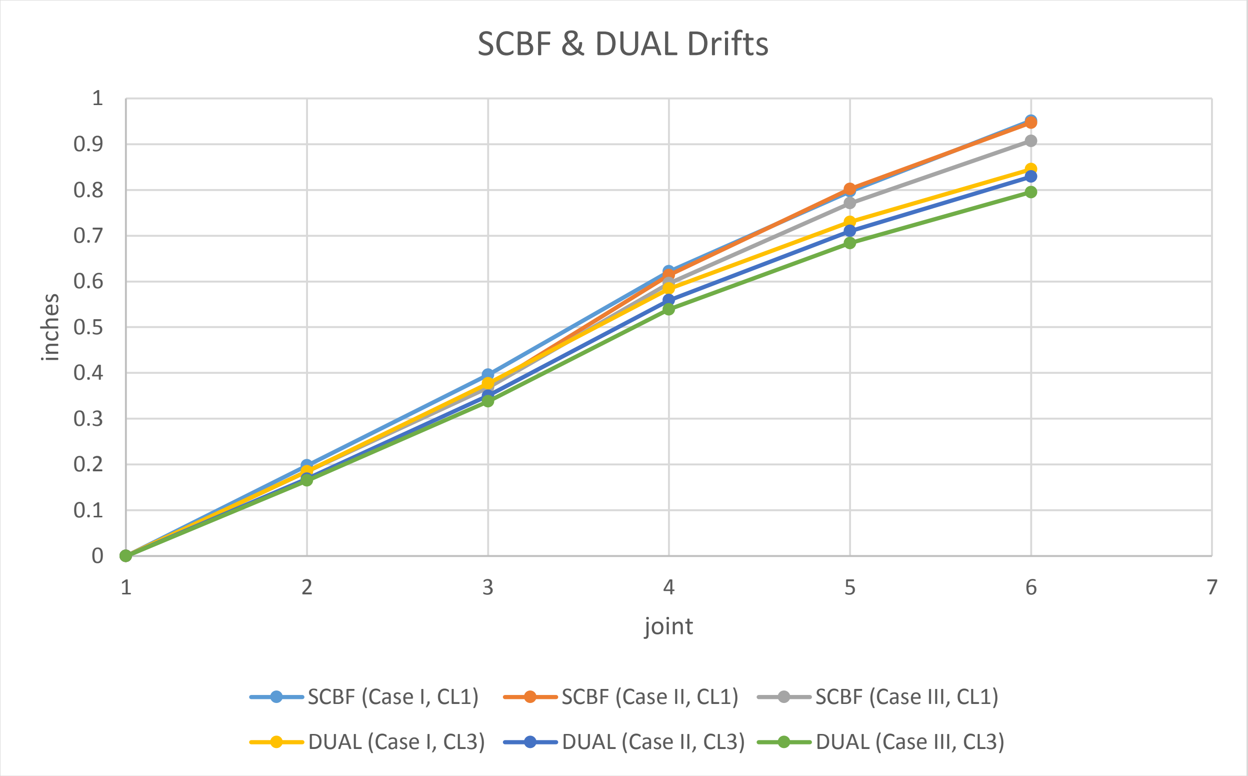


Figure 10: SCBF & DUAL Lateral Drifts (LC5)

Figure 10 displays the SCBF column line drifts with the SMRF-SCBF column line drifts. The closeness of these drifts is a sign that the designs of the SCBF systems accurately predicted the SCBF behavior within the SMRF-SCBF systems. The SMRF-SCBF system drifts trend to a lower maximum drift value than the SCBF systems. This indicates that there is additional stiffness in the SMRF-SCBF systems relative to the individual SCBF systems.

For the Case I designs, Figure 10 shows approximate deviations of: three two-hundredths of an inch (0.015") at the second story – one fiftieth of an inch (0.02") at the third story – one twenty-fifth of an inch (0.4") at the fourth story – three fiftieths of an inch (0.06") at the fifth story – one tenth of an inch (0.1") at the roof story.

For Case II designs, Figure 10 shows approximate deviations of: one one-hundredth of an inch (0.01") at the second story – one fiftieth of an inch (0.02") at the third story – one twentieth of an inch (0.05") at the fourth story – two twenty-fifths of an inch (0.08") at the fifth story – one tenth of an inch (0.1") at the roof story.

For Case III designs, Figure 10 shows approximate deviations of: three two-hundredths of an inch (0.015") at the second story – one fiftieth of an inch (0.02") at the third story – three fiftieths of an inch (0.06") at the fourth story – nine one-hundredths of an inch (0.09") at the fifth story – three twenty-fifths of an inch (0.12") at the roof story.

This data suggests that the designs pertaining to Case II behaved most similarly to one another. The designs pertaining to Case III behaved least similarly to one another here.

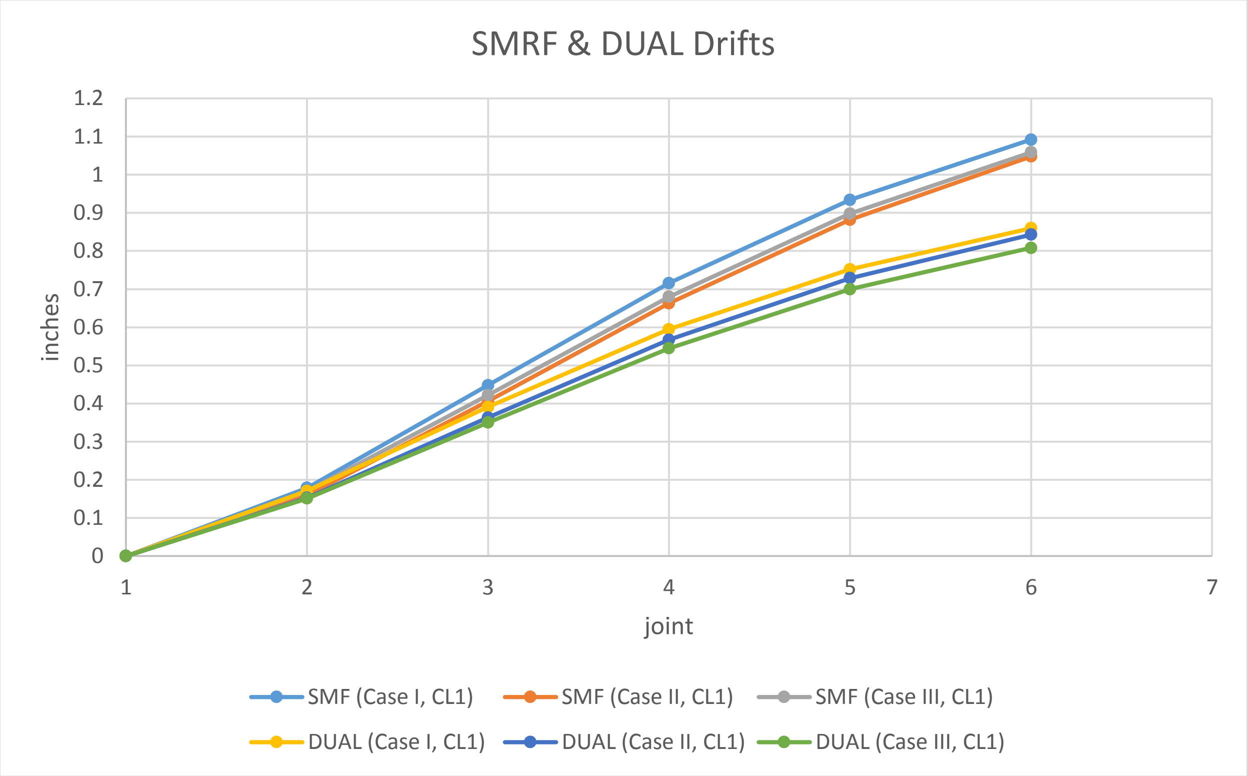


Figure 11: SMRF & DUAL Lateral Drifts (LC5)

Figure 11 shows SMRF-SCBF drifts with SMRF lateral drifts. Near the fourth story, some separation between the SMRF and SMRF-SCBF deflections begins to develop. The difference is largest at the sixth story where the magnitude of the difference is approximately a quarter of an inch (0.25"). This is a marginal difference for a five-story building subjected to high seismic forces.

For the Case I designs, Figure 11 shows approximate deviations of: one one-hundredth of an inch (0.01") at the second story – one twenty-fifth of an inch (0.04") at the third story – three twenty-fifths of an inch (0.12") at the fourth story – nine fiftieths of an inch (0.18") at the fifth story – one fourth of an inch (.25") at the roof story.

For Case II designs, Figure 11 shows approximate deviations of: less than one one-hundredth of an inch (0.01") at the second story – one twentieth of an inch (0.04") at the third story – nine hundredths of an inch (0.09”) at the fourth story – three twentieths of an inch (0.15”) at the fifth story – one fifth of an inch (0.2”) at the roof story.

For Case III designs, Figure 11 shows approximate deviations of: one one-hundredth of an inch (0.01”) at the second story – one twentieth of an inch (0.05”) at the third story – three twenty-fifths of an inch (0.12”) at the fourth story – one fifth of an inch (0.2”) at the fifth story – one quarter of an inch (0.25”) at the roof story.

This data suggests that the designs pertaining to Case II behaved most similarly to one another. The designs pertaining to Case III behaved least similarly to one another here.

Figure 9, Figure 10, Figure 11 are useful in evaluating the performance and behavior of the frames. However, these figures only address the lateral drifts of the SFRSs. Drifts are a primary indicator used in this study to evaluate the three designs’ accuracies; additional parameters can be of use. The overlapping that occurs within many of the figures can make it difficult to read and interpret many of the results from the study. For these reasons, the results of this portion of the study have been summarized in Table 4. In Table 4, horizontal drift is represented as the “X” direction and vertical deflection is represented as the “Y” direction.

Table 4: Summary of Elastic Drift Results (LC5)

COLUMN CL1

System	Parameter	Case I (80% - SCBF, 20% - SMRF)						Case II (75% - SCBF, 25% - SMRF)						Case III (70% - SCBF, 30% - SMRF)					
		Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof	Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof	Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof
SCBF	δ Lateral (in)	0	0.198	0.396	0.622	0.797	0.951	0	0.185	0.37	0.614	0.802	0.947	0	0.185	0.368	0.596	0.771	0.907
	δ Vertical (in)	0	-0.043	-0.064	-0.094	-0.107	-0.112	0	-0.046	-0.069	-0.1	-0.114	-0.119	0	-0.049	-0.075	-0.106	-0.121	-0.126
	% Deviation (X)	0.0%	7.0%	5.0%	6.5%	9.2%	12.5%	0.00%	9.47%	5.71%	9.84%	12.96%	14.23%	0.00%	12.12%	8.88%	10.58%	12.72%	14.09%
	% Deviation (Y)	0.0%	2.4%	4.9%	4.4%	4.9%	48.3%	0.00%	4.55%	4.55%	5.26%	5.56%	49.02%	0.00%	6.52%	8.70%	8.16%	8.04%	48.76%
DUAL (CL3)	δ Lateral (in)	0	0.185	0.377	0.584	0.73	0.845	0	0.169	0.35	0.559	0.71	0.829	0	0.165	0.338	0.539	0.684	0.795
	δ Vertical (in)	0	-0.042	-0.061	-0.09	-0.102	-0.107	0	-0.044	-0.066	-0.095	-0.108	-0.112	0	-0.046	-0.069	-0.098	-0.112	-0.116
SMRF	δ Lateral (in)	0	0.179	0.448	0.716	0.934	1.092	0	0.159	0.406	0.663	0.882	1.048	0	0.17	0.422	0.68	0.898	1.059
	δ Vertical (in)	0	-0.028	-0.046	-0.059	-0.066	-0.067	0	-0.022	-0.036	-0.046	-0.051	-0.052	0	-0.021	-0.034	-0.044	-0.049	-0.05
	% Deviation (X)	0.0%	4.7%	14.6%	20.3%	24.2%	27.0%	0.00%	3.92%	11.85%	16.93%	20.99%	24.32%	0.00%	12.58%	20.57%	24.77%	28.29%	31.06%
	% Deviation (Y)	0.0%	0.0%	-2.1%	-1.7%	-2.9%	-2.9%	0.00%	0.00%	-2.70%	-2.13%	-3.77%	-3.70%	0.00%	-4.55%	-5.56%	-4.35%	-5.77%	-5.66%
DUAL (CL1)	δ Lateral (in)	0	0.171	0.391	0.595	0.752	0.86	0	0.153	0.363	0.567	0.729	0.843	0	0.151	0.35	0.545	0.7	0.808
	δ Vertical (in)	0	-0.028	-0.047	-0.06	-0.068	-0.069	0	-0.022	-0.037	-0.047	-0.053	-0.054	0	-0.022	-0.036	-0.046	-0.052	-0.053

COLUMN CL2

System	Parameter	Case I (80% - SCBF, 20% - SMRF)						Case II (75% - SCBF, 25% - SMRF)						Case III (70% - SCBF, 30% - SMRF)					
		Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof	Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof	Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof
SCBF	δ Lateral (in)	0	0.198	0.414	0.622	0.799	0.951	0	0.185	0.391	0.615	0.804	0.947	0	0.186	0.388	0.596	0.773	0.907
	δ Vertical (in)	0	-0.13	-0.224	-0.275	-0.107	-0.314	0	-0.127	-0.22	-0.27	-0.114	-0.309	0	-0.125	-0.215	-0.265	-0.121	-0.303
	% Deviation (X)	0.00%	4.76%	7.25%	8.17%	9.18%	13.21%	0.00%	6.32%	8.31%	11.21%	12.96%	14.93%	0.00%	9.41%	10.54%	11.82%	12.72%	14.66%
	% Deviation (Y)	0.00%	49.43%	44.52%	48.65%	4.90%	48.82%	0.00%	49.41%	45.70%	49.17%	5.56%	50.00%	0.00%	50.60%	45.27%	48.88%	8.04%	50.00%
DUAL (CL4)	δ Lateral (in)	0	0.189	0.386	0.575	0.723	0.84	0	0.174	0.361	0.553	0.703	0.824	0	0.17	0.351	0.533	0.675	0.791
	δ Vertical (in)	0	-0.087	-0.155	-0.185	-0.209	-0.211	0	-0.085	-0.151	-0.181	-0.204	-0.206	0	-0.083	-0.148	-0.178	-0.201	-0.202
SMRF	δ Lateral (in)	0	0.181	0.448	0.716	0.933	1.089	0	0.16	0.406	0.663	0.882	1.046	0	0.171	0.422	0.68	0.898	1.057
	δ Vertical (in)	0	-0.035	-0.058	-0.073	-0.082	-0.084	0	-0.028	-0.047	-0.06	-0.067	-0.069	0	-0.029	-0.048	-0.061	-0.069	-0.07
	% Deviation (X)	0.0%	3.4%	15.2%	20.5%	24.7%	27.4%	0.00%	1.91%	12.15%	17.14%	21.49%	24.67%	0.00%	11.04%	20.92%	24.77%	28.65%	31.47%
	% Deviation (Y)	0.0%	2.9%	3.6%	1.4%	2.5%	2.4%	0.00%	0.00%	2.17%	3.45%	3.08%	2.99%	0.00%	3.57%	2.13%	3.39%	4.55%	2.94%
DUAL (CL2)	δ Lateral (in)	0	0.175	0.389	0.594	0.748	0.855	0	0.157	0.362	0.566	0.726	0.839	0	0.154	0.349	0.545	0.698	0.804
	δ Vertical (in)	0	-0.034	-0.056	-0.072	-0.08	-0.082	0	-0.028	-0.046	-0.058	-0.065	-0.067	0	-0.028	-0.047	-0.059	-0.066	-0.068

When calculating the percent deviations, the SMRF-SCBF systems are used as the base value; the equation used to calculate the percent deviations is shown below. A positive percentage indicates that the SCBF or SMRF had a greater deflection than the SMRF-SCBF at that same joint. Likewise, a negative percentage indicates that the SCBF or SMRF had a smaller deflection than the SMRF-SCBF at that same joint. This means that a positive percent deviation indicates conservative design. Conservative results suggest that the dual SFRS showed more stiffness than the design accounted for. While this is still an inaccuracy, conservative results, especially where the deviation is small, are not as concerning as non-conservative results are. Low percentages are an indication that the design accurately predicted the behavior of the relevant frame.

$$\%\ Deviation\ (X\ or\ Y) = \frac{\delta_{(SCBF\ or\ SMF)} - \delta_{SMR-CBF}}{\delta_{SMR-CBF}}$$

When examining the percent deviations, the magnitudes of the deflections should be considered. This is crucial when evaluating the results since some of the deflections are less than one tenth of an inch (0.1”). For example, the vertical percent deviation at the second story (Lvl. 2) of the SCBF under load Case I is close to fifty percent (50%). Alone, this percent deviation would indicate a large inaccuracy between the SCBF and SMRF-SCBF. However, considering the magnitudes show an approximate difference of only one twentieth of an inch (0.05”). This indicates that despite the relatively high percent deviation, the frames’ responses are nearly identical.

The vertical percent deviations for column CL-2 of the SCBFs are generally much greater in value compared to the vertical percent deviations for column CL-1. This trend is attributed to the difference in the vertical loads column CL-2 is subjected to between the SCBF RISA 3D models and the SMRF-SCBF RISA 3D models. For this reason, the vertical deflections and percent deviations pertaining to column CL-1 in the SCBFs are more valid compared to these same data pertaining to column CL-2.

Load Combination 7

The following data was tabulated using Load Combination 7 ((.9 - .2S_{DS})DL + ρE) from the ASCE/SEI 7-10 where (1):

- S_{DS} = design, 5% damped, spectral response acceleration parameter at short periods
- D = dead load
- ρ = redundancy factor based on the extent of structural redundancy present in a building
- E = earthquake load

In the figures below (Figure 12, Figure 13, and Figure 14), lateral deflections of the independent systems and the dual system are presented. These three graphs provide a side-by-side comparison of the different deflections along the column lines for the SFRSs analyzed.

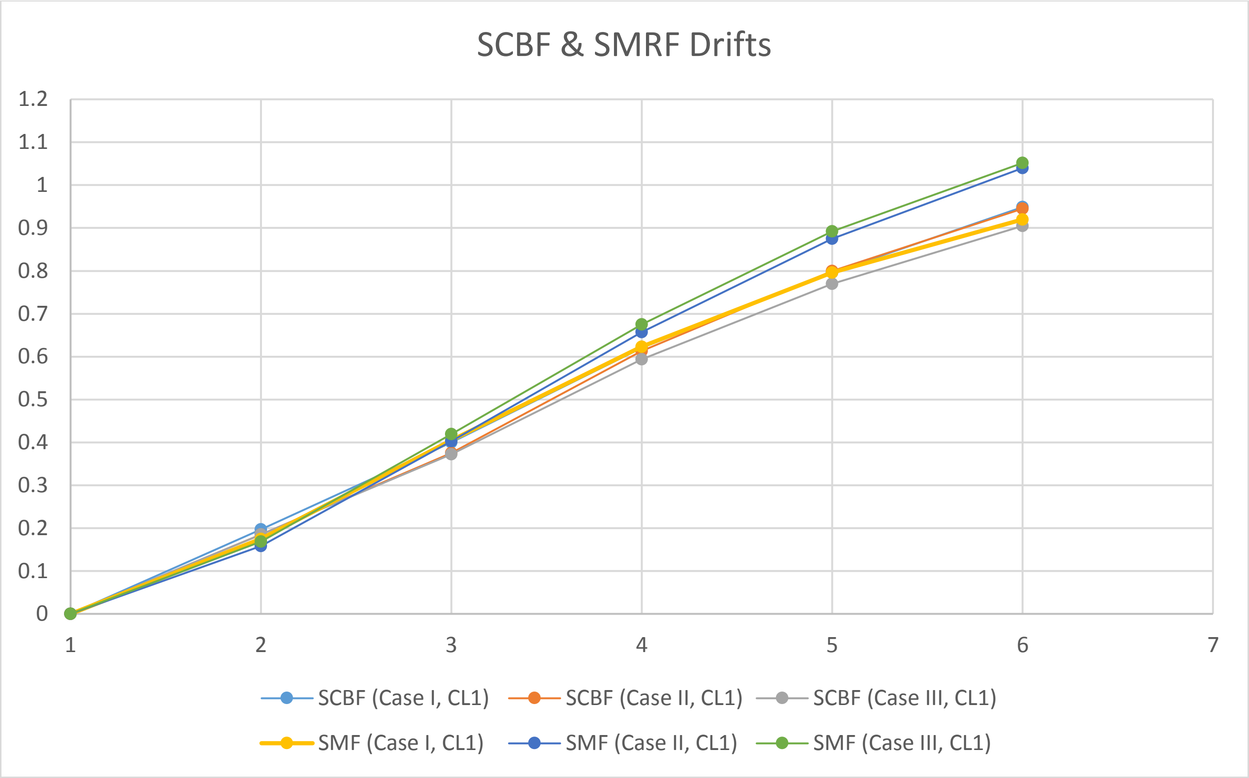


Figure 12: SCBF & SMRF Lateral Drifts (LC7)

Figure 12 displays the lateral deflections for the SCBFs and the SMRFs. As expected, the SCBFs consistently deflect less than the SMRFs. The difference between the SMRFs' and the SCBFs' collective deflections at any given story is not of significant magnitude.

For the Case I designs, Figure 12 shows approximate deviations of: one fiftieth of an inch (0.02") at the second story – one twenty-fifth of an inch (0.04") at the third story – nine one-hundredths of an inch (0.09") at the fourth story – thirteen one-hundredths of an inch (0.13") at the fifth story – seven fiftieths of an inch (0.14") at the roof story.

For Case II designs, Figure 12 shows approximate deviations of: one fiftieth of an inch (0.02") at the second story – three one-hundredths of an inch (0.03") at the third story – one twenty-fifth of an inch (0.04") at the fourth story – seven one-hundredths of an inch (0.07") at the fifth story – nine one-hundredths of an inch (0.09") at the roof story.

For Case III designs, Figure 12 shows approximate deviations of: one one-hundredth of an inch (0.01”) at the second story – one twenty-fifth of an inch (0.04”) at the third story – seven one-hundredths of an inch (0.07”) at the fourth story – three twenty-fifths of an inch (0.12”) at the fifth story – three twentieths of an inch (0.15”) at the roof story.

This data suggests that the designs pertaining to Case II behaved most similarly to one another. The designs pertaining to Case I behaved least similarly to one another here.

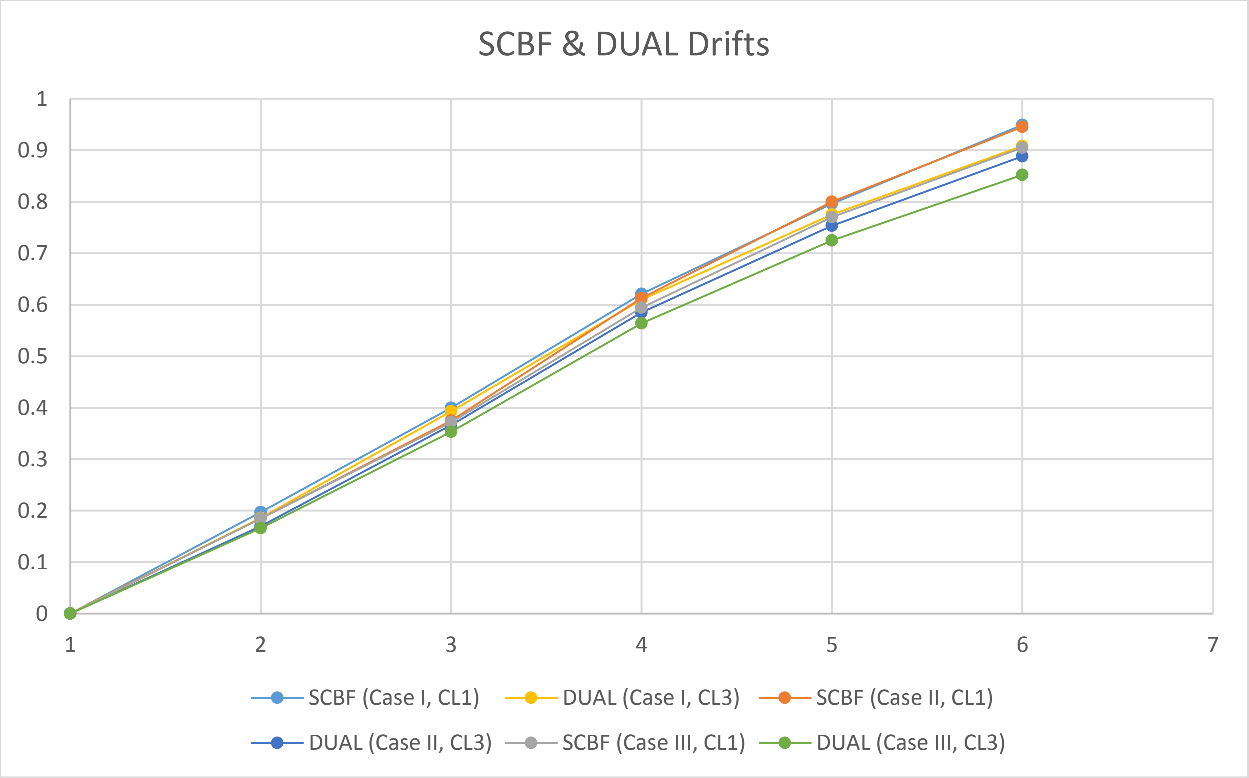


Figure 13: SCBF & DUAL Lateral Drifts (LC7)

Figure 13 shows the SCBF deflections and the SMRF-SCBF deflections alongside one another. Similar to Figure 10, the SCBFs consistently deflect a slightly greater amount than the SMRF-SCBFs. This small disparity indicates that the SCBFs that are integrated in the SMRF-SCBFs are behaving slightly stiffer than they were assumed to during design.

For the Case I designs, Figure 13 shows approximate deviations of: one one-hundredth of an inch (0.01”) at the second story – less than one one-hundredth of an inch (0.01”) at the third story – one one-hundredth of an inch (0.01”) at the fourth story – one fiftieth of an inch (0.02”) at the fifth story – one twenty-fifth of an inch (.04”) at the roof story.

For Case II designs, Figure 13 shows approximate deviations of: one one-hundredth of an inch (0.01”) at the second story – less than one one-hundredth of an inch (0.01”) at the third story – one fiftieth of an inch (0.02”) at the fourth story – one twenty-fifth of an inch (0.04”) at the fifth story – one twentieth of an inch (0.05”) at the roof story.

For Case III designs, Figure 13 shows approximate deviations of: one one-hundredth of an inch (0.01”) at the second story – one one-hundredth of an inch (0.01”) at the third story – one fiftieth of an inch (0.02”) at the fourth story – one twenty-fifth of an inch (0.04”) at the fifth story – one twentieth of an inch (0.05”) at the roof story.

This data suggests that the designs pertaining to Case I behaved most similarly to one another. The designs pertaining to Case III behaved least similarly to one another here.

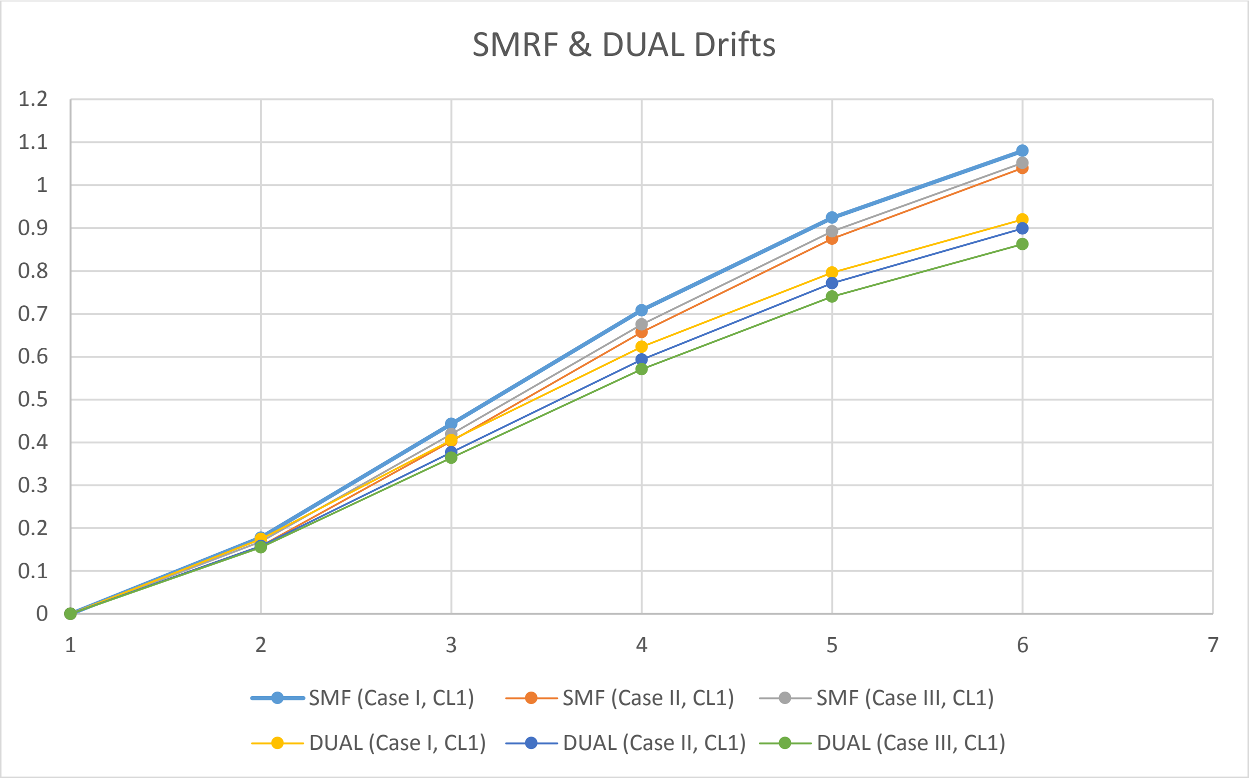


Figure 14: SMRF & DUAL Lateral Drifts (LC7)

The lateral deflections for the SMRFs and the SMRF-SCBFs are graphed in Figure 14. As was the case for Load Combination 5's results, the deflections of the independent SMRFs are greater than the SMRF-SCBFs' deflections. While the differences in deflections are small, Figure 14 shows the largest disparities between any individual SFRSs and the correlating SMRF-SCBF systems.

For the Case I designs, Figure 14 shows approximate deviations of: less than one one-hundredth of an inch (0.01") at the second story – three one-hundredths of an inch (0.03") at the third story – two twenty-fifths of an inch (0.08") at the fourth story – thirteen one-hundredths of an inch (0.13") at the fifth story – three twentieths of an inch (0.15") at the roof story.

For Case II designs, Figure 14 shows approximate deviations of: less than one one-hundredth of an inch (0.01") at the second story – one fiftieth of an inch (0.02") at the third story

– one twentieth of an inch (0.05”) at the fourth story – one one-hundredth of an inch (0.01”) at the fifth story – three twenty-fifths of an inch (0.15”) at the roof story.

For Case III designs, Figure 14 shows approximate deviations of: one one-hundredth of an inch (0.01”) at the second story – one twenty-fifth of an inch (0.04”) at the third story – one tenth of an inch (0.1”) at the fourth story – three twenty-fifths of an inch (0.15”) at the fifth story – nine fiftieths of an inch (0.18”) at the roof story.

This data suggests that the designs pertaining to Case II behaved most similarly to one another. The designs pertaining to Case III behaved least similarly to one another here.

As discussed in Load Combination 5, the preceding figures (Figure 12, Figure 13, and Figure 14) are useful, but an alternative presentation of this data is desirable. Furthermore, additional parameters can be useful in evaluating the accuracy and performance of the SFRSs. Both of these demands are satisfied in Table 5 below.

Table 5: Summary of Elastic Drift Results (LC7)

COLUMN CL1

System	Parameter	Case I (80% - SCBF, 20% - SMRF)						Case II (75% - SCBF, 25% - SMRF)						Case III (70% - SCBF, 30% - SMRF)					
		Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof	Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof	Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof
SCBF	δ Lateral (in)	0	0.197	0.4	0.621	0.796	0.949	0	0.185	0.375	0.613	0.8	0.945	0	0.185	0.372	0.594	0.77	0.905
	δ Vertical (in)	0	0.003	0.012	0.004	0.003	0	0	0	0.007	-0.002	-0.004	-0.006	0	-0.003	0.002	-0.008	-0.01	-0.013
	% Deviation (X)	0.0%	5.9%	1.8%	1.8%	2.7%	4.5%	0.00%	8.82%	2.46%	4.79%	6.24%	6.42%	0.00%	11.45%	5.38%	5.32%	6.21%	6.22%
	% Deviation (Y)	0.0%	-25.0%	-25.0%	-50.0%	-57.1%	NA	0.00%	NA	-36.36%	-166.7%	-300.0%	NA	0.00%	NA	-75.00%	NA	400.00%	160.00%
DUAL (CL3)	δ Lateral (in)	0	0.186	0.393	0.61	0.775	0.908	0	0.17	0.366	0.585	0.753	0.888	0	0.166	0.353	0.564	0.725	0.852
	δ Vertical (in)	0	0.004	0.016	0.008	0.007	0.005	0	0.002	0.011	0.003	0.002	0	0	0	0.008	0	-0.002	-0.005
SMRF	δ Lateral (in)	0	0.178	0.443	0.708	0.924	1.08	0	0.158	0.403	0.657	0.875	1.04	0	0.169	0.419	0.675	0.892	1.052
	δ Vertical (in)	0	-0.011	-0.018	-0.024	-0.027	-0.027	0	-0.008	-0.014	-0.018	-0.02	-0.02	0	-0.007	-0.012	-0.016	-0.018	-0.019
	% Deviation (X)	0.0%	1.7%	9.4%	13.6%	16.1%	17.4%	0.00%	0.00%	6.90%	10.79%	13.49%	15.68%	0.00%	9.03%	15.11%	18.21%	20.54%	22.04%
	% Deviation (Y)	0.0%	-8.3%	-5.3%	-4.0%	-3.6%	-6.9%	0.00%	-11.11%	-6.67%	-5.26%	-4.76%	-9.09%	0.00%	-12.50%	-14.29%	-11.11%	-10.00%	-9.52%
DUAL (CL1)	δ Lateral	0	0.175	0.405	0.623	0.796	0.92	0	0.158	0.377	0.593	0.771	0.899	0	0.155	0.364	0.571	0.74	0.862
	δ Vertical (in)	0	-0.012	-0.019	-0.025	-0.028	-0.029	0	-0.009	-0.015	-0.019	-0.021	-0.022	0	-0.008	-0.014	-0.018	-0.02	-0.021

COLUMN CL2

System	Parameter	Case I (80% - SCBF, 20% - SMRF)						Case II (75% - SCBF, 25% - SMRF)						Case III (70% - SCBF, 30% - SMRF)					
		Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof	Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof	Lvl. 1	Lvl. 2	Lvl. 3	Lvl. 4	Lvl. 5	Roof
SCBF	δ Lateral (in)	0	0.197	0.409	0.621	0.797	0.949	0	0.185	0.385	0.613	0.802	0.945	0	0.185	0.381	0.594	0.771	0.905
	δ Vertical (in)	0	-0.083	-0.147	-0.177	-0.199	-0.202	0	-0.081	-0.143	-0.172	-0.193	-0.196	0	-0.078	-0.138	-0.166	-0.187	-0.19
	% Deviation (X)	0.00%	4.23%	3.54%	3.16%	2.71%	4.98%	0.00%	6.32%	4.34%	6.06%	6.24%	6.90%	0.00%	9.47%	6.72%	6.64%	6.21%	6.72%
	% Deviation (Y)	0.00%	29.69%	26.72%	30.15%	30.07%	30.32%	0.00%	30.65%	27.68%	30.30%	30.41%	31.54%	0.00%	30.00%	26.61%	29.69%	29.86%	31.03%
DUAL (CL4)	δ Lateral (in)	0	0.189	0.395	0.602	0.768	0.904	0	0.174	0.369	0.578	0.745	0.884	0	0.169	0.357	0.557	0.716	0.848
	δ Vertical (in)	0	-0.064	-0.116	-0.136	-0.153	-0.155	0	-0.062	-0.112	-0.132	-0.148	-0.149	0	-0.06	-0.109	-0.128	-0.144	-0.145
SMRF	δ Lateral (in)	0	0.179	0.443	0.708	0.924	1.079	0	0.159	0.403	0.658	0.875	1.038	0	0.169	0.419	0.675	0.892	1.051
	δ Vertical (in)	0	-0.018	-0.03	-0.038	-0.043	-0.044	0	-0.015	-0.025	-0.032	-0.036	-0.037	0	-0.016	-0.026	-0.033	-0.037	-0.039
	% Deviation (X)	0.0%	0.6%	9.7%	13.8%	16.5%	17.7%	0.00%	-0.63%	7.18%	10.96%	13.78%	15.72%	0.00%	7.64%	15.43%	18.21%	20.70%	22.21%
	% Deviation (Y)	0.0%	0.0%	3.4%	2.7%	4.9%	2.3%	0.00%	0.00%	4.17%	3.23%	5.88%	5.71%	0.00%	6.67%	4.00%	3.13%	5.71%	8.33%
DUAL (CL2)	δ Lateral (in)	0	0.178	0.404	0.622	0.793	0.917	0	0.16	0.376	0.593	0.769	0.897	0	0.157	0.363	0.571	0.739	0.86
	δ Vertical (in)	0	-0.018	-0.029	-0.037	-0.041	-0.043	0	-0.015	-0.024	-0.031	-0.034	-0.035	0	-0.015	-0.025	-0.032	-0.035	-0.036

Generalities such as the significance of the numbers, how the deviation percentages were calculated, and keys to reading Table 5 are the same as they were for Table 4. For this information, please refer to the discussion directly under Table 4.

There are joints within the frames (that are not the column bases) which RISA 3D calculated no vertical deflection. RISA 3D calculates deflection values to the nearest one one-thousandth of an inch (.001”). This means that where RISA displays zero for the deflection, the calculated deflection is less than one two-thousandth of an inch (.0005”). The margin for error this rounding leaves is insignificant to the results of this study. Where the deflection was calculated to be zero, the percent deviation calculation cannot be used to draw any conclusions from the results. Instead, one must consider only the magnitudes of the deflections and the difference between them.

For a majority of the results, there is a greater variance in the performances and behaviors between the individual systems’ (SCBF or SMRF) and the SMRF-SCBF system’s in the results for load combination 7 compared to load combination 5. This trend is attributed to the reduction in gravity loads in load combination 7 relative to load combination 5. The additional gravity loads from load combination 5 dampen the frame, diminishing disparities between the SFRSs.

Column Base Reactions

This section analyzes results of the column base reactions of the various SFRSs from the study. The column base reactions of interest for this study are the shear, axial, and moment reactions.

It has been proven that when a structure is subjected to loading of any type, the forces will distribute themselves within that structure according to the stiffness’ of the elements that make up the structure - the stiffest elements in a structure will take the majority of the load, and the least stiff elements will take much less of the load. Forces will distribute within a structure directly proportional to the stiffness' of the structure's elements.

The SMRF and SCBF for each Case have been designed to take a quantified percentage of the lateral loads. When the systems are analyzed together in the same model, the forces should distribute proportional to the stiffness of the structural elements. The column base reactions of the models used for design and the reactions of the combined models can be compared to check the accuracy of the design's intended force distribution.

For Deflections, the relevant load combinations from the ASCE/SEI 7-10 were used. This was done to attain practical results that would be applicable for design purposes. For this section, no gravity loads will be considered, and only the seismic loads are applied. Gravity loads would introduce additional forces and because the parameters being discussed in this section are forces and not deflections, gravity forces would make interpreting the seismic force distribution within the frames more challenging than necessary.

Table 6 provides a summary of column base reactions for the study.

Table 6: Column Base Reactions (Seismic Loads Only)

COLUMN BASE REACTIONS

Case	Joint Label	Shear (k)					Axial (k)					Moment (k-ft)			
		DUAL	SCBF	SMRF	Deviation (k)	Deviation (%)	DUAL	SCBF	SMRF	Deviation (k)	Deviation (%)	DUAL	SMRF	Deviation (k-ft)	Deviation (%)
Case I	J1	-18.5		-16.2	2.3	-12.5%	-26.6		-29.5	-3.0	11.2%	240.3	229.1	11.2	-4.6%
	J7	-19.1		-16.1	3.0	-15.7%	26.6		29.5	-3.0	11.1%	244.7	229.1	15.7	-6.4%
	J13	-61.7	-64.7		-3.0	4.9%	-181.2	-179.1		2.1	-1.1%				
	J19	-62.1	-64.4		-2.3	3.7%	181.2	179.1		2.1	-1.1%				
Case II	J1	-23.2		-20.2	3.0	-12.8%	-32.2		-35.3	-3.1	9.7%	326.5	309.6	16.9	-5.2%
	J7	-24.1		-20.1	3.9	-16.3%	32.2		35.3	-3.1	9.7%	332.8	309.6	23.2	-7.0%
	J13	-56.8	-60.7		-3.8	6.8%	-169.7	-167.9		1.8	-1.1%				
	J19	-57.3	-60.4		-3.1	5.3%	169.7	167.9		1.8	-1.1%				
Case III	J1	-24.9		-24.3	0.6	-2.5%	-38.4		-44.5	-6.0	15.7%	328.8	340.4	-11.7	3.5%
	J7	-25.5		-24.2	1.3	-5.3%	38.4		44.5	-6.0	15.7%	333.4	340.4	-7.0	2.1%
	J13	-55.2	-56.6		-1.4	2.6%	-163.4	-156.7		6.7	-4.1%				
	J19	-55.8	-56.4		-0.5	1.0%	163.4	156.7		6.7	-4.1%				

The grayed cells in Table 6 represent non-applicable data points.

The deviations for Table 6 were calculated using absolute values. Because of this, the sign of the percent deviations has the same implications as the tables with the deflection results; a positive percent deviation represents a conservative design, and a visa versa for a negative percent deviation. The percent deviations were calculated using the same equation presented in the discussion under Table 4.

The same rules concerning conservative or non-conservative design do not apply for the magnitude-based deviations in Table 6. Instead, when evaluating these deviations, a positive value represents a non-conservative design, and a negative value represents a conservative design.

In the context of evaluating the results presented in Table 6 and future tables in thesis, a conservative design indicates that the reaction observed through the model used for design was of a greater magnitude than the reaction observed through the combined dual frame model. A non-conservative design indicates that the reaction observed through the design model was of a lesser magnitude than the reaction observed through the combined dual frame model.

In an effort to assist in reading Table 6, Case I, joint seven's (7) row (second row down from the column headings) will be broken down column by column. Starting from the left, the first column shows a value of -19.1. This is the shear value at joint seven (7) that was measured in the combined model. The next column is a greyed out cell which is insignificant and can be skipped over. The next column to the right shows a value of -16.1. This is the shear value at joint seven (7) that was measured in the individual model for the SMRF. The next column to the right shows a value of 3.0. This is the absolute difference between the two shear values that were presented in the columns to the left of this one (19.1 – 16.1 = 3.0). The next column to the right displays -15.7%. This is the percentage of deviation between the two shear values of the combined model and the individual model at joint seven (7). The next column over shows a value of 26.6. This is the axial load value at joint seven (7) that was measured in the combined model. The next column is a greyed out cell which is insignificant and can be skipped over. The next column to the right shows a value of 29.5. This is the axial load value at joint seven (7) that was measured in the individual model for the SMRF. The next column to the right shows a value of -3.0. This is the absolute difference between the two axial load values that were presented in the column to the left of this one (26.6 – 29.5 = -3.0, note that there is rounding occurring here). The next column displays 11.1%. This is the percentage of deviation between the two axial load values from the combined model and the individual model at joint seven (7). The next column over shows a value of 244.7. This is the moment value at joint seven (7) that was measured in the combined model. The next column is a greyed out cell which is insignificant and can be skipped over. The next column to the right shows a value of 229.1. This is the moment value at joint seven (7) that was measured in the individual model for the SMRF. The column next over shows a value of 15.7. This is the absolute difference between the two moment values that were presented in the column to the left of this one (244.7 – 229.1 = 15.7, note that there is rounding occurring here). The last column displays -6.4%. This is the percentage of deviation between the two moment values from the combined model and the individual model at joint seven (7).

These results suggest that the designs for Case III are the most accurate, and the designs for Case II are the least accurate. All of the Cases' designs produced satisfactory results, with the greatest percent deviations being around 10% or less.

Much of these results present non-conservative deviations for the SMRF designs with the largest percent deviation from Table 6 being 16.3% for the shear at joint seven (J7). This is of little concern, since the SMRF designs were all governed by drift and not capacity. The columns of the SFMs for every Case (joints J1 and J7) have approximately 80% - 85% spare capacity after checking combined loading, and even more spare capacity concerning shear.

The minor inaccuracies presented here should not impact the design of the SFRS members. The connection designs are more susceptible to being affected by the inaccuracies shown in Table 6. Failures such as flange or web local buckling, shear yielding, connecting element strength limitations, etc. could all occur where the results reflect a largely non-conservative design. This is far more likely to be an issue with the SCBF column design than the SMRF column design. This is because the SMRF columns are particularly robust due to the serviceability limitations for the design. The SCBF columns and connections should be designed for the maximum deliverable force from the braces and/or beams. If the member designs of the braces and/or beams remains unchanged and the SCBF columns have been properly designed, the likely hood that the column or connection will experience any type of failure should remain unaffected.

Brace Axial Loads

This section analyzes the axial loads carried by the bracing within the SCBF and the SMRF-SCBF systems.

For the same reasons discussed in Column Base Reactions, only the seismic loads were applied to attain the results for this section. The axial loads shown are the loads induced into the frames by the seismic design force without the overstrength factor applied. The reasons that brace axial loads are of interest relate back to the stiffness and force distribution discussion located toward the beginning of Column Base Reactions. Please see Table 7 below for the brace axial load data.

Table 7: Brace Axial Loads (Seismic Loads Only)

BRACE AXIAL LOADS (k)

Member Label	Case I				Case II				Case III			
	DUAL	SCBF	Deviation (k)	Deviation (%)	DUAL	SCBF	Deviation (k)	Deviation (%)	DUAL	SCBF	Deviation (k)	Deviation (%)
BR1	-84.7	-88.6	-3.9	4.6%	-78.1	-83.0	-5.0	6.4%	-75.8	-77.5	-1.7	2.3%
BR2	85.6	88.6	-3.0	3.6%	79.0	83.1	-4.0	5.1%	76.9	77.5	-0.6	0.8%
BR3	78.6	74.8	3.8	-4.8%	73.9	70.5	3.4	-4.6%	71.3	65.7	5.6	-7.9%
BR4	-77.6	-74.9	2.8	-3.6%	-72.9	-70.6	2.3	-3.2%	-70.1	-65.8	4.4	-6.2%
BR5	-61.5	-60.5	1.0	-1.6%	-57.3	-56.3	1.0	-1.7%	-54.8	-52.7	2.1	-3.8%
BR6	63.7	60.5	3.2	-5.0%	59.5	56.3	3.2	-5.4%	57.2	52.7	4.6	-8.0%
BR7	43.8	40.8	3.0	-6.8%	41.5	38.2	3.3	-8.0%	39.8	35.6	4.2	-10.5%
BR8	-41.5	-40.8	0.7	-1.7%	-39.2	-38.1	1.1	-2.8%	-37.2	-35.6	1.6	-4.4%
BR9	-12.5	-13.7	-1.2	9.8%	-13.5	-13.1	0.5	-3.6%	-12.7	-12.2	0.5	-4.1%
BR10	12.8	13.7	-0.9	6.8%	13.8	13.1	0.8	-5.6%	13.0	12.2	0.9	-6.5%

Refer to discussion under Table 6 to appropriately interpret the deviations.

In an effort to assist in reading Table 7, brace BR3's row (third row down from the column headings) will be broken down column by column. Starting on the left, the first column shows a value of 78.6. This is the axial load that brace BR3 in Case I's design developed in the combined model. The next column over shows a value of 74.8. This is the axial load that brace BR3 in Case I's design developed in the individual model. The next column to the right shows a value of 3.8. This is the absolute difference between the two values in the columns to the left (78.6 – 74.8 = 3.8). The next column over displays -4.8%. This is the percentage of deviation between the two axial load values from the combined model and the individual model for brace BR3 for Case I's design. The next column to the right shows a value of 73.9. This is the axial load that brace BR3 in Case II's design developed in the combined model. The next column to the right shows a value of 70.5. This is the axial load that brace BR3 in Case II's design developed in the individual model. The next column shows a value of 3.4. This is the absolute difference between the two values in the columns to the left (71.3 – 65.7 = 3.4). The next column displays -4.6%. This is the percentage of deviation between the two axial load values from the combined model and the individual model for brace BR3 for Case II's design. The next column to the right shows a value of 71.3. This is the axial load that brace BR3 in Case III's design developed in the combined model. The next column to the right shows a value of 65.7. This is the axial load that brace BR3 in Case III's design developed in the individual model. The next column over shows a value of 5.6. This is the absolute difference between the two values in the columns to the left (71.3 – 65.7 = 5.6). The next column displays -7.9%. This is the percentage of deviation between the two axial load values from the combined model and the individual model for brace BR3 for Case III's design.

The deviations pertaining to Case II designs are the most consistent, while the deviations pertaining to Case I seem to be the most overall accurate. All three Cases' designs produced good accuracy, with the largest percent deviation being -10.5% for BR7 for the Case III design. Remember that for this thesis, accuracy is defined as how closely the individual SFRS results match the dual SFRS results.

The negative deviations in Table 7 are slightly concerning since the sizes of the braces for all three designs were based off the member capacities. It is not a surprise that the deviations tend to reflect a non-conservative design since SCBFs are inherently much stiffer than SMRFs. Because the SCBF is stiffer, the bracing will take most all of the loading until either it loses stiffness via inelastic deformations or the drift becomes large enough that the SMRF engages. The largest magnitude of deviation is only 5.6 kips correlating to a percent deviation of -7.9%. This value is not concerning when the magnitude of the axial load is considered. The dual frame model showed an axial load magnitude of 71.3 kips compared to 65.7 kips from the individual model. The magnitudes of deviations in Table 7 are never of a great enough magnitude to be concerning, and are unlikely to influence the braces' designs.

Joint Moments

This section analyzes the internal moments at the joints within the SMRF and the SMRF-SCBF systems. The only joints within the SMRF-SCBF systems that have a non-zero moment are the joints associated with the SMRF portion of the system.

For the same reasons discussed in Column Base Reactions, only the seismic loads were applied to attain the results for this section. The reasons that joint moments are of interest relate back to the stiffness and force distribution discussion located towards the beginning of Column Base Reactions. See Table 8 below for the data concerning the joint moments data.

Table 8: Joint Moments (Seismic Loads Only)

JOINT MOMENTS (k-ft)

Joint Label	**Case I**				**Case II**				**Case III**			
	DUAL	SMRF	Deviation (k-ft)	Deviation (%)	DUAL	SMRF	Deviation (k-ft)	Deviation (%)	DUAL	SMRF	Deviation (k-ft)	Deviation (%)
J2	-102.2	-106.8	-4.6	4.5%	-120.6	-123.5	-2.9	2.4%	-150.1	-165.7	-15.6	10.4%
J3	-109.9	-123.2	-13.3	12.1%	-135.7	-148.2	-12.5	9.2%	-166.6	-193.5	-26.9	16.1%
J4	-86.1	-98.5	-12.5	14.5%	-99.4	-111.6	-12.2	12.3%	-117.5	-138.4	-20.9	17.8%
J5	-64.5	-73.3	-8.8	13.7%	-76.0	-87.2	-11.2	14.7%	-89.0	-105.8	-16.8	18.9%
J6	-36.7	-41.2	-4.4	12.1%	-51.5	-59.5	-8.0	15.5%	-53.5	-63.6	-10.1	18.9%
J8	102.0	106.8	-4.8	4.7%	120.5	123.5	-2.9	2.4%	150.0	165.7	-15.7	10.4%
J9	108.8	123.2	-14.4	13.3%	135.3	148.2	-12.9	9.5%	166.3	193.5	-27.2	16.4%
J10	86.0	98.5	-12.6	14.6%	99.3	111.6	-12.2	12.3%	117.5	138.4	-20.9	17.8%
J11	64.5	73.3	-8.8	13.6%	76.0	87.2	-11.2	14.7%	88.9	105.8	-16.8	18.9%
J12	37.0	41.2	-4.2	11.4%	51.7	59.5	-7.9	15.2%	53.6	63.6	-10.0	18.7%

Refer to discussion under Table 6 to appropriately interpret the deviations.

Every single deviation in Table 8 reflects conservative design. The predominantly conservative results from Table 8 are a product of the predominantly non-conservative results from Table 7. This again relates back to SCBFs being much stiffer systems than SMRFs.

In an effort to assist in reading Table 8, joint three's (3) row (second row down from the column headings) will be broken down column by column. Starting on the left, the first column shows a value of -109.9. This is the moment that developed at joint three (3) for Case I's design in the combined model. The next column to the right shows a value of -123.2. This is the moment that developed at joint three (3) for Case I's design in the individual SMRF model. The next column shows a value of -13.3. This is the absolute difference between the two values shown in the column to the left (109.9 – 123.2 = -13.3). The next column displays 12.1%. This is the percentage of deviation between the two moment values from the combined model and the individual model for joint three (3) for Case I's design. The next column to the right shows a value of -135.7. This is the moment that developed at joint three (3) for Case II's design in the combined model. The next column over shows a value of -148.2. This is the moment that developed at joint three (3) for Case II's design in the individual SMRF model. The next column shows a value of -12.5. This is the absolute difference between the two values shown in the column to the left (135.7 – 148.2 = -12.5). The next column to the right displays 9.2%. This is the percentage of deviation between the two moment values from the combined model and the individual model for joint three (3) for Case II's design. The next column shows a value of -166.6. This is the moment that developed at joint three (3) for Case III's design in the combined model. The next column to the right shows a value of -193.5. This is the moment that developed at joint three (3) for Case III's design in the individual SMRF model. The next column shows a value of -26.9. This is the absolute difference between the two values shown in the column to the left (166.6 – 193.5 = -26.9). The last column displays 16.1%. This is the percentage of deviation between the two moment values from the combined model and the individual model for joint three (3) for Case III's design.

The results presented in Table 8 suggest that the SMRF designed for Case II is the most accurate of the three designs. The SMRF pertaining to Case I also shows very accurate results with just slightly greater deviations than the SMRF designed for Case II. The Case III designs had the greatest differences regardless of the joint, with a maximum deviation of approximately 27 kip-ft. By comparison, the largest deviation for Cases I and II were approximately 14 kip-ft and 12 k-ft, respectively. Even if the deviations for the SMRFs reflect conservative design, large deviations still represent an inaccuracy, which should be slightly concerning.

Internal Story Shears

This section analyzes the internal story shears in the columns within the SMRF and the SMRF-SCBF systems. Internal story shears of only the SMRF columns within the SMRF-SCBF systems were analyzed.

The internal story shears shown are the induced column forces from application of the seismic design force without the overstrength factor applied. Internal story shears should show a story-by-story breakdown of the force distribution within the SMRF-SCBF. These values are then compared against the predicted internal story shears from the design models. The reason this comparison is of interest relates back to the stiffness and force distribution discussion located toward the beginning of Column Base Reactions.

Table 9: Internal Story Shears Deviations

STORY SHEARS DEVIATIONS

Level	**Case I**						**Case II**						**Case III**					
	CL-1		CL-2		Total		CL-1		CL-2		Total		CL-1		CL-2		Total	
	V (k)	V (%)	V (k)	V (%)	V (k)	V (%)	V (k)	V (%)	V (k)	V (%)	V (k)	V (%)	V (k)	V (%)	V (k)	V (%)	V (k)	V (%)
2	-2.3	-12.5%	-3.0	-15.7%	-5.3	-14.1%	-3.0	-12.9%	-3.9	-16.3%	-6.9	-14.6%	-0.6	-2.6%	-1.3	-5.2%	-2.0	-3.9%
3	2.4	19.4%	-3.4	-18.9%	-1.0	-3.4%	2.1	13.0%	3.1	20.3%	5.2	16.5%	3.8	20.8%	4.5	25.9%	8.3	23.3%
4	1.8	17.9%	1.5	14.2%	3.3	16.0%	1.6	12.4%	1.2	8.4%	2.8	10.3%	2.7	17.7%	2.3	14.8%	5.0	16.2%
5	1.2	17.5%	1.4	21.9%	2.6	19.6%	1.5	18.5%	1.8	22.9%	3.4	20.6%	2.1	21.5%	2.3	24.3%	4.4	22.9%
Roof	0.1	3.8%	-0.1	-3.0%	0.0	0.3%	0.6	24.2%	0.5	17.6%	1.1	20.8%	0.7	20.3%	0.6	16.1%	1.2	18.2%

The deviations presented in Table 9 show an interesting trend at and below the second story. A strong majority of the deviations reflect a non-conservative result. This is an expected result because of the stiffness differences between the SMRF and SCBF systems. This trend does not continue at the second story and below though. A potential cause for this phenomena is the fixed bases of the SMRF columns. As the column is subjected to lateral loads, it deflects accordingly. This deflection leads to column rotation and a change in slope of the column line. This rotation is not allowed at the base of the column though due to the fixed base. This effectively increases the stiffness of the column near its base. Thus, an increase in the columns' stiffnessess towards their bases results in an increase in the magnitude of forces the columns attract. The reasoning behind this argument relates back to the stiffness and force distribution discussion located toward the beginning of Column Base Reactions.

The data presented in Table 9 suggests that Case I's design produced the most accurate results. Case II's design resulted in more consistent results while Case III's design's accuracy varied widely.

The magnitudes of the story shears are presented in Table 10 below.

Table 10: Internal Story Shears

STORY SHEARS

Case	Level	DUAL				SMRF			
		V (k), CL1	V (k), CL2	V (k), Total	V_{base} (%)	V (k), CL1	V (k), CL2	V (k), Total	V_{base} (%)
Case I	2	18.5	19.2	37.6	11.7%	16.2	16.2	32.3	10.0%
	3	12.3	18.1	30.4	9.4%	14.7	14.7	29.3	9.1%
	4	10.1	10.4	20.6	6.4%	11.9	11.9	23.9	7.4%
	5	6.7	6.5	13.2	4.1%	7.9	7.9	15.8	4.9%
	R	2.6	2.7	5.3	1.6%	2.7	2.7	5.3	1.6%
Case II	2	23.2	24.1	47.3	14.6%	20.2	20.2	40.4	12.5%
	3	16.2	15.2	31.4	9.7%	18.3	18.3	36.6	11.3%
	4	13.3	13.7	27.0	8.4%	14.9	14.9	29.8	9.2%
	5	8.4	8.1	16.4	5.1%	9.9	9.9	19.8	6.1%
	R	2.7	2.8	5.5	1.7%	3.3	3.3	6.6	2.0%
Case III	2	24.9	25.6	50.4	15.6%	24.2	24.2	48.4	15.0%
	3	18.2	17.5	35.7	11.0%	22.0	22.0	44.0	13.6%
	4	15.2	15.6	30.8	9.5%	17.9	17.9	35.8	11.1%
	5	9.8	9.5	19.3	6.0%	11.9	11.9	23.7	7.4%
	R	3.3	3.4	6.7	2.1%	4.0	4.0	7.9	2.5%

The percent of the total base shear (V_{base} (%)) is the primary parameter of interest in Table 10. The SMRF total base shear percentages are the percentage of the base shear that the SMRF frame was designed to resist at each level. The DUAL total base shear percentages are the percentage of the base shear that the SMRF columns actually experienced when analyzed in the combined SMRF-SCBF model. Table 10 serves as an alternative presentation of the data presented in Table 9. Because of this, the results presented in Table 10 suggest the same accuracies as Table 9: Case I's design was the most accurate, Case III's design was the least accurate, and Case II's design produced the most consistent results.

Chapter 4 - Conclusion

This chapter highlights the main points of this research before further discussing the results of the parametric study and drawing conclusions pertaining to the results.

The aim of this research was to investigate the behavior and load distribution of a dual SMRF-SCBF SFRS. The ASCE/SEI 7-10 mandates that all dual LFRSs must include an MRF capable of resisting at least 25% of the lateral loads. The study performed for this thesis aimed to determine if the independent SFRSs which make up the dual SFRS behave as they are designed to.

If how a structure is idealized to behave and how it actually behaves are different, the structural integrity of the building is jeopardized. For this reason, it is crucial to any SFRS design that the structure is accurately designed and analyzed. In the case of the SMRF-SCBF system investigated in this thesis, deflections, column base reactions, brace axial loads, the moments occurring at joints, and the story shears within the SMRF columns of the designed SFRSs were compared to measure the design process's accuracy.

Of the SMRFs, the results for the SMRF designed for the Case II load distribution most closely aligns with the results from the relevant SMRF-SCBF. There are instances in which the SMRFs for Case I or III are more accurate. In most instances, the SMRF designed for Case III was the least accurate of the SMRFs.

The SCBFs' accuracies were generally more closely grouped than the SMRFs'. The SCBF designed for Case I appears to be the most accurate of the three SCBFs, while the SCBF pertaining to Case II appears to still be an accurate design and analysis. The SCBF designed for load distribution Case III seemed to be the least accurate of the three designs.

The presented results indicate that the Case II load distribution (SCBF – 75%, SMRF – 25%) most accurately predicts the structure's global response. The frames designed for the Case II load distribution produced the most consistent and accurate results.

Recommendations

The findings of this study show that the most accurate force distribution used for designing the SFRS coincides with the current code-mandated force distribution of 75%-25%. For this reason, it is recommended that designers use a 75%-25% force distribution when designing a dual LFRS. Furthermore, keeping at least one bay of separation between the SCBF and the SMRF is recommended. Providing at least one bay of separation between the two frames prevents additional design conflicts concerning column size, connection design and constructability, column base connection design, etc. from coming to fruition.

A large number of variables and directions within this research could influence the results and conclusions of the study. In light of this, there are many recommendations for future research in this area.

Studying more finely tuned designs could be an area of interest. This would mean varying the sizes of the braces of the SCBF and the beams of the SMRF at every level instead of grouping the 2nd and 3rd stories together and the 4th and 5th stories together. If column splices were considered, this would present an opportunity to possibly reduce the column size. These design steps could drive the design to be more economical. Additionally, the more economical design could result in smoother results whereas the results from this study, at times, show minor jumps in deflections between the 3rd and 4th stories.

Experimenting with additional load distributions for design could help industry have a better understanding of how the two frames are interacting with one another. The load distributions for this study were selected around the minimum capacity for the MRF within a dual LFRS. There is no reason this same system could not be designed with an even 50% distribution of the lateral forces to each frame. Furthermore, the load distribution could be flipped such that the MRF is designed to take a majority of the lateral load. This type of load distribution would be likely to result in a less economical design solution since MRF design is virtually always governed by serviceability and not capacity. For that reason, distributions between the 80% - 20% distribution and a 50% - 50% distribution are more highly recommended for future research.

Studying force distributions and design accuracy in a dual LFRS that does not include a SCBF is a recommended area of further research in this area. Different LFRS have different inherent stiffnesses due to their means of energy dissipation and/or configuration. There are many types of shear wall systems that are permitted to be included in a dual LFRS along with several brace frame types. Some examples of these systems include steel eccentric brace frames, special reinforced concrete shear walls, steel and concrete composite special concentric brace frames, etc. (1). In addition to researching an entirely different dual LFRS, different brace configurations could impact the results of this study. The two-story X brace pattern is inherently the stiffest SCBF configuration allowed by building code. Other configurations such as a zipper, chevron, or V configuration would alter the inherent stiffness of the SCBF potentially influencing the findings of this study.

Designing the MRF portion of the dual LFRS before designing the accompanying LFRS could result in different findings. The design presented in this thesis focused on balancing applicability and economics of the system. This led to the SCBF being designed prior to the

SMRF. Designing the MRF first could produce useful results purely intended to analyze the force distribution within a dual LFRS if applicability of the design is not of interest.

An applicable area of additional research is the connection design and what impact it may have on the rest of the system. For this particular study, the SMRF columns were large enough that additional reinforcement (continuity plates, etc.) was unnecessary for the limited instances that were checked. The only design that connections were at all checked for was Case I. However, the connections for the SCBF were not checked at all and these connections are more likely to impact the design. Connection design of the SCBF could result in different beam and column sizes. Even if member sizes remain constant, additional reinforcement (web stiffeners, etc.) will likely be necessary. This could have a considerable impact on the economics of the design.

Lastly, scaled testing of the designed frames is recommended for further research in this field. Computer models are fantastic for analyzing a frame or structure and trying to understand how it will behave. However, without scaled testing, the results and findings of the study are still theoretical in nature. It is for this reason that scaled testing of fully designed dual LFRSs is highly recommended.

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Appendix A - Load Calculations

GRAVITY LOADS

ALL REFERENCES PULLED FROM ASCE 7-10

Step	Computations			Reference
Find DL for Roofs	Roof			
	Materials	Weights		
	five-ply felt and gravel	6	psf	Tbl. C3-1
	2" fiberboard insulation	3	psf	Tbl. C3-1
	1.5 B20 type deck	2.5	psf	Vulcraft Manual
	Framing [beams(30'), girders(25')]	6	psf	Assumed
	Acoustic Ceiling	3	psf	Tbl. C3-1
	MEP	5	psf	Tbl. C3-1
	Miscellaneous	2	psf	Assumed
	DL =	**30**	psf	
Find DL for Floors	Floors			
	Materials	Weights		
	Floor finish	1	psf	Tbl. C3-1
	Steel deck and fill [5"(1.5VL20), NWC]	56	psf	Vulcraft Manual
	Framing [beams(30'), girders(25')]	6	psf	Assumed
	Acoustic Ceiling	3	psf	Tbl. C3-1
	MEP	5	psf	Tbl. C3-1
	Partition walls	10	psf	Tbl. C3-1
	Miscellaneous	2	psf	Assumed
	DL =	**85**	psf	
Find DL for Walls	Walls			
	Materials	Weights		
	Curtain wall system	20	psf	Tbl. C3-1
	DL =	**20**	psf	
Find Live Loads	Live Loads			
	LL_{ROOF} =	20	psf	Tbl. 4.1
	LL_{FLOOR} =	80	psf	Tbl. 4.1

SEISMIC LOADS

4/21/2017	ARE 899 - Master's Thesis	Christopher Wearing
	PAGE 1	ALL REFERENCES PULLED FROM ASCE 7-10
Step	Computations	Reference
Site information pretaining to seismic	Site Class: D	Assumed/given
	S_S = 0.626 g	USGS Design Maps
	S_1 = 0.184 g	
	I_e = 1	Tbl. 1.5-2
	h = 62 ft	
	Select System: All other systems	
	T_L = 6 s	Fig. 22-(12-16)
	R = 7	Tbl. 12.2-1
	Ω = 2.5	
	C_d = 5.5	
	L_{NS} = 75 ft	
	L_{EW} = 120 ft	
Site coefficients	F_a = 1.299	Tbl. 11.4-1
	F_v = 2.066	Tbl. 11.4-2
Spectral response acceleration paramerters	S_{MS} = 0.813 g	Eq. 11.4-1
	S_{M1} = 0.380 g	Eq. 11.4-2
Design spectral response acc. para.	S_{DS} = 0.542 g	Eq. 11.4-3
	S_{D1} = 0.253 g	Eq. 11.4-4
Seismic design category	SDC = D	Tbl. 11.6-1 Tbl. 11.6-2
Approximate fundamental period	C_t = 0.02	Tbl. 12.8-2
	x = 0.75	
	T_a = 0.442 s	Eq. 12.8-7
Seismic Response Coeefficient	(EQUATION) C_s = 0.077 USE THIS VALUE	Eq. 12.8-2
	(CHECK) C_s = 0.082 DO NOT USE THIS VALUE	Eq. 12.8-(3 or 4)
	(CHECK) C_s = 0.024 DO NOT USE THIS VALUE	Eq. 12.8-5
	(CHECK) C_s = 0.013 DO NOT USE THIS VALUE	Eq. 12.8-6
	C_s = 0.0774	

PAGE 2

ALL REFERENCES PULLED FROM ASCE 7-10

Step	Computations	Reference

Effective Seismic Weight

Level	DL_f (psf)	W_f (k)	h_f (ft)	W_W (k)	W_x (k)
Roof	30	270	10	78	348
5th floor	85	765	12	93.6	858.6
4th floor	85	765	12	93.6	858.6
3rd floor	85	765	12	93.6	858.6
2nd floor	85	765	13	101.4	866.4

A_f = 9000 ft^2

DL_w = 20 psf

L_w = 390 ft

Base shear

V= 293.5 kips 234.8261 — Eq. 12.8-1

Vertical Distribution

Level	W_x (k)	h_x (ft)	$h^k_x W_x$(k-ft)	C_{vx}	F_x (k)	V_x (k)
Roof	348	62	21576	0.164	48.1	48.1
5th floor	858.6	50	42930	0.326	95.8	143.9
4th floor	858.6	38	32627	0.248	72.8	216.7
3rd floor	858.6	26	22324	0.170	49.8	266.5
2nd floor	866.4	14	12130	0.092	27.1	293.5

Eq. 12.8-12

Eq. 12.8-13

k = 1 — Sect. 12.8.3

Torsional Shear

e_{NS} = 3.75 ft

e_{EW} = 6 ft

Sect. 12.8.4.2

Level	Direction	M (k-ft)	V_T (k)
Roof	N/S	180.5	2.41
	E/W	288.8	2.41
5th floor	N/S	359.1	4.79
	E/W	574.6	4.79
4th floor	N/S	272.9	3.64
	E/W	436.7	3.64
3rd floor	N/S	186.7	2.49
	E/W	298.8	2.49
2nd floor	N/S	101.5	1.35
	E/W	162.3	1.35
		Total:	29.35

ALL REFERENCES PULLED FROM ASCE 7-10

Step | Computations | Reference

Combined System Shear Distributions

Level	V_D (k)	Direction	V_T (k)	$V_{comb.}$ (k)
Roof	24.1	N/S	2.41	26.5
		E/W	2.41	26.5
5th floor	47.9	N/S	4.79	52.7
		E/W	4.79	52.7
4th floor	36.4	N/S	3.64	40.0
		E/W	3.64	40.0
3rd floor	24.9	N/S	2.49	27.4
		E/W	2.49	27.4
2nd floor	13.5	N/S	1.35	14.9
		E/W	1.35	14.9
Checks:	293.5		29.4	322.9

Frame Forces

Case I - SCBF 80% - MRF 20%

Level	Direction	V_{SCBF} (k)	V_{MRF} (k)
Roof	N/S	21.2	5.3
	E/W	21.2	5.3
5th floor	N/S	42.1	10.5
	E/W	42.1	10.5
4th floor	N/S	32.0	8.0
	E/W	32.0	8.0
3rd floor	N/S	21.9	5.5
	E/W	21.9	5.5
2nd floor	N/S	11.9	3.0
	E/W	11.9	3.0
Checks:		258.3	64.6

Case II - SCBF 75% - MRF 25%

Level	Direction	V_{SCBF} (k)	V_{MRF} (k)
Roof	N/S	19.9	6.6
	E/W	19.9	6.6
5th floor	N/S	39.5	13.2
	E/W	39.5	13.2
4th floor	N/S	30.0	10.0
	E/W	30.0	10.0
3rd floor	N/S	20.5	6.8
	E/W	20.5	6.8
2nd floor	N/S	11.2	3.7
	E/W	11.2	3.7
Checks:		242.2	80.7

ALL REFERENCES PULLED FROM ASCE 7-10

Step	Computations	Reference

Frame Forces cont.

Case III - SCBF 70% - MRF 30%

Level	Direction	V_{SCBF} (k)	V_{MRF} (k)
Roof	N/S	18.5	7.9
	E/W	18.5	7.9
5th floor	N/S	36.9	15.8
	E/W	36.9	15.8
4th floor	N/S	28.0	12.0
	E/W	28.0	12.0
3rd floor	N/S	19.2	8.2
	E/W	19.2	8.2
2nd floor	N/S	10.4	4.5
	E/W	10.4	4.5
Checks:		226.0	96.9

Additional Loads

Roof

P_{DL} = 13.7 k

P_{LL} = 7.5 k

Floor

P_{DL} = 39.1 k

P_{LL} = 30.0 k

See Gravity Sheets

Redundancy Factor

ρ = 1.0

Sect. 12.3.4.2

Combined DL Factor

$1.2+.2S_{DS}$ = 1.308 LC 5

$.9+.2S_{DS}$ = 0.792 LC 7

Sect. 12.4.2.3 &
Sect. 12.4.3.2

ALL REFERENCES PULLED FROM ASCE 7-10

Step	Computations	Reference

Story Drifts

Case I

GOOD	Level	h_x (ft)	From Model		Δ_{design} (in)	Δ_{all} (in)
			δ_x (in)	Δ_x (in)		
	Roof	10	0.96	0.15	0.83	2.40
	5th floor	12	0.80	0.17	0.92	2.88
	4th floor	12	0.64	0.22	1.23	2.88
	3rd floor	12	0.41	0.21	1.16	2.88
	2nd floor	14	0.20	0.20	1.11	3.36

Case II

GOOD	Level	h_x (ft)	From Model		Δ_{design} (in)	Δ_{all} (in)
			δ_x (in)	Δ_x (in)		
	Roof	10	0.97	0.15	0.84	2.40
	5th floor	12	0.82	0.19	1.05	2.88
	4th floor	12	0.63	0.24	1.34	2.88
	3rd floor	12	0.38	0.20	1.08	2.88
	2nd floor	14	0.19	0.19	1.03	3.36

Case III

GOOD	Level	h_x (ft)	From Model		Δ_{design} (in)	Δ_{all} (in)
			δ_x (in)	Δ_x (in)		
	Roof	10	0.93	0.14	0.79	2.40
	5th floor	12	0.79	0.18	0.97	2.88
	4th floor	12	0.61	0.22	1.22	2.88
	3rd floor	12	0.39	0.20	1.09	2.88
	2nd floor	14	0.19	0.19	1.03	3.36

Appendix B - Special Concentric-Brace Frame Design

BRACES

PAGE 1

Step	Computations	Reference

Loads

CASE I - 80% SCBF - 20% MRF

Level	P_t (k)	P_c (k)	$P_{E,T}$ (k)
Roof	11.3	17.6	13.7
5th floor	42.3	44.4	42.8
4th floor	55.2	78.3	62.6
3rd floor	65.2	99.3	75.7
2nd floor	73.2	124.1	89.3

RISA Model CASE I

CASE II - 75% SCBF - 25% MRF

Level	P_t (k)	P_c (k)	$P_{E,T}$ (k)
Roof	10.5	16.9	12.9
5th floor	36.8	40.0	36.7
4th floor	48.2	72.3	54.9
3rd floor	59.2	95.0	69.8
2nd floor	66.1	118.5	82.3

RISA Model CASE II

CASE III - 70% SCBF - 30% MRF

Level	P_t (k)	P_c (k)	$P_{E,T}$ (k)
Roof	9.6	16.1	11.9
5th floor	36.9	37.4	37.2
4th floor	47.3	68.6	54.8
3rd floor	55.7	88.3	66.3
2nd floor	62.0	111.2	78.1

RISA Model CASE III

Uniform Member Properties

F_y = 42 ksi

L_{BAY} = 30 ft

$h_{BR,3\text{-}R}$ = 12 ft

$h_{BR,2}$ = 14 ft

$L_{BR,3\text{-}R}$ = 19.2 ft

$L_{BR,2}$ = 20.5 ft

R_y = 1.4

$n_{stories}$ = 5

α = 1

AISC Manual Tbl. 2-4

AISC Sesmic Prov. Tbl. A3.1

AISC Manual App. 8.2

Step	Computations	Reference

Members and their Properties

CASE I - 80% SCBF - 20% MRF

Level	Section (round HSS)	t_{des} (in)	A_g (in^2)	D/t	I (in^4)	r (in)
Roof	4.00x.226	0.21	2.50	19.0	4.5	1.34
5th floor	6.00x.312	0.291	5.22	20.6	21.3	2.02
4th floor	6.00x.312	0.291	5.22	20.6	21.3	2.02
3rd floor	7.50x.312	0.291	6.59	25.8	42.9	2.55
2nd floor	7.50x.312	0.291	6.59	25.8	42.9	2.55

AISC Manual Tbl. 1-13

CASE II - 75% SCBF - 25% MRF

Level	Section (round HSS)	t_{des} (in)	A_g (in^2)	D/t	I (in^4)	r (in)
Roof	4.00x.220	0.205	2.44	19.5	4.41	1.34
5th floor	6.00x.280	0.26	4.69	23.1	19.3	2.03
4th floor	6.00x.280	0.26	4.69	23.1	19.3	2.03
3rd floor	7.50x.312	0.291	6.59	25.8	42.9	2.55
2nd floor	7.50x.312	0.291	6.59	25.8	42.9	2.55

AISC Manual Tbl. 1-13

CASE III - 70% SCBF - 30% MRF

Level	Section (round HSS)	t_{des} (in)	A_g (in^2)	D/t	I (in^4)	r (in)
Roof	4.00x.220	0.205	2.44	19.5	4.41	1.34
5th floor	6.00x.250	0.233	4.22	25.8	17.6	2.04
4th floor	6.00x.250	0.233	4.22	25.8	17.6	2.04
3rd floor	7.00x.312	0.291	6.13	24.1	34.6	2.37
2nd floor	7.00x.312	0.291	6.13	24.1	34.6	2.37

AISC Manual Tbl. 1-13

Step	Computations	Reference

Check width-to-thickness ratio & $.3V_{story} > P_{H\text{-}E,T} > .7V_{story}$

CASE I - 80% SCBF - 20% MRF

Level	D/t	λ_{hd}	Check	$P_{E,T}$ (k)	V_{story} (k)	Check
Roof	19	26.2	GOOD	13.7	21.2	GOOD
5th floor	20.6	26.2	GOOD	42.8	63.3	GOOD
4th floor	20.6	26.2	GOOD	62.6	95.3	GOOD
3rd floor	25.8	26.2	GOOD	75.7	117.2	GOOD
2nd floor	25.8	26.2	GOOD	89.3	129.2	GOOD

AISC Seismic Prov.
Sect. F2.5a &
Sect. F2.4a

CASE II - 75% SCBF - 25% MRF

Level	D/t	λ_{hd}	Check	$P_{E,T}$ (k)	V_{story} (k)	Check
Roof	19.5	26.2	GOOD	12.9	19.9	GOOD
5th floor	23.1	26.2	GOOD	36.7	59.4	GOOD
4th floor	23.1	26.2	GOOD	54.9	89.4	GOOD
3rd floor	25.8	26.2	GOOD	69.8	109.9	GOOD
2nd floor	25.8	26.2	GOOD	82.3	121.1	GOOD

AISC Seismic Prov.
Sect. F2.5a &
Sect. F2.4a

CASE III - 70% SCBF - 30% MRF

Level	D/t	λ_{hd}	Check	$P_{E,T}$ (k)	V_{story} (k)	Check
Roof	19.5	26.2	GOOD	11.9	18.5	GOOD
5th floor	25.8	26.2	GOOD	37.2	55.4	GOOD
4th floor	25.8	26.2	GOOD	54.8	83.4	GOOD
3rd floor	24.1	26.2	GOOD	66.3	102.6	GOOD
2nd floor	24.1	26.2	GOOD	78.1	113.0	GOOD

AISC Seismic Prov.
Sect. F2.5a &
Sect. F2.4a

Check Slenderness

CASE I - 80% SCBF - 20% MRF

Level	KL/r	Limit	Check
Roof	172.0	200	GOOD
5th floor	114.1	200	GOOD
4th floor	114.1	200	GOOD
3rd floor	90.4	200	GOOD
2nd floor	96.6	200	GOOD

AISC Seismic Prov.
Sect. F2.5b(1)

CASE II - 75% SCBF - 25% MRF

Level	KL/r	Limit	Check
Roof	172.0	200	GOOD
5th floor	113.6	200	GOOD
4th floor	113.6	200	GOOD
3rd floor	90.4	200	GOOD
2nd floor	96.6	200	GOOD

AISC Seismic Prov.
Sect. F2.5b(1)

CASE III - 70% SCBF - 30% MRF

Level	KL/r	Limit	Check
Roof	172.0	200	GOOD
5th floor	113.0	200	GOOD
4th floor	113.0	200	GOOD
3rd floor	97.3	200	GOOD
2nd floor	103.9	200	GOOD

AISC Seismic Prov.
Sect. F2.5b(1)

Step	Computations	Reference

Calculate B_2
CASE I

CASE I - 80% SCBF - 20% MRF

ROOF

P_{mf} = 0 k
P_{story} = 394.1 k
R_M = 1
H = 42.4 k
L = 12 ft
Δ_H = 0.15 in
$P_{e\ story}$ = 40391 k
B_2 = 1.01

5th FLOOR

P_{mf} = 0 k
P_{story} = 1877.5 k
R_M = 1
H = 126.6 k
L = 12 ft
Δ_H = 0.17 in
$P_{e\ story}$ = 108538 k
B_2 = 1.02

4th FLOOR

P_{mf} = 0 k
P_{story} = 3360.9 k
R_M = 1
H = 190.7 k
L = 12 ft
Δ_H = 0.22 in
$P_{e\ story}$ = 122577 k
B_2 = 1.03

3rd FLOOR

P_{mf} = 0 k
P_{story} = 4844.3 k
R_M = 1
H = 234.5 k
L = 12 ft
Δ_H = 0.21 in
$P_{e\ story}$ = 160798 k
B_2 = 1.03

2nd FLOOR

P_{mf} = 0 k
P_{story} = 6327.7 k
R_M = 1
H = 258.3 k
L = 14 ft
Δ_H = 0.20 in
$P_{e\ story}$ = 214831 k
B_2 = 1.03

AISC Manual
App. 8.2 &
Eq. A-8-8

Frame Forces

Story Drifts

AISC Manual
Eq. A-8-7 &
Eq. A-8-6

AISC Manual
App. 8.2 &
Eq. A-8-8

Frame Forces

Story Drifts

AISC Manual
Eq. A-8-7 &
Eq. A-8-6

AISC Manual
App. 8.2

Eq. A-8-8

Frame Forces

Story Drifts

AISC Manual
App. 8.2 &
Eq. A-8-8

Step	Computations	Reference

Calculate B_2
CASE II

CASE II - 75% SCBF - 25% MRF

ROOF

P_{mf} = 0 k
P_{story} = 394.1 k
R_M = 1
H = 39.7 k
L = 12 ft
Δ_H = 0.15 in
$P_{e\ story}$ = 37618 k
B_2 = 1.01

5th FLOOR

P_{mf} = 0 k
P_{story} = 1877.5 k
R_M = 1
H = 118.7 k
L = 12 ft
Δ_H = 0.19 in
$P_{e\ story}$ = 89972 k
B_2 = 1.02

AISC Manual
App. 8.2 &
Eq. A-8-8

Frame Forces

Story Drifts

AISC Manual
Eq. A-8-7 &
Eq. A-8-6

4th FLOOR

P_{mf} = 0 k
P_{story} = 3360.9 k
R_M = 1
H = 178.8 k
L = 12 ft
Δ_H = 0.24 in
$P_{e\ story}$ = 105931 k
B_2 = 1.03

3rd FLOOR

P_{mf} = 0 k
P_{story} = 4844.3 k
R_M = 1
H = 219.8 k
L = 12 ft
Δ_H = 0.20 in
$P_{e\ story}$ = 160696 k
B_2 = 1.03

AISC Manual
App. 8.2 &
Eq. A-8-8

Frame Forces

Story Drifts

AISC Manual
Eq. A-8-7 &
Eq. A-8-6

2nd FLOOR

P_{mf} = 0 k
P_{story} = 6327.7 k
R_M = 1
H = 242.2 k
L = 14 ft
Δ_H = 0.19 in
$P_{e\ story}$ = 217559 k
B_2 = 1.03

AISC Manual
App. 8.2

Eq. A-8-8

Frame Forces

Story Drifts

AISC Manual
App. 8.2 &
Eq. A-8-8

Step | Computations | Reference

Calculate B_2
CASE III

CASE III - 70% SCBF - 30% MRF

ROOF

P_{mf} = 0 k
P_{story} = 394.1 k
R_M = 1
H = 37.1 k
L = 12 ft
Δ_H = 0.14 in
$P_{e\ story}$ = 37319 k
B_2 = 1.01

5th FLOOR

P_{mf} = 0 k
P_{story} = 1877.5 k
R_M = 1
H = 110.8 k
L = 12 ft
Δ_H = 0.18 in
$P_{e\ story}$ = 90142 k
B_2 = 1.02

AISC Manual
App. 8.2 &
Eq. A-8-8

Frame Forces

Story Drifts

AISC Manual
App. 8.2 &
Eq. A-8-8

4th FLOOR

P_{mf} = 0 k
P_{story} = 3360.9 k
R_M = 1
H = 166.8 k
L = 12 ft
Δ_H = 0.22 in
$P_{e\ story}$ = 108221 k
B_2 = 1.03

3rd FLOOR

P_{mf} = 0 k
P_{story} = 4844.3 k
R_M = 1
H = 205.2 k
L = 12 ft
Δ_H = 0.20 in
$P_{e\ story}$ = 148476 k
B_2 = 1.03

AISC Manual
App. 8.2 &
Eq. A-8-8

Frame Forces

Story Drifts

AISC Manual
App. 8.2 &
Eq. A-8-8

2nd FLOOR

P_{mf} = 0 k
P_{story} = 6327.7 k
R_M = 1
H = 226.0 k
L = 14 ft
Δ_H = 0.19 in
$P_{e\ story}$ = 201975 k
B_2 = 1.03

App. 8.2

Eq. A-8-8

Frame Forces

Story Drifts

AISC Manual
App. 8.2 &
Eq. A-8-8

Available Compressive & Tensil Strengths

CASE I - 80% SCBF - 20% MRF

Member	Section	$\phi_c P_n$ (k)	P_c (k)	$\phi_t P_n$ (k)	P_t (k)	Check
BR-9,10	**4.00x.226**	18.0	**17.8**	94.5	**11.3**	GOOD
BR-7,8	**6.00x.312**	84.4	**45.2**	197.0	**42.3**	GOOD
BR-5,6	**6.00x.312**	84.4	**80.5**	197.0	**55.2**	GOOD
BR-3,4	**7.50x.312**	146.5	**102.4**	249.0	**65.2**	GOOD
BR-1,2	**7.50x.312**	141.1	**127.8**	249.0	**73.2**	GOOD

AISC Manual
Tbl. 4-5 &
Tbl. 5-6

CASE II - 75% SCBF - 25% MRF

Member	Section	$\phi_c P_n$ (k)	P_c (k)	$\phi_t P_n$ (k)	P_t (k)	Check
BR-9,10	**4.00x.220**	17.6	**17.1**	92.2	**10.5**	GOOD
BR-7,8	**6.00x.280**	76.5	**40.8**	177.0	**36.8**	GOOD
BR-5,6	**6.00x.280**	76.5	**74.7**	177.0	**48.2**	GOOD
BR-3,4	**7.50x.312**	146.5	**98.0**	249.0	**59.2**	GOOD
BR-1,2	**7.50x.312**	141.1	**122.1**	249.0	**66.1**	GOOD

AISC Manual
Tbl. 4-5 &
Tbl. 5-6

CASE III - 70% SCBF - 30% MRF

Member	Section	$\phi_c P_n$ (k)	P_c (k)	$\phi_t P_n$ (k)	P_t (k)	Check
BR-9,10	**4.00x.220**	17.6	**16.2**	92.2	**9.6**	GOOD
BR-7,8	**6.00x.250**	76.5	**38.2**	177.0	**36.9**	GOOD
BR-5,6	**6.00x.250**	76.5	**70.8**	177.0	**47.3**	GOOD
BR-3,4	**7.00x.312**	124.7	**91.3**	232.0	**55.7**	GOOD
BR-1,2	**7.00x.312**	119.6	**114.8**	232.0	**62.0**	GOOD

AISC Manual
Tbl. 4-5 &
Tbl. 5-6

Step | Computations | Reference

Expected Brace Strengths in Tension

CASE I

Member	P_T (k)
BR-9,10	147.0
BR-7,8	306.9
BR-5,6	306.9
BR-3,4	387.5
BR-1,2	387.5

CASE II

Member	P_T (k)
BR-9,10	143.5
BR-7,8	275.8
BR-5,6	275.8
BR-3,4	387.5
BR-1,2	387.5

AISC Seismic Prov. Sect. F2.3

CASE III

Member	P_T (k)
BR-9,10	143.5
BR-7,8	248.1
BR-5,6	248.1
BR-3,4	360.4
BR-1,2	360.4

AISC Seismic Prov. Sect. F2.3

Expected Brace Strengths in Compression

CASE I - 80% SCBF - 20% MRF

Member	L_{actual} (ft)	KL/r	F_e (ksi)	F_{cre} (ksi)	P_C (k)	.3P_C (k)
BR-9,10	14	125.4	18.2	16.0	45.5	13.7
BR-7,8	14	83.2	41.4	32.4	193.0	57.9
BR-5,6	14	83.2	41.4	32.4	193.0	57.9
BR-3,4	14	65.9	65.9	40.5	304.1	91.2
BR-1,2	14	65.9	65.9	40.5	304.1	91.2

AISC Seismic Prov. Sect. F2.3

AISC Manual Eq. E3-2, Eq. E3-3, & Eq. E3-4

CASE II - 75% SCBF - 25% MRF

Member	L_{actual} (ft)	KL/r	F_e (ksi)	F_{cre} (ksi)	P_C (k)	.3P_C (k)
BR-9,10	14	125.4	18.2	16.0	44.4	13.3
BR-7,8	14	82.8	41.8	32.6	174.5	52.3
BR-5,6	14	82.8	41.8	32.6	174.5	52.3
BR-3,4	14	65.9	65.9	40.5	304.1	91.2
BR-1,2	14	65.9	65.9	40.5	304.1	91.2

AISC Seismic Prov. Sect. F2.3

AISC Manual Eq. E3-2, Eq. E3-3, & Eq. E3-4

CASE III - 70% SCBF - 30% MRF

Member	L_{actual} (ft)	KL/r	F_e (ksi)	F_{cre} (ksi)	P_C (k)	.3P_C (k)
BR-9,10	14	125.4	18.2	16.0	44.4	13.3
BR-7,8	14	82.4	42.2	32.8	157.9	47.4
BR-5,6	14	82.4	42.2	32.8	157.9	47.4
BR-3,4	14	70.9	57.0	38.2	266.7	80.0
BR-1,2	14	70.9	57.0	38.2	266.7	80.0

AISC Seismic Prov. Sect. F2.3

AISC Manual Eq. E3-2, Eq. E3-3, & Eq. E3-4

COLUMNS

Step	Computations	Reference

Expected Forces from Braces Buckling/Yielding

CASE I - 80% SCBF - 20% MRF

Member	P_T (k)	P_C (k)
BR-9,10	**147.0**	**45.5**
BR-7,8	**306.9**	**193.0**
BR-5,6	**306.9**	**193.0**
BR-3,4	**387.5**	**304.1**
BR-1,2	**387.5**	**304.1**

CASE II - 75% SCBF - 25% MRF

Member	P_T (k)	P_C (k)
BR-9,10	**143.5**	**44.4**
BR-7,8	**275.8**	**174.5**
BR-5,6	**275.8**	**174.5**
BR-3,4	**387.5**	**304.1**
BR-1,2	**387.5**	**304.1**

CASE III - 70% SCBF - 30% MRF

Member	P_T (k)	P_C (k)
BR-9,10	**143.5**	**44.4**
BR-7,8	**248.1**	**157.9**
BR-5,6	**248.1**	**157.9**
BR-3,4	**360.4**	**266.7**
BR-1,2	**360.4**	**266.7**

See Bracing Design

Loads from RISA using Overstrength Factor

CASE I - 80% SCBF - 20% MRF

Member	P_T (k)	P_C (k)
BR-9,10	34.0	34.1
BR-7,8	97.0	107.0
BR-5,6	146.7	156.4
BR-3,4	185.4	189.2
BR-1,2	219.6	223.3

CASE II - 75% SCBF - 25% MRF

Member	P_T (k)	P_C (k)
BR-9,10	32.2	32.3
BR-7,8	91.6	99.5
BR-5,6	137.3	144.9
BR-3,4	174.5	178.2
BR-1,2	205.7	209.3

See Applicable RISA Models, Both P_C & P_T values from EL ONLY WITH Ω Load Combo

CASE III - 70% SCBF - 30% MRF

Member	P_T (k)	P_C (k)
BR-9,10	30.1	30.2
BR-7,8	85.5	92.8
BR-5,6	128.4	135.5
BR-3,4	162.8	166.0
BR-1,2	192.1	195.2

See Applicable RISA Models, Both P_C & P_T values from EL ONLY WITH Ω Load Combo

Inputs

ϒ = 38.7 °

Step	Computations	Reference

Max Compression Loading from Buckling/Yielding

CASE I - 80% SCBF - 20% MRF

$V_{B,ROOF}$ = -31.7 k
$V_{B,5th\ Fl.}$ = -3.9 k
$V_{B,4th\ Fl.}$ = 0.0 k
$V_{B,3rd\ Fl.}$ = 9.5 k
$V_{B,2nd\ Fl.}$ = 0.0 k
$P_{E,C}$ = 608.9 k

CASE II - 75% SCBF - 25% MRF

$V_{B,ROOF}$ = -30.9 k
$V_{B,5th\ Fl.}$ = -0.7 k
$V_{B,4th\ Fl.}$ = 0.0 k
$V_{B,3rd\ Fl.}$ = 5.6 k
$V_{B,2nd\ Fl.}$ = 0.0 k
$P_{E,C}$ = 577.1 k

CASE III - 70% SCBF - 30% MRF

$V_{B,ROOF}$ = -30.9 k
$V_{B,5th\ Fl.}$ = 2.7 k
$V_{B,4th\ Fl.}$ = 0.0 k
$V_{B,3rd\ Fl.}$ = -1.1 k
$V_{B,2nd\ Fl.}$ = 0.0 k
$P_{E,C}$ = 535.8 k

Max Compression Loading from LC's using Ω

CASE I - 80% SCBF - 20% MRF

$V_{B,ROOF}$ = 0.0 k
$V_{B,5th\ Fl.}$ = 3.1 k
$V_{B,4th\ Fl.}$ = -0.1 k
$V_{B,3rd\ Fl.}$ = -1.9 k
$V_{B,2nd\ Fl.}$ = 0.0 k
$P_{E,C}$ = 294.3 k

CASE II - 75% SCBF - 25% MRF

$V_{B,ROOF}$ = 0.0 k
$V_{B,5th\ Fl.}$ = 2.4 k
$V_{B,4th\ Fl.}$ = -0.1 k
$V_{B,3rd\ Fl.}$ = -1.2 k
$V_{B,2nd\ Fl.}$ = 0.0 k
$P_{E,C}$ = 275.9 k

CASE III - 70% SCBF - 30% MRF

$V_{B,ROOF}$ = 0.0 k
$V_{B,5th\ Fl.}$ = 2.3 k
$V_{B,4th\ Fl.}$ = -0.1 k
$V_{B,3rd\ Fl.}$ = -1.2 k
$V_{B,2nd\ Fl.}$ = 0.0 k
$P_{E,C}$ = 257.6 k

Applicable B2 Factors

CASE I

B_2 = 1.03

CASE II

B_2 = 1.03

CASE III

B_2 = 1.03

See Bracing Calculations

Step	Computations	Reference

Design Compression Loads

CASE I - 80% SCBF - 20% MRF

P_{DL+LL} = 238.4 k

$P_{E,C}$ = 303.2 k

P_U = 541.7 k

CASE II - 75% SCBF - 25% MRF

P_{DL+LL} = 238.4 k

$P_{E,C}$ = 284.1 k

P_U = 522.6 k

CASE III - 70% SCBF - 30% MRF

P_{DL+LL} = 238.4 k

$P_{E,C}$ = 266.0 k

P_U = 504.4 k

Select Section

CASE I - 80% SCBF - 20% MRF

Select: W14x68

$\phi_c P_n$ = 640 k

Check: GOOD

CASE II - 75% SCBF - 25% MRF

Select: W14x68

$\phi_c P_n$ = 640 k

Check: GOOD

AISC Manual
Tbl. 1-1 &
Tbl. 4-1

CASE III - 70% SCBF - 30% MRF

Select: W14x68 -

$\phi_c P_n$ = 640 k

Check: GOOD

AISC Manual
Tbl. 1-1 &
Tbl. 4-1

BEAMS

(CASE I)

Step	Computations	Reference

Expected Forces from Braces Buckling/Yielding

CASE I - 80% SCBF - 20% MRF

Member	P_T (k)	P_C (k)
BR-9,10	147.0	45.5
BR-7,8	306.9	193.0
BR-5,6	306.9	193.0
BR-3,4	387.5	304.1
BR-1,2	387.5	304.1

See Bracing Design

Expected Forces from Braces Post-Buckling/Yielding

CASE I - 80% SCBF - 20% MRF

Member	P_T (k)	P_C (k)
BR-9,10	147.0	13.7
BR-7,8	306.9	57.9
BR-5,6	306.9	57.9
BR-3,4	387.5	91.2
BR-1,2	387.5	91.2

See Bracing Design

Inputs

Υ = 38.7 °
α = 43.0 °
L_x = 30 ft
L_z = 10 ft
L_z = 15 ft
K = 1
F_y = 50 ksi

Step	Computations	Reference

Dead Loads + Live Loads Only

LOAD COMBINATION 5 [(1.2 + .2S_{DS})DL + .5LL]

CASE I - 80% SCBF - 20% MRF

Member	V_{D+L} (k)	M_{D+L} (k-ft)
BM-5	4.0	30.2
BM-4	14.2	106.1
BM-3	14.2	106.1
BM-2	14.2	106.1
BM-1	14.2	106.1

Applicable RISA Model

LOAD COMBINATION 7 [(.9 - .2S_{DS})DL]

CASE I - 80% SCBF - 20% MRF

Member	V_{D+L} (k)	M_{D+L} (k-ft)
BM-5	2.4	18.3
BM-4	7.1	52.9
BM-3	7.1	52.9
BM-2	7.1	52.9
BM-1	7.1	52.9

Applicable RISA Model

Required Strengths *__due to Seismic Loading__* (Buckling, Post-Buckling)/Yielding

CASE I - 80% SCBF - 20% MRF

Member	Buckling/Yielding			Post-Buckling/Yielding		
	P_E (k)	V_E (k)	M_E (k-ft)	P_E (k)	V_E (k)	M_E (k-ft)
BM-5	**75.2**	**31.7**	**475.5**	**62.7**	**41.6**	**624.5**
BM-4	**120.0**	**0.0**	**0.0**	**149.3**	**0.0**	**0.0**
BM-3	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
BM-2	**77.0**	**0.0**	**0.0**	**212.9**	**0.0**	**0.0**
BM-1	**17.2**	**2.4**	**36.0**	**11.9**	**8.5**	**128.1**

Step	Computations							Reference
Member Selections and Properties	CASE I - 80% SCBF - 20% MRF							
	BM-5 W24x84	A_g =	24.7	in^2	S_x =	196	in^3	AISC Manual Tbl. 1-1
		d =	24.1	in	r_x =	9.8	in	
		t_w =	0.470	in	Z_x =	224	in^3	
		b_f =	9.02	in	I_y =	94.4	in^4	
		t_f =	0.770	in	r_y =	1.95	in	
		k_{des} =	1.27	in	h_O =	23.3	in	
		h/t_w =	45.9		J =	3.7	in^4	
		I_x =	2370	in^4	C_w =	12800	in^6	
	BM-4 W24x68	A_g =	20.1	in^2	S_x =	154	in^3	AISC Manual Tbl. 1-1
		d =	23.7	in	r_x =	9.55	in	
		t_w =	0.415	in	Z_x =	177	in^3	
		b_f =	8.97	in	I_y =	70.4	in^4	
		t_f =	0.589	in	r_y =	1.87	in	
		k_{des} =	1.09	in	h_O =	23.1	in	
		h/t_w =	52		J =	1.87	in^4	
		I_x =	1830	in^4	C_w =	9430	in^6	
	BM-3 W21x44	A_g =	13	in^2	S_x =	81.6	in^3	AISC Manual Tbl. 1-1
		d =	20.7	in	r_x =	8.1	in	
		t_w =	0.350	in	Z_x =	95.4	in^3	
		b_f =	6.50	in	I_y =	20.7	in^4	
		t_f =	0.450	in	r_y =	1.26	in	
		k_{des} =	0.95	in	h_O =	20.3	in	
		h/t_w =	53.6		J =	0.77	in^4	
		I_x =	843	in^4	C_w =	2110	in^6	
	BM-2 W24x68	A_g =	20.1	in^2	S_x =	154	in^3	AISC Manual Tbl. 1-1
		d =	23.7	in	r_x =	9.55	in	
		t_w =	0.415	in	Z_x =	177	in^3	
		b_f =	8.97	in	I_y =	70.4	in^4	
		t_f =	0.589	in	r_y =	1.87	in	
		k_{des} =	1.09	in	h_O =	23.1	in	
		h/t_w =	52		J =	1.87	in^4	
		I_x =	1830	in^4	C_w =	9430	in^6	
	BM-1 W21x44	A_g =	13	in^2	S_x =	81.6	in^3	AISC Manual Tbl. 1-1
		d =	20.7	in	r_x =	8.1	in	
		t_w =	0.350	in	Z_x =	95.4	in^3	
		b_f =	6.50	in	I_y =	20.7	in^4	
		t_f =	0.450	in	r_y =	1.26	in	
		k_{des} =	0.95	in	h_O =	20.3	in	
		h/t_w =	53.6		J =	0.77	in^4	
		I_x =	843	in^4	C_w =	2110	in^6	

Step	Computations	Reference
	MEMBER: **BM-1** *(CASE I - 80% SCBF - 20% MRF)*	
Available Compressive Strength	Q = 1.0 (assumed value)	
	$(KL/r)_x$ = 44.66501 113.4 $= 4.71\sqrt{E/QF_y}$	
	F_e = 143.5 ksi	AISC Manual Eq. E3-4
	F_y/F_e = 0.349 2.25	
	F_{cr} = 43.2 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 41.9 ksi	AISC Manual Eq. 8-3
	F_y/F_e = 1.192 2.25	
	F_{cr} = 30.4 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	$b = h$ = 18.8 in	
	f = 30.4 ksi	
	b_e = 16.71 in	AISC Manual Eq. E7-17
	A_e = 12.26721 in^2	
	Q_a = 0.944	AISC Manual Eq. E7-16
	Q_s = 1.0	
	Q = 0.943631	
	F_e = 41.9 ksi	
	QF_y/F_e = 1.125 2.25	
	F_{cr} = 28.6 ksi	
	$\phi_c P_n$ = 335.2 k	
Second Order Effects	P_{el} = 1861.7 k	AISC Manual Eq. A-8-5
	C_m = 1.0	
	$B_{1,BUCKLING}$ = 1.01	AISC Manual Eq. A-8-3
	$B_{1,POST-BUCKLING}$ = 1.01	

Step	Computations	Reference
Member Forces	MEMBER: **BM-1** *(CASE I - 80% SCBF - 20% MRF)*	

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	**17.4**	k	P_u =	**17.4**	k
	V_u =	**16.6**	k	V_u =	**9.5**	k
	(+) M_u =	**142.5179**	k-ft	(+) M_u =	**89.30491**	k-ft
	(-) M_u =	**69.75809**	k-ft	(-) M_u =	**16.54509**	k-ft
Post-Buckling / Yielding	P_u =	**12.0**	k	P_u =	**12.0**	k
	V_u =	**22.7**	k	V_u =	**15.6**	k
	(+) M_u =	**235.0165**	k-ft	(+) M_u =	**181.8035**	k-ft
	(-) M_u =	**-22.7405**	k-ft	(-) M_u =	**-75.9535**	k-ft

Step	Computations	Reference
	MEMBER: **BM-2** *(CASE I - 80% SCBF - 20% MRF)*	
Available Compressive Strength	Q = 1.0 (assumed value)	
	$(KL/r)_x$ = 37.69634 113.4 $= 4.71vE/QF_y$	
	F_e = 201.4 ksi	AISC Manual Eq. E3-4
	F_y/F_e = 0.248 2.25	
	F_{cr} = 45.1 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 40.6 ksi	AISC Manual Eq. 8-3
	F_y/F_e = 1.233 2.25	
	F_{cr} = 29.8 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	$b = h$ = 21.52 in	
	f = 29.8 ksi	
	b_e = 19.76 in	AISC Manual Eq. E7-17
	A_e = 19.36988 in^2	
	Q_a = 1.0	AISC Manual Eq. E7-16
	Q_s = 1.0	
	Q = 0.963676	
	F_e = 40.6 ksi	
	QF_y/F_e = 1.188 2.25	
	F_{cr} = 28.8 ksi	
	ϕ_cP_n = 520.3 k	
Second Order Effects	P_{el} = 4041.5 k	AISC Manual Eq. A-8-5
	C_m = 1.0	
	$B_{1,BUCKLING}$ = 1.00	AISC Manual Eq. A-8-3
	$B_{1,POST\text{-}BUCKLING}$ = 1.06	

Step	Computations	Reference
Member Forces	MEMBER: **BM-2** *(CASE I - 80% SCBF - 20% MRF)*	

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	77.4	k	P_u =	77.4	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft
Post-Buckling / Yielding	P_u =	224.7	k	P_u =	224.7	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft

Step	Computations	Reference
	MEMBER: **BM-3** *(CASE I - 80% SCBF - 20% MRF)*	
Available Compressive Strength	Q = 1.0 (assumed value)	
	$(KL/r)_x$ = 44.66501 113.4 = $4.71\sqrt{E/QF_y}$	
	F_e = 143.5 ksi	AISC Manual Eq. E3-4
	F_y/F_e = 0.349 2.25	
	F_{cr} = 43.2 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 20.8 ksi	AISC Manual Eq. 8-3
	F_y/F_e = 2.408 2.25	
	F_{cr} = 18.2 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	b = h = 18.8 in	
	f = 18.2 ksi	
	b_e = 18.80 in	AISC Manual Eq. E7-17
	A_e = 13 in^2	
	Q_a = 1.0	AISC Manual Eq. E7-16
	Q_s = 1.0	
	Q = 1.0	
	F_e = 20.8 ksi	
	QF_y/F_e = 2.408 2.25	
	F_{cr} = 18.2 ksi	
	$\phi_c P_n$ = 213.1 k	
Second Order Effects	P_{el} = 1861.7 k	AISC Manual Eq. A-8-5
	C_m = 1.0	
	$B_{1,BUCKLING}$ = 1.00	AISC Manual Eq. A-8-3
	$B_{1,POST\text{-}BUCKLING}$ = 1.00	

Step	Computations	Reference
Member Forces	MEMBER: **BM-3** *(CASE I - 80% SCBF - 20% MRF)*	

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	0.0	k	P_u =	0.0	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft
Post-Buckling / Yielding	P_u =	0.0	k	P_u =	0.0	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft

Step	Computations	Reference
Available Compressive Strength	MEMBER: **BM-4** *(CASE I - 80% SCBF - 20% MRF)*	
	Q = 1.0 (assumed value)	
	$(KL/r)_x$ = 37.69634 113.4 $= 4.71\sqrt{E/QF_y}$	
	F_e = 201.4 ksi	AISC Manual Eq. E3-4
	F_y/F_e = 0.248 2.25	
	F_{cr} = 45.1 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 40.6 ksi	AISC Manual Eq. 8-3
	F_y/F_e = 1.233 2.25	
	F_{cr} = 29.8 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	b = h = 21.52 in	
	f = 29.8 ksi	
	b_e = 19.76 in	AISC Manual Eq. E7-17
	A_e = 19.36988 in^2	
	Q_a = 1.0	AISC Manual Eq. E7-16
	Q_s = 1.0	
	Q = 0.963676	
	F_e = 40.6 ksi	
	QF_y/F_e = 1.188 2.25	
	F_{cr} = 28.8 ksi	
	$\phi_c P_n$ = 520.3 k	
Second Order Effects	P_{el} = 4041.5 k	AISC Manual Eq. A-8-5
	C_m = 1.0	
	$B_{1,BUCKLING}$ = 1.03	AISC Manual Eq. A-8-3
	$B_{1,POST\text{-}BUCKLING}$ = 1.04	

Step	Computations	Reference
Member Forces	MEMBER: **BM-4** *(CASE I - 80% SCBF - 20% MRF)*	

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	123.7	k	P_u =	123.7	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft
Post-Buckling / Yielding	P_u =	155.0	k	P_u =	155.0	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft

Step	Computations	Reference
	MEMBER: **BM-5** *(CASE I - 80% SCBF - 20% MRF)*	
Available Compressive Strength	Q = 1.0 (assumed value)	
	$(KL/r)_x$ = 36.77222 113.4 = $4.71\sqrt{E/QF_y}$	
	F_e = 211.7 ksi	AISC Manual Eq. E3-4
	F_y/F_e = 0.236 2.25	
	F_{cr} = 45.3 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 45.5 ksi	AISC Manual Eq. 8-3
	F_y/F_e = 1.098 2.25	
	F_{cr} = 31.6 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	b = h = 21.56 in	
	f = 31.6 ksi	
	b_e = 21.20 in	AISC Manual Eq. E7-17
	A_e = 24.5325 in^2	
	Q_a = 1.0	AISC Manual Eq. E7-16
	Q_s = 1.0	
	Q = 0.993219	
	F_e = 45.5 ksi	
	QF_y/F_e = 1.090 2.25	
	F_{cr} = 31.4 ksi	
	$\phi_c P_n$ = 697.3 k	
Second Order Effects	P_{el} = 5234.1 k	AISC Manual Eq. A-8-5
	C_m = 1.0	
	$B_{1,BUCKLING}$ = 1.01	AISC Manual Eq. A-8-3
	$B_{1,POST-BUCKLING}$ = 1.01	

Step	Computations	Reference
Member Forces	MEMBER: **BM-5** *(CASE I - 80% SCBF - 20% MRF)*	

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	121.8	k	P_u =	76.3	k
	V_u =	46.3	k	V_u =	39.2	k
	(+) M_u =	588.6152	k-ft	(+) M_u =	535.4022	k-ft
	(-) M_u =	-376.339	k-ft	(-) M_u =	-429.552	k-ft
Post-Buckling / Yielding	P_u =	63.5	k	P_u =	63.5	k
	V_u =	56.3	k	V_u =	49.2	k
	(+) M_u =	738.2543	k-ft	(+) M_u =	685.0413	k-ft
	(-) M_u =	-525.978	k-ft	(-) M_u =	-579.191	k-ft

Step	Computations	Reference

Available Strengths

CASE I - 80% SCBF - 20% MRF

Member	(+) $\phi_b M_n$ (k-ft)	(-) $\phi_b M_n$ (k-ft)	$\phi_c P_n$ (k)	$\phi_v V_n$ (k)
BM-5	840	765	697.3	340
BM-4	574	NA	520.3	306
BM-3	1050	900	213.1	404
BM-2	574	NA	520.3	306
BM-1	358	264	335.2	217

MEMBER: **BM-1** *(CASE I - 80% SCBF - 20% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.051873	Eq. H1-1b	P_r/P_c =	0.051873	Eq. H1-1b
	V:	0.076393	GOOD	V:	0.043697	GOOD
	(+) M:	0.424	GOOD	(+) M:	0.275	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0.035801	Eq. H1-1b	P_r/P_c =	0.035801	Eq. H1-1b
	V:	0.104811	GOOD	V:	0.072115	GOOD
	(+) M:	0.674	GOOD	(+) M:	0.526	GOOD
	(-) M:	0.104	GOOD	(-) M:	0.306	GOOD

MEMBER: **BM-2** *(CASE I - 80% SCBF - 20% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.14868	Eq. H1-1b	P_r/P_c =	0.14868	Eq. H1-1b
	V:	0.046248	GOOD	V:	0.023062	GOOD
	(+) M:	0.259	GOOD	(+) M:	0.167	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0.431924	Eq. H1-1a	P_r/P_c =	0.431924	Eq. H1-1a
	V:	0.046248	GOOD	V:	0.023062	GOOD
	(+) M:	0.596	GOOD	(+) M:	0.514	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA

Step	Computations	Reference

Combined Loading & Shear Checks

MEMBER: **BM-3** *(CASE I - 80% SCBF - 20% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0	Eq. H1-1b	P_r/P_c =	0	Eq. H1-1b
	V:	0.03503	GOOD	V:	0.017468	GOOD
	(+) M:	0.101	GOOD	(+) M:	0.050	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0	Eq. H1-1b	P_r/P_c =	0	Eq. H1-1b
	V:	0.03503	GOOD	V:	0.017468	GOOD
	(+) M:	0.101	GOOD	(+) M:	0.050	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA

MEMBER: **BM-4** *(CASE I - 80% SCBF - 20% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.23772	Eq. H1-1a	P_r/P_c =	0.23772	Eq. H1-1a
	V:	0.046248	GOOD	V:	0.023062	GOOD
	(+) M:	0.402	GOOD	(+) M:	0.320	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0.297863	Eq. H1-1a	P_r/P_c =	0.297863	Eq. H1-1a
	V:	0.046248	GOOD	V:	0.023062	GOOD
	(+) M:	0.462	GOOD	(+) M:	0.380	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA

MEMBER: **BM-5** *(CASE I - 80% SCBF - 20% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.174632	Eq. H1-1b	P_r/P_c =	0.109358	Eq. H1-1b
	V:	0.136227	GOOD	V:	0.115359	GOOD
	(+) M:	0.788	GOOD	(+) M:	0.692	GOOD
	(-) M:	0.579	GOOD	(-) M:	0.616	GOOD
Post-Buckling / Yielding	P_r/P_c =	0.091074	Eq. H1-1b	P_r/P_c =	0.091074	Eq. H1-1b
	V:	0.165568	GOOD	V:	0.1447	GOOD
	(+) M:	0.924	GOOD	(+) M:	0.861	GOOD
	(-) M:	0.733	GOOD	(-) M:	0.803	GOOD

BEAMS
(CASE I)

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Step	Computations	Reference

Expected Forces from Braces Buckling/Yielding

CASE II - 75% SCBF - 25% MRF

Member	P_T (k)	P_C (k)
BR-9,10	143.5	44.4
BR-7,8	275.8	174.5
BR-5,6	275.8	174.5
BR-3,4	387.5	304.1
BR-1,2	387.5	304.1

See Bracing Design

Expected Forces from Braces Post-Buckling/Yielding

CASE II - 75% SCBF - 25% MRF

Member	P_T (k)	P_C (k)
BR-9,10	143.5	13.3
BR-7,8	275.8	52.3
BR-5,6	275.8	52.3
BR-3,4	387.5	91.2
BR-1,2	387.5	91.2

Inputs

Υ = 38.7 °

α = 43.0 °

L_x = 30 ft

L_x = 10 ft

L_z = 15 ft

K = 1

F_y = 50 ksi

Step	Computations	Reference

Dead Loads + Live Loads Only

LOAD COMBINATION 5 [(1.2 + .2S_{DS})DL + .5LL]

CASE II - 75% SCBF - 25% MRF

Member	V_{D+L} (k)	M_{D+L} (k-ft)
BM-5	4.0	30.2
BM-4	14.2	106.1
BM-3	14.2	106.1
BM-2	14.2	106.1
BM-1	14.2	106.1

Applicable RISA Model

LOAD COMBINATION 7 [(.9 - .2S_{DS})DL]

CASE II - 75% SCBF - 25% MRF

Member	V_{D+L} (k)	M_{D+L} (k-ft)
BM-5	2.4	18.3
BM-4	7.1	52.9
BM-3	7.1	52.9
BM-2	7.1	52.9
BM-1	7.1	52.9

Applicable RISA Model

Required Strengths *due to Seismic Loading* (Buckling, Post-Buckling)/Yielding

CASE II - 75% SCBF - 25% MRF

Member	Buckling/Yielding			Post-Buckling/Yielding		
	P_E (k)	V_E (k)	M_E (k-ft)	P_E (k)	V_E (k)	M_E (k-ft)
BM-5	**73.4**	**31.0**	**464.3**	**61.2**	**40.7**	**610.0**
BM-4	**102.4**	**0.0**	**0.0**	**138.1**	**0.0**	**0.0**
BM-3	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
BM-2	**94.2**	**0.0**	**0.0**	**202.9**	**0.0**	**0.0**
BM-1	**17.2**	**2.4**	**36.0**	**11.9**	**8.5**	**128.1**

Step	Computations							Reference
	CASE II - 75% SCBF - 25% MRF							
Member Selections and Properties	BM-5 W24x68	A_g =	20.1	in^2	S_x =	154	in^3	Tbl. 1-1
		d =	23.7	in	r_x =	9.55	in	
		t_w =	0.415	in	Z_x =	177.000	in^3	
		b_f =	8.97	in	I_y =	70.40	in^4	
		t_f =	0.585	in	r_y =	1.870	in	
		k_{des} =	1.09	in	h_O =	23.1	in	
		h/t_w =	52		J =	1.87	in^4	
		I_x =	1830	in^4	C_w =	9430	in^6	
	BM-4 W24x55	A_g =	16.2	in^2	S_x =	114	in^3	Tbl. 1-1
		d =	23.6	in	r_x =	9.11	in	
		t_w =	0.395	in	Z_x =	134	in^3	
		b_f =	7.01	in	I_y =	29.1	in^4	
		t_f =	0.505	in	r_y =	1.34	in	
		k_{des} =	1.01	in	h_O =	23.1	in	
		h/t_w =	54.6		J =	1.18	in^4	
		I_x =	1350	in^4	C_w =	3870	in^6	
	BM-3 W21x44	A_g =	13	in^2	S_x =	81.6	in^3	Tbl. 1-1
		d =	20.7	in	r_x =	8.1	in	
		t_w =	0.350	in	Z_x =	95.4	in^3	
		b_f =	6.50	in	I_y =	20.7	in^4	
		t_f =	0.450	in	r_y =	1.26	in	
		k_{des} =	0.95	in	h_O =	20.3	in	
		h/t_w =	53.6		J =	0.77	in^4	
		I_x =	843	in^4	C_w =	2110	in^6	
	BM-2 W24x55	A_g =	16.2	in^2	S_x =	114	in^3	Tbl. 1-1
		d =	23.6	in	r_x =	9.11	in	
		t_w =	0.395	in	Z_x =	134	in^3	
		b_f =	7.01	in	I_y =	29.1	in^4	
		t_f =	0.505	in	r_y =	1.34	in	
		k_{des} =	1.01	in	h_O =	23.1	in	
		h/t_w =	54.6		J =	1.18	in^4	
		I_x =	1350	in^4	C_w =	3870	in^6	
	BM-1 W21x44	A_g =	13	in^2	S_x =	81.6	in^3	Tbl. 1-1
		d =	20.7	in	r_x =	8.1	in	
		t_w =	0.350	in	Z_x =	95.4	in^3	
		b_f =	6.50	in	I_y =	20.7	in^4	
		t_f =	0.450	in	r_y =	1.26	in	
		k_{des} =	0.95	in	h_O =	20.3	in	
		h/t_w =	53.6		J =	0.77	in^4	
		I_x =	843	in^4	C_w =	2110	in^6	

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Step	Computations	Reference
Available Compressive Strength	MEMBER: **BM-1** *(CASE II - 75% SCBF - 25% MRF)*	
	Q = 1.0 (assumed value) $(KL/r)_x$ = 44.66501 113.4 $= 4.71\sqrt{E/QF_y}$	
	F_e = 143.5 ksi F_y/F_e = 0.349 2.25	AISC Manual Eq. E3-4
	F_{cr} = 43.2 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 41.9 ksi F_y/F_e = 1.192 2.25	AISC Manual Eq. 8-3
	F_{cr} = 30.4 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	$b = h$ = 18.8 in f = 30.4 ksi	
	b_e = 16.71 in A_e = 12.26721 in^2	AISC Manual Eq. E7-17
	Q_a = 0.9 Q_s = 1.0	AISC Manual Eq. E7-16
	Q = 0.94	
	F_e = 41.9 ksi QF_y/F_e = 1.125 2.25	
	F_{cr} = 28.6 ksi	
	$\phi_c P_n$ = 335.2 k	
Second Order Effects	P_{el} = 1861.7 k C_m = 1.0	AISC Manual Eq. A-8-5
	$B_{1,BUCKLING}$ = 1.01 $B_{1,POST\text{-}BUCKLING}$ = 1.01	AISC Manual Eq. A-8-3

Step	Computations	Reference
Member Forces		

MEMBER: **BM-1** *(CASE II - 75% SCBF - 25% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	17.4	k	P_u =	17.4	k
	V_u =	16.6	k	V_u =	9.5	k
	(+) M_u =	142.5179	k-ft	(+) M_u =	89.30491	k-ft
	(-) M_u =	69.75809	k-ft	(-) M_u =	16.54509	k-ft
Post-Buckling / Yielding	P_u =	12.0	k	P_u =	12.0	k
	V_u =	22.7	k	V_u =	15.6	k
	(+) M_u =	235.0165	k-ft	(+) M_u =	181.8035	k-ft
	(-) M_u =	-22.7405	k-ft	(-) M_u =	-75.9535	k-ft

Step	Computations	Reference
	MEMBER: **BM-2** *(CASE II - 75% SCBF - 25% MRF)*	
Available Compressive Strength	Q = 1.0 (assumed value)	
	$(KL/r)_x$ = 39.51701 113.4 = 4.71√E/QF$_y$	
	F_e = 183.3 ksi	AISC Manual Eq. E3-4
	F_y/F_e = 0.273 2.25	
	F_{cr} = 44.6 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 22.9 ksi	AISC Manual Eq. 8-3
	F_y/F_e = 2.184 2.25	
	F_{cr} = 20.0 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	b = h = 21.58 in	
	f = 20.0 ksi	
	b_e = 21.58 in	AISC Manual Eq. E7-17
	A_e = 16.2 in^2	
	Q_a = 1.0	AISC Manual Eq. E7-16
	Q_s = 1.0	
	Q = 1	
	F_e = 22.9 ksi	
	QF_y/F_e = 2.184 2.25	
	F_{cr} = 20.0 ksi	
	$\phi_c P_n$ = 292.2 k	
Second Order Effects	P_{el} = 2981.4 k	AISC Manual Eq. A-8-5
	C_m = 1.0	
	$B_{1,BUCKLING}$ = 1.01	AISC Manual Eq. A-8-3
	$B_{1,POST\text{-}BUCKLING}$ = 1.00	

Step	Computations	Reference
Member Forces		

MEMBER: **BM-2** *(CASE II - 75% SCBF - 25% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	94.8	k	P_u =	94.8	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft
Post-Buckling / Yielding	P_u =	203.8	k	P_u =	203.8	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft

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Step	Computations	Reference
	MEMBER: **BM-3** *(CASE II - 75% SCBF - 25% MRF)*	
Available Compressive Strength	Q = 1.0 (assumed value)	
	$(KL/r)_x$ = 44.66501 113.4 = 4.71√E/QF$_y$	
	F_e = 143.5 ksi	AISC Manual Eq. E3-4
	F_y/F_e = 0.349 2.25	
	F_{cr} = 43.2 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 20.8 ksi	AISC Manual Eq. 8-3
	F_y/F_e = 2.408 2.25	
	F_{cr} = 18.2 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	b = h = 18.8 in	
	f = 18.2 ksi	
	b_e = 18.80 in	AISC Manual Eq. E7-17
	A_e = 13 in^2	
	Q_a = 1.0	AISC Manual Eq. E7-16
	Q_s = 1.0	
	Q = 1	
	F_e = 20.8 ksi	
	QF_y/F_e = 2.408 2.25	
	F_{cr} = 18.2 ksi	
	$\phi_c P_n$ = 213.1 k	
Second Order Effects	P_{el} = 1861.7 k	AISC Manual Eq. A-8-5
	C_m = 1.0	
	$B_{1,BUCKLING}$ = 1.00	AISC Manual Eq. A-8-3
	$B_{1,POST\text{-}BUCKLING}$ = 1.00	

Member Forces

MEMBER: **BM-3** *(CASE II - 75% SCBF - 25% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	0.0	k	P_u =	0.0	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft
Post-Buckling / Yielding	P_u =	0.0	k	P_u =	0.0	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft

Step	Computations	Reference
	MEMBER: **BM-4** *(CASE II - 75% SCBF - 25% MRF)*	
Available Compressive Strength	Q = 1.0 (assumed value)	
	$(KL/r)_x$ = 39.51701 113.4 = $4.71\sqrt{E/QF_y}$	
	F_e = 183.3 ksi F_y/F_e = 0.273 2.25	AISC Manual Eq. E3-4
	F_{cr} = 44.6 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 22.9 ksi F_y/F_e = 2.184 2.25	AISC Manual Eq. 8-3
	F_{cr} = 20.0 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	b = h = 21.58 in f = 20.0 ksi	
	b_e = 21.58 in A_e = 16.2 in^2	AISC Manual Eq. E7-17
	Q_a = 1.0 Q_s = 1.0	AISC Manual Eq. E7-16
	Q = 1	
	F_e = 22.9 ksi QF_y/F_e = 2.184 2.25	
	F_{cr} = 20.0 ksi	
	$\phi_c P_n$ = 292.2 k	
Second Order Effects	P_{el} = 2981.4 k C_m = 1.0	AISC Manual Eq. A-8-5
	$B_{1,BUCKLING}$ = 1.04 $B_{1,POST\text{-}BUCKLING}$ = 1.05	AISC Manual Eq. A-8-3

Step	Computations	Reference
Member Forces		

MEMBER: **BM-4** *(CASE II - 75% SCBF - 25% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	106.1	k	P_u =	106.1	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft
Post-Buckling / Yielding	P_u =	144.8	k	P_u =	144.8	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft

Step	Computations	Reference
	MEMBER: **BM-5** *(CASE II - 75% SCBF - 25% MRF)*	
Available Compressive Strength	Q = 1.0 (assumed value)	
	$(KL/r)_x$ = 37.69634 113.4 $= 4.71\sqrt{E/QF_y}$	
	F_e = 201.4 ksi	AISC Manual Eq. E3-4
	F_y/F_e = 0.248 2.25	
	F_{cr} = 45.1 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 40.6 ksi	AISC Manual Eq. 8-3
	F_y/F_e = 1.233 2.25	
	F_{cr} = 29.8 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	$b = h$ = 21.52 in	
	f = 29.8 ksi	
	b_e = 19.76 in	AISC Manual Eq. E7-17
	A_e = 19.36988 in^2	
	Q_a = 1.0	AISC Manual Eq. E7-16
	Q_s = 1.0	
	Q = 0.963676	
	F_e = 40.6 ksi	
	QF_y/F_e = 1.188 2.25	
	F_{cr} = 28.8 ksi	
	$\phi_c P_n$ = 520.3 k	
Second Order Effects	P_{el} = 4041.5 k	AISC Manual Eq. A-8-5
	C_m = 1.0	
	$B_{1,BUCKLING}$ = 1.02	AISC Manual Eq. A-8-3
	$B_{1,POST\text{-}BUCKLING}$ = 1.02	

Step	Computations	Reference
Member Forces		

MEMBER: **BM-5** *(CASE II - 75% SCBF - 25% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	**74.7**	k	P_u =	**74.7**	k
	V_u =	**45.7**	k	V_u =	**38.6**	k
	(+) M_u =	**579.0266**	k-ft	(+) M_u =	**525.8136**	k-ft
	(-) M_u =	**-366.751**	k-ft	(-) M_u =	**-419.964**	k-ft
Post-Buckling / Yielding	P_u =	**62.2**	k	P_u =	**62.2**	k
	V_u =	**55.4**	k	V_u =	**48.4**	k
	(+) M_u =	**725.5352**	k-ft	(+) M_u =	**672.3222**	k-ft
	(-) M_u =	**-513.259**	k-ft	(-) M_u =	**-566.472**	k-ft

Step	Computations	Reference

Available Strengths

CASE II - 75% SCBF - 25% MRF

Member	(+) $\phi_b M_n$ (k-ft)	(-) $\phi_b M_n$ (k-ft)	$\phi_c P_n$ (k)	$\phi_v V_n$ (k)
BM-5	840	765	520.3	340
BM-4	574	NA	292.2	306
BM-3	358	264	213.1	217
BM-2	574	NA	292.2	306
BM-1	358	264	335.2	217

Combined Loading & Shear Checks

MEMBER: **BM-1** *(CASE II - 75% SCBF - 25% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.051873	Eq. H1-1b	P_r/P_c =	0.051873	Eq. H1-1b
	V:	0.076393	GOOD	V:	0.043697	GOOD
	(+) M:	0.424	GOOD	(+) M:	0.275	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0.035801	Eq. H1-1b	P_r/P_c =	0.035801	Eq. H1-1b
	V:	0.104811	GOOD	V:	0.072115	GOOD
	(+) M:	0.674	GOOD	(+) M:	0.526	GOOD
	(-) M:	0.104	GOOD	(-) M:	0.306	GOOD

MEMBER: **BM-2** *(CASE II - 75% SCBF - 25% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.324317	Eq. H1-1a	P_r/P_c =	0.324317	Eq. H1-1a
	V:	0.046248	GOOD	V:	0.023062	GOOD
	(+) M:	0.489	GOOD	(+) M:	0.406	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0.324317	Eq. H1-1a	P_r/P_c =	0.697384	Eq. H1-1a
	V:	0.046248	GOOD	V:	0.023062	GOOD
	(+) M:	0.862	GOOD	(+) M:	0.779	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA

Step	Computations	Reference
Combined Loading & Shear Checks		

MEMBER: **BM-3** *(CASE II - 75% SCBF - 25% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0	Eq. H1-1b	P_r/P_c =	0	Eq. H1-1b
	V:	0.065217	GOOD	V:	0.032521	GOOD
	(+) M:	0.296	GOOD	(+) M:	0.148	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0	Eq. H1-1b	P_r/P_c =	0	Eq. H1-1b
	V:	0.065217	GOOD	V:	0.032521	GOOD
	(+) M:	0.296	GOOD	(+) M:	0.148	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA

MEMBER: **BM-4** *(CASE II - 75% SCBF - 25% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.363116	Eq. H1-1a	P_r/P_c =	0.363116	Eq. H1-1a
	V:	0.046248	GOOD	V:	0.023062	GOOD
	(+) M:	0.527	GOOD	(+) M:	0.445	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0.363116	Eq. H1-1a	P_r/P_c =	0.495596	Eq. H1-1a
	V:	0.046248	GOOD	V:	0.023062	GOOD
	(+) M:	0.660	GOOD	(+) M:	0.578	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA

MEMBER: **BM-5** *(CASE II - 75% SCBF - 25% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.143599	Eq. H1-1b	P_r/P_c =	0.143599	Eq. H1-1b
	V:	0.134347	GOOD	V:	0.113479	GOOD
	(+) M:	0.761	GOOD	(+) M:	0.698	GOOD
	(-) M:	0.551	GOOD	(-) M:	0.621	GOOD
Post-Buckling / Yielding	P_r/P_c =	0.143599	Eq. H1-1b	P_r/P_c =	0.143599	Eq. H1-1b
	V:	0.163074	GOOD	V:	0.142206	GOOD
	(+) M:	0.923	GOOD	(+) M:	0.860	GOOD
	(-) M:	0.731	GOOD	(-) M:	0.800	GOOD

BEAMS
(CASE III)

Step	Computations	Reference

Expected Forces from Braces Buckling/Yielding

CASE III - 70% SCBF - 30% MRF

Member	P_T (k)	P_C (k)
BR-9,10	143.5	44.4
BR-7,8	248.1	157.9
BR-5,6	248.1	157.9
BR-3,4	360.4	266.7
BR-1,2	360.4	266.7

See Bracing Design

Expected Forces from Braces Post-Buckling/Yielding

CASE III - 70% SCBF - 30% MRF

Member	P_T (k)	P_C (k)
BR-9,10	143.5	13.3
BR-7,8	248.1	47.4
BR-5,6	248.1	47.4
BR-3,4	360.4	80.0
BR-1,2	360.4	80.0

Inputs

Υ = 38.7 °
α = 43.0 °
L_x = 30 ft
L_z = 12.5 ft
K = 1
F_y = 50 ksi

Step	Computations	Reference

Dead Loads + Live Loads Only

LOAD COMBINATION 5 [$(1.2 + .2S_{DS})$DL + .5LL]

CASE III - 70% SCBF - 30% MRF

Member	V_{D+L} (k)	M_{D+L} (k-ft)
BM-5	4.0	30.2
BM-4	14.2	106.1
BM-3	14.2	106.1
BM-2	14.2	106.1
BM-1	14.2	106.1

Applicable RISA Model

LOAD COMBINATION 7 [$(.9 - .2S_{DS})$DL]

CASE III - 70% SCBF - 30% MRF

Member	V_{D+L} (k)	M_{D+L} (k-ft)
BM-5	2.4	18.3
BM-4	7.1	52.9
BM-3	7.1	52.9
BM-2	7.1	52.9
BM-1	7.1	52.9

Applicable RISA Model

Required Strengths *due to Seismic Loading* (Buckling, Post-Buckling)/Yielding

CASE III - 70% SCBF - 30% MRF

Member	Buckling/Yielding			Post-Buckling/Yielding		
	P_E (k)	V_E (k)	M_E (k-ft)	P_E (k)	V_E (k)	M_E (k-ft)
BM-5	**73.4**	**30.9**	**464.1**	**61.2**	**40.7**	**609.8**
BM-4	**85.2**	**0.0**	**0.0**	**129.2**	**0.0**	**0.0**
BM-3	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
BM-2	**86.4**	**0.0**	**0.0**	**187.9**	**0.0**	**0.0**
BM-1	**15.6**	**2.7**	**40.5**	**11.0**	**8.1**	**121.2**

Step	Computations							Reference
Member Selections and Properties	CASE III - 70% SCBF - 30% MRF							
	BM-5 W24x68	A_g =	20.1	in^2	S_x =	154	in^3	Tbl. 1-1
		d =	23.7	in	r_x =	9.55	in	
		t_w =	0.415	in	Z_x =	177.000	in^3	
		b_f =	8.97	in	I_y =	70.40	in^4	
		t_f =	0.585	in	r_y =	1.870	in	
		k_{des} =	1.09	in	h_O =	23.1	in	
		h/t_w =	52		J =	1.87	in^4	
		I_x =	1830	in^4	C_w =	9430	in^6	
	BM-4 W21x44	A_g =	13	in^2	S_x =		in^3	Tbl. 1-1
		d =	20.7	in	r_x =	8.1	in	
		t_w =	0.350	in	Z_x =	95.4	in^3	
		b_f =	6.50	in	I_y =	20.7	in^4	
		t_f =	0.450	in	r_y =	1.26	in	
		k_{des} =	0.95	in	h_O =	20.3	in	
		h/t_w =	53.6		J =	0.77	in^4	
		I_x =	843	in^4	C_w =	2110	in^6	
	BM-3 W21x44	A_g =	13	in^2	S_x =	81.6	in^3	Tbl. 1-1
		d =	20.7	in	r_x =	8.1	in	
		t_w =	0.350	in	Z_x =	95.4	in^3	
		b_f =	6.50	in	I_y =	20.7	in^4	
		t_f =	0.450	in	r_y =	1.26	in	
		k_{des} =	0.95	in	h_O =	20.3	in	
		h/t_w =	53.6		J =	0.77	in^4	
		I_x =	843	in^4	C_w =	2110	in^6	
	BM-2 W21x44	A_g =	13	in^2	S_x =		in^3	Tbl. 1-1
		d =	20.7	in	r_x =	8.1	in	
		t_w =	0.350	in	Z_x =	95.4	in^3	
		b_f =	6.50	in	I_y =	20.7	in^4	
		t_f =	0.450	in	r_y =	1.26	in	
		k_{des} =	0.95	in	h_O =	20.3	in	
		h/t_w =	53.6		J =	0.77	in^4	
		I_x =	843	in^4	C_w =	2110	in^6	
	BM-1 W21x44	A_g =	13	in^2	S_x =		in^3	Tbl. 1-1
		d =	20.7	in	r_x =	8.1	in	
		t_w =	0.350	in	Z_x =	95.4	in^3	
		b_f =	6.50	in	I_y =	20.7	in^4	
		t_f =	0.450	in	r_y =	1.26	in	
		k_{des} =	0.95	in	h_O =	20.3	in	
		h/t_w =	53.6		J =	0.77	in^4	
		I_x =	843	in^4	C_w =	2110	in^6	

Step	Computations	Reference
Available Compressive Strength	MEMBER: **BM-1** *(CASE III - 70% SCBF - 30% MRF)*	
	Q = 1.0 (assumed value) $(KL/r)_x$ = 44.66501 113.4 = $4.71\sqrt{E/QF_y}$	
	F_e = 143.5 ksi F_y/F_e = 0.349 2.25	AISC Manual Eq. E3-4
	F_{cr} = 43.2 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 28.2 ksi F_y/F_e = 1.772 2.25	AISC Manual Eq. 8-3
	F_{cr} = 23.8 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	b = h = 18.8 in f = 23.8 ksi	
	b_e = 18.27 in A_e = 12.81432 in^2	AISC Manual Eq. E7-17
	Q_a = 1.0 Q_s = 1.0	AISC Manual Eq. E7-16
	Q = 0.985717	
	F_e = 28.2 ksi QF_y/F_e = 1.746 2.25	
	F_{cr} = 23.5 ksi	
	$\phi_c P_n$ = 274.7 k	
Second Order Effects	P_{el} = 1861.7 k C_m = 1.0	AISC Manual Eq. A-8-5
	$B_{1,BUCKLING}$ = 1.01 $B_{1,POST\text{-}BUCKLING}$ = 1.01	AISC Manual Eq. A-8-3

Step	Computations	Reference
Member Forces		

MEMBER: **BM-1** *(CASE III - 70% SCBF - 30% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	15.8	k	P_u =	15.8	k
	V_u =	16.9	k	V_u =	9.8	k
	(+) M_u =	146.9741	k-ft	(+) M_u =	93.76105	k-ft
	(-) M_u =	65.30195	k-ft	(-) M_u =	12.08895	k-ft
Post-Buckling / Yielding	P_u =	11.0	k	P_u =	11.0	k
	V_u =	22.3	k	V_u =	15.2	k
	(+) M_u =	228.0466	k-ft	(+) M_u =	174.8336	k-ft
	(-) M_u =	-15.7706	k-ft	(-) M_u =	-68.9836	k-ft

Step	Computations	Reference
Available Compressive Strength	MEMBER: **BM-2** *(CASE III - 70% SCBF - 30% MRF)*	
	Q = 1.0 (assumed value)	
	$(KL/r)_x$ = 44.66501 113.4 = $4.71\sqrt{E/QF_y}$	
	F_e = 143.5 ksi F_y/F_e = 0.349 2.25	AISC Manual Eq. E3-4
	F_{cr} = 43.2 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 28.2 ksi F_y/F_e = 1.772 2.25	AISC Manual Eq. 8-3
	F_{cr} = 23.8 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	b = h = 18.8 in f = 23.8 ksi	
	b_e = 18.27 in A_e = 12.81432 in^2	AISC Manual Eq. E7-17
	Q_a = 1.0 Q_s = 1.0	AISC Manual Eq. E7-16
	Q = 0.985717	
	F_e = 28.2 ksi QF_y/F_e = 1.746 2.25	
	F_{cr} = 23.5 ksi	
	$\phi_c P_n$ = 274.7 k	
Second Order Effects	P_{el} = 1861.7 k C_m = 1.0	AISC Manual Eq. A-8-5
	$B_{1,BUCKLING}$ = 1.01 $B_{1,POST\text{-}BUCKLING}$ = 1.01	AISC Manual Eq. A-8-3

Member Forces

MEMBER: **BM-2** *(CASE III - 70% SCBF - 30% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	**87.1**	k	P_u =	**87.1**	k
	V_u =	**14.2**	k	V_u =	**7.1**	k
	(+) M_u =	**106.138**	k-ft	(+) M_u =	**52.925**	k-ft
	(-) M_u =	**106.138**	k-ft	(-) M_u =	**52.925**	k-ft
Post-Buckling / Yielding	P_u =	**189.0**	k	P_u =	**189.0**	k
	V_u =	**14.2**	k	V_u =	**7.1**	k
	(+) M_u =	**106.138**	k-ft	(+) M_u =	**52.925**	k-ft
	(-) M_u =	**106.138**	k-ft	(-) M_u =	**52.925**	k-ft

Step	Computations	Reference
Available Compressive Strength	MEMBER: **BM-3** *(CASE III - 70% SCBF - 30% MRF)*	
	$Q =$ 1.0 (assumed value)	
	$(KL/r)_x =$ 44.66501 113.4 $= 4.71\sqrt{E/QF_y}$	
	$F_e =$ 143.5 ksi	AISC Manual Eq. E3-4
	$F_y/F_e =$ 0.349 2.25	
	$F_{cr} =$ 43.2 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	$F_e =$ 28.2 ksi	AISC Manual Eq. 8-3
	$F_y/F_e =$ 1.772 2.25	
	$F_{cr} =$ 23.8 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	$b = h =$ 18.8 in	
	$f =$ 23.8 ksi	
	$b_e =$ 18.27 in	AISC Manual Eq. E7-17
	$A_e =$ 12.81432 in^2	
	$Q_a =$ 1.0	AISC Manual Eq. E7-16
	$Q_s =$ 1.0	
	$Q =$ 0.985717	
	$F_e =$ 28.2 ksi	
	$QF_y/F_e =$ 1.746 2.25	
	$F_{cr} =$ 23.5 ksi	
	$\phi_c P_n =$ 274.7 k	
Second Order Effects	$P_{el} =$ 1861.7 k	AISC Manual Eq. A-8-5
	$C_m =$ 1.0	
	$B_{1,BUCKLING} =$ 1.00	AISC Manual Eq. A-8-3
	$B_{1,POST\text{-}BUCKLING} =$ 1.00	

Step	Computations	Reference
Member Forces		

MEMBER: **BM-3** *(CASE III - 70% SCBF - 30% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	0.0	k	P_u =	0.0	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft
Post-Buckling / Yielding	P_u =	0.0	k	P_u =	0.0	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft

Step	Computations	Reference
Available Compressive Strength	MEMBER: **BM-4** *(CASE III - 70% SCBF - 30% MRF)*	
	Q = 1.0 (assumed value)	
	$(KL/r)_x$ = 44.66501 113.4 = $4.71\sqrt{E/QF_y}$	
	F_e = 143.5 ksi	AISC Manual Eq. E3-4
	F_y/F_e = 0.349 2.25	
	F_{cr} = 43.2 ksi ← Critical buckling stress about the X-X axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	F_e = 28.2 ksi	AISC Manual Eq. 8-3
	F_y/F_e = 1.772 2.25	
	F_{cr} = 23.8 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0	AISC Manual Eq. E3-2 or E3-3
	$b = h$ = 18.8 in	
	f = 23.8 ksi	
	b_e = 18.27 in	AISC Manual Eq. E7-17
	A_e = 12.81432 in^2	
	Q_a = 1.0	AISC Manual Eq. E7-16
	Q_s = 1.0	
	Q = 0.985717	
	F_e = 28.2 ksi	
	QF_y/F_e = 1.746 2.25	
	F_{cr} = 23.5 ksi	
	$\phi_c P_n$ = 274.7 k	
Second Order Effects	P_{el} = 1861.7 k	AISC Manual Eq. A-8-5
	C_m = 1.0	
	$B_{1,BUCKLING}$ = 1.05	AISC Manual Eq. A-8-3
	$B_{1,POST\text{-}BUCKLING}$ = 1.07	

Step | Computations | Reference

Member Forces

MEMBER: **BM-4** *(CASE III - 70% SCBF - 30% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	89.2	k	P_u =	89.2	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft
Post-Buckling / Yielding	P_u =	138.8	k	P_u =	138.8	k
	V_u =	14.2	k	V_u =	7.1	k
	(+) M_u =	106.138	k-ft	(+) M_u =	52.925	k-ft
	(-) M_u =	106.138	k-ft	(-) M_u =	52.925	k-ft

Step	Computations	Reference

MEMBER: **BM-5** *(CASE III - 70% SCBF - 30% MRF)*

Available Compressive Strength

Q = 1.0 (assumed value)
$(KL/r)_x$ = 37.69634 | 113.4 | $= 4.71\sqrt{E/QF_y}$

F_e = 201.4 ksi
F_y/F_e = 0.248 | 2.25

AISC Manual Eq. E3-4

F_{cr} = 45.1 ksi ← Critical buckling stress about the X-X axis with Q = 1.0

AISC Manual Eq. E3-2 or E3-3

F_e = 56.5 ksi
F_y/F_e = 0.886 | 2.25

AISC Manual Eq. 8-3

F_{cr} = 34.5 ksi ← Critical buckling stress about the Z-Z axis with Q = 1.0

AISC Manual Eq. E3-2 or E3-3

$b = h$ = 21.52 in
f = 34.5 ksi

b_e = 18.71 in
A_e = 18.93262 in^2

AISC Manual Eq. E7-17

Q_a = 0.9
Q_s = 1.0

AISC Manual Eq. E7-16

Q = 0.941921

F_e = 56.5 ksi
QF_y/F_e = 0.834 | 2.25

F_{cr} = 32.5 ksi

$\phi_c P_n$ = 588.1 k

Second Order Effects

P_{el} = 4041.5 k
C_m = 1.0

AISC Manual Eq. A-8-5

$B_{1,BUCKLING}$ = 1.02
$B_{1,POST\text{-}BUCKLING}$ = 1.02

AISC Manual Eq. A-8-3

Step	Computations	Reference
Member Forces		

MEMBER: **BM-5** *(CASE III - 70% SCBF - 30% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_u =	74.7	k	P_u =	74.7	k
	V_u =	45.7	k	V_u =	38.6	k
	(+) M_u =	578.7972	k-ft	(+) M_u =	525.5842	k-ft
	(-) M_u =	-366.521	k-ft	(-) M_u =	-419.734	k-ft
Post-Buckling / Yielding	P_u =	62.2	k	P_u =	62.2	k
	V_u =	55.4	k	V_u =	48.3	k
	(+) M_u =	725.2782	k-ft	(+) M_u =	672.0652	k-ft
	(-) M_u =	-513.002	k-ft	(-) M_u =	-566.215	k-ft

Step	Computations	Reference

Available Strengths

CASE III - 70% SCBF - 30% MRF

Member	(+) $\phi_b M_n$ (k-ft)	(-) $\phi_b M_n$ (k-ft)	$\phi_c P_n$ (k)	$\phi_v V_n$ (k)
BM-5	840	765	588.1	340
BM-4	574	NA	274.7	306
BM-3	358	264	274.7	217
BM-2	574	NA	274.7	306
BM-1	358	264	274.7	217

MEMBER: **BM-1** *(CASE III - 70% SCBF - 30% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.057	Eq. H1-1b	P_r/P_c =	0.057	Eq. H1-1b
	V:	0.078	GOOD	V:	0.045	GOOD
	(+) M:	0.439	GOOD	(+) M:	0.291	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0.040	Eq. H1-1b	P_r/P_c =	0.040	Eq. H1-1b
	V:	0.103	GOOD	V:	0.070	GOOD
	(+) M:	0.657	GOOD	(+) M:	0.508	GOOD
	(-) M:	0.080	GOOD	(-) M:	0.281	GOOD

MEMBER: **BM-2** *(CASE III - 70% SCBF - 30% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.317	Eq. H1-1a	P_r/P_c =	0.317	Eq. H1-1a
	V:	0.065	GOOD	V:	0.023	GOOD
	(+) M:	0.481	GOOD	(+) M:	0.399	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0.688	Eq. H1-1a	P_r/P_c =	0.688	Eq. H1-1a
	V:	0.046	GOOD	V:	0.023	GOOD
	(+) M:	0.852	GOOD	(+) M:	0.770	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA

Step	Computations	Reference
Combined Loading & Shear Checks		

MEMBER: **BM-3** *(CASE III - 70% SCBF - 30% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.000	Eq. H1-1b	P_r/P_c =	0.000	Eq. H1-1b
	V:	0.065	GOOD	V:	0.033	GOOD
	(+) M:	0.296	GOOD	(+) M:	0.148	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0.000	Eq. H1-1b	P_r/P_c =	0.000	Eq. H1-1b
	V:	0.065	GOOD	V:	0.033	GOOD
	(+) M:	0.296	GOOD	(+) M:	0.148	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA

MEMBER: **BM-4** *(CASE III - 70% SCBF - 30% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.325	Eq. H1-1a	P_r/P_c =	0.325	Eq. H1-1a
	V:	0.046	GOOD	V:	0.023	GOOD
	(+) M:	0.489	GOOD	(+) M:	0.407	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA
Post-Buckling / Yielding	P_r/P_c =	0.505	Eq. H1-1a	P_r/P_c =	0.505	Eq. H1-1a
	V:	0.046	GOOD	V:	0.023	GOOD
	(+) M:	0.670	GOOD	(+) M:	0.587	GOOD
	(-) M:	NA	NA	(-) M:	NA	NA

MEMBER: **BM-5** *(CASE III - 70% SCBF - 30% MRF)*

	Load Combination 5			Load Combination 7		
Buckling / Yielding	P_r/P_c =	0.127	Eq. H1-1b	P_r/P_c =	0.127	Eq. H1-1b
	V:	0.134	GOOD	V:	0.113	GOOD
	(+) M:	0.753	GOOD	(+) M:	0.689	GOOD
	(-) M:	0.543	GOOD	(-) M:	0.612	GOOD
Post-Buckling / Yielding	P_r/P_c =	0.106	Eq. H1-1b	P_r/P_c =	0.106	Eq. H1-1b
	V:	0.163	GOOD	V:	0.142	GOOD
	(+) M:	0.916	GOOD	(+) M:	0.853	GOOD
	(-) M:	0.723	GOOD	(-) M:	0.793	GOOD

Appendix C - Special Moment-Resisting Frame Design

MEMBER PROPERTIES

Step	Computations	Reference
Members and Member Properties		

CASE I - 80% SCBF - 20% MRF

2nd Floor Member - BM-1

Try:	W24x84		F_y =	50	ksi
A_g =	24.7	in^2	h/t_w =	45.9	
d =	24.1	in	I_x =	2370	in^4
t_w =	0.47	in	S_x =	196	in^3
b_f =	9.02	in	r_x =	9.79	in
t_f =	0.77	in	Z_x =	224	in^3
k_{des} =	1.27	in	I_y =	94.4	in^4
k_{det} =	1.6875	in	r_y =	1.95	in
k_1 =	1.0625	in	h_0 =	23.3	in
T =	20.75	in	J =	3.7	in^4
$b_f/2t_f$ =	5.86		C_w =	12800	in^6

3rd Floor Member - BM-2

Try:	W24x84		F_y =	50	ksi
A_g =	24.7	in^2	h/t_w =	45.9	
d =	24.1	in	I_x =	2370	in^4
t_w =	0.47	in	S_x =	196	in^3
b_f =	9.02	in	r_x =	9.79	in
t_f =	0.77	in	Z_x =	224	in^3
k_{des} =	1.27	in	I_y =	94.4	in^4
k_{det} =	1.6875	in	r_y =	1.95	in
k_1 =	1.0625	in	h_0 =	23.3	in
T =	20.75	in	J =	3.7	in^4
$b_f/2t_f$ =	5.86		C_w =	12800	in^6

4th Floor Member - BM-3

Try:	W24x76		F_y =	50	ksi
A_g =	22.4	in^2	h/t_w =	49	
d =	23.9	in	I_x =	2100	in^4
t_w =	0.44	in	S_x =	176	in^3
b_f =	8.99	in	r_x =	9.69	in
t_f =	0.68	in	Z_x =	200	in^3
k_{des} =	1.18	in	I_y =	82.5	in^4
k_{det} =	1.5625	in	r_y =	1.92	in
k_1 =	1.0625	in	h_0 =	23.2	in
T =	20.75	in	J =	2.68	in^4
$b_f/2t_f$ =	6.61		C_w =	11100	in^6

Step | Computations | Reference

Members and Member Properties (cont.)

CASE I - 80% SCBF - 20% MRF (cont.)

5th Floor Member - BM-4

Try: W24x76

A_g = 22.4 in^2

d = 23.9 in

t_w = 0.44 in

b_f = 8.99 in

t_f = 0.68 in

k_{des} = 1.18 in

k_{det} = 1.5625 in

k_1 = 1.0625 in

T = 20.75 in

$b_f/2t_f$ = 6.61

F_y = 50 ksi

h/t_w = 49

I_x = 2100 in^4

S_x = 176 in^3

r_x = 9.69 in

Z_x = 200 in^3

I_y = 82.5 in^4

r_y = 1.92 in

h_0 = 23.2 in

J = 2.68 in^4

C_w = 11100 in^6

Roof Member - BM-5

Try: W24x62

A_g = 18.2 in^2

d = 23.7 in

t_w = 0.43 in

b_f = 7.04 in

t_f = 0.59 in

k_{des} = 1.09 in

k_{det} = 1.5 in

k_1 = 1.0625 in

T = 20.75 in

$b_f/2t_f$ = 5.97

F_y = 50 ksi

h/t_w = 50.1

I_x = 1550 in^4

S_x = 131 in^3

r_x = 9.23 in

Z_x = 153 in^3

I_y = 34.5 in^4

r_y = 1.38 in

h_0 = 23.1 in

J = 1.71 in^4

C_w = 4620 in^6

Columns

Try: W24x207

A_g = 60.7 in^2

d = 25.7 in

t_w = 8.7 in

b_f = 13 in

t_f = 1.57 in

k_{des} = 2.07 in

k_{det} = 2.5 in

k_1 = 1.25 in

T = 20.75 in

$b_f/2t_f$ = 4.14

F_y = 50 ksi

h/t_w = 24.8

I_x = 6820 in^4

S_x = 531 in^3

r_x = 10.6 in

Z_x = 606 in^3

I_y = 578 in^4

r_y = 3.08 in

h_0 = 24.1 in

J = 38.3 in^4

C_w = 84100 in^6

Step	Computations	Reference
Members and Member Properties	***CASE II - 75% SCBF - 25% MRF***	

2nd Floor Member - BM-1

Try:	W24x103		F_y =	50	ksi
A_g =	30.3	in^2	h/t_w =	39.2	
d =	24.5	in	I_x =	3000	in^4
t_w =	0.55	in	S_x =	245	in^3
b_f =	9	in	r_x =	10	in
t_f =	0.98	in	Z_x =	280	in^3
k_{des} =	1.48	in	I_y =	119	in^4
k_{det} =	1.785	in	r_y =	1.99	in
k_1 =	1.125	in	h_0 =	23.5	in
T =	20.75	in	J =	7.07	in^4
$b_f/2t_f$ =	4.59		C_w =	16600	in^6

3rd Floor Member - BM-2

Try:	W24x103		F_y =	50	ksi
A_g =	30.3	in^2	h/t_w =	39.2	
d =	24.5	in	I_x =	3000	in^4
t_w =	0.55	in	S_x =	245	in^3
b_f =	9	in	r_x =	10	in
t_f =	0.98	in	Z_x =	280	in^3
k_{des} =	1.48	in	I_y =	119	in^4
k_{det} =	1.785	in	r_y =	1.99	in
k_1 =	1.125	in	h_0 =	23.5	in
T =	20.75	in	J =	7.07	in^4
$b_f/2t_f$ =	4.59		C_w =	16600	in^6

4th Floor Member - BM-3

Try:	W24x84		F_y =	50	ksi
A_g =	24.7	in^2	h/t_w =	45.9	
d =	24.1	in	I_x =	2370	in^4
t_w =	0.47	in	S_x =	196	in^3
b_f =	9.02	in	r_x =	9.79	in
t_f =	0.77	in	Z_x =	224	in^3
k_{des} =	1.27	in	I_y =	94.4	in^4
k_{det} =	1.6875	in	r_y =	1.95	in
k_1 =	1.0625	in	h_0 =	23.3	in
T =	20.75	in	J =	3.7	in^4
$b_f/2t_f$ =	5.86		C_w =	12800	in^6

Step	Computations	Reference

Members and Member Properties (cont.)

CASE II - 75% SCBF - 25% MRF

5th Floor Member - BM-4

Try:	W24x84		F_y =	50	ksi
A_g =	24.7	in^2	h/t_w =	45.9	
d =	24.1	in	I_x =	2370	in^4
t_w =	0.47	in	S_x =	196	in^3
b_f =	9.02	in	r_x =	9.79	in
t_f =	0.77	in	Z_x =	224	in^3
k_{des} =	1.27	in	I_y =	94.4	in^4
k_{det} =	1.6875	in	r_y =	1.95	in
k_1 =	1.0625	in	h_0 =	23.3	in
T =	20.75	in	J =	3.7	in^4
$b_f/2t_f$ =	5.86		C_w =	12800	in^6

Roof Member - BM-5

Try:	W24x76		F_y =	50	ksi
A_g =	22.4	in^2	h/t_w =	49	
d =	23.9	in	I_x =	2100	in^4
t_w =	0.44	in	S_x =	176	in^3
b_f =	8.99	in	r_x =	9.69	in
t_f =	0.68	in	Z_x =	200	in^3
k_{des} =	1.18	in	I_y =	82.5	in^4
k_{det} =	1.5625	in	r_y =	1.92	in
k_1 =	1.0625	in	h_0 =	23.2	in
T =	20.75	in	J =	2.68	in^4
$b_f/2t_f$ =	6.61		C_w =	11100	in^6

Columns

Try:	W27x258		F_y =	50	ksi
A_g =	76.1	in^2	h/t_w =	24.4	
d =	29	in	I_x =	10800	in^4
t_w =	0.98	in	S_x =	745	in^3
b_f =	14.3	in	r_x =	11.9	in
t_f =	1.77	in	Z_x =	852	in^3
k_{des} =	2.56	in	I_y =	859	in^4
k_{det} =	2.6875	in	r_y =	3.36	in
k_1 =	1.3125	in	h_0 =	27.2	in
T =	23.625	in	J =	61.6	in^4
$b_f/2t_f$ =	4.03		C_w =	159000	in^6

Step	Computations	Reference

Members and Member Properties

CASE III - 70% SCBF - 30% MRF

2nd Floor Member - BM-1

Try:	W24x131		F_y =	50	ksi
A_g =	38.6	in^2	h/t_w =	35.6	
d =	24.5	in	I_x =	4020	in^4
t_w =	0.605	in	S_x =	329	in^3
b_f =	13.9	in	r_x =	10.2	in
t_f =	0.96	in	Z_x =	370	in^3
k_{des} =	1.46	in	I_y =	340	in^4
k_{det} =	1.785	in	r_y =	2.97	in
k_1 =	1.125	in	h_0 =	23.5	in
T =	20.75	in	J =	9.5	in^4
$b_f/2t_f$ =	6.7		C_w =	47100	in^6

3rd Floor Member - BM-2

Try:	W24x131		F_y =	50	ksi
A_g =	38.6	in^2	h/t_w =	35.6	
d =	24.5	in	I_x =	4020	in^4
t_w =	0.605	in	S_x =	329	in^3
b_f =	13.9	in	r_x =	10.2	in
t_f =	0.96	in	Z_x =	370	in^3
k_{des} =	1.46	in	I_y =	340	in^4
k_{det} =	1.785	in	r_y =	2.97	in
k_1 =	1.125	in	h_0 =	23.5	in
T =	20.75	in	J =	9.5	in^4
$b_f/2t_f$ =	6.7		C_w =	47100	in^6

4th Floor Member - BM-3

Try:	W24x103		F_y =	50	ksi
A_g =	30.3	in^2	h/t_w =	39.2	
d =	24.5	in	I_x =	3000	in^4
t_w =	0.55	in	S_x =	245	in^3
b_f =	9	in	r_x =	10	in
t_f =	0.98	in	Z_x =	280	in^3
k_{des} =	1.48	in	I_y =	119	in^4
k_{det} =	1.785	in	r_y =	1.99	in
k_1 =	1.125	in	h_0 =	23.5	in
T =	20.75	in	J =	7.07	in^4
$b_f/2t_f$ =	4.59		C_w =	16600	in^6

Members and
Member Properties
(cont.)

CASE III - 70% SCBF - 30% MRF

5th Floor Member - BM-4

Try:	W24x103		F_y =	50	ksi
A_g =	30.3	in^2	h/t_w =	39.2	
d =	24.5	in	I_x =	3000	in^4
t_w =	0.55	in	S_x =	245	in^3
b_f =	9	in	r_x =	10	in
t_f =	0.98	in	Z_x =	280	in^3
k_{des} =	1.48	in	I_y =	119	in^4
k_{det} =	1.785	in	r_y =	1.99	in
k_1 =	1.125	in	h_0 =	23.5	in
T =	20.75	in	J =	7.07	in^4
$b_f/2t_f$ =	4.59		C_w =	16600	in^6

Roof Member - BM-5

Try:	W24x84		F_y =	50	ksi
A_g =	24.7	in^2	h/t_w =	45.9	
d =	24.1	in	I_x =	2370	in^4
t_w =	0.47	in	S_x =	196	in^3
b_f =	9.02	in	r_x =	9.79	in
t_f =	0.77	in	Z_x =	224	in^3
k_{des} =	1.27	in	I_y =	94.4	in^4
k_{det} =	1.6875	in	r_y =	1.95	in
k_1 =	1.0625	in	h_0 =	23.3	in
T =	20.75	in	J =	3.7	in^4
$b_f/2t_f$ =	5.86		C_w =	12800	in^6

Columns

Try:	W27x258		F_y =	50	ksi
A_g =	76.1	in^2	h/t_w =	24.4	
d =	29	in	I_x =	10800	in^4
t_w =	0.98	in	S_x =	745	in^3
b_f =	14.3	in	r_x =	11.9	in
t_f =	1.77	in	Z_x =	852	in^3
k_{des} =	2.56	in	I_y =	859	in^4
k_{det} =	2.6875	in	r_y =	3.36	in
k_1 =	1.3125	in	h_0 =	27.2	in
T =	23.625	in	J =	61.6	in^4
$b_f/2t_f$ =	4.03		C_w =	159000	in^6

STABILITY CHECK

Step	Computations	Reference
Inputs	C_d = 5.5 I_e = 1.0 θ_{max} = 0.0909 rad	ASCE/SEI 7-10 Tbl. 12.2-1 ASCE/SEI 7-10 Tbl. 1.5-2

Step	Computations	Reference
Check Frame Stability & Story Drifts	***CASE I - 80% SCBF - 20% MRF***	

BM-1 & 2 (all units for immediately below are inches)

a = 5.5	b = 18	c = 2
$.5b_f$ = 4.51	.65d = 15.67	$.1b_f$ = 0.90
$.75b_f$ = 6.77	.85d = 20.49	$.25b_f$ = 2.26
GOOD	GOOD	GOOD

AISC 358 Eq. 5.8-(1, 2, & 3)

A_Δ = 1.089

AISC 358 Sect. 5.8

$\delta_{MRF,2}$ = 0.181 in	$\delta_{MRF,3}$ = 0.449 in
h_{s2} = 168 in	h_{s3} = 144 in
δ_2 = 0.40 in	δ_3 = 0.90 in
Δ_2 = 2.19 in	Δ_3 = 2.76 in
Δ_{all} = 3.36 in	Δ_{all} = 2.88 in
GOOD	GOOD

RISA Models
ASCE/SEI 7-10
Eq. 12.8-15 &
Tbl. 12.12-1

P_2 = 215 k	P_3 = 164 k
V_2 = 323 k	V_3 = 293.2 k
θ_2 = 0.0016 rad	θ_3 = 0.0019 rad
$\theta_{2,ADJ}$ = 0.0016 rad	$\theta_{3,ADJ}$ = 0.0019 rad
GOOD	GOOD

RISA Models & Load Calculations
ASCE/SEI 7-10
Eq. 12.8-16 &
Eq. 12.8-17

BM-3 & 4 (all units for below are inches)

a = 5.5	b = 18	c = 2
$.5b_f$ = 4.50	.65d = 15.54	$.1b_f$ = 0.90
$.75b_f$ = 6.74	.85d = 20.32	$.25b_f$ = 2.25
GOOD	GOOD	GOOD

AISC 358 Eq. 5.8-(1, 2, & 3)

A_Δ = 1.089

AISC 358 Sect. 5.8

$\delta_{MRF,4}$ = 0.717 in	$\delta_{MRF,5}$ = 0.935 in
h_{s4} = 144 in	h_{s5} = 144 in
δ_4 = 1.42 in	δ_5 = 1.82 in
Δ_4 = 2.84 in	Δ_5 = 2.23 in
Δ_{all} = 2.88 in	Δ_{all} = 2.88 in
GOOD	GOOD

RISA Models
ASCE/SEI 7-10
Eq. 12.8-15 &
Tbl. 12.12-1

P_4 = 114 k	P_5 = 63 k
V_4 = 238.4 k	V_5 = 158.3 k
θ_4 = 0.0017 rad	θ_5 = 0.0011 rad
$\theta_{4,ADJ}$ = 0.0017 rad	$\theta_{5,ADJ}$ = 0.0011 rad
GOOD	GOOD

RISA Models & Load Calculations
ASCE/SEI 7-10
Eq. 12.8-16 &
Eq. 12.8-17

Step	Computations	Reference

Check Frame Stability & Story Drifts (cont.)

CASE I - 80% SCBF - 20% MRF

BM-5 (all units for below are inches)

a = 4.25	b = 18	c = 1.5
$.5b_f$ = 3.52	.65d = 15.41	$.1b_f$ = 0.70
$.75b_f$ = 5.28	.85d = 20.15	$.25b_f$ = 1.76
GOOD	GOOD	GOOD

AISC 358
Eq. 5.8-(1, 2, & 3)

A_Δ = 1.085

AISC 358 Sect. 5.8

$\delta_{MRF,R}$ = 1.093 in
h_{sR} = 144 in
δ_R = 2.14 in
Δ_R = 1.75 in
Δ_{all} = 2.88 in
GOOD

RISA Models
ASCE/SEI 7-10
Eq. 12.8-15 &
Tbl. 12.12-1

P_R = 13 k
V_R = 53 k
θ_R = 0.0005 rad
$\theta_{R,ADJ}$ = 0.0005 rad
GOOD

RISA Models & Load Calculations
ASCE/SEI 7-10
Eq. 12.8-16 &
Eq. 12.8-17

Step	Computations	Reference

Check Frame Stability & Story Drifts

CASE II - 75% SCBF - 25% MRF

BM-1 & 2 (all units for immediately below are inches)

a = 5.5	b = 18	c = 2
$.5b_f$ = 4.50	.65d = 15.93	$.1b_f$ = 0.90
$.75b_f$ = 6.75	.85d = 20.83	$.25b_f$ = 2.25
GOOD	GOOD	GOOD

AISC 358
Eq. 5.8-(1, 2, & 3)

A_Δ = 1.089

AISC 358 Sect. 5.8

$\delta_{MRF,2}$ = 0.16 in	$\delta_{MRF,3}$ = 0.407 in
h_{s2} = 168 in	h_{s3} = 144 in
δ_2 = 0.36 in	δ_3 = 0.83 in
Δ_2 = 1.99 in	Δ_3 = 2.56 in
Δ_{all} = 3.36 in	Δ_{all} = 2.88 in
GOOD	GOOD

RISA Models
ASCE/SEI 7-10
Eq. 12.8-15 &
Tbl. 12.12-1

P_2 = 215 k	P_3 = 164 k
V_2 = 323 k	V_3 = 293.2 k
θ_2 = 0.0014 rad	θ_3 = 0.0018 rad
$\theta_{2,ADJ}$ = 0.0014 rad	$\theta_{3,ADJ}$ = 0.0018 rad
GOOD	GOOD

RISA Models & Load Calculations
ASCE/SEI 7-10
Eq. 12.8-16 &
Eq. 12.8-17

BM-3 & 4 (all units for below are inches)

a = 5.5	b = 18	c = 1.5
$.5b_f$ = 4.51	.65d = 15.67	$.1b_f$ = 0.90
$.75b_f$ = 6.77	.85d = 20.49	$.25b_f$ = 2.26
GOOD	GOOD	GOOD

AISC 358
Eq. 5.8-(1, 2, & 3)

A_Δ = 1.067

AISC 358 Sect. 5.8

$\delta_{MRF,4}$ = 0.664 in	$\delta_{MRF,5}$ = 0.88 in
h_{s4} = 144 in	h_{s5} = 144 in
δ_4 = 1.34 in	δ_5 = 1.76 in
Δ_4 = 2.79 in	Δ_5 = 2.31 in
Δ_{all} = 2.88 in	Δ_{all} = 2.88 in
GOOD	GOOD

RISA Models
ASCE/SEI 7-10
Eq. 12.8-15 &
Tbl. 12.12-1

P_4 = 114 k	P_5 = 63 k
V_4 = 238.4 k	V_5 = 158.3 k
θ_4 = 0.0017 rad	θ_5 = 0.0012 rad
$\theta_{4,ADJ}$ = 0.0017 rad	$\theta_{5,ADJ}$ = 0.0012 rad
GOOD	GOOD

RISA Models & Load Calculations
ASCE/SEI 7-10
Eq. 12.8-16 &
Eq. 12.8-17

Step	Computations	Reference
Check Frame Stability & Story Drifts (cont.)	***CASE II - 75% SCBF - 25% MRF*** BM-5 (all units for below are inches)	
	$a =$ 5.5 $b =$ 18 $c =$ 1.5 $.5b_f =$ 4.50 $.65d =$ 15.54 $.1b_f =$ 0.90 $.75b_f =$ 6.74 $.85d =$ 20.32 $.25b_f =$ 2.25 GOOD GOOD GOOD	AISC 358 Eq. 5.8-(1, 2, & 3)
	$A_\Delta =$ 1.067	AISC 358 Sect. 5.8
	$\delta_{MRF,R} =$ 1.049 in $h_{sR} =$ 144 in $\delta_R =$ 2.09 in $\Delta_R =$ 1.83 in $\Delta_{all} =$ 2.88 in GOOD	RISA Models ASCE/SEI 7-10 Eq. 12.8-15 & Tbl. 12.12-1
	$P_R =$ 13 k $V_R =$ 53 k $\theta_R =$ 0.0006 rad $\theta_{R,ADJ} =$ 0.0006 rad GOOD	RISA Models & Load Calculations ASCE/SEI 7-10 Eq. 12.8-16 & Eq. 12.8-17

Step	Computations	Reference

Check Frame Stability & Story Drifts

CASE III - 70% SCBF - 30% MRF

BM-1 & 2 (all units for immediately below are inches)

a =	7	b =	18	c =	2
$.5b_f$ =	6.95	.65d =	15.93	$.1b_f$ =	1.39
$.75b_f$ =	10.43	.85d =	20.83	$.25b_f$ =	3.48
	GOOD		GOOD		GOOD

AISC 358
Eq. 5.8-(1, 2, & 3)

A_Δ = 1.058

AISC 358 Sect. 5.8

$\delta_{MRF,2}$ =	0.17	in	$\delta_{MRF,3}$ =	0.423	in
h_{s2} =	168	in	h_{s3} =	144	in
δ_2 =	0.37	in	δ_3 =	0.83	in
Δ_2 =	2.02	in	Δ_3 =	2.57	in
Δ_{all} =	3.36	in	Δ_{all} =	2.88	in
	GOOD			GOOD	

RISA Models
ASCE/SEI 7-10
Eq. 12.8-15 &
Tbl. 12.12-1

P_2 =	215	k	P_3 =	164	k
V_2 =	323	k	V_3 =	293.2	k
θ_2 =	0.0015	rad	θ_3 =	0.0018	rad
$\theta_{2,ADJ}$ =	0.0015	rad	$\theta_{3,ADJ}$ =	0.0018	rad
	GOOD			GOOD	

RISA Models & Load Calculations
ASCE/SEI 7-10
Eq. 12.8-16 &
Eq. 12.8-17

BM-3 & 4 (all units for below are inches)

a =	7.5	b =	18	c =	1.75
$.5b_f$ =	4.50	.65d =	15.93	$.1b_f$ =	0.90
$.75b_f$ =	6.75	.85d =	20.83	$.25b_f$ =	2.25
	NG		GOOD		GOOD

AISC 358
Eq. 5.8-(1, 2, & 3)

A_Δ = 1.078

AISC 358 Sect. 5.8

$\delta_{MRF,4}$ =	0.681	in	$\delta_{MRF,5}$ =	0.9	in
h_{s4} =	144	in	h_{s5} =	144	in
δ_4 =	1.34	in	δ_5 =	1.76	in
Δ_4 =	2.80	in	Δ_5 =	2.27	in
Δ_{all} =	2.88	in	Δ_{all} =	2.88	in
	GOOD			GOOD	

RISA Models
ASCE/SEI 7-10
Eq. 12.8-15 &
Tbl. 12.12-1

P_4 =	114	k	P_5 =	63	k
V_4 =	238.4	k	V_5 =	158.3	k
θ_4 =	0.0017	rad	θ_5 =	0.0011	rad
$\theta_{4,ADJ}$ =	0.0017	rad	$\theta_{5,ADJ}$ =	0.0011	rad
	GOOD			GOOD	

RISA Models & Load Calculations
ASCE/SEI 7-10
Eq. 12.8-16 &
Eq. 12.8-17

Step	Computations	Reference
Check Frame Stability & Story Drifts (cont.)	***CASE III - 70% SCBF - 30% MRF*** BM-5 (all units for below are inches)	
	a = 5.5 $.5b_f$ = 4.51 $.75b_f$ = 6.77 GOOD b = 18 $.65d$ = 15.67 $.85d$ = 20.49 GOOD c = 1.5 $.1b_f$ = 0.90 $.25b_f$ = 2.26 GOOD	AISC 358 Eq. 5.8-(1, 2, & 3)
	A_Δ = 1.067	AISC 358 Sect. 5.8
	$\delta_{MRF,R}$ = 1.06 in h_{sR} = 144 in δ_R = 2.06 in Δ_R = 1.67 in Δ_{all} = 2.88 in GOOD	RISA Models ASCE/SEI 7-10 Eq. 12.8-15 & Tbl. 12.12-1
	P_R = 13 k V_R = 53 k θ_R = 0.0005 rad $\theta_{R,ADJ}$ = 0.0005 rad GOOD	RISA Models & Load Calculations ASCE/SEI 7-10 Eq. 12.8-16 & Eq. 12.8-17

COLUMNS

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Step	Computations	Reference
Inputs	F_y = 50 ksi ϕ_c = 0.9	AISC Manual Tbl. 2-4 AISC Manual Sect. G1

Step	Computations	Reference
Column Strength Check	***CASE I - 80% SCBF - 20% MRF***	
	P_u = 337 k V_u = 21.1 k P_u = 293 k M_u = 246 k-ft	ASCE/SEI 7-10 Load Comb. 5: 12.4.2.3 & 12.4.3.2
	k = 1.2 h = 14 ft kL/r_y = **65.45** $4.71\sqrt{E/F_y}$ = **113.43**	
	F_e = **66.81** ksi F_{cr} = **36.55** ksi	AISC Manual Eq. E3-(2, 3), 4, 1
	$\phi_c P_n$ = **1997** k GOOD	
	$\phi_v V_n$ = 671 k GOOD	AISC Manual Tbl. 3-2
	$\phi_b M_n$ = 2180 k-ft	AISC Manual Tbl. 3-10
	P_r/P_c = **0.15** Use Eq. H1-1b	
	Comb. Loading: **0.19** GOOD	AISC Manual Eq. H1-1(a,b)
	CASE II - 75% SCBF - 25% MRF	
	P_u = 352 k V_u = 24.7 k P_u = 298 k M_u = 326 k-ft	ASCE/SEI 7-10 Load Comb. 5: 12.4.2.3 & 12.4.3.2
	k = 1.2 h = 14 ft kL/r_y = **60.00** $4.71\sqrt{E/F_y}$ = **113.43**	
	F_e = **79.51** ksi F_{cr} = **38.43** ksi	AISC Manual Eq. E3-(2, 3), 4, 1
	$\phi_c P_n$ = **2632** k GOOD	
	$\phi_v V_n$ = 853 k GOOD	AISC Manual Tbl. 3-2
	$\phi_b M_n$ = 3120 k-ft	AISC Manual Tbl. 3-10
	P_r/P_c = **0.11** Use Eq. H1-1b	
	Comb. Loading: **0.16** GOOD	AISC Manual Eq. H1-1(a,b)

Step	Computations	Reference
Column Strength Check (cont.)	***CASE III - 70% SCBF - 30% MRF***	
	P_u = 337 k V_u = 21.1 k P_u = 293 k M_u = 246 k-ft	ASCE/SEI 7-10 Load Comb. 5: 12.4.2.3 & 12.4.3.2
	k = 1.2 h = 14 ft kL/r_y = 60.00 $4.71\sqrt{E/F_y}$ = 113.43	
	F_e = 79.51 ksi F_{cr} = 38.43 ksi	AISC Manual Eq. E3-(2, 3), 4, 1
	$\phi_c P_n$ = 2632 k GOOD	
	$\phi_v V_n$ = 853 k GOOD	AISC Manual Tbl. 3-2
	$\phi_b M_n$ = 3120 k-ft	AISC Manual Tbl. 3-10
	P_r/P_c = 0.11 Use Eq. H1-1b	
	Comb. Loading: 0.13 GOOD	AISC Manual Eq. H1-1(a,b)

BEAMS

Step	Computations	Reference
Inputs	F_y = 50 ksi ϕ_b = 0.9	AISC Manual Sect. F1(1) & Tbl. 2-4
	R_y = 1.1	AISC Seismic Prov. Tbl. A3.1
	C_d = 1 ϕ_c = 0.75	AISC Manual App. 6.3.1a

Step	Computations					Reference
	CASE I - 80% SCBF - 20% MRF					
Beam Strength Check	BM-1 & BM -2					
	M_u =	195	k-ft	2340	k-in	ASCE/SEI 7-10 Load Comb. 5: 12.4.2.3 & 12.4.3.2
	V_u =	23	k			
	$M_{u,RBS}$ =	175	k-ft	2100	k-in	
	P_u =	0	k			
	L =	30	ft			
	s =	25	ft			AISC Seismic Prov. Tbl. 4-2
	$P_{\text{all for ductility}}$ =	506	k	GOOD		
	R =	21.25	in			
	$L_{b,\ max}$=	8.11	ft			AISC Seismic Prov. Tbl. 4-2
	Provide bracing at:	4th points				
	L_b=	7.5	ft	GOOD		
	$\phi_b M_n$ =	825	k-ft	GOOD		AISC Manual Tbl. 3-10
	$Z_{x,RBS}$ =	152.1436	in^3			AISC 358 Eq. 5.8-4
	$\phi_b M_n$ = 6846.462 k-in	570.5385	k-ft	GOOD		AISC Manal Eq. F2-1 & AISC Manal Tbl. 3-2
	$\phi_v V_n$ =	340	k	GOOD		
	M_r =	12320	k-in			AISC Seismic Prov. Eq. D1-1a
	P_{urb} =	10.58	k-in			AISC Seismic Prov. Eq. A-6-7, 8
	β_{br} =	78.33	k/in			
	Try: L5x5x5/16	A =	3.07	in^2		AISC Manual Tbl. 1-1
	L_{br} =	300.9	in	25.08	ft	
	θ =	0.078	rad			
	k =	294.1	k/in	GOOD		

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Step	Computations	Reference
Beam Strength Check (cont.)	***CASE I - 80% SCBF - 20% MRF*** BM-3 & BM -4	
	M_u = 170 k-ft 2040 k-in V_u = 21 k $M_{u,RBS}$ = 160 k-ft 1920 k-in P_u = 0 k	ASCE/SEI 7-10 Load Comb. 5: 12.4.2.3 & 12.4.3.2
	L = 30 ft s = 25 ft $P_{all\ for\ ductility}$ = 290 k GOOD	AISC Seismic Prov. Tbl. 4-2
	R = 21.25 in $L_{b,\ max}$= 7.98 ft	AISC Seismic Prov. Tbl. 4-2
	Provide bracing at: 4th points L_b= 7.5 ft GOOD	
	$\phi_b M_n$ = 735 k-ft GOOD	AISC Manual Tbl. 3-10
	$Z_{x,RBS}$ = 136.8416 in^3 $\phi_b M_n$ = 6157.872 k-in 513.156 k-ft GOOD	AISC 358 Eq. 5.8-4 AISC Manal Eq. F2-1 & AISC Manal Tbl. 3-2
	$\phi_v V_n$ = 315 k GOOD	
	M_r = 11000 k-in	AISC Seismic Prov. Eq. D1-1a
	P_{urb} = 9.48 k-in β_{br} = 70.24 k/in	AISC Seismic Prov. Eq. A-6-7, 8
	Try: L5x5x5/16 A = 3.07 in^2	AISC Manual Tbl. 1-1
	L_{br} = 300.9 in 25.07 ft θ = 0.077 rad k = 294.1 k/in GOOD	

Step	Computations	Reference
Beam Strength Check (cont.)	***CASE I - 80% SCBF - 20% MRF***	
	BM-5	
	M_u = 62 k-ft 744 k-in	ASCE/SEI 7-10 Load Comb. 5: 12.4.2.3 & 12.4.3.2
	V_u = 6.9 k	
	$M_{u,RBS}$ = 58 k-ft 696 k-in	
	P_u = 0 k	
	L = 30 ft	
	s = 25 ft	AISC Seismic Prov. Tbl. 4-2
	$P_{all\ for\ ductility}$ = 187 k GOOD	
	R = 27.75 in	AISC Seismic Prov. Tbl. 4-2
	$L_{b,\ max}$= 5.74 ft	
	Provide bracing at: 6th points	
	L_b= 5 ft GOOD	
	$\phi_b M_n$ = 570 k-ft GOOD	AISC Manual Tbl. 3-10
	$Z_{x,RBS}$ = 112.0953 in^3	AISC 358 Eq. 5.8-4
	$\phi_b M_n$ = 5044.289 k-in 420.3574 k-ft GOOD	AISC Manal Eq. F2-1 & AISC Manal Tbl. 3-2
	$\phi_v V_n$ = 306 k GOOD	
	M_r = 8415 k-in	AISC Seismic Prov. Eq. D1-1a
	P_{urb} = 7.29 k-in	AISC Seismic Prov. Eq. A-6-7, 8
	β_{br} = 80.95 k/in	
	Try: L5x5x5/16 A = 3.07 in^2	AISC Manual Tbl. 1-1
	L_{br} = 300.9 in 25.07 ft	
	θ = 0.077 rad	
	k = 294.1 k/in GOOD	

Step	Computations						Reference
	CASE II - 75% SCBF - 25% MRF						
Beam Strength Check	BM-1 & BM -2						
	M_u =	220	k-ft	2640	k-in		ASCE/SEI 7-10 Load Comb. 5: 12.4.2.3 & 12.4.3.2
	V_u =	24.5	k				
	$M_{u,RBS}$ =	205	k-ft	2460	k-in		
	P_u =	0	k				
	L =	30	ft				
	s =	25	ft				AISC Seismic Prov. Tbl. 4-2
	$P_{all\ for\ ductility}$ =	1110	k			GOOD	
	R =	21.25	in				
	$L_{b,\ max}$=	8.27	ft				AISC Seismic Prov. Tbl. 4-2
	Provide bracing at:	4th points					
	L_b=	7.5	ft			GOOD	
	$\phi_b M_n$ =	1035	k-ft			GOOD	AISC Manual Tbl. 3-10
	$Z_{x,RBS}$ =	187.8016	in^3				AISC 358 Eq. 5.8-4
	$\phi_b M_n$ =	8451.072	k-in	704.256	k-ft	GOOD	AISC Manal Eq. F2-1 & AISC Manal Tbl. 3-2
	$\phi_v V_n$ =	404	k			GOOD	
	M_r =	15400	k-in				AISC Seismic Prov. Eq. D1-1a
	P_{urb} =	13.11	k-in				AISC Seismic Prov. Eq. A-6-7, 8
	β_{br} =	97.08	k/in				
	Try:	L5x5x5/16		A = 3.07	in^2		AISC Manual Tbl. 1-1
	L_{br} =	300.9	in	25.08	ft		
	θ =	0.078	rad				
	k =	294.1	k/in			GOOD	

Step	Computations	Reference

Beam Strength Check (cont.)

CASE II - 75% SCBF - 25% MRF

BM-3 & BM -4

M_u = 184 k-ft 2208 k-in
V_u = 22 k
$M_{u,RBS}$ = 170 k-ft 2040 k-in
P_u = 0 k

ASCE/SEI 7-10
Load Comb. 5:
12.4.2.3 & 12.4.3.2

L = 30 ft
s = 25 ft
$P_{\text{all for ductility}}$ = 506 k GOOD

AISC Seismic Prov.
Tbl. 4-2

R = 27.75 in

AISC Seismic Prov.
Tbl. 4-2

$L_{b,\ max}$= 8.11 ft

Provide bracing at: 4th points

L_b= 7.5 ft GOOD

AISC Manual
Tbl. 3-10

$\phi_b M_n$ = 825 k-ft GOOD

AISC 358
Eq. 5.8-4

$Z_{x,RBS}$ = 170.1077 in^3

AISC Manal
Eq. F2-1 &
AISC Manal
Tbl. 3-2

$\phi_b M_n$ = 7654.847 k-in 637.9039 k-ft GOOD

$\phi_v V_n$ = 340 k GOOD

AISC Seismic Prov. Eq.
D1-1a

M_r = 12320 k-in
P_{urb} = 10.58 k-in

AISC Seismic Prov. Eq.
A-6-7, 8

β_{br} = 78.33 k/in

AISC Manual
Tbl. 1-1

Try: L5x5x5/16 A = 3.07 in^2

L_{br} = 300.9 in 25.08 ft

θ = 0.078 rad

k = 294.1 k/in GOOD

Step	Computations	Reference
Beam Strength Check (cont.)	***CASE II - 75% SCBF - 25% MRF***	
	BM-5	
	M_u = 160 k-ft 1920 k-in	ASCE/SEI 7-10 Load Comb. 5: 12.4.2.3 & 12.4.3.2
	V_u = 20.5 k	
	$M_{u,RBS}$ = 141 k-ft 1692 k-in	
	P_u = 0 k	
	L = 30 ft	
	s = 25 ft	AISC Seismic Prov. Tbl. 4-2
	$P_{\text{all for ductility}}$ = 290 k GOOD	
	R = 27.75 in	AISC Seismic Prov. Tbl. 4-2
	$L_{b,\ max}$= 7.89 ft	
	Provide bracing at: 6th points	
	L_b= 5 ft GOOD	
	$\phi_b M_n$ = 750 k-ft GOOD	AISC Manual Tbl. 3-10
	$Z_{x,RBS}$ = 152.6312 in^3	AISC 358 Eq. 5.8-4
	$\phi_b M_n$ = 6868.404 k-in 572.367 k-ft GOOD	AISC Manal Eq. F2-1 & AISC Manal Tbl. 3-2
	$\phi_v V_n$ = 315 k GOOD	
	M_r = 11000 k-in	AISC Seismic Prov. Eq. D1-1a
	P_{urb} = 9.48 k-in	AISC Seismic Prov. Eq. A-6-7, 8
	β_{br} = 105.36 k/in	
	Try: L5x5x5/16 A = 3.07 in^2	AISC Manual Tbl. 1-1
	L_{br} = 300.9 in 25.07 ft	
	θ = 0.077 rad	
	k = 294.1 k/in GOOD	

Step	Computations	Reference
Beam Strength Check	***CASE III - 70% SCBF - 30% MRF***	
	BM-1 & BM -2	
	M_u = 254 k-ft 3048 k-in	ASCE/SEI 7-10 Load Comb. 5: 12.4.2.3 & 12.4.3.2
	V_u = 26.5 k	
	$M_{u,RBS}$ = 238 k-ft 2856 k-in	
	P_u = 0 k	
	L = 30 ft	
	s = 25 ft	AISC Seismic Prov. Tbl. 4-2
	$P_{all\ for\ ductility}$ = 1E+12 k GOOD	
	R = 21.25 in	AISC Seismic Prov. Tbl. 4-2
	$L_{b,\ max}$= 12.3 ft	
	Provide bracing at: 3rd points	
	L_b= 10 ft GOOD	
	$\phi_b M_n$ = 1390 k-ft GOOD	AISC Manual Tbl. 3-10
	$Z_{x,RBS}$ = 279.6064 in^3	AISC 358 Eq. 5.8-4
	$\phi_b M_n$ = 12582.29 k-in 1048.524 k-ft GOOD	AISC Manal Eq. F2-1 & AISC Manal Tbl. 3-2
	$\phi_v V_n$ = 477 k GOOD	
	M_r = 20350 k-in	AISC Seismic Prov. Eq. D1-1a
	P_{urb} = 17.32 k-in	AISC Seismic Prov. Eq. A-6-7, 8
	β_{br} = 96.22 k/in	
	Try: L5x5x5/16 A = 3.07 in^2	AISC Manual Tbl. 1-1
	L_{br} = 300.9 in 25.08 ft	
	θ = 0.078 rad	
	k = 294.1 k/in GOOD	

Step	Computations	Reference
Beam Strength Check (cont.)	***CASE III - 70% SCBF - 30% MRF***	
	BM-3 & BM -4	
	M_u = 214 k-ft 2568 k-in	ASCE/SEI 7-10 Load Comb. 5: 12.4.2.3 & 12.4.3.2
	V_u = 24 k	
	$M_{u,RBS}$ = 199 k-ft 2388 k-in	
	P_u = 0 k	
	L = 30 ft	
	s = 25 ft	AISC Seismic Prov. Tbl. 4-2
	$P_{all\ for\ ductility}$ = 1110 k GOOD	
	R = 24.01786 in	
	$L_{b,\ max}$= 8.27 ft	AISC Seismic Prov. Tbl. 4-2
	Provide bracing at: 4th points	
	L_b= 7.5 ft GOOD	
	$\phi_b M_n$ = 735 k-ft GOOD	AISC Manual Tbl. 3-10
	$Z_{x,RBS}$ = 199.3264 in^3	AISC 358 Eq. 5.8-4
	$\phi_b M_n$ = 8969.688 k-in 747.474 k-ft GOOD	AISC Manal Eq. F2-1 & AISC Manal Tbl. 3-2
	$\phi_v V_n$ = 404 k GOOD	
	M_r = 15400 k-in	AISC Seismic Prov. Eq. D1-1a
	P_{urb} = 13.11 k-in	AISC Seismic Prov. Eq. A-6-7, 8
	β_{br} = 97.08 k/in	
	Try: L5x5x5/16 A = 3.07 in^2	AISC Manual Tbl. 1-1
	L_{br} = 300.9 in 25.08 ft	
	θ = 0.078 rad	
	k = 294.1 k/in GOOD	

Step	Computations	Reference
Beam Strength Check (cont.)	***CASE III - 70% SCBF - 30% MRF***	

BM-5

M_u = 85 k-ft 1020 k-in

V_u = 8.5 k

$M_{u,RBS}$ = 79 k-ft 948 k-in

P_u = 0 k

ASCE/SEI 7-10 Load Comb. 5: 12.4.2.3 & 12.4.3.2

L = 30 ft

s = 25 ft

$P_{all\ for\ ductility}$ = 506 k GOOD

AISC Seismic Prov. Tbl. 4-2

R = 27.75 in

$L_{b,\ max}$= 8.11 ft

AISC Seismic Prov. Tbl. 4-2

Provide bracing at: 4th points

L_b= 7.5 ft GOOD

$\phi_b M_n$ = 825 k-ft GOOD

AISC Manual Tbl. 3-10

$Z_{x,RBS}$ = 170.1077 in^3

AISC 358 Eq. 5.8-4

$\phi_b M_n$ = 7654.847 k-in 637.9039 k-ft GOOD

AISC Manal Eq. F2-1 & AISC Manal Tbl. 3-2

$\phi_v V_n$ = 340 k GOOD

M_r = 12320 k-in

AISC Seismic Prov. Eq. D1-1a

P_{urb} = 10.58 k-in

β_{br} = 78.33 k/in

AISC Seismic Prov. Eq. A-6-7, 8

Try: L5x5x5/16 A = 3.07 in^2

AISC Manual Tbl. 1-1

L_{br} = 300.8 in 25.07 ft

θ = 0.078 rad

k = 294.2 k/in GOOD