Three-dimensional non-linear analyses of steel special moment resisting frames

by

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Abstract

A special moment resisting frame (SMRF) is a type of lateral force resisting system that is used to protect structures from extreme seismic events. This report evaluates the performance of a steel SMRF with an emphasis on the lower story elements. Previous research evaluating the performance of a reinforced concrete SMRF concluded that two-dimensional finite element analyses (2D-FEA) and three-dimensional finite element analyses (3D-FEA) have varying results. These varying results were so extreme in two instances that a change in design standards and codes was deemed necessary. In order to determine if the same conclusions can be drawn when analyzing a steel SMRF, this report used 2D-FEA and 3D-FEA programs to analyze a steel SMRF from Section 5.2 of the *FEMA 451, NEHRP Recommended Provisions: Design Examples.* The lower story member forces were gathered from both programs at 28 locations, then the percent difference was calculated. Analysis showed that the beam axial forces varied by more than 50 percent and that the beams varied in behavior. One form of analysis showed that the beams were undergoing axial compression while the other showed that the beams were experiencing axial tension. Upon discovering these differences, the beam axial forces were further studied by comparing the lower story beam displacements. The beam elongations differed by more than 1000 percent in some locations. These findings affirmed previous research by showing a large difference in 2D-FEA and 3D-FEA results. Though this research proved that the two forms of analyses vary greatly, this is only true of the axial forces. Even then, the variances equated to less than a hundredth of an inch. This variance is minimal in the overall scheme of design and it does not warrant a change in design standards, contrary to the reinforced concrete SMRF analyzed in previous research.

Table of Contents

List of Figures

List of Tables

Chapter 1 - Introduction

Earthquakes have become a common occurrence in parts of the world today. The severity of past earthquakes and the potential severity of future earthquakes has required engineers to incorporate building systems capable of withstanding extreme loads and conditions caused by seismic activity. A Special Moment Resisting Frame (SMRF) is a type of lateral force resisting system that is often used to protect structures from these extreme events. Typically, these frames are designed and analyzed using two-dimensional finite element analyses (2D-FEA). This type of analyses is done with compatible computer programs, and it is very common due to the programs' accessibility and ease of use. However, recent research proves that three-dimensional finite element analysis (3D-FEA) outperforms 2D-FEA when analyzing structures experiencing extreme loads. This report evaluates the performance of a steel SMRF described in Section 5.2 of the *FEMA 451, NEHRP Recommended Provisions: Design Examples* using both 2D-FEA and 3D-FEA software.

The basis of this report stems from an article titled *Three-Dimensional Non-Linear Analyses of Special Reinforced Concrete Moment Frames* by Donald Phillippi and Gabrielle Liuzza and a thesis titled *The Elongation of Beams in Reinforced Concrete Special Moment Resisting Frames* by Gabrielle Liuzza. This article discussed research conducted that compared a special reinforced concrete moment frame analyzed using 2D-FEA and 3D-FEA. Ideally 2D-FEA and 3D-FEA should provide similar results. Though 3D-FEA often provides more realistic data, 2D-FEA is based off of 3D-FEA and is used for the design and analysis of most structures today. The research discussed in the article shows differently. There were two major differences between the 2D-FEA and 3-FEA. First, the shear force distribution at the end columns when using 3D-FEA is approximately double the shear force distribution when analyzing with 2D-

FEA. Additionally, 2D-FEA shows that beams stay the same length or get shorter under extreme lateral loads while 3D-FEA showed that the concrete beams actually experience elongation. This research shows drastic differences between using 2D-FEA and 3D-FEA when analyzing special reinforced concrete moment frames, as seen in Table 1.

Table 1. Reinforced concrete SMRF 2D-FEA vs. 3D-FEA (Phillippi & Liuzza, 2017).

Method	Parameter	First Floor Columns					
		1	2	3	4	5	6
2D-FEA	Axial (kN)	381	-1300	-1287	-1285	-1255	-1665
	Shear (kN)	294	419	427	427	437	323
	Top M (kN-m)	-137	-472	-461	-461	-474	-230
	Base M (kN-m)	1119	1391	1361	1360	1390	1148
	Joint Δ (mm)	8.59	8.60	8.60	8.59	8.57	8.53
	Joint θ (rad.)	0.00278	0.00238	0.00239	0.00238	0.00237	0.00263
3D-FEA	Axial (kN)	265	-1237	-1282	-1275	-1224	-1812
	Shear (kN)	65	281	357	447	550	647
	Top M (kN-m)	186	-19	-131	-240	-342	-356
	Base M (kN-m)	379	956	1061	1257	1476	1801
	Joint Δ (mm)	5.84	7.25	8.57	10.01	11.71	14.50
	Joint θ (rad.)	0.00178	0.00180	0.00193	0.00217	0.00243	0.00358

Method	Parameter	Second Floor Beams									
		1L	1R	2L	2R	3L	3R	4L	4R	5L	5R
2D-FEA	Axial (kN)	0	0	-8	-8	-22	-22	-35	-35	-40	-40
	Shear (kN)	96	287	80	278	86	278	86	277	93	284
	M (kN-m)	589	-787	534	-768	534	-767	532	-765	549	-807
	Elongation (mm)	0.01		0.00		-0.01		-0.02		-0.04	
3D-FEA	Axial (kN)	-81	-94	-197	-235	-334	-331	-371	-380	-337	-355
	Shear (kN)	59	217	115	262	127	267	163	331	196	349
	M (kN-m)	323	-615	461	-735	534	-820	594	-864	649	-1102
	Elongation (mm)	1.41		1.32		1.44		1.70		2.79	

Method	Parameter	Second Floor Columns					
		1	2	3	4	5	6
2D-FEA	Axial (kN)	286	-1099	-1095	-1093	-1071	-1380
	Shear (kN)	219	445	441	439	442	283
	Base M (kN-m)	463	879	870	867	870	587
3D-FEA	Axial (kN)	222	-1139	-1078	-1078	-1082	-1464
	Shear (kN)	131	427	430	451	488	288
	Base M (kN-m)	426	898	912	899	878	390

Now, the idea in question is whether or not a difference in 2D-FEA and 3D-FEA exists in other structure types. This report conducts a similar comparison in analyses for a steel special moment frame in order to determine variances in 2D-FEA and 3D-FEA for the analysis of steel structures. For this report, a parametric study has been done using a SMRF example from *FEMA*

451, NEHRP Recommended Provisions: Design Examples. The frame was modeled and analyzed in RISA and LS-DYNA. Axial, shear and moment forces of the members were gathered from both programs, and then compared. Difference in force comparisons eventually led to a beam displacement comparison as well.

Chapter 2 - Literature Review

There is a significant difference between designing a lateral force resisting system for wind loads and for seismic loads. When seismic activity occurs, a series of excitations act on a structure. These excitations require a structure to respond inelastically in order to dissipate the energy. For this reason, deformation/ductility demands control the design. Achieving required ductility in a structure is very complex. This complexity has spurred the implementation of special lateral force resisting systems (Rafezy, 2017).

2.1 Material Properties of Steel

The seismic demand for ductility in structures has made steel a prominent building material in regions at risk to seismic activity. Typical steel structures are designed so that the load applied to the structure does not exceed the yield strength of the steel. However, steel structures in high seismic areas are designed so that the seismic loading exceeds the yield strength. After steel yields, the structure behaves inelastically. This inelastic behavior is desirable when resisting seismic forces because it allows the structure to dissipate energy present from seismic excitation forces.

Some materials maintain a level of structural integrity after yielding, while others rupture quickly after yielding. This is the difference between ductile and brittle materials. Ductile materials have the ability to absorb energy after yielding, as can be seen in Figure 1. A materials ability to deform and absorb energy determines it ability to perform under the occurrence of seismic excitation.

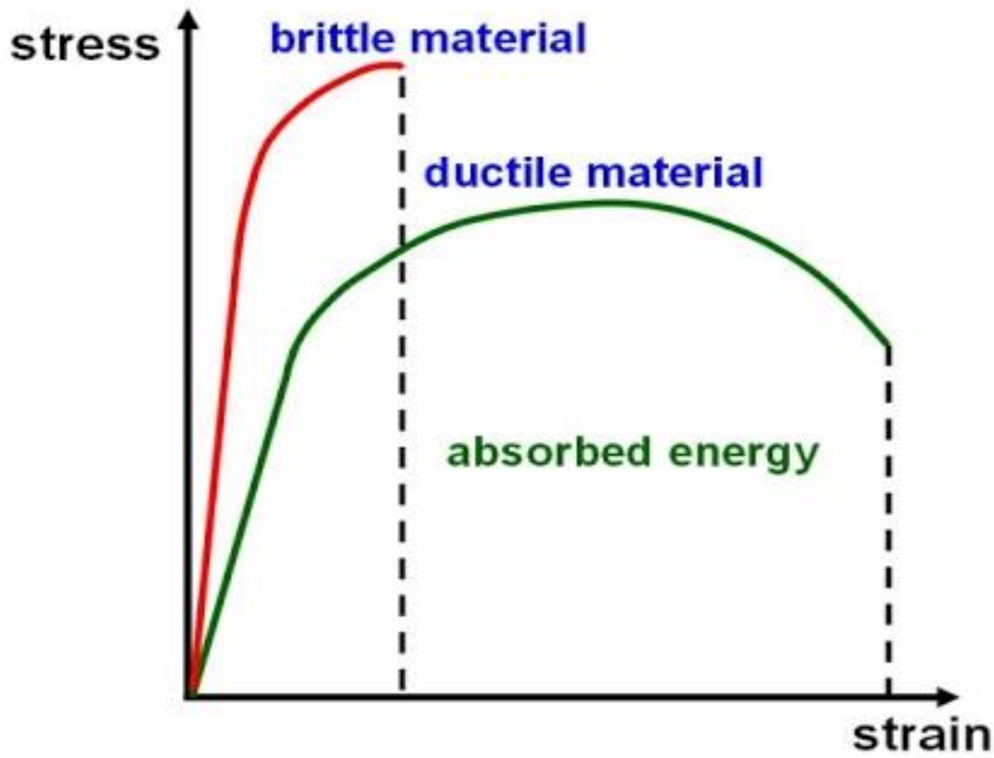

Figure 1. Brittle vs. ductile materials (Ashkenazi, 2019).

Steel is a ductile building material that actually increases in strength after yielding through a process called strain hardening. Strain hardening is the use of permanent deformation to increase the strength of a metal. Upon strain hardening, steel reaches its ultimate strength. This process can be seen in Figure 2. This process allows steel lateral force resisting systems to dissipate seismic energy and still maintain structural integrity for the purpose of life safety.

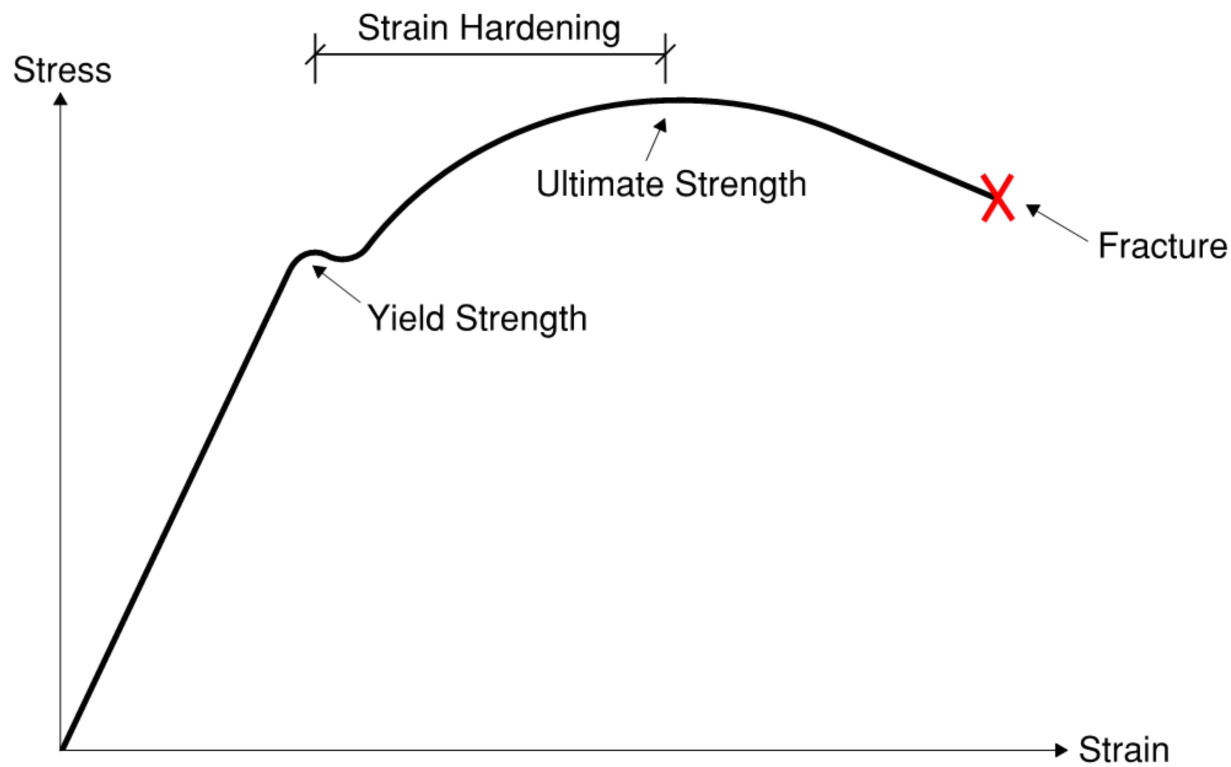

Figure 2. Steel stress vs. strain curve.

2.2 Structural Steel Special Moment Resisting Frames

Structural steel special moment frames often are used as part of the lateral force resisting systems in buildings designed to resist seismic loading. These moment frames have been utilized since the 19th century, but they were not added to the building code until later. The 1988 Edition of the Uniform Building Code was the first published code to include special steel moment frames. These frames were coined special due to the special design criteria and their superior performance. Initially, the special design criteria applied primarily to the frame connections, but more requirements were added as time progressed. These additional requirements included the need for strong-column/weak-beam behavior, the balance between panel zone shear strength and beam flexural capacity, section compactness criteria, and lateral bracing criteria (Hamburger, 2009).

2.2.1 Strong-Column Weak-Beam

Strong-column weak-beam is a design philosophy utilized to control the failure location of a structure. As the name implies, the weak beams should fail before the strong columns. This is done in order to prevent the collapse of a structure. If the beams were stronger than the columns, a lower level column may fail resulting in the collapse of a structure. Alternatively, if the beams are stronger, then the beams will fail first which only effects a single level. This concept is depicted in Figure 3.

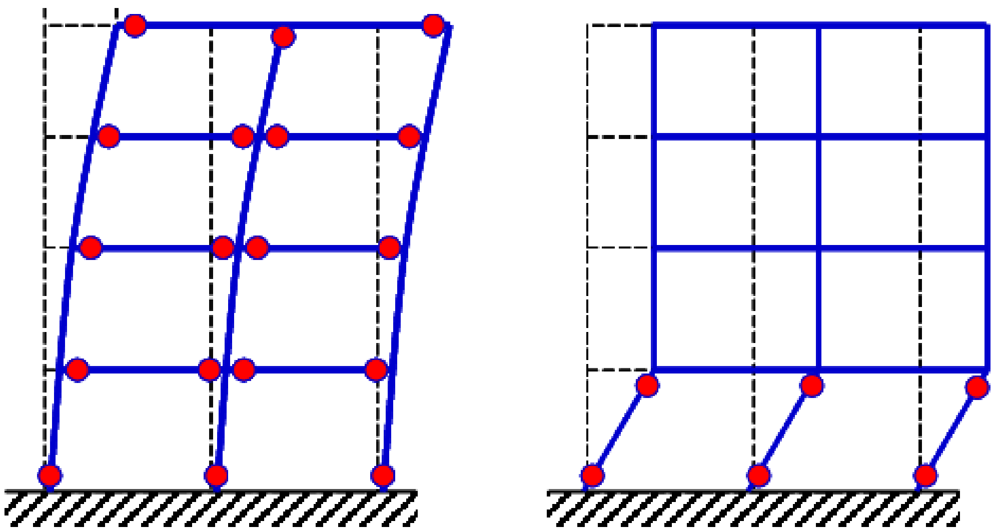

Figure 3. Strong column vs. strong beam (Ye & Qu, 2011).

As per the AISC Seismic Provisions, to ensure strong-column weak beam design, the moment capacity of the columns must be greater than the moment capacity of the beams. Doing so ensures that the beams will fail first in the event of an earthquake.

2.2.2 Plastic Hinges

A plastic hinge refers to the deformation of a structural member wherever plastic bending occurs. This mechanism is considered a hinge because it has no capability to resist moment. In other words, plastic hinges permit free rotation. The development of plastic hinges is inevitable in structures located in high seismic regions, but the location of these plastic hinges can be controlled. The strong-column weak-beam design method is important in dictating the location of plastic hinges. Because the beams are weaker than the columns, failure will occur through plastic hinges in the beams. Furthermore, the location of plastic hinges along a beam can be controlled. Beams and moment connections are designed in order to ensure that plastic hinges will occur at the end of each beam but before the moment connection. The typical location of plastic hinges can be seen in Figure 4.

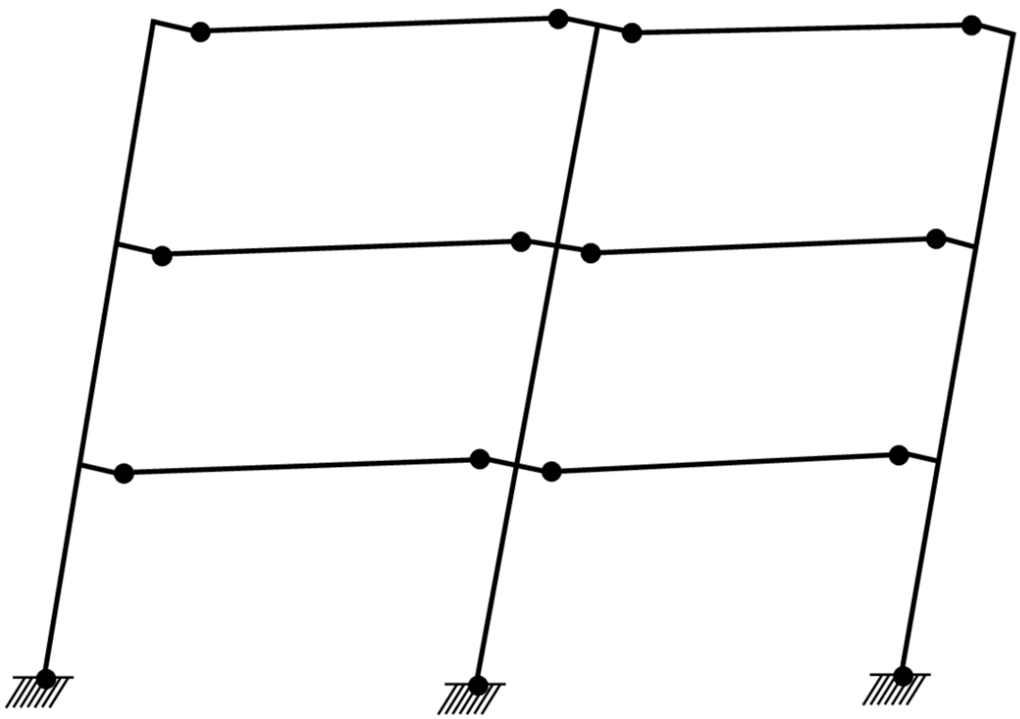

Figure 4. Ductile moment frame plastic hinge location (AISC, 2012).

2.2.3 Moment Connections

Steel special moment resisting frames require the use of moment connections. Moment connections are a joint that allows the transfer of bending moment forces between a column and a beam. If a beam has an internal moment then a moment connection should have the ability to transmit the load from the moment to the column. The purpose of this connection is to simulate a completely fixed joint (Liang, 2019). In other words, moment connections are rigid connections in all translation and rotational directions. Additionally, moment connections are important in dictating the failure location of a structure. The strong-column weak-beam design method is in place to permit failure in the beams rather than the columns. Moment connections are similar in the sense that the beam should fail before the connection should.

The American Institute of Steel Construction (AISC) has published a design standard, AISC 358 *Prequalified connection for Special and Intermediate Steel Moment Frames for Seismic Applications.* The standard presents materials, design, detailing, fabrication and inspection requirements for multiple different types of moment connections. Each connection is unique, so the applicability of each connection varies.

Reduced Beam Section Moment Connection

A reduced beam section (RBS) moment connection is a connection in which portions of the beam flanged are trimmed in the region beside the actual beam to column connection. This connection ensures the location of yielding in the beam and thus dictates the plastic hinge location (Carter, 2010). An example of this type of connection is illustrated in Figure 5.

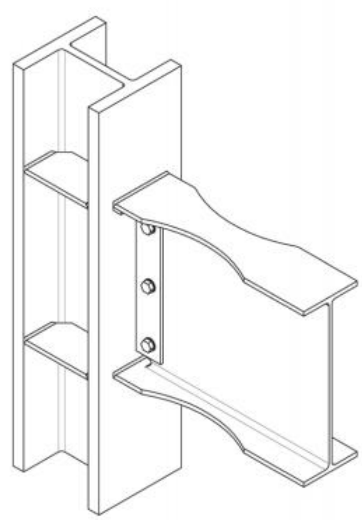

Figure 5. Reduced beam section moment connection (NIST, 2016).

Extended End Plate Moment Connections

There are two types of end plate moment connections: bolted unstiffened extended end-plate (BUEEP) and bolted stiffened extended end-plate (BSEEP). In this connection the beam is welded to an extended end-plate that is then bolted to the column in one of three ways. The three differences are illustrated in Figure 6.

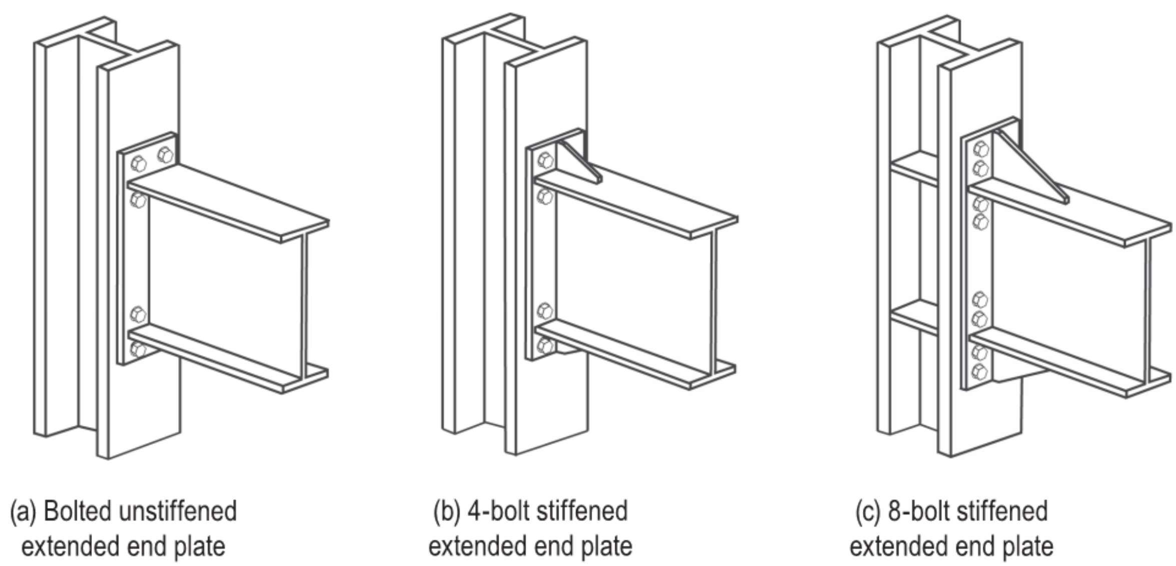

Figure 6. Types of extended end plate moment connections (NIST, 2016)

Bolted Flange Plate Moment Connection

This type of connection, also known as a BFP moment connection, uses plates welded to column flanges and bolted to beam flanges. The beam web is connected to the column flange using a plate shear connection. Inelastic rotation that occurs is intended to occur in the beam near the end of the flange plates. Similar to RBS connections, this controls where the plastic hinges occur and ensures that the beam fails before the connection does (Carter, 2010). Different examples of this type of connection are illustrated in Figure 6.

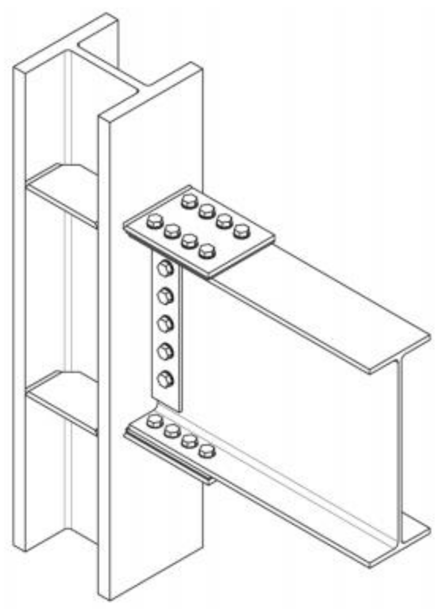

Figure 7. Bolted flange plate moment connections (NIST, 2016).

Welded Unreinforced Flange-Welded Web Moment Connection

This connection type, also known as a WUF-W moment connection, utilizes welds to connect the beam flanges to the column flanges. The beam web is bolted to a single plate shear connection for erection purposes. Inelastic rotation for this connection is intended to occur in the beam adjacent to the column face. The moment connection maintains integrity due to special detailing requirements associated with the welds joining the flanges (Carter, 2010). This type of connection is illustrated in Figure 7.

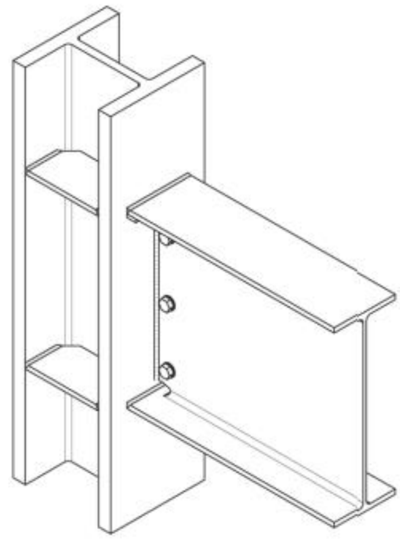

Figure 8. Welded unreinforced flange-welded web moment connection (NIST, 2016).

Kaiser Bolted Bracket Moment Connection

Kaiser Bolted Bracket (KBB) connections use high strength steel brackets that are fastened to each beam flange and then bolted to the column. The connection of the bracket to the beam flange can be done by welding or bolting the bracket. The inelastic rotation in the beam is intended to occur at the end of the brackets, much like the BFP connection (Carter, 2010). An example of two KBB connections are illustrated in Figure 8.

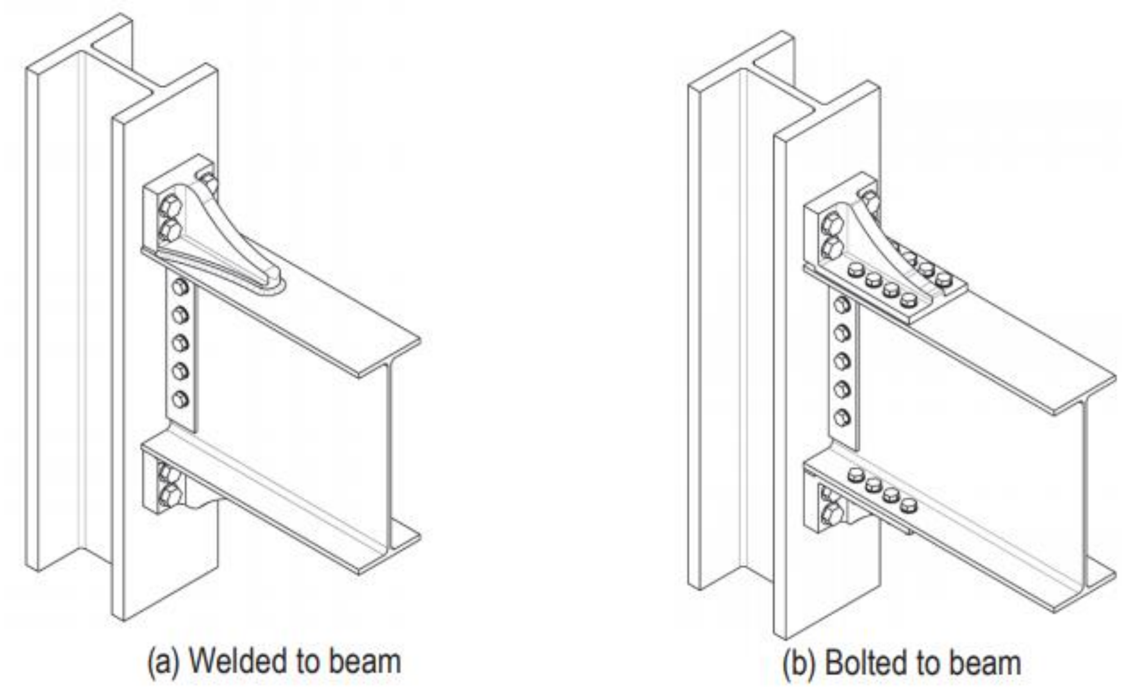

Figure 9. Kaiser bolted bracket moment connection (NIST, 2016).

Other Moment Connection Types

In addition to the moment connections addressed in the previous sections, there are other prequalified connections per AISC 358. These additional connection types include a bolted double tee connection, a welded double tee connection, the ConXtech ConXL connection, various end plate connections, and more (NIST, 2016)

. For a more detailed explanation of moment connections, please refer to the AISC 358 standard.

Chapter 3 - Research Background

Before discussing the parametric study, it is important to discuss the research leading up to the study. This chapter will discuss the example utilized for this report and the two programs in which the frame performance was analyzed. The chapter will also address relevant codes and regulations.

3.1 Example

This report evaluates the performance of a steel SMRF described in Section 5.2 of the *FEMA 451, NEHRP Recommended Provisions: Design Examples*. The example is a seven-story office building with SMRFs on each perimeter wall. The north and south perimeter walls have a seven-bay frame while the east and west walls have a five-bay frame. The moment frame locations can be seen in Figure 5. This report will focus on the five-bay SMRF.

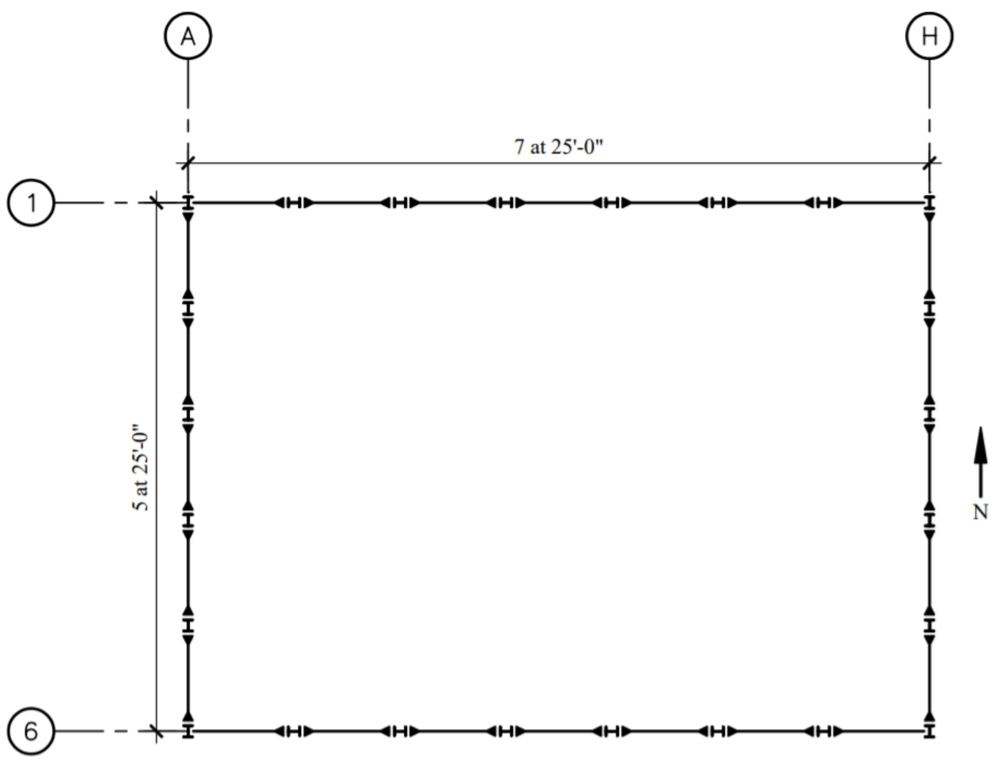

Figure 10. Design example SMRF framing plan (FEMA, 2006).

The design example locates the structure in Los Angeles, California. The location experiences frequent seismic activity, so the spectral response parameters are design restrictive. In other words, the seismic parameters require that a special lateral force resisting system be implemented. The example location spectral response parameters can be seen in Table 2. These parameters were then used to calculate the seismic forces acting on the frame. The seismic forces utilized in the example were given, and they were later utilized in the 2D-FEA and 3D-FEA portion of this report in order to conduct an accurate comparison.

Table 2: Seismic design parameters (FEMA, 2006).

Site Class	D
SS	1.5
S1	0.6
SDS	1.0
SD1	0.6
Seismic Design Category	D

Section 5.2 of the *FEMA 451, NEHRP Recommended Provisions: Design Examples* also discusses the gravity loads acting on the building. Dead loads and live loads were calculated considering construction materials and building use. Those loads were then combined with the seismic load parameters to properly select member sizes for the SMRF. The proper sizes were selected and displayed in the design example as shown in Figure 6.

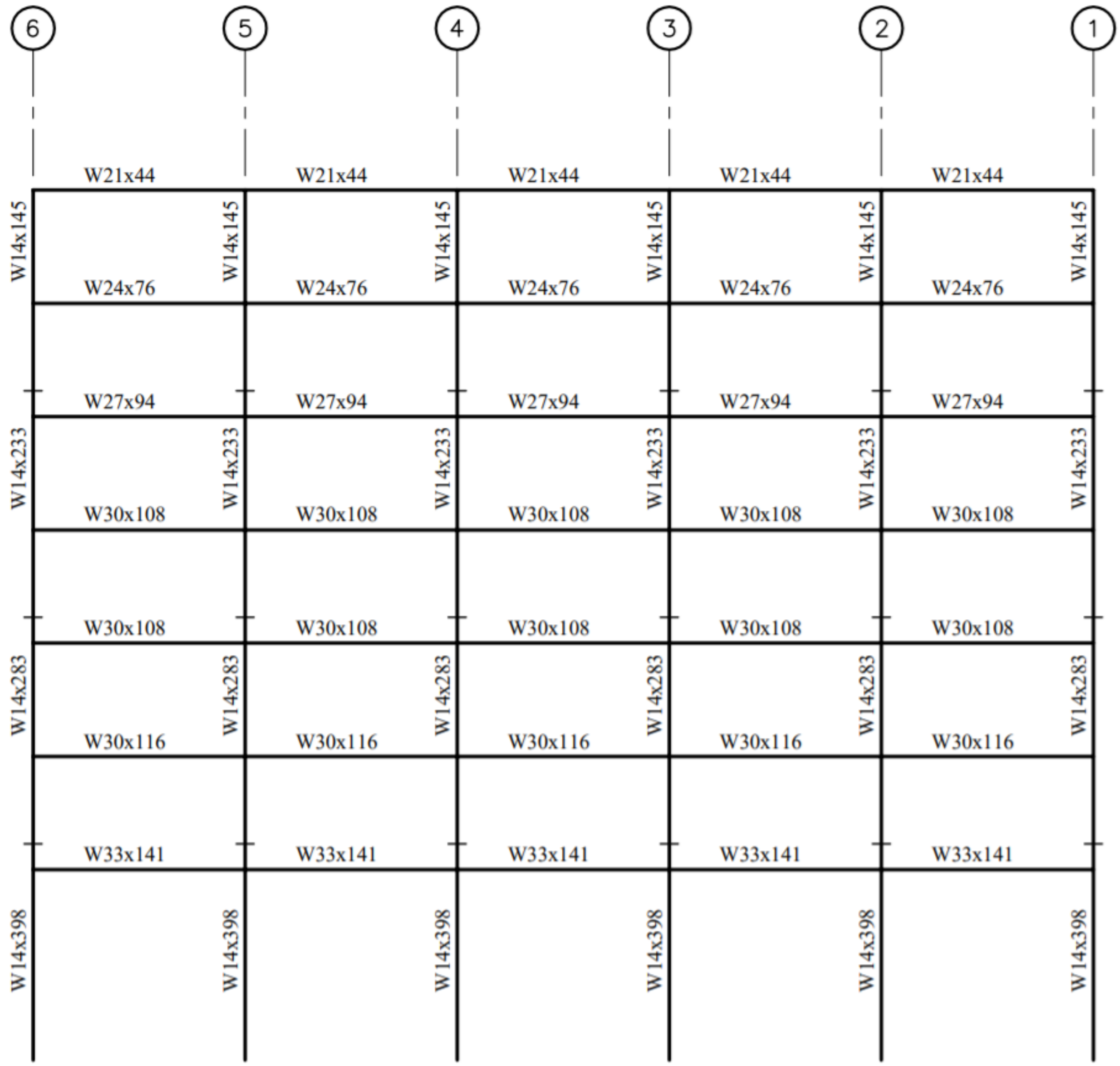

Figure 11. Five-bay seven-story SMRF member sizes (FEMA, 2006).

This example was chosen for this research because it a component of a widely accepted publication. Using a proven publication causes less research error and makes it easier for others to continue or add to the research being done. In order to gain a better understanding of the design example laid out in Section 5.2 of the *FEMA 451, NEHRP Recommended Provisions: Design Examples*, see Appendix A.

3.2 Two-Dimensional Finite Element Analysis through RISA

RISA is a 2D-FEA software that was used to analyze the SMRF described in Section 5.2 of the *FEMA 451, NEHRP Recommended Provisions: Design Examples.* Before explaining how the program was used to analyze the given frame, it is important to gain a better understanding of the program itself. The following sections provide an overview of this program as well as the manner in which it was utilized for this report.

3.2.1 RISA Program Overview

RISA is a common design and analysis program used in the structural engineering industry. RISA has multiple products that work together and the program has the ability to analyze all types of building materials. These properties make the program simple for everyday analysis and design. RISA also has the ability to create three-dimensional models as shown in Figure 7, which can then be used to properly analyze a structure.

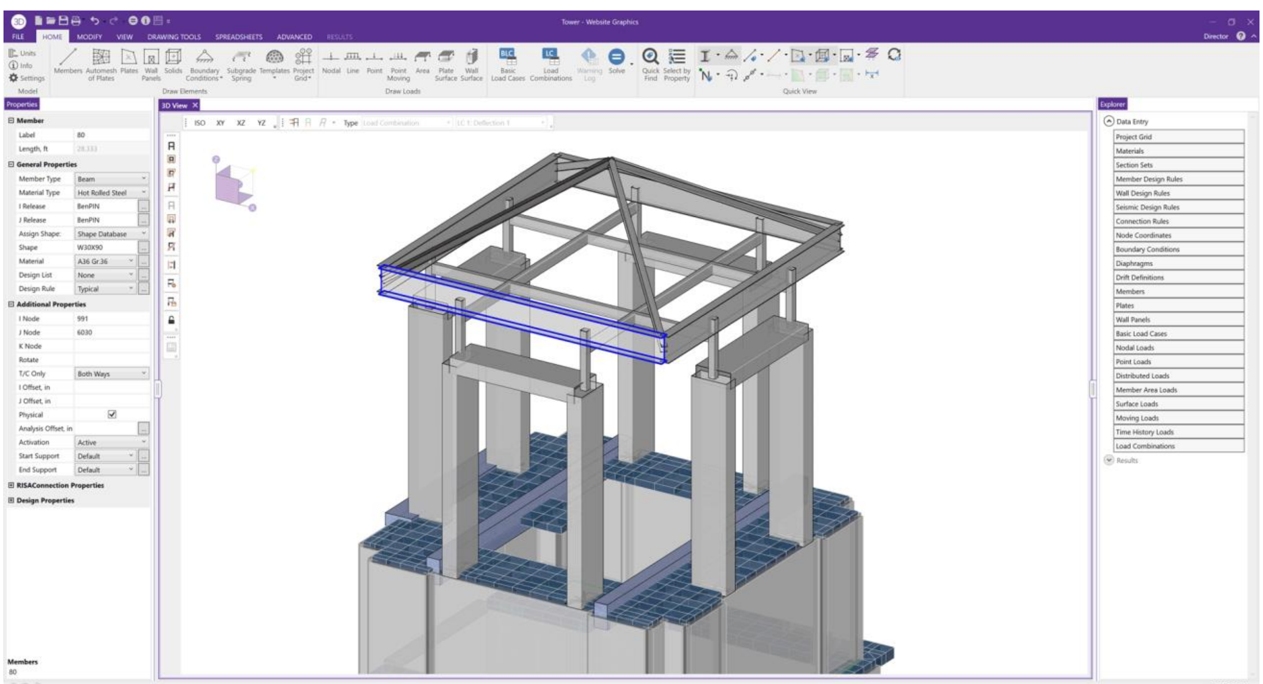

Figure 12: Diagram of RISA model (RISA, n.d.).

Benefits

RISA was the first program to introduce a graphical interface that was adept for creating both large and small structures. For this reason, many companies in industry started utilizing RISA. As the program grew in popularity, more components were added, making the program even better than before. RISA now provides the ability to customize the shape properties of structural components and elements. Another notable component of RISA is the program's user interface. The program is very user friendly and it provides helpful program tutorials; this helps expedite the design and analysis process.

Drawbacks

RISA lacks the three-dimensional aspects of structural behavior. RISA is a three-dimensional program in the sense that a structure can be modeled in three-dimensions and the elements have three-dimensional properties. However, the program does not take the three-dimensional movement of a structure into effect. In other words, RISA is not a dynamic analysis program.

3.2.2 Design Example Analysis

This research required a 2D-FEA program in order to model and analyze the example in Section 5.2 of the *FEMA 451, NEHRP Recommended Provisions: Design Examples*. RISA was selected as the proper 2D-FEA program for the research. The SMRF was modeled and constructed of already made elements in the program using code suggested settings. The material properties in the program include material type, material grade, and section shape. The example SMRF modeled in RISA can be seen in Figure 8.

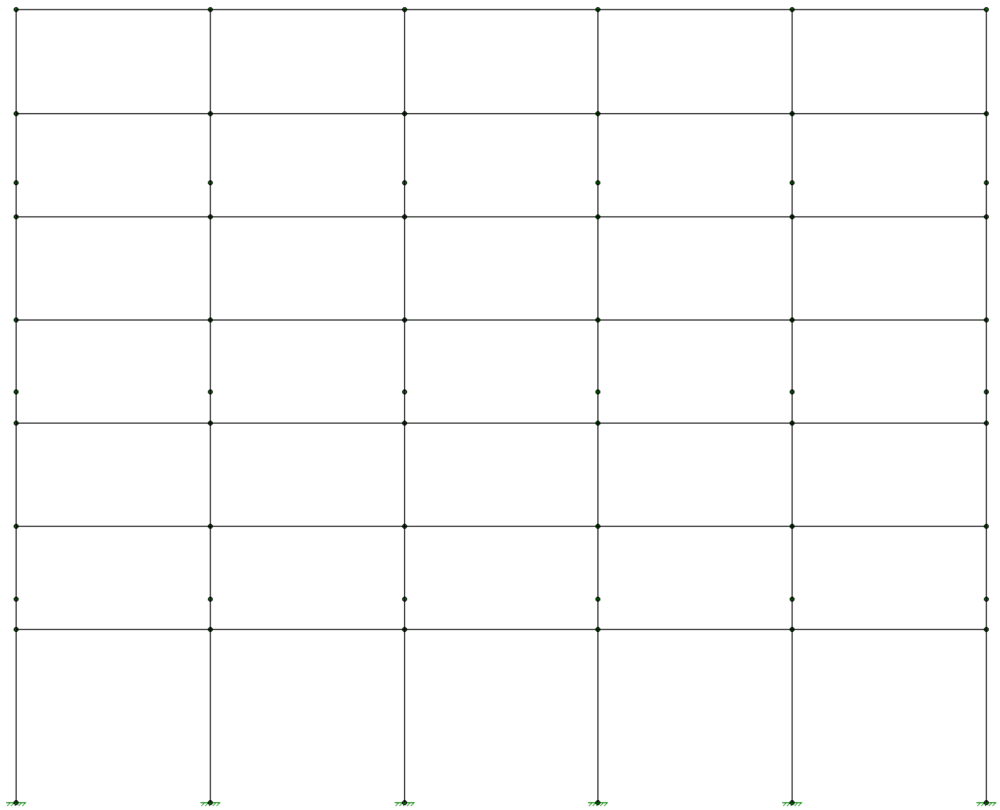

Figure 13. Five-bay seven-story SMRF modeled in RISA.

3.3 Three-Dimensional Finite Element Analysis through LS-DYNA

LS-DYNA is a 3D-FEA software that was used to analyze the SMRF described in Section 5.2 of the *FEMA 451, NEHRP Recommended Provisions: Design Examples.* Before explaining how the program was used to analyze the given frame, it is important to gain a better understanding of the program itself. The following sections provide an overview of this program as well as the manner in which it was utilized for this report.

3.3.1 LS-DYNA Program Overview

LS-DYNA is a three-dimensional general purpose FEA program that uses material models and integration methods to solve complex three-dimensional problems associated with non-linear behavior. The program has commonly been used in the automobile industry to analyze collision data, but as the program has grown in popularity, structural dynamics have been added into the software. LS-DYNA can create three-dimensional models with the use of a grid system, as can be seen in the Figure 9.

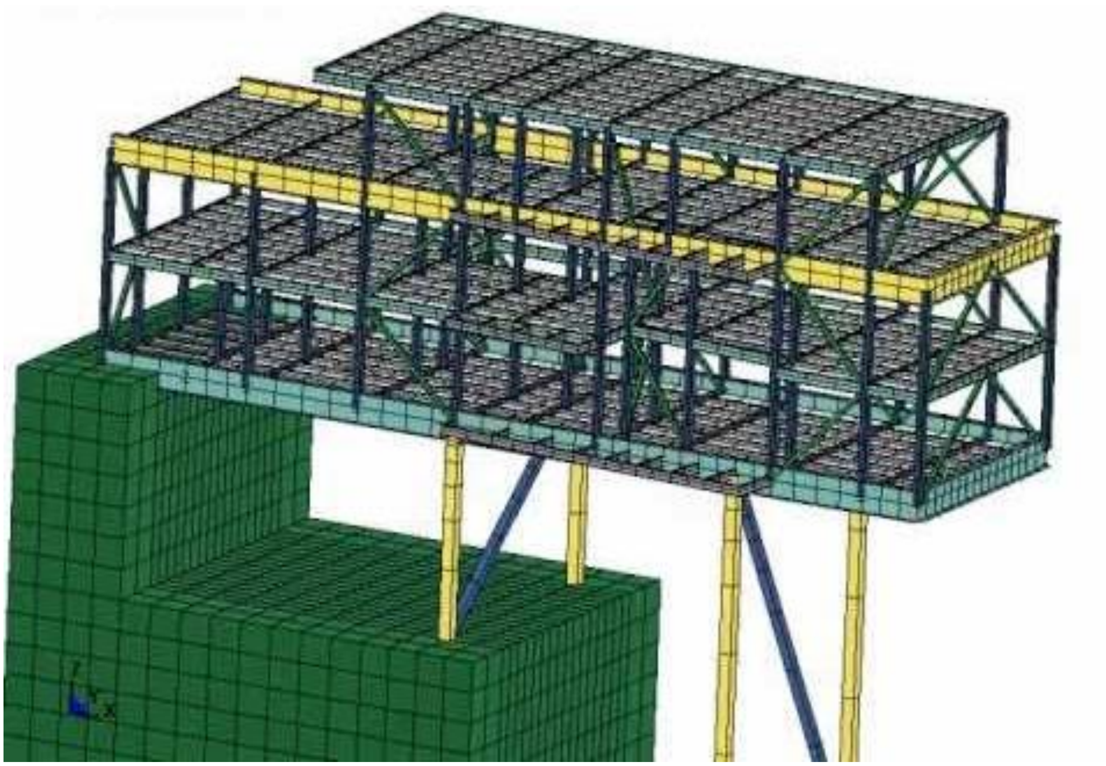

Figure 14. Diagram of LS-DYNA model (Seismic analysis in LS-DYNA, 2012)

Benefits

LS-DYNA has the ability to determine the equation of motion for a given structure or structural component. This allows the program to then calculate displacement, velocity, and acceleration in terms of time. According to Livermore Software Technology Corporation (LSTC, 2011), "in LS-DYNA, the study area is spatially discretized by 2D or 3D finite elements. At each time step, the equations of motion for the considered dynamic system are solved at the integration points of each element. The strain increments are determined based on the calculated nodal displacements". Thus, making LS-DYNA a highly advanced dynamic analysis program.

Dynamic analysis is of the upmost importance when designing structures that may reach inelastic behavior. Examples of such behavior may occur when blast loads, fluid loads, impact loads, or seismic loads are applied to a structure.

Drawbacks

Using LS-DYNA to design and analyze a structure can be a lengthy process. Before using the program, an alternative program must be utilized to assemble a three-dimensional mesh that resembles the specimen being analyzed. This alternative program must also be used to set boundary conditions and establish materials. Upon completing this process, a data file can be imported into LS-DYNA where loads can be applied, and a dynamic simulation can be launched. Depending on the type of simulation, the program may require a large amount of time to perform the analysis. This process makes LS-DYNA an impractical design tool, but it can be a useful research tool.

3.3.2 Design Example Analysis

In addition to a 2D-FEA program, this research required a 3D-FEA program in order to model and analyze the example in Section 5.2 of the *FEMA 451, NEHRP Recommended Provisions: Design Examples.* LS-DYNA was selected as the proper 3D-FEA program for the research. TrueGrid was used to generate a mesh, set boundary conditions, and establish materials. The TrueGrid data file was then used to model the SMRF in LS-DYNA. There, loads were applied, and a simulation was created. The example SMRF modeled in LS-DYNA can be seen in Figure 10. The lines on the figure are an indication of the mesh discussed above.

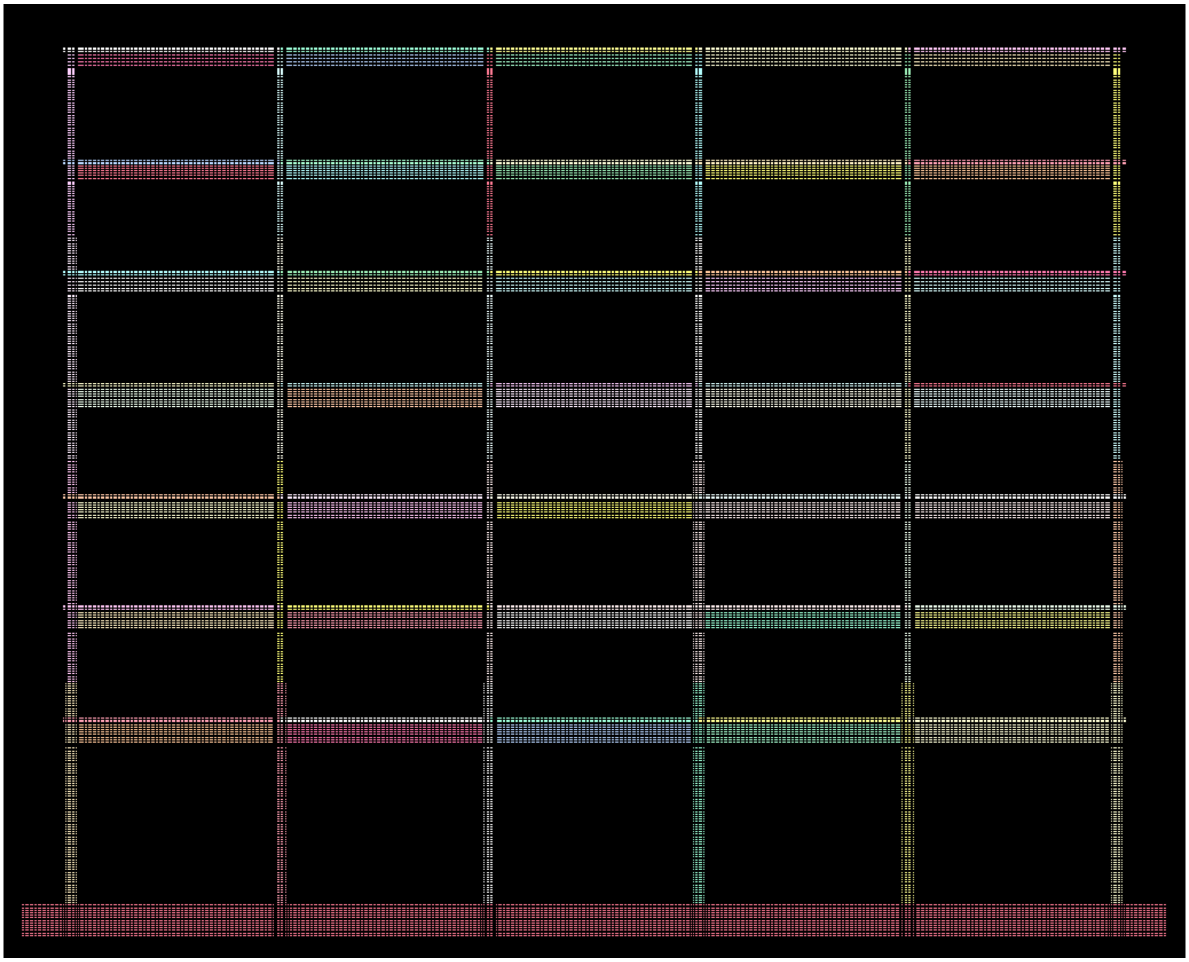

Figure 15. Five-bay seven-story SMRF modeled in LS-DYNA.

3.4 Governing Regulations

Governing regulations are in place to ensure the safety, health, and welfare of the occupant. The design example from Section 5.2 of *FEMA 451, NEHRP Recommended Provisions: Design Examples* often refers to the provisions set forth by *FEMA 450, NEHRP Recommended Provisions for Seismic Regulation of New Buildings and Other Structures*. In addition to these provisions, the design example referenced building codes set for by the International Code Council (ICC) and the American Institute of Steel Construction (AISC). Though newer codes have become available, the design codes used in the example from *FEMA 451, NEHRP Recommended Provisions: Design Examples* will be addressed here.

3.4.1 NEHRP Recommended Provisions

FEMA 450, NEHRP Recommended Provisions for Seismic Regulation of New Buildings and Other Structures is a standard that presents criteria for the design and construction of structures to resist earthquake ground motions. The provisions provide minimum design requirements in order to improve the capability of structures in the event of an earthquake (FEMA, 2004).

3.4.2 International Building Code

The *International Building Code* establishes minimum regulations for building systems using prescriptive and performance related provisions. The building codes sets forth general requirements for structural design, it prescribes the need for special inspections, and it addressed requirements in regard to every structural material type (ICC, 2014). This code also refers to other design standards such as the *Minimum Design Loads for Buildings and Other Structures, ASCE 7*. The ASCE 7 gives more detailed information used to calculate building loads (ASCE, 2017), The *International Building Code* refers to other codes and standards such as *the AISC Steel Construction Manual* and *Seismic Design Manual*. Essentially, this document sets forth the initial basis of design and provides information to progress further in design.

3.4.3 AISC Steel Construction Manual

The *AISC Steel Construction Manual, 15rd Edition* is a standard for the design of structural steel buildings. These regulations on design help with the integrity of a structure. The manual consists of 17 parts that address various topics related to steel building design and construction (AISC, 2017).

3.4.4 AISC Seismic Design Manual

The *AISC Seismic Design Manual, 2st Edition* is a standard intended to assist designers in properly applying AISC standards and provisions in the design of steel lateral force resisting systems. It is intended to be used in conjunction with the *AISC Steel Construction Manual*. The manual consists of 10 parts that address various topics related to the design and construction of seismic force resisting systems of structural steel and structural steel acting compositely with reinforced concrete (AISC, 2012).

Chapter 4 - Parametric Study

As mentioned, this report evaluates the performance of a steel special moment resisting frame described in Section 5.2 of *FEMA 451, NEHRP Recommended Provisions: Design Examples.* Using RISA and LS-DYNA forces were simulated to act laterally on the frame, then the lower story elements of the frame were studied. The effect of the load applied in LS-DYNA is illustrated in Figure 11.

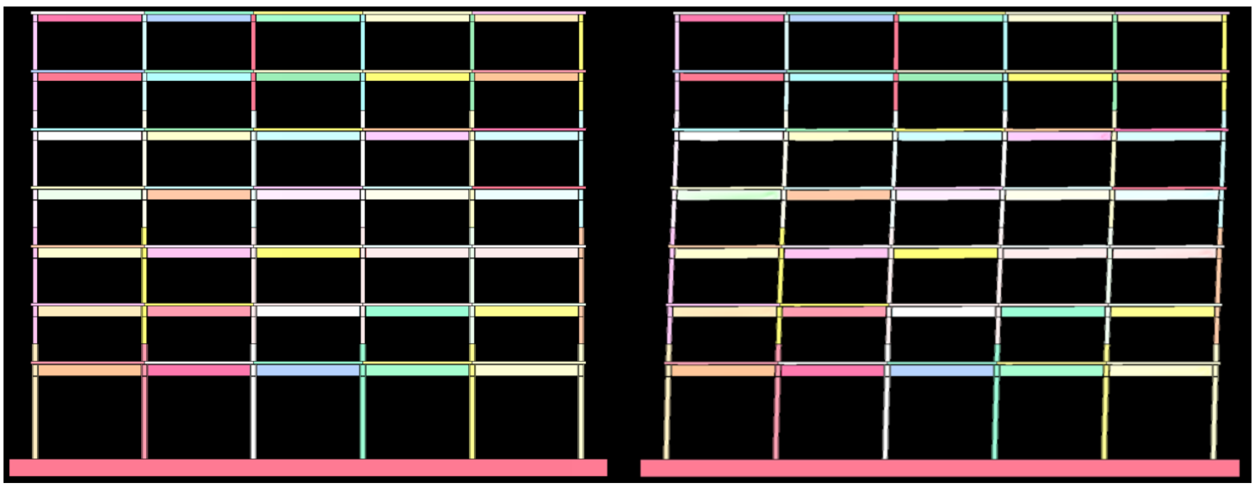

Figure 16. LS-DYNA model displacement animation.

Since, LS-DYNA solves for an equation of motion, data produced from the program and structural mechanics can be used to create basic steel analysis. The results of this analysis were then used for the parametric study.

4.1 Time Comparison

Before collecting any data, the base shears of the two models had to be compared in order to determine at what time step LS-DYNA was comparable to RISA. The shear reactions at each of the six columns were taken from RISA. Next, a force vs. time graph was plotted in LS-DYNA. The shear reactions were used in order to determine at what time LS-DYNA showed the exact same reaction. This was done at each of the six columns so that the total base shear in both programs would be equivalent. Last, the average time collected for all six columns was calculated. The results can be seen in Table 3, the average time was 1.87 seconds.

Table 3. Average time at which base shears of each model is equal.

Column Gridline	RISA Base Shear (lbs)	Filter n = 9 Time (sec)	Filter n = 6 Time (sec)	Filter n = 3 Time (sec)
1	-84264	1.96	1.96	1.97
2	-99599	1.93	1.93	1.94
3	-98786	1.89	1.9	1.89
4	-98867	1.84	1.84	1.85
5	-100499	1.82	1.83	1.81
6	-81310	1.75	1.75	1.75
	average time	1.87	1.87	1.87

This was done three different times with three different filters. The filters get rid of some of the noise that appears in the plot, due to the nature of the program. The effect of the different filters can be seen in Figure 12. The n = 9 filter was the most readable while the n = 3 graph was closest to reality. Though the individual time at each column varied per filter, the average time for all three cases ended up being 1.87 seconds.

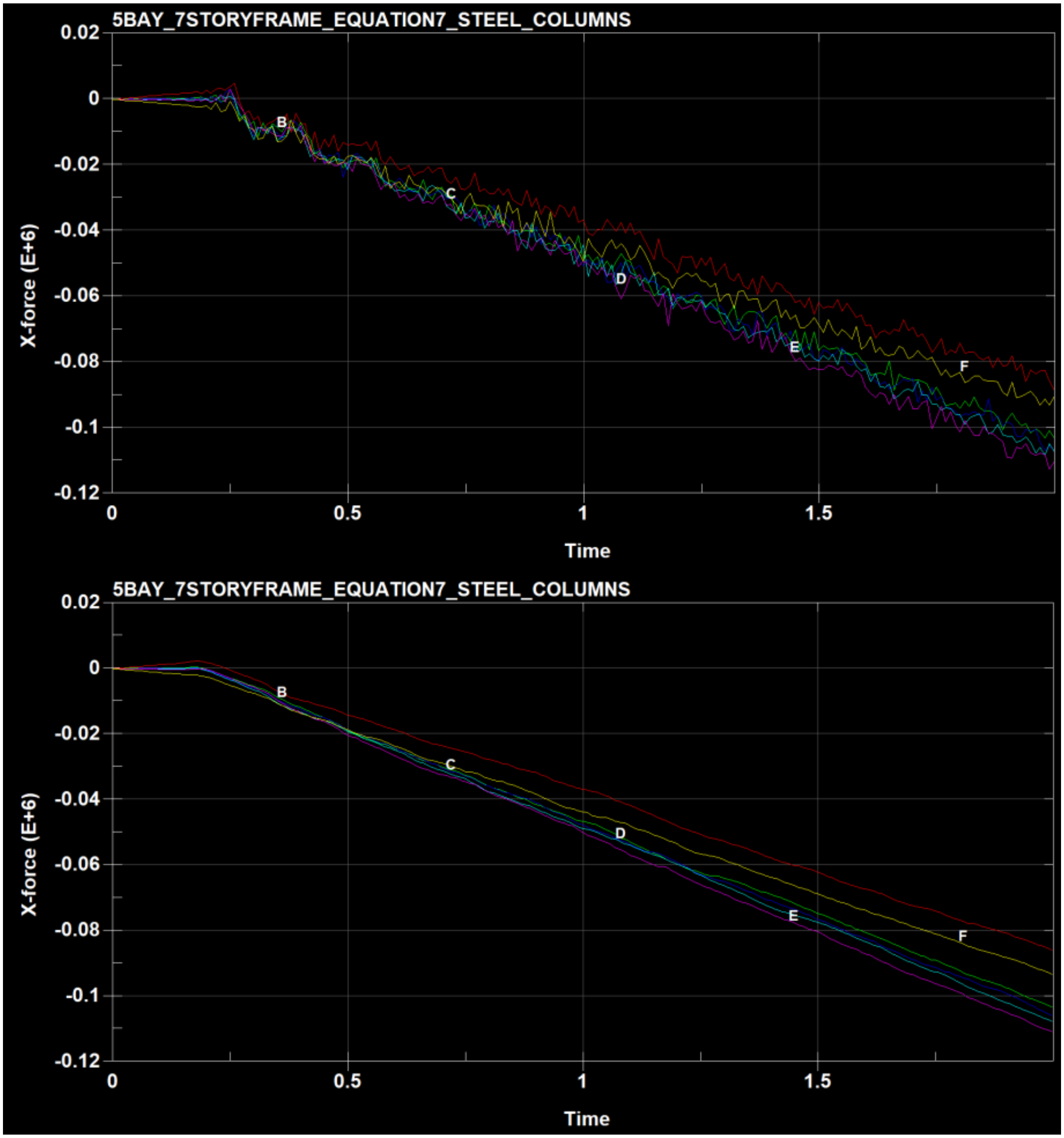

Figure 17. Difference in filters of base shear plot.

The average time gathered from the different plots was 1.87 seconds, as previously mentioned. This time was then used to gather data throughout the rest of the research.

4.2 Axial Comparison

After the proper time of base shear equivalency was determined, the base axial forces were compared at the same time. Like the shear forces, the axial forces needed to be the same in order to make further comparisons. The initial axial force comparison at 1.87 seconds showed that the LS-DYNA axial forces were 337% different than the RISA axial forces, as can be seen in Table 4.

Table 4. RISA vs. LS-DYNA total axial reactions.

Column Gridline	RISA Axial	LS-DYNA Axial	Percent Difference
1	-190201	-114000	-
2	266971	449000	-
3	234632	410000	-
4	236487	409000	-
5	204773	414000	-
6	423213	488000	-
Total	1175875	2056000	337%

As a response to the difference in axial loads, the dead loads and live loads in RISA were increased to result in a 0% difference between axial loads. The adjusted axial forces can be seen in Table 5. Once the time, shear and axial forces were constant, other components could be compared between the two models.

Table 5. RISA vs LS-DYNA adjusted axial reactions.

Column Gridline	RISA Adjusted Axial	LS-DYNA Axial	Percent Difference
1	-106578	-114000	-
2	444288	449000	-
3	411285	410000	-
4	413174	409000	-
5	381264	414000	-
6	514347	488000	-
Total	2057781	2056000	0%

4.3 Force vs. Time

The first comparison done in order to find any major differences between 2D-FEA and 3D-FEA was a force vs. time comparison. This comparison was specifically done at the lower story level of the frame. Forces were gathered at 28 different locations on the frame. The first 18 locations were on column elements and the second 10 locations were on beam elements. The data locations are indicated on Figure 13.

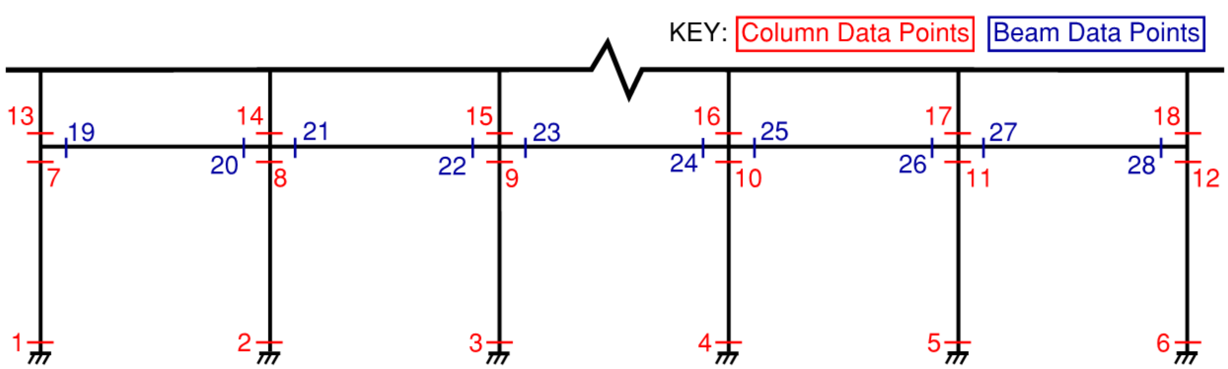

Figure 18. Frame data point locations.

After the data points were selected axial, shear and moment forces were collected from both RISA and LS-DYNA. It is important to note that these two programs have different sign conventions for compression and tension. For this reason, some of the forces collected from RISA were multiplied by negative one in order to change the sign and have a more accurate comparison. This was only true of the columns. The axial forces also showed different signs, but in an inconsistent manner. This is due to a difference in behavior rather than a difference in program sign conventions. When comparing the forces of the two programs the forces were very similar for the columns. When comparing the forces in the beams the shear and moment forces seemed comparable while the axial forces proved to be drastically different. Any values that differed by more than 50% were focused on. These items of interest are highlighted in red in Table 6.

Table 6. RISA vs. LS-DYNA member forces.

	RISA-3D			LS-DYNA			% Difference		
	Axial (lbs)	Shear (lbs)	Moment (lb-in)	Axial (lbs)	Shear (lbs)	Moment (lb-in)	Axial	Shear	Moment
1	-106362	-83509	-13775200	-114000	-81800	-10900000	7%	2%	21%
2	444265	-99208	-15646000	449000	-95200	-12600000	1%	4%	19%
3	411286	-98433	-15572700	410000	-98000	-12800000	0%	0%	18%
4	413174	-98575	-15594400	409000	-101000	-13100000	1%	2%	16%
5	381287	-100288	-15706100	414000	-101000	-13300000	9%	1%	15%
6	514131	-82807	-14307500	488000	-81000	-12300000	5%	2%	14%
7	-106362	-83509	8306320	-112000	-78900	6930000	5%	6%	17%
8	444265	-99208	12179700	457000	-100000	9600000	3%	1%	21%
9	411286	-98433	11956700	422000	-97800	9680000	3%	1%	19%
10	413174	-98575	11979700	419000	-97600	9890000	1%	1%	17%
11	381287	-100288	12236600	409000	-104000	10300000	7%	4%	16%
12	514131	-82807	9318920	490000	-85400	7730000	5%	3%	17%
13	-51492	-48291	-4114670	-50000	-60100	-4110000	3%	24%	0%
14	368154	-113337	-10143600	406000	-115000	-7370000	10%	1%	27%
15	350962	-108655	-9647890	373000	-108000	-7260000	6%	1%	25%
16	350369	-108368	-9622470	374000	-108000	-7260000	7%	0%	25%
17	332343	-112784	-10071100	373000	-118000	-7850000	12%	5%	22%
18	399598	-65849	-5624780	377000	-51500	-3170000	6%	22%	44%
19	-34722	-54880	12421000	-21000	-59900	14000000	40%	9%	13%
20	-30627	-116450	-13278500	-14400	-109000	-9420000	53%	6%	29%
21	-19558	-40328	9044740	1990	-48800	10700000	110%	21%	18%
22	-15462	-101898	-12289200	4580	-97100	-9310000	130%	5%	24%
23	-8188	-41574	9315380	16800	-49600	10700000	305%	19%	15%
24	-4092	-103144	-12392400	9920	-98000	-9680000	342%	5%	22%
25	2760	-40339	9209800	18500	-46700	10700000	570%	16%	16%
26	6856	-101908	-12127200	20400	-94600	-9500000	198%	7%	22%
27	16594	-52963	10180500	38900	-66300	12100000	134%	25%	19%
28	20690	-114532	-14943700	29700	-115000	-13300000	44%	0%	11%

The difference in signs between axial forces is notable here. The negative sign implies that the beam is in compression while the positive sign shows that the beam is experiencing tension. At the first six beam data points RISA shows that the beam is in compression while LS-DYNA only shows that the beam is in compression and the first data point.

4.4 Force vs. Displacement

As a result of the force vs. time comparison, a force vs. displacement comparison was done for the beam elements. If LS-DYNA shows a difference in beam axial forces then it also likely shows a difference in beam displacement based on the forces the beam is experiencing. In order to do this comparison, the displacement of each node was gathered from RISA and from LS-DYNA. These displacements where then used to determine the elongation or the shrinkage of each lower level beam. The LS-DYNA results were drastically different as can be seen in Table 7.

Table 7. Beam x-displacement comparison

Beam Span (grid to grid)	RISA X-Disp (in)	LS-DYNA X-Disp (in)	Percent Difference
1 to 2	0.010	0.029	190%
2 to 3	0.006	0.022	267%
3 to 4	0.002	0.019	850%
4 to 5	-0.002	0.019	1050%
5 to 6	-0.006	0.016	367%

RISA showed that that four beams elongated by a maximum of a hundredth of an inch while two beams shrank by a maximum of six thousandths of an inch. On the contrary, LS-DYNA shows that each beam elongated by a minimum of a hundredth of an inch. Though a difference is shown, it is still very minor in the overall scheme of the beam design. LS-DYNA is showing a greater elongation, but that elongation is still less than one percent of the beam’s overall length. In other words, the effects are negligible and most likely do not need to be considered in the design of the beams.

Chapter 5 - Conclusion

The focus of this report was to study variances between 2D-FEA and 3D-FEA when analyzing a steel SMRF. This was done by evaluating member forces at 28 different locations in both RISA and LS-DYNA. In the research done for this report, LS-DYNA showed the same results as RISA in nearly all instances. The only variance that occurred was within the beam axial forces. 2D-FEA showed that multiple members were in compression, however, 3D-FEA showed than nearly all of the beams were experiencing tension. LS-DYNA has the ability to analyze dynamic behavior, which is likely the reason why it was able to gather different behavioral results than what RISA predicted.

As a result of the difference in axial forces, the beam displacements were investigated. LS-DYNA showed much more beam elongation than the RISA analysis showed. The difference in elongation between the two programs was anywhere from 190% different to 1050% different. Though these differences in results are very large, they are minimal in the scheme of overall beam design. The largest elongation shown by either program was 0.029 inches. This elongation accounts for less than one percent of the overall beam length. The effects of the elongation are essentially negligible. Steel beam design is dictated by shear forces, moment forces, and deflection. The difference in axial forces, though large, are still not enough to impact design. In other words, the variance in results do not warrant a change to current design codes or standards.

The results of this research do not demand a change in current structural design procedures. This is good news for today's engineering industry. This research shows that the SMRF's that have been constructed in years past are structurally sound. The research also shows that steel SMRF’s currently being designed and constructed will have structural integrity. Economically speaking, the results from this research were the desired results for the engineering industry. Designs of the past are still good, and the designs of the future do not have to change.

As stated before, the research done for this report stemmed from similar research that compared a reinforced concrete SMRF with 2D-FEA and 3D-FEA. That research showed two very major differences in analyses that required action to be taken in regard to design codes and standards. The idea in question was whether or not those same large variances would occur between 2D-FEA and 3D-FEA when analyzing a steel SMRF. Though some differences were seen, 2D-FEA proved to be much more accurate when analyzing a steel SMRF. Moving forward, updates in 2D-FEA concrete analysis should be the focus.

This research studied a single steel SMRF experiencing a single load case. In order to gain a better understanding of the results, this research should be done a multitude of times with various other examples. Further research will provide more accuracy and further research will provide a better explanation of program variances. As this subject is researched more, 2D-FEA programs can be adjusted in order to more accurately reflect reality, and engineers can utilize said programs to design structure with integrity.

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Appendix A - FEMA 451

5.2 SEVEN-STORY OFFICE BUILDING, LOS ANGELES, CALIFORNIA

Three alternative framing arrangements for a seven-story office building are illustrated.

5.2.1 Building Description

5.2.1.1 General Description

This seven-story office building of rectangular plan configuration is 177 ft, 4 in. long in the E-W direction and 127 ft, 4 in. wide in the N-S direction (Figure 5.2-1). The building has a penthouse. It extends a total of 118 ft, 4 in. above grade. It is framed in structural steel with 25-ft bays in each direction. The story height is 13 ft, 4 in. except for the first story which is 22 ft, 4 in. high. The penthouse extends 16 ft above the roof level of the building and covers the area bounded by gridlines C, F, 2, and 5 in Figure 5.2-1. Floors consist of 3-1/4 in. lightweight concrete placed on composite metal deck. The elevators and stairs are located in the central three bays. The building is planned for heavy filing systems (350 psf) covering approximately four bays on each floor.

5.2.1.2 Alternatives

This example features three alternatives – a steel moment-resisting frame, concentrically braced frame, and a dual system with a moment-resisting frame at the perimeter and a concentrically braced frame at the core area – as follows:

1. Alternative A – Seismic force resistance is provided by special moment frames located on the perimeter of the building (on lines A, H, 1, and 6 in Figure 5.2-1, also illustrated in Figure 5.2-2).

2. Alternative B – Seismic force resistance is provided by four special concentrically braced frames in each direction. They are located in the elevator core walls between columns 3C and 3D, 3E and 3F, 4C and 4D, and 4E and 4F in the E-W direction and between columns 3C and 4C, 3-D and 4D, 3E and 4E, and 3F and 4F in the N-S direction (Figure 5.2-1). The braced frames in an X configuration are designed for both diagonals being effective in tension and compression. The braced frames are not identical, but are arranged to accommodate elevator door openings. Braced frame elevations are shown in Figure 5.2-3.

3. Alternative C – Seismic force resistance is provided by a dual system with the special moment frames at the perimeter of the building and a special concentrically braced frames at the core. The moment frames are shown in Figure 5.2-2 and the braced frames are shown in Figure 5.2-3.

5.2.1.3 Scope

The example covers:

1. Seismic design parameters
2. Analysis of perimeter moment frames
3. Beam and column proportioning
4. Analysis of concentrically braced frames
5. Proportioning of braces
6. Analysis and proportioning of the dual system

Figure 19. Design Example 5.2 from FEMA 451

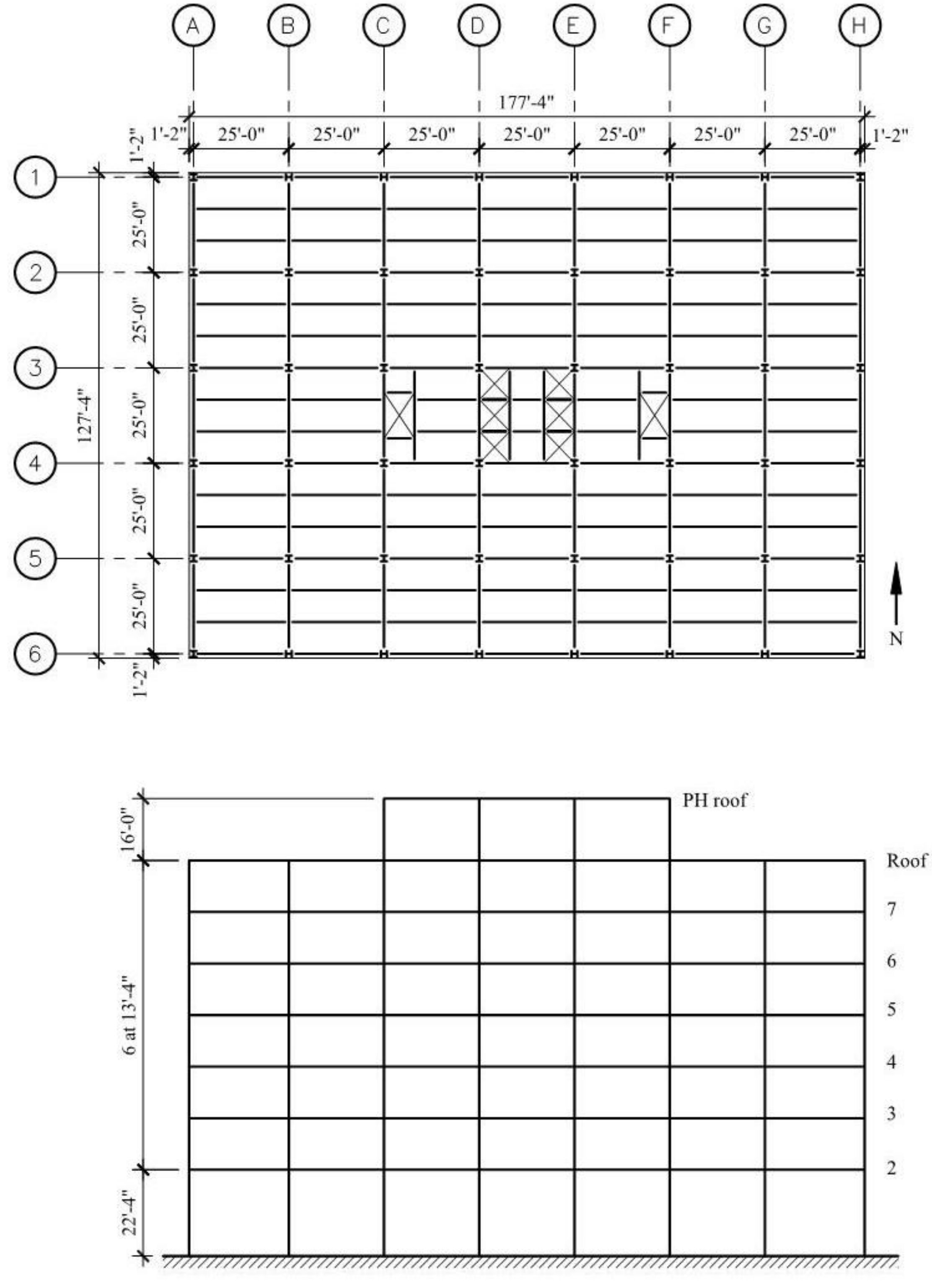


Figure 5.2-1 Typical floor framing plan and building section (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

Figure 20. Design Example 5.2 from FEMA 451

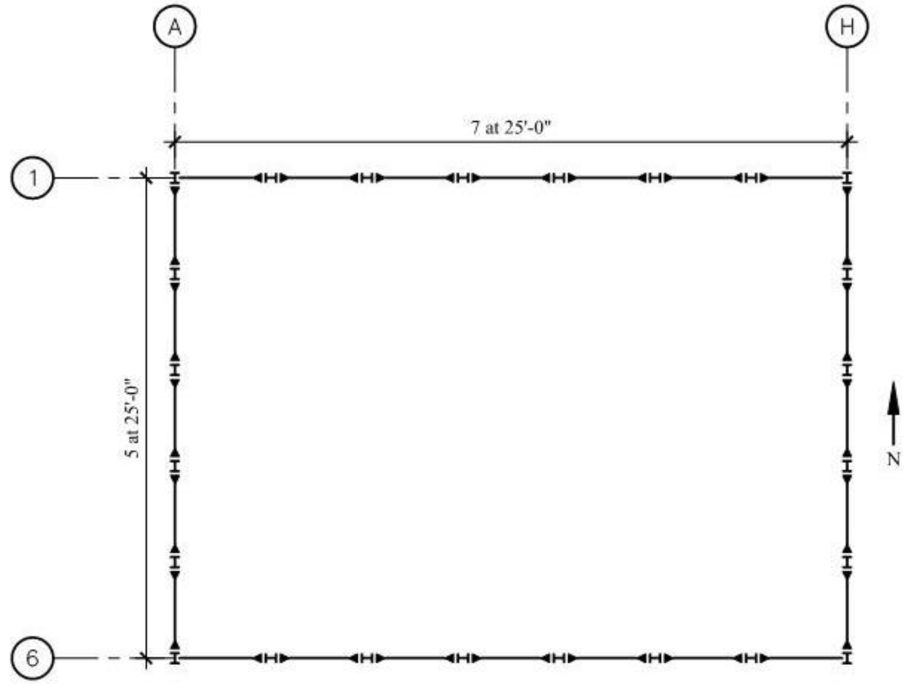


Figure 5.2-2 Framing plan for special moment frame (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

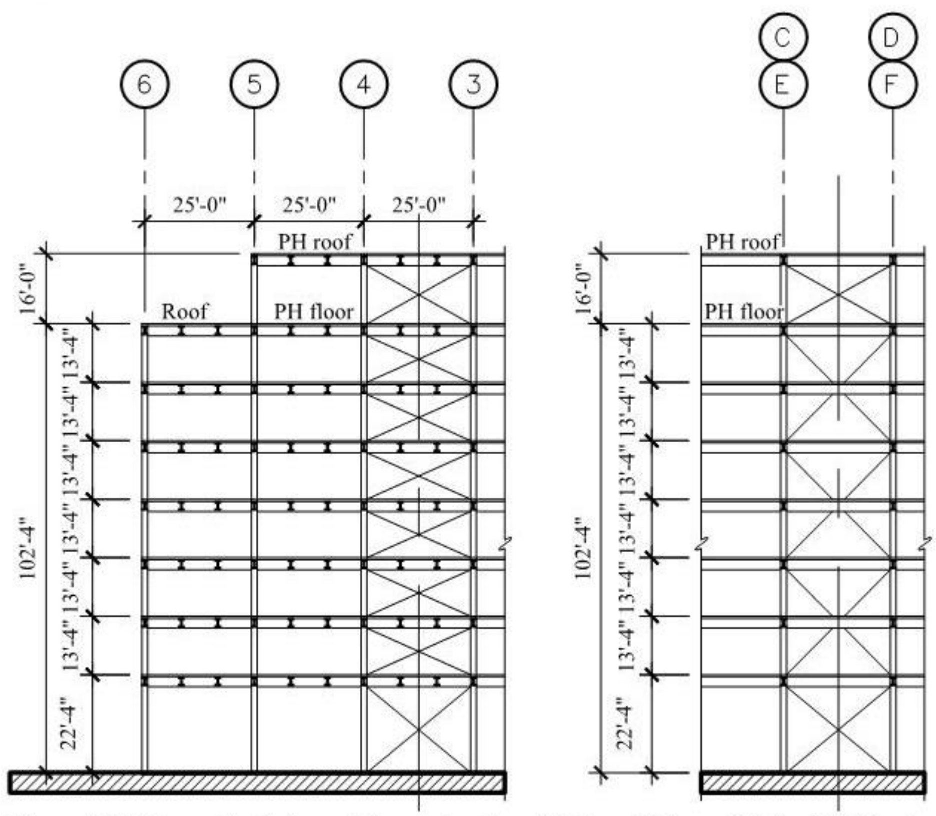


Figure 5.2-3 Concentrically braced frame elevations (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

Figure 21. Design Example 5.2 from FEMA 451

5.2.2 Basic Requirements

5.2.2.1 *Provisions* Parameters

Site Class = D	(*Provisions* Sec. 4.1.2.1 [3.5])
$S_S = 1.5$	(*Provisions* Map 9 [Figure 3.3-3])
$S_1 = 0.6$	(*Provisions* Map 10 [Figure 3.3-4])
$F_a = 1.0$	(*Provisions* Table 4.1.2.4a [3.3-1])
$F_v = 1.5$	(*Provisions* Table 4.1.2.4b [3.3-2])
$S_{MS} = F_a S_S = 1.5$	(*Provisions* Eq. 4.1.2.4-1 [3.3-1])
$S_{M1} = F_v S_1 = 0.9$	(*Provisions* Eq. 4.1.2.4-2 [3.3-2])
$S_{DS} = 2/3\ S_{MS} = 1.0$	(*Provisions* Eq. 4.1.2.5-1 [3.3-3])
$S_{D1} = 2/3\ S_{M1} = 0.6$	(*Provisions* Eq. 4.1.2.5-2 [3.3-4])
Seismic Use Group = I	(*Provisions* Sec. 1.3 [1.2])
Seismic Design Category = D	(*Provisions* Sec. 4.2.1 [1.4])

Alternative A, Special Steel Moment Frame (*Provisions* Table 5.2.2 [4.3-1])

$R = 8$
$\Omega_0 = 3$
$C_d = 5.5$

Alternative B, Special Steel Concentrically Braced Frame (*Provisions* Table 5.2.2 [4.3-1])

$R = 6$
$\Omega_0 = 2$
$C_d = 5$

Alternative C, Dual System of Special Steel Moment Frame Combined with Special Steel Concentrically Braced Frame (*Provisions* Table 5.2.2 [4.3-1])

$R = 8$
$\Omega_0 = 2.5$
$C_d = 6.5$

5.2.2.2 Loads

Roof live load (L)	= 25 psf
Penthouse roof dead load (D)	= 25 psf
Exterior walls of penthouse	= 25 psf of wall
Roof *DL* (roofing, insulation, deck beams, girders, fireproofing, ceiling, M&E)	= 55 psf
Exterior wall cladding	= 25 psf of wall
Penthouse floor D	= 65 psf
Floor L	= 50 psf
Floor D (deck, beams, girders, fireproofing, ceiling, M&E, partitions)	= 62.5 psf
Floor L reductions per the IBC	

5.2.2.3 Materials

Concrete for drilled piers	$f_c' = 5$ ksi, normal weight (NW)
Concrete for floors	$f_c' = 3$ ksi, lightweight (LW)

Figure 22. Design Example 5.2 from FEMA 451

All other concrete — $f_c' = 4$ ksi, NW

Structural steel

- Wide flange sections — ASTM A992, Grade 50
- HSS — ASTM A500, Grade B
- Plates — ASTM A36

5.2.3 Structural Design Criteria

5.2.3.1 Building Configuration

The building is considered vertically regular despite the relatively tall height of the first story. The exception of *Provisions* Sec. 5.2.3.3 [4.3.2.3]is taken in which the drift ratio of adjacent stories are compared rather than the stiffness of the stories. In the 3-D analysis, it will be shown that the first story drift ratio is less than 130 percent of the story above. Because the building is symmetrical in plan, plan irregularities would not be expected. Analysis reveals that Alternatives B and C are torsionally irregular, which is not uncommon for core-braced buildings.

5.2.3.2 Redundancy

According to *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*], the reliability factor, (ρ) for a Seismic Design Category D structure is:

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{A_x}}$$

In a typical story, the floor area, $A_x = 22{,}579$ ft.^{2}

The ratio of the design story shear resisted by the single element carrying the most shear force in the story to the total story shear is r_{max_x} as defined in *Provisions* Sec. 5.2.4.2.

Preliminary ρ factors will be determined for use as multipliers on design force effects. These preliminary ρ factors will be verified by subsequent analyses.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. For a building assigned to Seismic Design Category D, $\rho = 1.0$ as long as it can be shown that failure of beam-to-column connections at both ends of a single beam (moment frame system) or failure of an individual brace (braced frame system) would not result in more than a 33 percent reduction in story strength or create an extreme torsional irregularity. Alternatively, if the structure is regular in plan and there are at least two bays of perimeter framing on each side of the structure in each orthogonal direction, it is permitted to use $\rho = 1.0$. Per 2003 *Provisions* Sec. 4.3.1.4.3, special moment frames in Seismic Design Category D must be configured such that the structure satisfies the criteria for $\rho = 1.0$. There are no reductions in the redundancy factor for dual systems. Based on the preliminary design, $\rho = 1.0$ for Alternative A because it has a perimeter moment frame and is regular. The determination of ρ for Alternatives B and C (which are torsionally irregular) requires the evaluation of connection and brace failures per 2003 *Provisions* Sec. 4.3.3.2.]

5.2.3.2.1 Alternative A (moment frame)

Figure 23. Design Example 5.2 from FEMA 451

For a moment-resisting frame, r_{max_x} is taken as the maximum of the sum of the shears in any two adjacent columns divided by the total story shear. The final calculation of ρ will be deferred until the building frame analysis, which will include the effects of accidental torsion, is completed. At that point, we will know the total shear in each story and the shear being carried by each column at every story. See Sec. 5.2.4.3.1.

Provisions Sec. 5.2.4.2 requires that the configuration be such that ρ shall not exceed 1.25 for special moment frames. [1.0 in the 2003 *Provisions*] (There is no limit for other structures, although ρ need not be taken larger than 1.50 in the design.) Therefore, it is a good idea to make a preliminary estimate of ρ, which was done here. In this case, ρ was found to be 1.11 and 1.08 in the N-S and E-W directions, respectively. A method for a preliminary estimate is explained in Alternative B.

Note that ρ is a multiplier that applies only to the force effects (strength of the members and connections), not to displacements. As will be seen for this moment-resisting frame, drift, and not strength, will govern the design.

5.2.3.2.2 Alternative B (concentrically braced frame)

Again, the following preliminary analysis must be refined by the final calculation. For the braced frame system, there are four braced-bay braces subject to shear at each story, so the direct shear on each line of braces is equal to $V_x/4$. The effects of accidental torsion will be estimated as:

The torsional moment $M_{ta} = (0.05)(175)(V_x) = 8.75V_x$.
The torsional force applied to either grid line C or F is $V_t = M_{ta}Kd / \Sigma Kd^2$.

Assuming all frame rigidity factors (K) are equal:

$$V_t = \frac{M_{ta}(37.5)}{\left[(2)(37.5)^2+(6)(12.5)^2\right]} = 0.01M_{ta}$$

$$V_t = (0.01)(8.75\ V_x) = 0.0875V_x$$

The amplification of torsional shear (A_x) must be considered in accordance with *Provisions* Sec. 5.4.4.1.3 [5.2.4.3]. Without dynamic amplification of torsion, the direct shear applied to each line of braces is $V_x/4$ and the torsional shear, $V_t = 0.0875\ V_x$. Thus, the combined shear at Grid C is $0.25V_x$ - $0.0875V_x$ = $0.1625V_x$, and the combined shear at Grid F is $0.25V_x + 0.0875V_x = 0.3375V_x$. As the torsional deflections will be proportional to the shears and extrapolating to Grids A and H, the deflection at A can be seen to be proportional to $0.250V_x + (0.0875V_x.)(87.5/37.5) = 0.454V_x$. Likewise, the deflection at H is proportional to $0.250V_x - (0.0875V_x)(87.5/37.5) = 0.046V_x$. The average deflection is thus proportional to $[(0.454 + 0.046)/2]V_x = 0.250V_x$. These torsional effects are illustrated in Figure 5.2-4.

Figure 24. Design Example 5.2 from FEMA 451

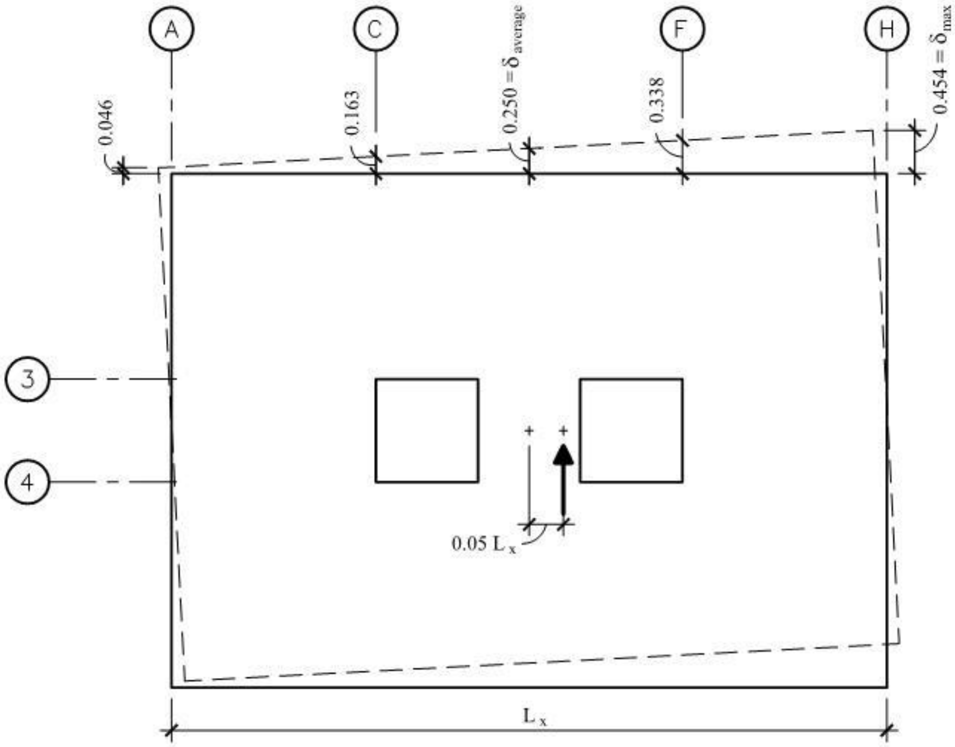


Figure 5.2-4 Approximate effect of accidental of torsion (1.0 in. = 25.4 mm).

From the above estimation of deflections, the torsional amplification can be determined per *Provisions* Eq. 5.4.4.1.3.1 [5.2-13] as:

$$A_x = \left(\frac{\delta_{max}}{1.2\ \delta_{avg}}\right)^2 = \left(\frac{0.454}{(1.2)(0.250)}\right)^2 = 2.29$$

The total shear in the N-S direction on Gridlines C or F is the direct shear plus the amplified torsional shear equal to:

$$V_x/4 + A_x V_t = [0.250 + (2.29)(0.0875)]V_x = 0.450 V_x$$

As there are two braces in each braced bay (one in tension and the other in compression):

$$r_{max_x} = \frac{0.450}{2} = 0.225$$

and

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{Ax}} = 2 - \frac{20}{(0.225)\sqrt{22{,}579}} = 1.41$$

Figure 25. Design Example 5.2 from FEMA 451

Therefore, use $\rho = 1.41$ for the N-S direction. In a like manner, the ρ factor for the E-W direction is determined to be $\rho = 1.05$. These preliminary values will be verified by the final calculations.
5.2.3.2.3 Alternative C (dual system)

For the dual system, the preliminary value for ρ is taken as 1.0. The reason for this decision is that, with the dual system, the moment frame will substantially reduce the torsion at any story, so torsional amplification will be low. The combined redundancy of the braced frame combined with the moment frame (despite the fact that the moment frame is more flexible) will reduce ρ from either single system. Finally, *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*] calls for taking only 80 percent of the calculated ρ value when a dual system is used. Thus, we expect the final value to fall below 1.0, for which we will take $\rho = 1.0$. This will be verified by analysis later.

5.2.3.3 Orthogonal Load Effects

A combination of 100 percent of the seismic forces in one direction with 30 percent seismic forces in orthogonal direction is required for structures in Seismic Design Category D (*Provisions* Sec. 5.2.5.2.3 and 5.2.5.2.2 [4.4.2.2]).

5.2.3.4 Structural Component Load Effects

The effect of seismic load is be defined by *Provisions* Eq, 5.2.7-1 [4.2-1] as:

$$E = \rho Q_E + 0.2 S_{DS} D$$

Recall that $S_{DS} = 1.0$. As stated above, ρ values are preliminary estimates to be checked and, if necessary, refined later.

For Alternative A

N-S direction $E = (1.11)Q_E \pm (0.2)D$
E-W direction $E = (1.08)Q_E \pm (0.2)D$

Alternative B

N-S direction $E = (1.41)Q_E \pm (0.2)D$
E-W direction $E = (1.05)Q_E \pm (0.2)D$

Alternative. C

N-S direction $E = (1.00)Q_E \pm (0.2)D$
E-W direction $E = (1.00)Q_E \pm (0.2)D$

5.2.3.5 Load Combinations

Load combinations from ASCE 7 are:

$$1.2D + 1.0E + 0.5L + 0.2S$$

and

$$0.9D + 1.0E + 1.6H$$

Figure 26. Design Example 5.2 from FEMA 451

To each of these load combinations, substitute E as determined above, showing the maximum additive and minimum negative. Recall that Q_E acts both east and west (or north and south):

Alternative A

N-S	$1.4D + 1.11Q_E + 0.5L$ and $0.7D + 1.11Q_E$
E-W	$1.4D + 1.08Q_E + 0.5L$ and $0.7D + 1.08Q_E$

Alternative B

N-S	$1.4D + 1.41Q_E + 0.5L$ and $0.7D + 1.41Q_E$
E-W	$1.4D + 1.05Q_E + 0.5L$ and $0.7D + 1.05Q_E$

Alternative C

N-S	$1.4D + Q_E + 0.5L$ and $0.7D + Q_E$
E-W	$1.4D + Q_E + 0.5L$ and $0.7D + Q_E$

5.2.3.6 Drift Limits

The allowable story drift per *Provisions* Sec. 5.2.8 [4.5-1] is $\Delta_a = 0.02h_{sx}$.

The allowable story drift for the first floor is $\Delta_a = (0.02)(22.33 \text{ ft})(12 \text{ in./ft}) = 5.36$ in.
The allowable story drift for a typical story is $\Delta_a = (0.02)(13.33 \text{ ft})(12 \text{ in./ft}) = 3.20$ in.

Remember to adjust calculated story drifts by the appropriate C_d factor from Sec. 5.2.2.1.

Consider that the maximum story drifts summed to the roof of the seven-story building, (102 ft-4 in. main roof/penthouse floor) is 24.56 in.

5.2.3.7 Basic Gravity Loads

Penthouse roof

Roof slab = (0.025)(75)(75)	= 141 kips
Walls = (0.025)(8)(300)	= 60 kips
Columns = (0.110)(8)(16)	= 14 kips
Total	= 215 kips

Lower roof

Roof slab = $(0.055)[(127.33)(177.33) - (75)^2]$	= 932 kips
Penthouse floor = (0.065)(75)(75)	= 366 kips
Walls = 60 + (0.025)(609)(6.67)	= 162 kips
Columns = 14 + (0.170)(6.67)(48)	= 68 kips
Equipment (allowance for mechanical equipment in penthouse)	= 217 kips
Total	= 1,745 kips

Figure 27. Design Example 5.2 from FEMA 451

Typical floor

Floor = (0.0625)(127.33)(177.33)	= 1,412 kips
Walls = (0.025)(609)(13.33)	= 203 kips
Columns = (0.285)(13.33)(48)	= 182 kips
Heavy storage = (0.50)(4)(25 x 25)(350)	= 438 kips
Total	= 2,235 kips

Total weight of building = 215 + 1,745 + 6(2,235) = 15,370 kips

Note that this office building has heavy storage in the central bays of 280 psf over five bays. Use 50 percent of this weight as effective seismic mass. (This was done to add seismic mass to this example thereby making it more interesting. It is not meant to imply that the authors believe such a step is necessary for ordinary office buildings.)

5.2.4 Analysis

5.2.4.1 Equivalent Lateral Force Analysis

The equivalent lateral force (ELF) procedure will be used for each alternative building system. The seismic base shear will be determined for each alternative in the following sections.

5.2.4.1.1 ELF Analysis for Alternative A, Moment Frame

First determine the building period (T) per *Provisions* Eq. 5.4.2.1-1 [5.2-6]:

$$T_a = C_r h_n^x = (0.028)(102.3)^{0.8} = 1.14 \text{ sec}$$

where h_n, the height to the main roof, is conservatively taken as 102.3 ft. The height of the penthouse (the penthouse having a smaller contribution to seismic mass than the main roof or the floors) will be neglected.

The seismic response coefficient (C_s) is determined from *Provisions* Eq. 5.4.1.1-1 [5.2-2] as:

$$C_S = \frac{S_{DS}}{R/I} = \frac{1}{(8/1)} = 0.125$$

However, *Provisions* Eq. 5.4.1.1-2 [5.2-3] indicates that the value for C_s need not exceed:

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.6}{1.14(8/1)} = 0.066$$

and the minimum value for C_s per *Provisions* Eq. 5.4.1.1-3 [not applicable in the 2003 *Provisions*] is:

$$C_s = 0.044 I S_{DS} = (0.044)(1)(1) = 0.044$$

Therefore, use C_s = 0.066.

Figure 28. Design Example 5.2 from FEMA 451

Seismic base shear is computed per *Provisions* Eq. 5.4.1 [5.2-1] as:

$$V = C_S W = (0.066)(15,370) = 1014 \text{ kips}$$

5.2.4.1.2 ELF Analysis for Alternative B, Braced Frame

As above, first find the building period (T) using *Provisions* Eq. 5.4.2.1-1 [5.2-6]:

$$T_a = C_r h_n^x = (0.02)(102.3)^{0.75} = 0.64 \text{ sec}$$

The seismic response coefficient (C_s) is determined from *Provisions* Eq. 5.4.1.1-1 [5.2-2] as:

$$C_S = \frac{S_{DS}}{R/I} = \frac{1}{(6/1)} = 0.167$$

However, *Provisions* Eq. 5.4.1.1-2 [5.2-3] indicates that the value for C_s need not exceed:

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.6}{(0.64)(6/1)} = 0.156$$

and the minimum value for C_s per *Provisions* Eq. 5.4.1.1-3 [not applicable in 2003 *Provisions*] is:

$$C_s = 0.044 I S_{DS} = (0.044)(1)(1) = 0.044$$

Use $C_s = 0.156$.

Seismic base shear is computed using *Provisions* Eq. 5.4.1 [5.2-1] as:

$$V = C_S W = (0.156)(15,370) = 2,398 \text{ kips}$$

5.2.4.1.3 ELF Analysis for Alternative C, Dual System

The building period (T) is the same as for the braced frame (*Provisions* Eq. 5.4.2.1-1 [5.2-6]):

$$T_a = C_r h_n^x = (0.02)(102.3)^{0.75} = 0.64 \text{ sec}$$

The seismic response coefficient (C_s) is determined as (*Provisions* Eq. 5.4.1.1-1 [5.2-2]):

$$C_S = \frac{S_{DS}}{R/I} = \frac{1}{(8/1)} = 0.125$$

However, the value for C_s need not exceed (*Provisions* Eq. 5.4.1.1-2 [5.2-3]):

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.6}{(0.64)(8/1)} = 0.117$$

and the minimum value for C_s is (*Provisions* Eq. 5.4.1.1-3 [not applicable in the 2003 *Provisions*]):

$$C_s = 0.044 I S_{DS} = (0.044)(1)(1) = 0.044$$

Therefore, use $C_s = 0.117$.

Figure 29. Design Example 5.2 from FEMA 451

Seismic base shear is computed as (*Provisions* Eq. 5.4.1 [5.2-1]):

$V = C_s W = (0.117)(15{,}370) = 1{,}798$ kips

5.2.4.2 Vertical Distribution of Seismic Forces

Provisions Sec. 5.4.3 [5.2.3] provides the procedure for determining the portion of the total seismic load that goes to each floor level. The floor force F_x is calculated using *Provisions* Eq. 5.4.3-1 [5.2-10] as:

$$F_x = C_{vx} V$$

where (*Provisions* Eq. 5.4.3-2 [5.2-11])

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

For Alternative A

$T = 1.14$ secs, thus $k = 1.32$

For Alternatives B and C

$T = 0.64$ sec, thus $k = 1.07$

Using *Provisions* Eq. 5.4.4 [5.2-12], the seismic design shear in any story is computed as:

$$V_x = \sum_{i=x}^{n} F_i$$

The story overturning moment is computed from *Provisions* Eq. 5.4.5 [5.2-14]:

$$M_x = \sum_{i=x}^{n} F_i (h_i - h_x)$$

The application of these equations for the three alternative building frames is shown in Tables 5.2-1, 5.2-2, and 5.1-3.

Figure 30. Design Example 5.2 from FEMA 451

Table 5.2-1 Alternative A, Moment Frame Seismic Forces and Moments by Level

Level (x)	W_x (kips)	h_x (ft)	$W_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
PH Roof	215	118.33	117,200	0.03	32	32	514
Main roof	1,745	102.33	785,200	0.21	215	247	3,810
Story 7	2,235	89.00	836,500	0.23	229	476	10,160
Story 6	2,235	75.67	675,200	0.18	185	661	18,980
Story 5	2,235	62.33	522,700	0.14	143	805	29,710
Story 4	2,235	49.00	380,500	0.10	104	909	41,830
Story 3	2,235	35.67	250,200	0.07	69	977	54,870
Story 2	2,235	22.33	134,800	0.04	37	1,014	77,520
Σ	15,370		3,702,500	1.00	1,014		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m.

Table 5.2-2 Alternative B, Braced Frame Seismic Forces and Moments by Level

Level (x)	W_x (kips)	h_x (ft)	$W_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
PH Roof	215	118.33	35,500	0.03	67	67	1,070
Main roof	1,745	102.33	246,900	0.19	463	530	8,130
Story 7	2,235	89.00	272,300	0.21	511	1,041	22,010
Story 6	2,235	75.67	228,900	0.18	430	1,470	41,620
Story 5	2,235	62.33	186,000	0.15	349	1,819	65,870
Story 4	2,235	49.00	143,800	0.11	270	2,089	93,720
Story 3	2,235	35.67	102,400	0.08	192	2,281	124,160
Story 2	2,235	22.33	62,000	0.05	116	2,398	177,720
Σ	15,370		1,278,000	1.00	2,398		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m.

Figure 31. Design Example 5.2 from FEMA 451

Table 5.2-3 Alternative C, Dual System Seismic Forces and Moments by Level

Level (x)	W_x (kips)	h_x (ft)	$W_x h_x^k$ (ft-kips)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
PH Roof	215	118.33	35,500	0.03	50	50	800
Main roof	1,745	102.33	246,900	0.19	347	397	6,100
Story 7	2,235	89.00	272,350	0.21	383	781	16,500
Story 6	2,235	75.67	228,900	0.18	322	1,103	31,220
Story 5	2,235	62.33	186,000	0.15	262	1,365	49,400
Story 4	2,235	49.00	143,800	0.11	202	1,567	70,290
Story 3	2,235	35.67	102,386	0.08	144	1,711	93,120
Story 2	2,235	22.33	62,000	0.05	87	1,798	133,270
Σ	15,370		1,278,000	1.00	1,798		

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m.

Be sure to note that the seismic mass at any given level, which includes the lower half of the wall above that level and the upper half of the wall below that level, produces the shear applied at that level and that shear produces the moment which is applied at the top of the next level down. Resisting the overturning moment is the weight of the building above that level combined with the moment resistance of the framing at that level. Note that the story overturning moment is applied to the level below the level that receives the story shear. (This is illustrated in Figure 9.2-4 in the masonry examples.)

5.2.4.3 Size Members

At this point we are ready to select the sizes of the framing members. The method for each alternative is summarized below.

Alternative A, Special Moment Frame:

1. Select preliminary member sizes
2. Check deflection and drift (*Provisions* Sec. 5.2.8 [5.4.1])
3. Check torsional amplification (*Provisions* Sec. 5.4.4.1.3 [5.2.4.3])
4. Check the column-beam moment ratio rule (AISC Seismic Sec. 9.6)
5. Check shear requirement at panel-zone (AISC Seismic Sec. 9.3; FEMA 350 Sec. 3.3.3.2)
6. Check redundancy (*Provisions* Sec. 5.2.4.2 [5.3.3])
7. Check strength

Reproportion member sizes as necessary after each check. The most significant criteria for the design are drift limits, relative strengths of columns and beams, and the panel-zone shear.

Figure 32. Design Example 5.2 from FEMA 451

Alternative B, Special Concentrically Braced Frame:

1. Select preliminary member sizes
2. Check strength
3. Check drift (*Provisions* Sec. 5.2.8 [4.5.1])
4. Check torsional amplification (*Provisions* Sec. 5.4.4.1 [5.2.4.3])
5. Check redundancy (*Provisions* Sec. 5.2.4.2 [4.3.3])

Reproportion member sizes as necessary after each check. The most significant criteria for this design is torsional amplification.

Alternative C, Dual System:

1. Select preliminary member sizes
2. Check strength of moment frame for 25 percent of story shear (*Provisions* Sec. 5.2.2.1 [4.3.1.1])
3. Check strength of braced frames
4. Check drift for total building (*Provisions* Sec. 5.2.8 [4.5.1])
5. Check torsional amplification (*Provisions* Sec. 5.4.4.1 [5.2.4.3])
6. Check redundancy (*Provisions* Sec. 5.2.4.2 [4.3.3])

Reproportion member sizes as necessary after each check.

5.2.4.3.1 Size Members for Alternative A, Moment Frame

1. Select Preliminary Member Sizes – The preliminary member sizes are shown for the moment frame in the X-direction (7 bays) in Figure 5.2-5 and in the Y direction (5 bays) in Figure 5.2-6.

   Check Local Stability – Check beam flange stability in accordance with AISC Seismic Table I-9-1 [I-8-1] (same as FEMA 350 Sec. 3.3.1.1) and beam web stability in accordance with AISC Seismic Table I-9-1 [I-8-1]. (FEMA 350 Sec.3.3.1.2.is more restrictive for cases with low $P_u/\phi_b P_y$, such as in this example.) Beam flange slenderness ratios are limited to $52/\sqrt{F_y}$ and beam web height-to-thickness ratios are limited to $418/\sqrt{F_y}$.

   [The terminology for local stability has been revised in the 2002 edition of AISC Seismic. The limiting slenderness ratios in AISC Seismic use the notation λ_{ps} ("seismically compact") to differentiate them from λ_p in AISC LRFD. In addition, the formulas appear different because the elastic modulus, E_s, has been added as a variable. Both of these changes are essentially editorial, but Table I-8-1 in the 2002 edition of AISC Seismic has also been expanded to include more elements than in the 1997 edition.]

   Be careful because certain shapes found in the AISC Manual will not be permitted for Grade 50 steel (but may have been permitted for Grade 36 steel) because of these restrictions. For Grade 50, *b/t* is limited to 7.35.

   Further note that for columns in special steel moment frames such as this example, AISC Seismic 9.4b [I-8-1] requires that when the column moment strength to beam moment strength ratio is less than or equal to 2.0, the more stringent λ_p requirements apply for *b/t*, and when $P_u/\phi_b P_y$ is less than or equal to 0.125, the more stringent *h/t* requirements apply.

Figure 33. Design Example 5.2 from FEMA 451

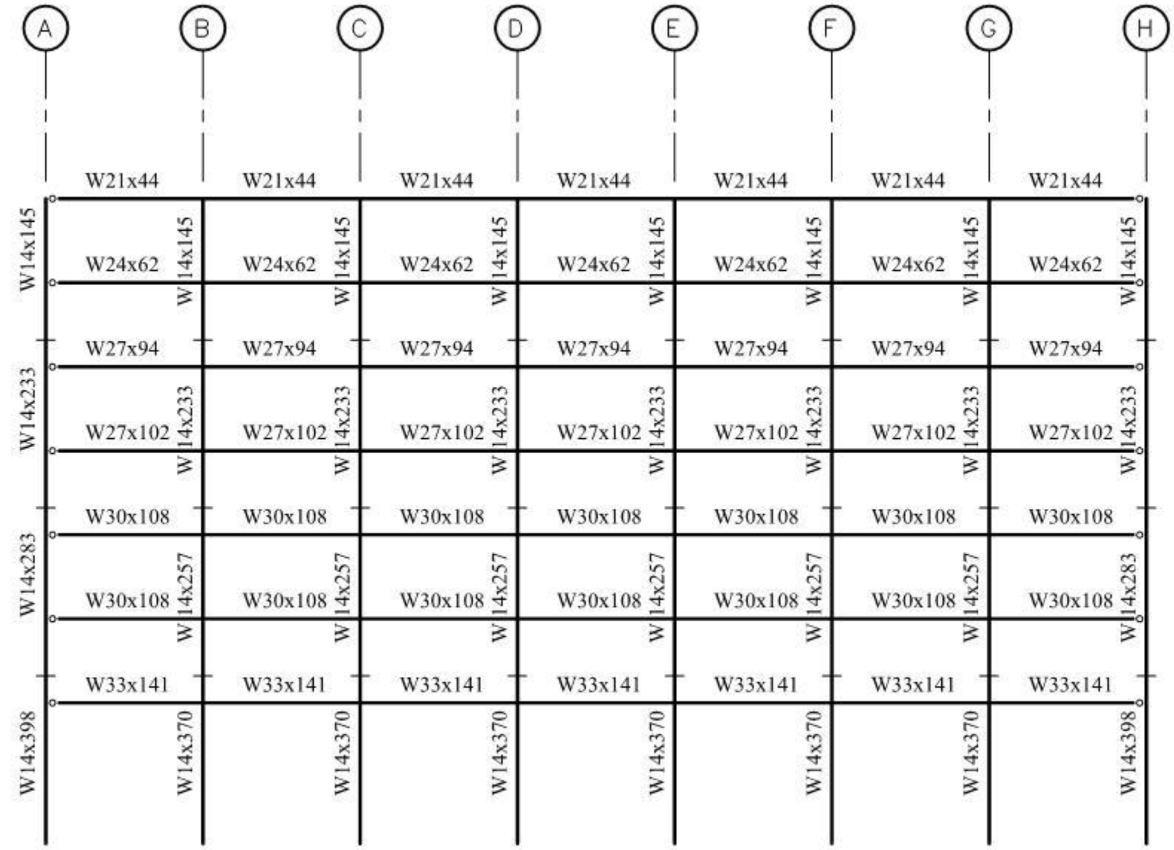


Figure 5.2-5 SMRF frame in E-W direction (penthouse not shown).

2. Check Drift – Check drift in accordance with *Provisions* Sec. 5.2.8 [4.5.1]. The building was modeled in 3-D using RAMFRAME. Displacements at the building centroid are used here because the building is not torsionally irregular (see the next paragraph regarding torsional amplification). Calculated story drifts and C_d amplification factors are summarized in Table 5.2-4. P-delta effects are included.

   All story drifts are within the allowable story drift limit of $0.020h_{sx}$ per *Provisions* Sec. 5.2.8 [4.5.1] and Sec. 5.2.3.6 of this chapter.

   As indicated below, the first story drift ratio is less than 130 percent of the story above (*Provisions* Sec. 5.2.3.3 [4.3.2.3]):

$$\frac{C_d\Delta_{x\ \text{story 2}}}{C_d\Delta_{x\ \text{story 3}}} = \frac{\left(\dfrac{5.17\text{ in.}}{268\text{ in.}}\right)}{\left(\dfrac{3.14\text{ in.}}{160\text{ in.}}\right)} = 0.98 < 1.30$$

   Therefore, there is no vertical irregularity.

Figure 34. Design Example 5.2 from FEMA 451

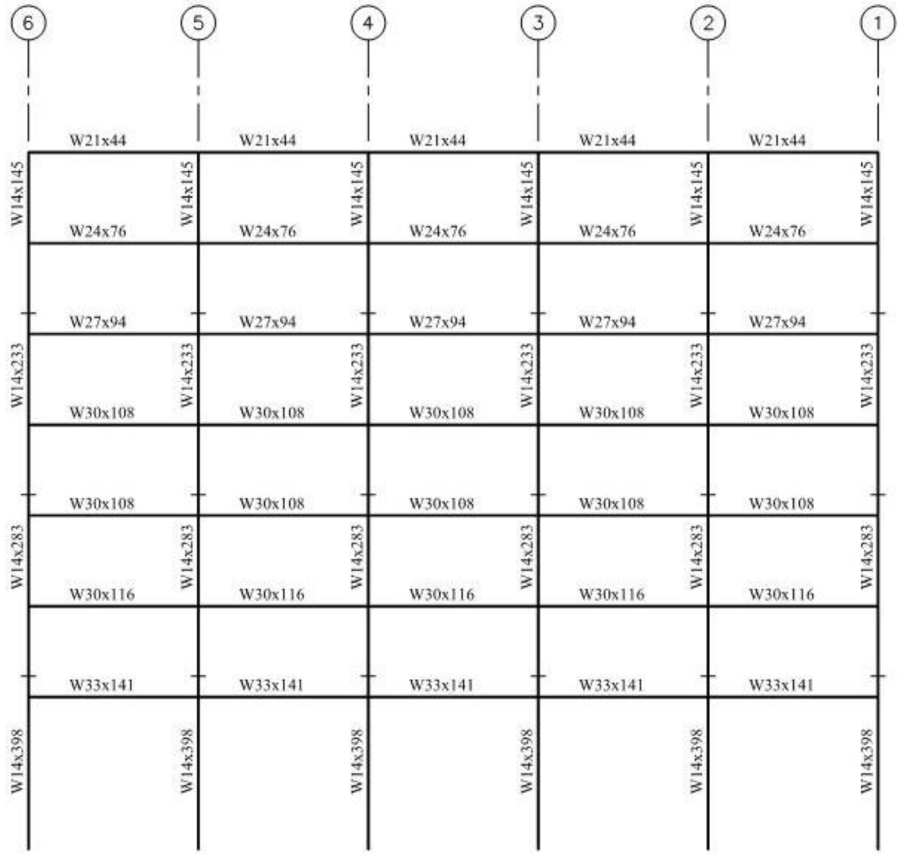


Figure 5.2-6 SMRF frame in N-S direction (penthouse not shown).

Table 5.2-4 Alternative A (Moment Frame) Story Drifts under Seismic Loads

	Total Displacement at Building Centroid (86.5, 62.5)		Story Drift from 3-D Elastic Analysis at Building Centroid		C_d	(C_d) x (Elastic Story Drift)		Allowable Story Drift
	δE-W (in.)	δN-S (in.)	ΔE-W (in.)	ΔN-S (in.)	5.5	ΔE-W (in.)	ΔN-S (in.)	Δ (in.)
Roof	4.24	4.24	0.48	0.47	5.5	2.64	2.59	3.20
Floor 7	3.76	3.77	0.57	0.58	5.5	3.14	3.19	3.20
Floor 6	3.19	3.19	0.54	0.53	5.5	2.97	2.92	3.20
Floor 5	2.65	2.66	0.57	0.58	5.5	3.14	3.19	3.20
Floor 4	2.08	2.08	0.57	0.58	5.5	3.14	3.19	3.20
Floor 3	1.51	1.50	0.57	0.57	5.5	3.14	3.14	3.20
Floor 2	0.94	0.93	0.94	0.93	5.5	5.17	5.12	5.36

1.0 in. = 25.4 mm.

Figure 35. Design Example 5.2 from FEMA 451

3. Check Torsional Amplification – The torsional amplification factor per *Provisions* Eq. 5.4.4.1.3-1 [5.2-13] is:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2$$

If $A_x < 1.0$, then torsional amplification need not be considered. It is readily seen that if the ratio of $\delta_{max}/\delta_{avg}$ is less that 1.2, then torsional amplification will not be necessary.

The 3-D analysis provided the story deflections listed in Table 5.2-5. Because none of the ratios for $\delta_{max}/\delta_{avg}$ exceed 1.2, torsional amplification of forces is not necessary for the moment frame alternative.

Table 5.2-5 Alternative A Torsional Analysis

	Torsion Checks			
	$\delta_{EW_{max}}$ (in.) (175,0)	$\delta_{NS_{max}}$ (in.) (125,0)	$\delta_{EW_{max}} / \delta_{EW_{avg}}$	$\delta_{NS_{max}} / \delta_{NS_{avg}}$
Roof	4.39	4.54	1.04	1.07
Story 7	3.89	4.04	1.04	1.07
Story 6	3.30	3.42	1.04	1.07
Story 5	2.75	2.85	1.03	1.07
Story 4	2.16	2.23	1.04	1.07
Story 3	1.57	1.62	1.04	1.08
Story 2	0.98	1.00	1.04	1.08

1.0 in. = 25.4 mm.

Member Design Considerations – Because $P_u/\phi P_n$ is typically less than 0.4 for the columns (re: AISC Seismic Sec. 8.2 [8.3]), combinations involving Ω_0 factors do not come into play for the special steel moment frames. In sizing columns (and beams) for strength we will satisfy the most severe value from interaction equations. However, this frame is controlled by drift. So, with both strength and drift requirements satisfied, we will check the column-beam moment ratio and the panel zone shear.

4. Check the Column-Beam Moment Ratio – Check the column-beam moment ratio per AISC Seismic Sec. 9.6. For purposes of this check, the plastic hinge was taken to occur at $0.5d_b$ from the face or the column in accordance with FEMA 350 for WUF-W connections (see below for description of these connections). The expected moment strength of the beams were projected from the plastic hinge location to the column centerline per the requirements of AISC Seismic Sec. 9.6. This is illustrated in Figure 5.2-7. For the columns, the moments at the location of the beam flanges is projected to the column-beam intersection as shown in Figure 5.2-8.

Figure 36. Design Example 5.2 from FEMA 451

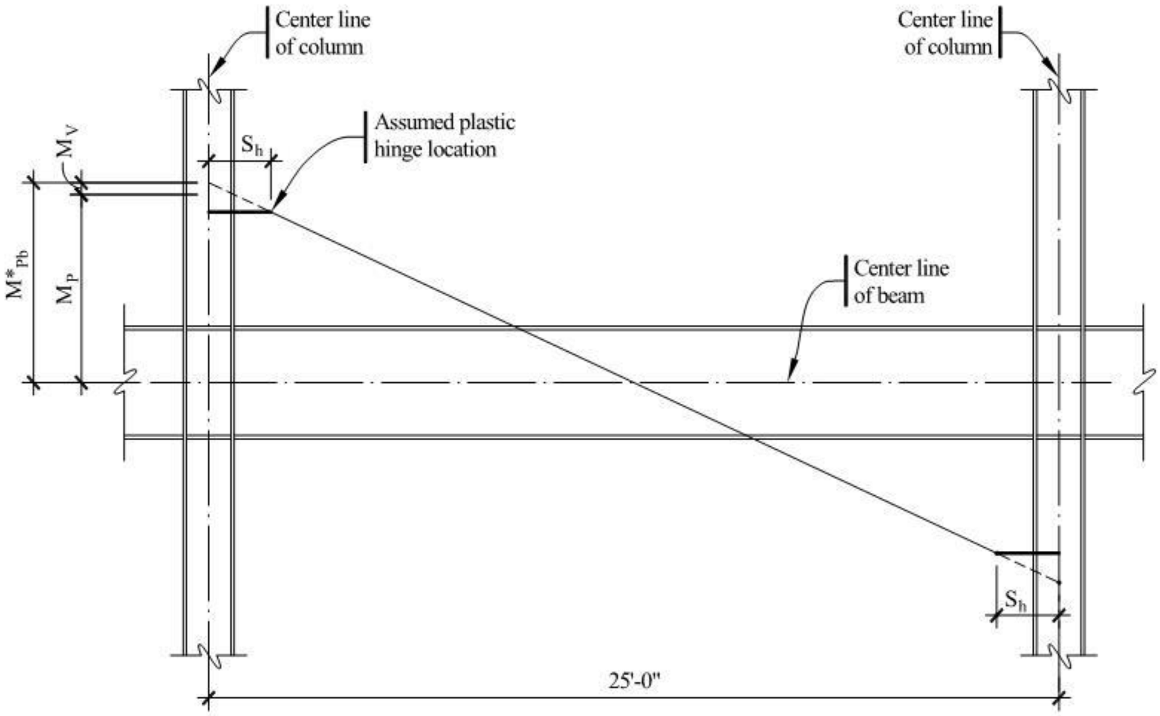


Figure 5.2-7 Projection of expected moment strength of beam (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

The column-beam strength ratio calculation is illustrated for the lower level in the E-W direction, Level 2, at gridline G (W14×370 column and W33×141 beam). For the columns:

$$\Sigma M^*_{pc} = \Sigma Z_c \left(F_{yc} - \frac{P_{uc}}{A_g} \right)$$

$$\Sigma M^*_{pc} = 2\left[736 \text{ in.}^3 \left(50 \text{ ksi} - \frac{500 \text{ kips}}{109 \text{ in.}^2} \right) \right] = 66{,}850 \text{ ft-kips}$$

Adjust this by the ratio of average story height to average clear height between beams, or (268 + 160)/ (251.35 + 128.44) = 1.13. Therefore, ΣM^*_{pc} = (1.13)(66,850) = 75,300 ft-kips. For the beams,

$$\Sigma M^*_{pb} = \Sigma(1.1 R_y M_p + M_v)$$

where

R_y = 1.1 for Grade 50 steel
$M_p = F_y Z$ = (50) (514) = 25,700 in.-kips
$M_v = V_p S_h$
S_h = Distance from column centerline to plastic hinge = $d_c/2 + d_b/2$ = 25.61 in.
V_p = Shear at plastic hinge location

Figure 37. Design Example 5.2 from FEMA 451

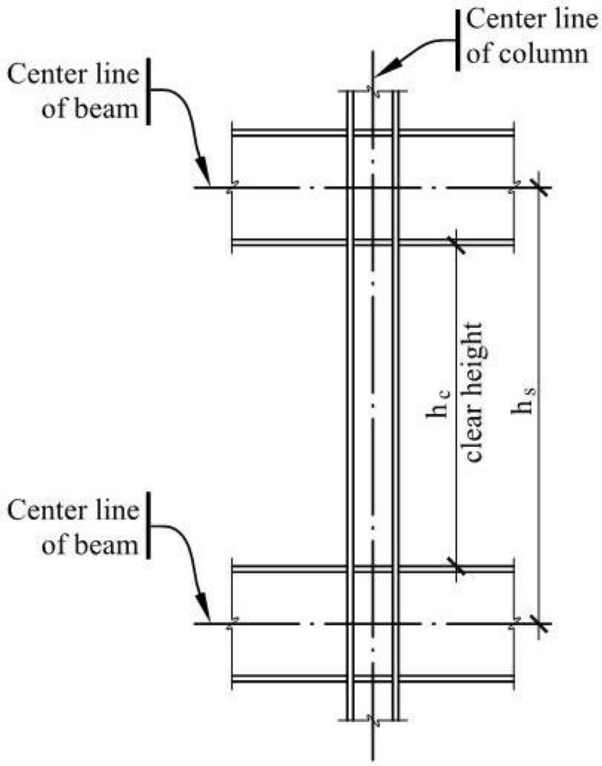


Figure 5.2-8 Story height and clear height.

The shear at the plastic hinge (Figure 5.2-9) is computed as:

$V_p = [2M_p + (wL'^{\,2}/2] / L'$

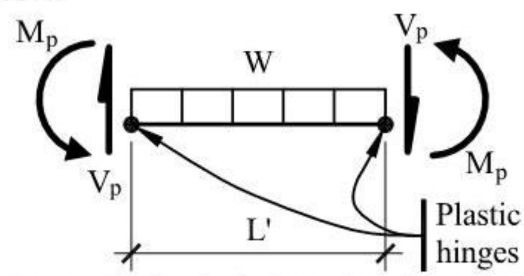


Figure 5.2-9 Free body diagram bounded by plastic hinges.

where

L' = Distance between plastic hinges = 248.8 in.
w = Factored uniform gravity load along beam
$w = 1.4D + 0.5L = 1.4(0.0625\text{ ksf})(12.5\text{ ft})$
$\quad + 0.5(0.050\text{ ksf})(12.5\text{ ft}) = 1.406\text{ klf}$

Therefore,

$$V_p = \frac{2M_p + \frac{wL'^{\,2}}{2}}{L'} = \frac{(2)(25{,}700) + \left(\frac{(1.406)}{12}\frac{(248.8)^2}{2}\right)}{248.8} = 221.2\text{ kips}$$

and

$M_v = V_p S_h = (221.2)(25.61) = 5{,}665$ in.-kips

Finally, $\Sigma M^*_{pb} = \Sigma(1.1R_yM_p + M_v) = 2[(1.1)(1.1)(25{,}700) + 5{,}665] = 73{,}500$ in.-kips.

The ratio of column moment strengths to beam moment strengths is computed as:

Figure 38. Design Example 5.2 from FEMA 451

$$\text{Ratio} = \frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} = \frac{76{,}900}{73{,}500} = 1.05 > 1.0 \qquad \text{OK}$$

The column-beam strength ratio for all the other stories is determined in a similar manner. They are summarized in Table 5.2-4 for the E-W direction (seven-bay) frame and in Table 5.2-5 for the N-S direction (five-bay) frame. All cases are acceptable because the column-beam moment ratios are all greater than 1.00.

Table 5.2-4 Column-Beam Moment Ratios for Seven-Bay Frame (N-S Direction)

Story	Member		ΣM^*_{pc} (in.-kips)	ΣM^*_{pb} (in.-kips)	Column-Beam Ratio
7	column beam	W14×145 W24×62	29,000	21,300	1.36
5	column beam	W14×233 W27×102	40,000	42,600	1.15
3	column beam	W14×257 W30×108	53,900	48,800	1.11
2	column beam	W14×370 W33×141	75,300	73,500	1.02

For levels with the same size column, the one with the larger beam size will govern; only these are shown. 1.0 in.-kip = 0.113 kN-m.

Table 5.2-5 Column-Beam Moment Ratios for Five-Bay Frame (N-S Direction)

Story	Member		ΣM^*_{pc} (in.-kips)	ΣM^*_{pb} (in.-kips)	Column-Beam Ratio
7	column beam	W14×145 W24×76	29,400	27,700	1.06
5	column beam	W14×233 W30×108	50,700	48,700	1.04
3	column beam	W14×283 W30×116	63,100	53,900	1.17
2	column beam	W14×398 W33×141	85,900	74,100	1.16

For levels with the same size column, the one with the larger beam size will govern; only these are shown. 1.0 in.-kip = 0.113 kN-m.

5. Check Panel Zone – The *Provisions* defers to AISC Seismic for the panel zone shear calculation. Because the two methods for calculating panel zone shear (AISC Seismic and FEMA 350) differ, both are illustrated below.

Figure 39. Design Example 5.2 from FEMA 451

AISC Seismic Method

Check the shear requirement at the panel zone in accordance with AISC Seismic Sec. 9.3. The factored shear R_u is determined from the flexural strength of the beams connected to the column. This depends on the style of connection. In its simplest form, the shear in the panel zone (R_u) is

$$R_u = \Sigma \frac{M_f}{d_b - t_{fb}}$$

M_f is the moment at the column face determined by projecting the expected moment at the plastic hinge points to the column faces (see Figure 5.2-10).

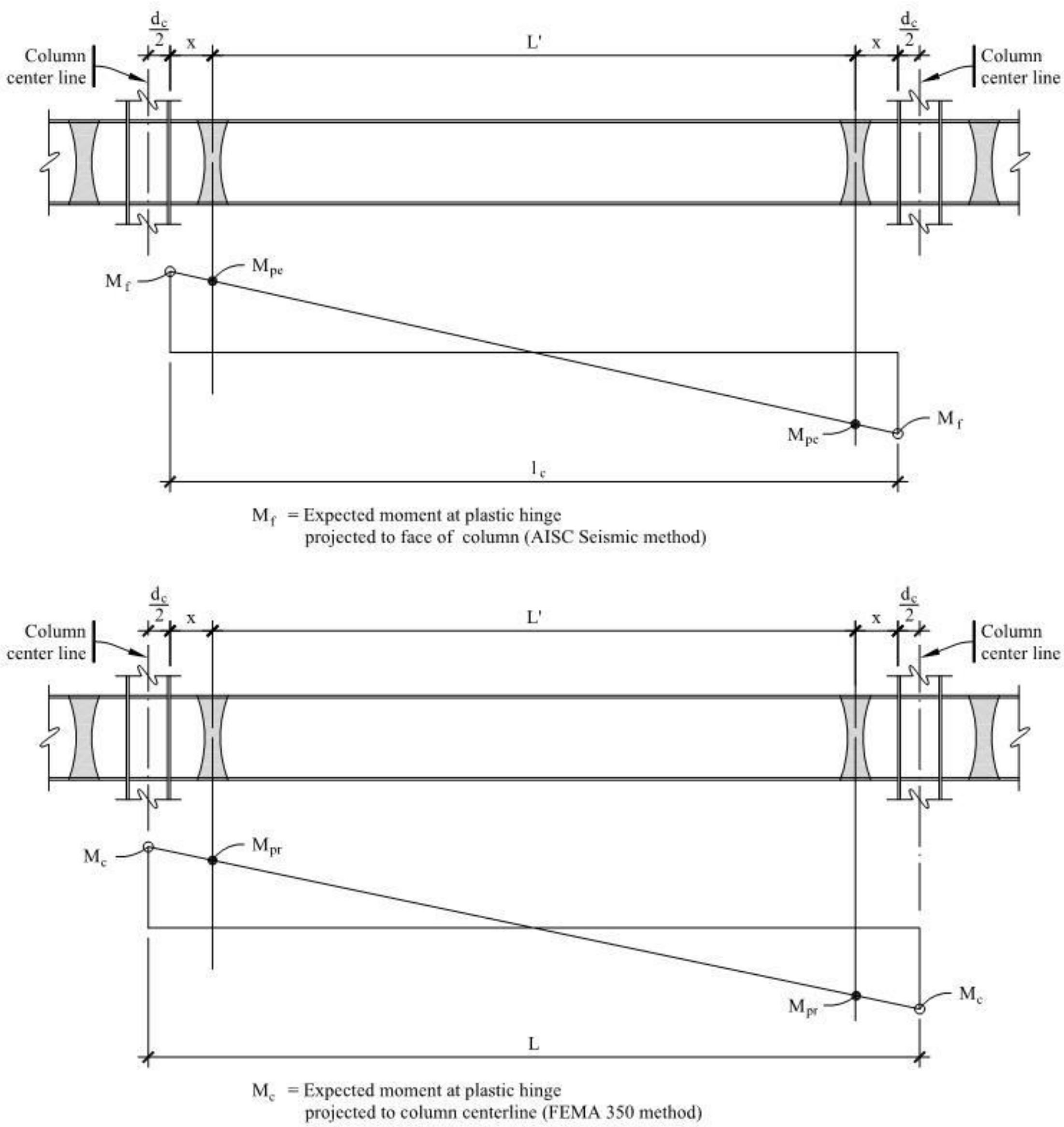


Figure 5.2-10 Illustration of AISC Seismic vs. FEMA 350 methods for panel zone shear.

Figure 40. Design Example 5.2 from FEMA 451

For a column with equal beams of equal spans framing into opposite faces (such as on Grids C, D, E, F, 2, 3, 4, and 5), the effect of gravity loads offset, and

$$\Sigma M_f = 2R_y F_y Z_x \left[\frac{l_c}{l_c - 2x}\right]$$

where l_c = the clear span and x = distance from column face to plastic hinge location.

For Grids 1 and 6, only one beam frames into the column; at Grids B and G, the distance x is different on one side; at Grids A and H, there is no moment because the beams are pin-connected to the corner columns. For all these cases, the above relationship needs to be modified accordingly.

For W33×141 beams framing into each side of a W14×370 column (such as Level 2 at Grid F):

$$\Sigma M_f = (2)(1.1)(50)(514)\left[\frac{282.1}{282.1-(2)(16.55)}\right] = 64{,}056 \text{ in.-kips}$$

$$R_u = \frac{64{,}056}{33.30 - 0.96} = 1{,}981 \text{ kips}$$

The shear transmitted to the joint from the story above opposes the direction of R_u and may be used to reduce the demand. From analysis, this is 98 kips at this location. Thus the frame R_u = 1,981 - 98 = 1,883 kips.

The panel zone shear calculation for Story 2 of the frame in the E-W direction at Grid F (column: W14×370; beam: W33×141) is from AISC Seismic Eq. 9-1:

$$R_v = 0.6F_y d_c t_p \left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right]$$

$$R_v = (0.6)(50)(17.92)(t_p)\left[1 + \frac{(3)(16.475)(2.660)^2}{(33.30)(17.92)(t_p)}\right]$$

$$R_v = 537.6t_p\left[1 + \frac{0.586}{t_p}\right]$$

$$R_v = 537.6t_p + 315$$

The required total (web plus doubler plate) thickness is determined by:

$R_v = \phi R_u$

Therefore, $537.6t_p + 315 = (1.0)(1883)$ and $t_p = 2.91$ in.

Because the column web thickness is 1.655 in., the required doubler plate thickness is 1.26 in. Use a plate thickness of 1-1/4 in.

Figure 41. Design Example 5.2 from FEMA 451

Both the column web thickness and the doubler plate thickness are checked for shear buckling during inelastic deformations by AISC Seismic Eq. 9-2. If necessary, the doubler plate may be plug-welded to the column web as indicated by AISC Seismic Commentary Figure C-9.2. For this case, the minimum individual thickness as limited by local buckling is:

$$t \geq (d_z + w_z)/90$$

$$t \geq \frac{(31.38+12.6)}{90} = 0.49 \text{ in.}$$

Because both the column web thickness and the doubler plate thicknesses are greater than 0.49 in., plug welding of the doubler plate to the column web is not necessary.

In the case of thick doubler plates, to avoid thick welds, two doubler plates (each of half the required thickness) may be used, one on each side of the column web. For such cases, buckling also must be checked using AISC Seismic Eq. 9-2 as doubler plate buckling would be a greater concern. Also, the detailing of connections that may be attached to the (thinner) doubler plate on the side of the weld needs to be carefully reviewed for secondary effects such as undesirable out-of-plane bending or prying.

FEMA 350 Method

For the FEMA 350 method, see FEMA 350 Sec. 3.3.3.2, "Panel Zone Strength," to determine the required total panel zone thickness (t):

$$t = \sum \frac{C_y M_c \left[\frac{h - d_b}{h}\right]}{(0.9)(0.6) F_{yc} R_{yc} d_c (d_b - t_{fb})}$$

(Please note the Σ; its omission from FEMA 350 Eq. 3-7 is an inadvertent typographical error.)

The term M_c refers to the expected beam moment projected to the centerline of the column; whereas AISC Seismic uses the expected beam moment projected to the face of the column flange. (This difference is illustrated in Figure 5.2-10.) The term $\left[\frac{h - d_b}{h}\right]$ is an adjustment similar to reducing R_u by the direct shear in the column, where h is the average story height. C_y is a factor that adjusts the force on the panel down to the level at which the beam begins to yield in flexure (see FEMA 350 Sec. 3.2.7) and is computed from FEMA 350 Eq. 3-4:

$$C_y = \frac{1}{C_{pr} \frac{Z_{be}}{S_b}}$$

C_{pr}, a factor accounting for the peak connection strength, includes the effects of strain hardening and local restraint, among others (see FEMA 350 Sec. 3.2.4) and is computed from FEMA 350 Eq. 3-2:

$$C_{pr} = \frac{(F_y + F_u)}{2F_y}$$

For the case of a W33×141 beam and W14×370 column (same as used for the above AISC Seismic method), values for the variables are:

Figure 42. Design Example 5.2 from FEMA 451

Distance from column centerline to plastic hinge, $S_h = d_c/2 + d_b/2 = 17.92/2 + 33.30/2 = 25.61$ in.

Span between plastic hinges, $L' = 25$ ft - 2(25.61 in.)/12 = 20.73 ft

$M_{pr} = C_{pr}R_yZ_eF_y$ (FEMA 350 Figure 3-4)
$M_{pr} = (1.2)(1.1)(514)(50) = 33{,}924$ in.-kips (FEMA 350, Figure 3-4)

$$V_p = \frac{\left[2M_{pr} + \left(\frac{wL'^2}{2}\right)\right]}{L'}$$

$$V_p = \frac{\left[(2)(33{,}924) + \left(\frac{(1.266)(20.73)^2}{(12)(2)}\right)\right]}{(20.73)(12)} = 273 \text{ kips}$$

$M_c = M_{pr} + V_p(x + d_c/2)$ (FEMA 350 Figure 3-4)

$M_c = 33{,}924 + (273)(17.92/2 + 25.61/2) = 40{,}916$ in.-kips

$$C_y = \frac{1}{C_{pr}\frac{Z_{be}}{S_b}} = \frac{1}{(1.2)\frac{514}{448}} = 0.73$$

Therefore,

$$t = 2\left[\frac{(0.73)(40{,}916)\left[\frac{(214)-(33.30)}{(214)}\right]}{(0.9)(0.6)(50)(1.1)(17.92)(33.30-0.96)}\right] = 2.93 \text{ in.}$$

The required doubler plate thickness is equal to $t - t_{cw} = 2.93$ in. - 1.655 in. = 1.27 in. Thus, the doubler plate thickness for 1.27 in. by FEMA 350 is close to the thickness of 1.26 by AISC Seismic.

6. Check Redundancy – Return to the calculation of r_x for the moment frame. In accordance with *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*], r_{max_x} is taken as the maximum of the sum of the shears in any two adjacent columns in the plane of a moment frame divided by the story shear. For columns common to two bays with moment resisting connections on opposite sides of the column at the level under consideration, 70 percent of the shear in that column may be used in the column shear summation (Figures 5.2-11 and 5.2-12).

Figure 43. Design Example 5.2 from FEMA 451

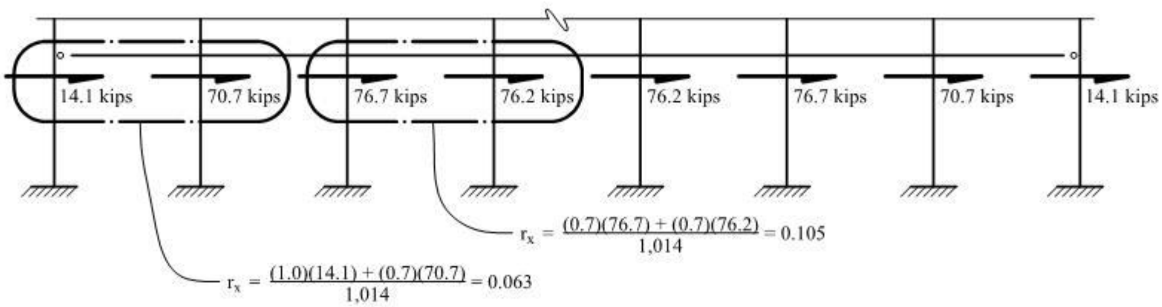


Figure 5.2-11 Column shears for E-W direction (partial elevation, Level 2) (1.0 kip = 4.45 kN).

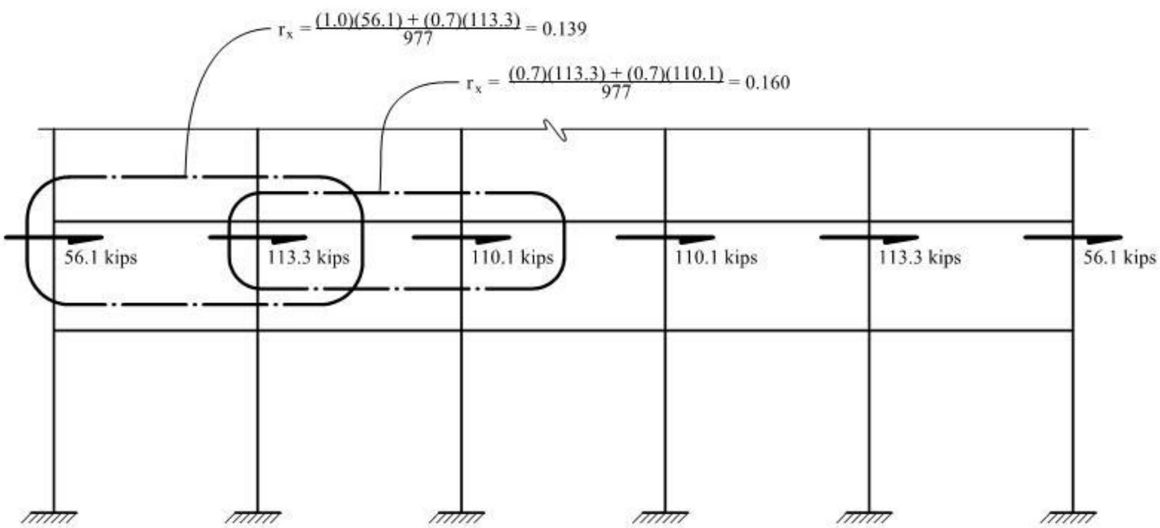


Figure 5.2-12 Column shears for N-S direction (partial elevation, Level 3) (1.0 kip = 4.45 kN).

For this example, r_x was computed for every column pair at every level in both directions. The shear carried by each column comes from the RAMFRAME analysis, which includes the effect of accidental torsion. Selected results are illustrated in the figures. The maximum value of r_{max_x} in the N-S direction is 0.160, and ρ is now determined using *Provisions* Eq.5.2.4.2 [not applicable in the 2003 *Provisions*]:

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{A_x}}$$

$$\rho = 2 - \frac{20}{0.160\sqrt{21{,}875\ \text{ft}^2}} = 1.15$$

Because 1.15 is less than the limit of 1.25 for special moment frames per the exception in the *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*], use $\rho = 1.15$. (If $\rho > 1.25$, then the framing would have to be reconfigured until $\rho < 1.25$.)

Figure 44. Design Example 5.2 from FEMA 451

In the E-W direction, $r_{max_x} = 0.105$ and $\rho = 0.71$, which is less than 1.00, so use $\rho = 1.00$. All design force effects (axial force, shear, moment) obtained from analysis must be increased by the ρ factors. (However, drift controls the design in this example. Drift and deflections are not subject to the ρ factor.)

7. Connection Design – One beam-to-column connection for the moment-resisting frame is now designed to illustrate the FEMA 350 method for a prequalified connection. The welded unreinforced flanges-welded web (WUF-W) connection is selected because it is prequalified for special moment frames with members of the size used in this example. FEMA 350 Sec. 3.5.2 notes that the WUF-W connection can perform reliably provided all the limitations are met and the quality assurance requirements are satisfied. While the discussion of the design procedure below considers design requirements, remember that the quality assurance requirements are a vital part of the total requirements and must be enforced.

   Figure 5.2-13 illustrates the forces at the beam-to-column connection.

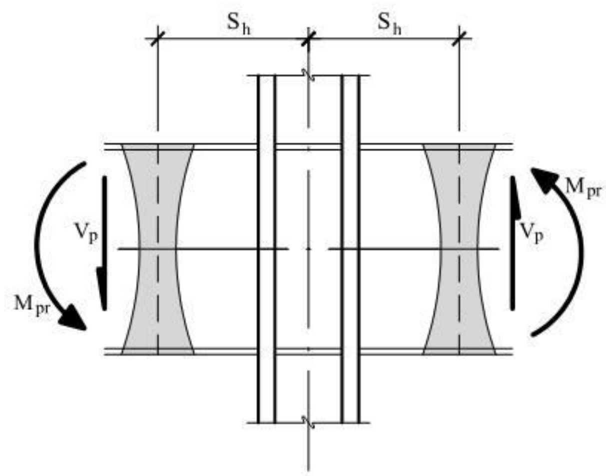


Figure 5.2-13 Forces at beam/column connection.

   First review FEMA 350 Table 3-3 for prequalification data. Our case of a W36×135 beam connected to a W14×398 column meets all of these. (Of course, here the panel zone strength requirement is from FEMA 350, not the AISC Seismic method.)

   The connection, shown in Figure 5.2-14, is based on the general design shown in FEMA 350 Figure 3-8. The design procedure outlined in FEMA 350 Sec. 3.5.2.1 for this application is reviewed below. All other beam-to-column connections in the moment frame will be similar.

   The procedure outlined above for the FEMA 350 method for panel zone shear is repeated here to determine S_h, M_{pr}, V_p, M_c, C_y and the required panel zone thickness.

   Continuity plates are required in accordance with FEMA 350 Sec. 3.3.3.1:

$$t_{cf} < 0.4\sqrt{1.8 b_f t_f \frac{F_{yb} R_{yb}}{F_{yc} R_{yc}}}$$

Figure 45. Design Example 5.2 from FEMA 451

$$t_{cf} < 0.4\sqrt{(1.8)(11.950)(0.790)\frac{(50)(1.1)}{(50)(1.1)}} = 1.65 \text{ in. required}$$

$t_{cf} = 2.845$ in. > actual OK

Therefore, continuity plates are not necessary at this connection because the column flange is so thick. But we will provide them anyway to illustrate continuity plates in the example. At a minimum, continuity plates should be at least as thick as the beam flanges. Provide continuity plates of 7/8 in. thickness, which is thicker than the beam flange of 0.79 in.

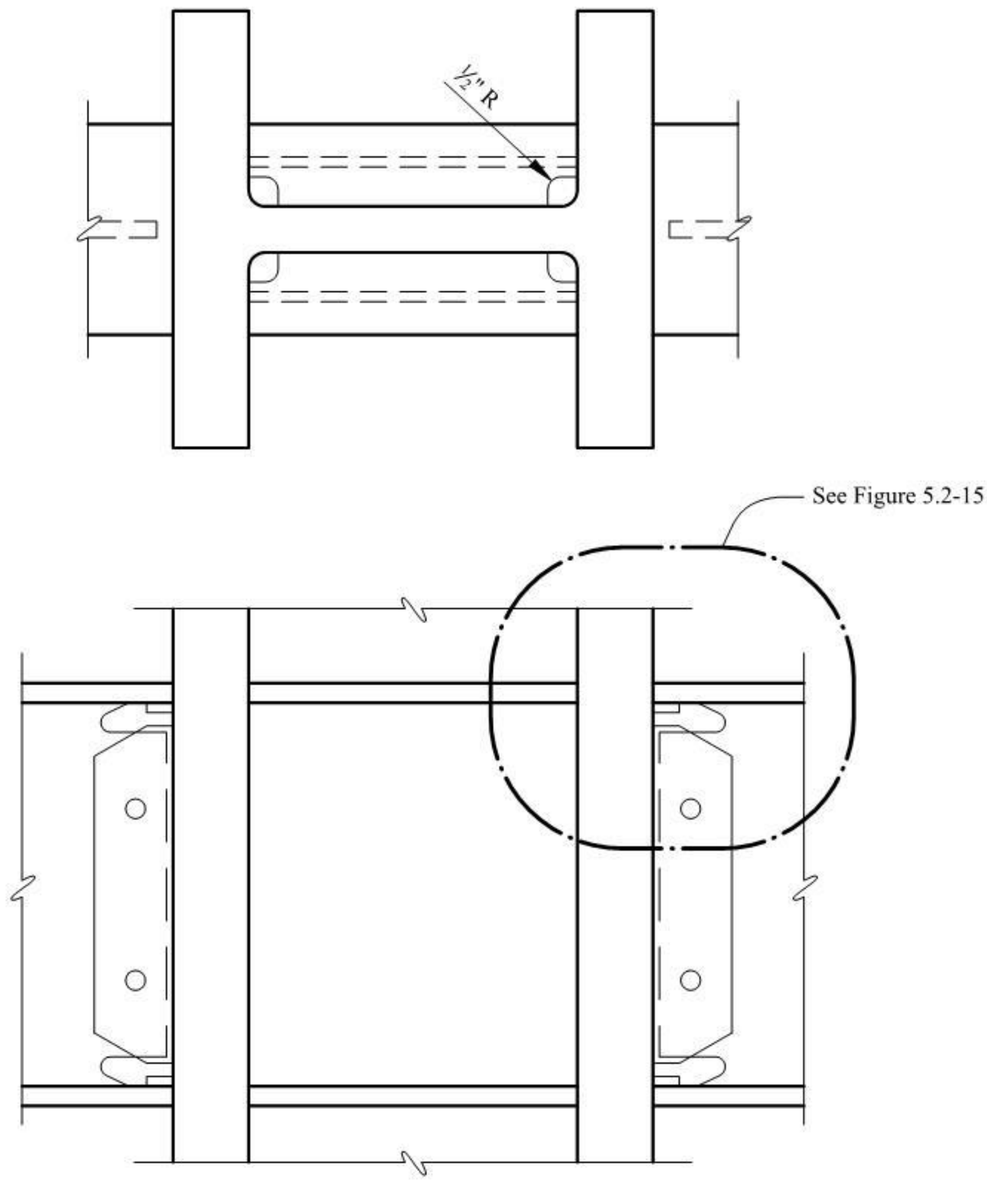


Figure 5.2-14 WUF-W connection, Second level, NS-direction (1.0 in. = 25.4 mm).

Figure 46. Design Example 5.2 from FEMA 451

Check AISC LRFD K1.9:

Width of stiffener $+\frac{t_{cw}}{2} \geq \frac{b_{bf}}{3}$

$$\left(5+\frac{1.77}{2}\right) = 5.88 \text{ in.} > 3.98 \text{ in.} = \frac{11.950}{3} \qquad \text{OK}$$

$$t_{stiffener} \geq \frac{b_f}{2}$$

$$0.875 \text{ in.} > 0.395 \text{ in.} = \frac{0.79}{2} \qquad \text{OK}$$

$$t_{stiffener} > w_{stiffener} \frac{\sqrt{F_y}}{95}$$

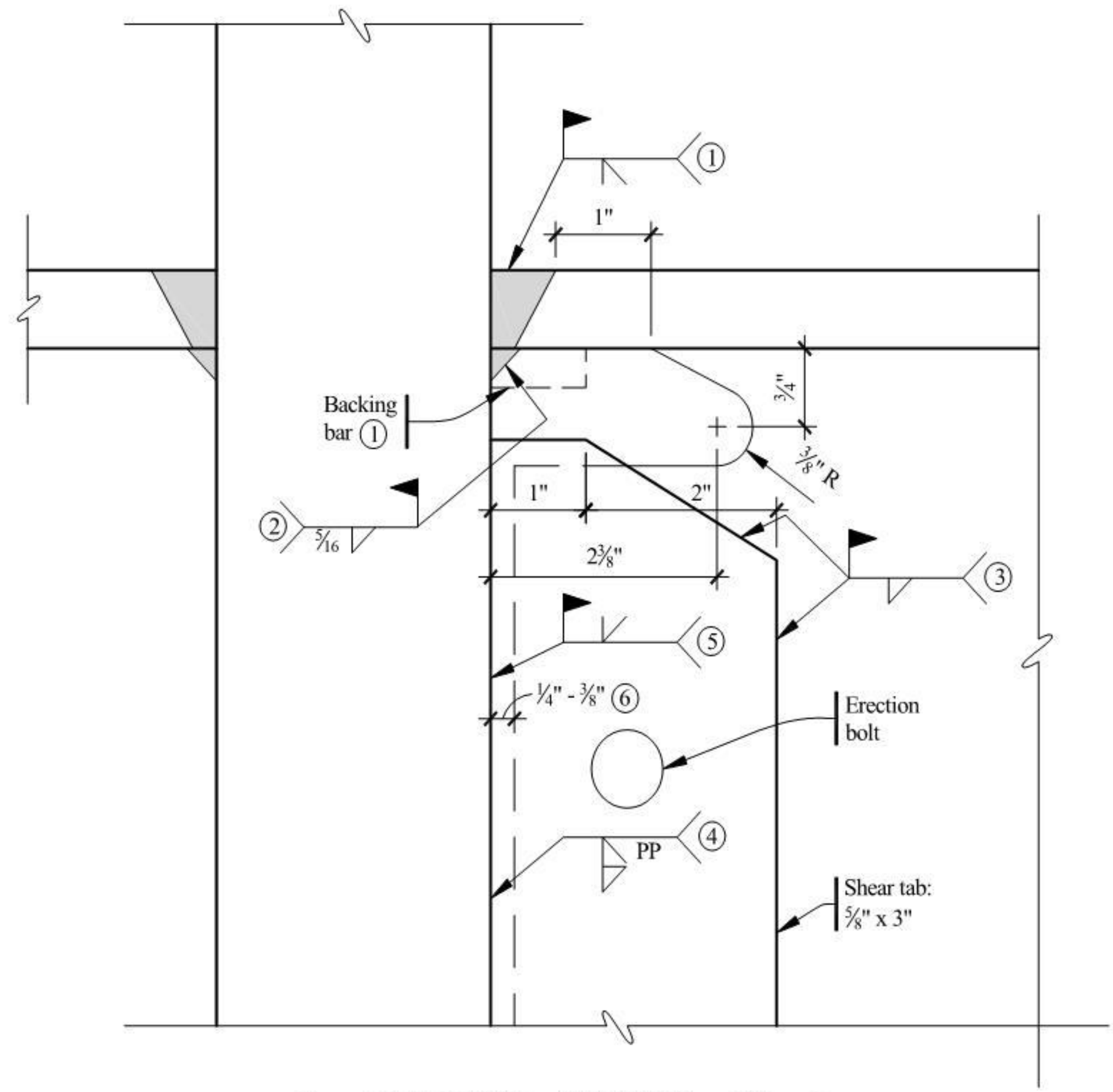


Figure 5.2-15 WUF-W weld detail (1.0 in. = 25.4 mm).

Figure 47. Design Example 5.2 from FEMA 451

$$0.875 \text{ in.} > 0.37 \text{ in.} = (5)\left(\frac{\sqrt{50}}{95}\right) \qquad \text{OK}$$

The details shown in Figures 5.2-14 and 5.2-15 conform to the requirements of FEMA 350 for a WUF-W connection in a special moment frame.

Notes for Figure 5.2-15 (indicated by circles in the figure) are:

1. CJP groove weld at top and bottom flanges, made with backing bar.
2. Remove backing bar, backgouge, and add fillet weld.
3. Fillet weld shear tab to beam web. Weld size shall be equal to thickness of shear tab minus 1/16 in. Weld shall extend over the top and bottom third of the shear tab height and extend across the top and bottom of the shear tab.
4. Full depth partial penetration weld from far side. Then fillet weld from near side. These are shop welds of shear tab to column.
5. CJP groove weld full length between weld access holes. Provide non-fusible weld tabs, which shall be removed after welding. Grind end of weld smooth at weld access holes.
6. Root opening between beam web and column prior to starting weld 5.

See also FEMA 350 Figure 3-8 for more elaboration on the welds.

5.2.4.3.2 Size Members for Alternative B, Braced Frame

1. Select Preliminary Member Sizes – The preliminary member sizes are shown for the braced frame in the E-W direction (seven bays) in Figure 5.2-16 and in the N-S direction (five bays) in Figure 5.2-17. The arrangement is dictated by architectural considerations regarding doorways into the stairwells.

2. Check Strength – First, check slenderness and width-to-thickness ratios – the geometrical requirements for local stability. In accordance with AISC Seismic Sec. 13.2, bracing members must satisfy

$$\frac{kl}{r} \le \frac{1000}{\sqrt{F_y}} = \frac{1000}{\sqrt{50}} = 141$$

The columns are all relatively heavy shapes, so *kl/r* is assumed to be acceptable and is not examined in this example.

Wide flange members and channels must comply with the width-to-thickness ratios contained in AISC Seismic Table I-9-1 [I-8-1]. Flanges must satisfy:

$$\frac{b}{2t} \le \frac{52}{\sqrt{F_y}} = \frac{52}{\sqrt{50}} = 7.35$$

Webs in combined flexural and axial compression (where $P_u/\phi_b P_y < 0.125$, which is the case in this example) must satisfy:

Figure 48. Design Example 5.2 from FEMA 451

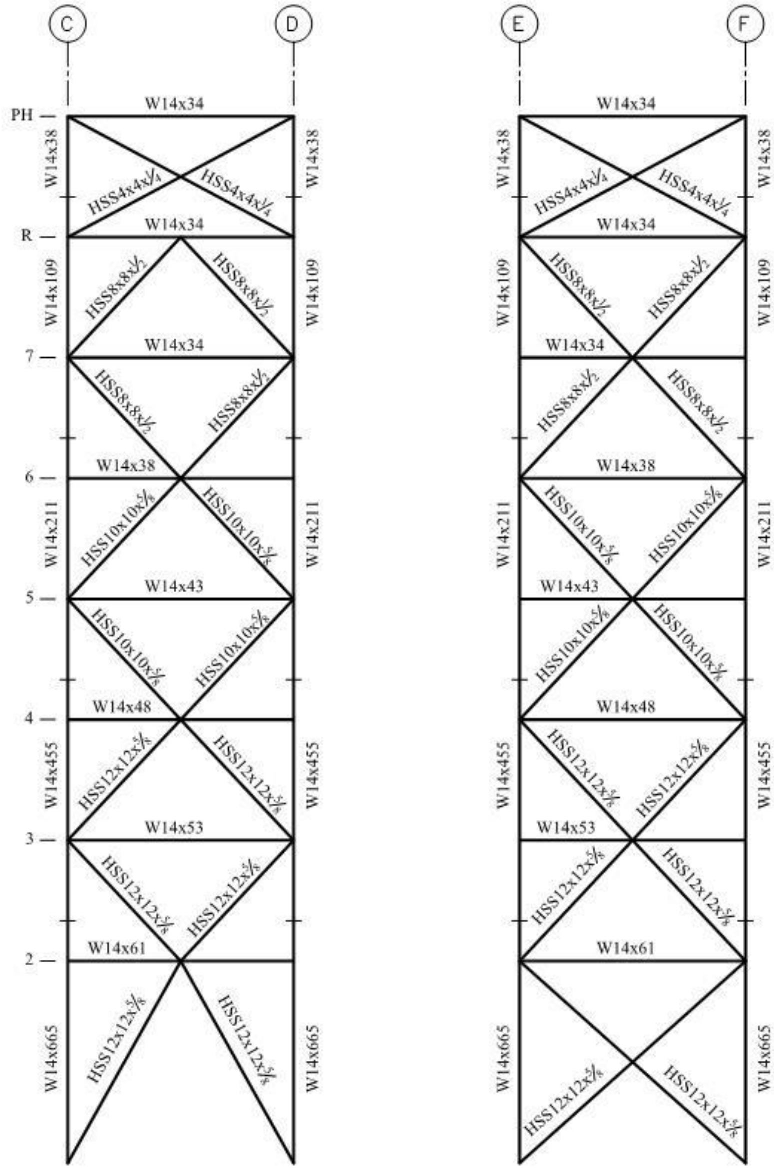


Figure 5.2-16 Braced frame in E-W direction.

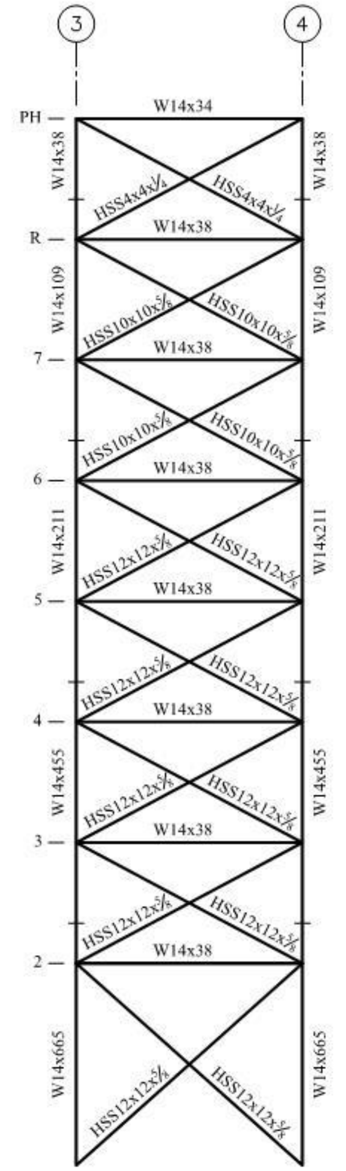


Figure 5.2-17 Braced frame in N-S direction.

$$\frac{h_c}{t_w} \leq \frac{520}{\sqrt{F_y}}\left[1 - 1.54\frac{P_u}{\phi_b P_y}\right]$$

Rectangular HSS members must satisfy:

$$\frac{b}{t} \leq \frac{110}{\sqrt{F_y}} = \frac{110}{\sqrt{46}} = 16.2$$

Figure 49. Design Example 5.2 from FEMA 451

Selected members are checked below:

W14×38: $b/2t = 6.6 < 7.35$ OK

W14×34: $b/2t = 7.4 > 7.35$, but is acceptable for this example. Note that the W14×34 is at the penthouse roof, which is barely significant for this braced frame.

HSS12×12×5/8: $$\frac{kl}{r} = \frac{(1)\left(\frac{28.33 \times 12}{2}\right)}{4.62} = 36.8 < 141 \quad \text{OK}$$

$$\frac{b}{t} = \frac{9.4}{0.581} = 16.17 < 16.2 \quad \text{OK}$$

Also note that t for the HSS is actual, not nominal. The corner radius of HSS varies somewhat, which affects the dimension b. The value of b used here, 9.40 in., depends on a corner radius slightly larger than $2t$, and it would have to be specified for this tube to meet the b/t limit.

3. Check Drift – Check drift in accordance with *Provisions* Sec. 5.2.8 [4.5]. The building was modeled in 3-D using RAMFRAME. Maximum displacements at the building corners are used here because the building is torsionally irregular. Displacements at the building centroid are also calculated because these will be the average between the maximum at one corner and the minimum at the diagonally opposite corner. Use of the displacements at the centroid as the average displacements is valid for a symmetrical building. Calculated story displacements are used to determine A_x, the torsional amplification factor. This is summarized in Table 5.2-6. P-delta effects are included.

Figure 50. Design Example 5.2 from FEMA 451

Table 5.2-6 Alternative B Amplification of Accidental Torsion

	Average Elastic Displacement = Displacement at Building Centroid (in.)		Maximum Elastic Displacement at Building Corner* (in.)		$\frac{\delta_{max}}{\delta_{avg}}$ **		Torsional Amplification Factor = $A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2$		Amplified Eccentricity = $A_x(0.05\ L)$*** (ft)	
	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S
R	2.38	2.08	3.03	3.37	1.28	1.62	1.13	1.82	7.08	15.95
7	2.04	1.79	2.62	2.93	1.29	1.64	1.15	1.88	7.20	16.41
6	1.65	1.47	2.15	2.44	1.30	1.67	1.18	1.93	7.37	16.86
5	1.30	1.16	1.70	1.96	1.32	1.69	1.2	1.99	7.52	17.41
4	0.95	0.86	1.27	1.48	1.33	1.72	1.23	2.06	7.71	17.99
3	0.66	0.59	0.89	1.03	1.34	1.75	1.25	2.14	7.80	18.70
2	0.39	0.34	0.53	0.60	1.35	1.79	1.26	2.23	7.89	19.57

* These values are taken directly from the analysis. Accidental torsion is not amplified here.
** Amplification of accidental torsion is required because this term is greater than 1.2 (*Provisions* Table 5.2.3.2 Item 1a [4.3-2, Item 1a]). The building is *torsionally irregular* in plan. *Provisions* Table 5.2.5.1 [4.4-1] indicates that an ELF analysis is "not permitted" for torsionally irregular structures. However, given rigid diaphragms and symmetry about both axes, a modal analysis will not give any difference in results than an ELF analysis insofar as accidental torsion is concerned unless one arbitrarily offsets the center of mass. The *Provisions* does not require an arbitrary offset for center of mass in dynamic analysis nor is it common practice to do so. One reason for this is that the computed period of vibration would lengthen, which, in turn, would reduce the overall seismic demand. See Sec. 9.2 and 9.3 of this volume of design examples for a more detailed examination of this issue.
*** The initial eccentricities of 0.05 in the E-W and N-S directions are multiplied by A_x to determine the amplified eccentricities. These will be used in the next round of analysis.
1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m.

4. Check Torsional Amplification – A second RAMFRAME 3-D analysis was made, using the amplified eccentricity for accidental torsion instead of merely 0.05L for accidental torsion. The results are summarized in Table 5.2-7.

Figure 51. Design Example 5.2 from FEMA 451

Table 5.2-7 Alternative B Story Drifts under Seismic Load

	Max. Elastic Displacement at Building Corners (in.)		Elastic Story Drift at Location of Max. Displacement (at corners) (in.)		C_d	$(C_d) \times$ (Elastic Story Drift) (in.)		Allowable Story Drift (in.)
	E-W	N-S	ΔE-W	ΔN-S		ΔE-W	ΔN-S	Δ
R	3.14	4.50	0.42	0.55	5	2.10	2.75	3.20
7	2.72	3.95	0.49	0.64	5	2.46	3.19	3.20
6	2.23	3.32	0.45	0.63	5	2.27	3.16	3.20
5	1.77	2.68	0.45	0.64	5	2.25	3.18	3.20
4	1.32	2.05	0.40	0.61	5	1.98	3.07	3.20
3	0.93	1.43	0.38	0.59	5	1.89	2.93	3.20
2	0.55	0.85	0.55	0.85	5	2.75	4.24	5.36

1.0 in. = 25.4 mm

All story drifts are within the allowable story drift limit of $0.020h_{sx}$ in accordance with *Provisions* Sec. 5.2.8 [4.5-1] and the allowable deflections for this building from Sec. 5.2.3.6 above. This a good point to reflect on the impact of accidental torsion and its amplification on the design of this core-braced structure. The sizes of members were increased substantially to bring the drift within the limits (note how close the N-S direction drifts are). For the final structure, the elastic displacements at the main roof are:

At the centroid	= 2.08 in.
At the corner with accidental torsion	= 3.37 in.
At the corner with amplified accidental torsion	= 4.50 in.

The two effects of torsional irregularity (in this case, it would more properly be called torsional flexibility) of amplifying the accidental torsion and checking the drift limits at the corners combine to create a demand for substantially more stiffness in the structure. Even though many braced frames are controlled by strength, this is an example of how the *Provisions* places significant stiffness demands on some braced structures.

5. Check Redundancy – Now return to the calculation of r_x for the braced frame. Per *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*], r_{max_x} for braced frames is taken as the lateral force component in the most heavily loaded brace element divided by the story shear (Figure 5.2-18).

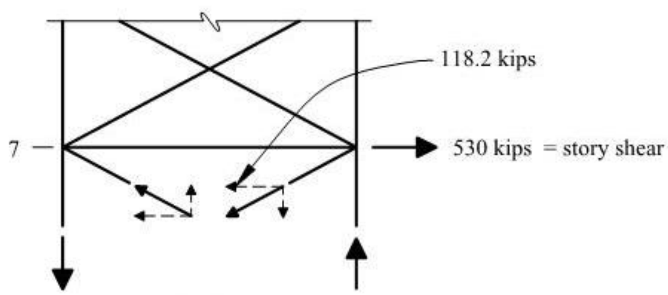
A value for r_x was determined for every brace element at every level in both directions. The lateral component carried by each brace element comes from the RAMFRAME analysis, which includes the effect of amplified accidental torsion. Selected results are illustrated in the figures. The maximum r_x was found to be 0.223 below Level 7 in the NS-direction. The reliability factor (ρ) is now determined using *Provisions* Eq. 5.2.4.2 [not applicable in the 2003 *Provisions*]:

Figure 52. Design Example 5.2 from FEMA 451

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{A_x}} = 2 - \frac{20}{0.223\sqrt{21{,}875\ \text{ft}^2}} = 1.39$$

In the N-S direction, all design force effects (axial forces, shears, moments) obtained from analysis must be increased by the ρ factor of 1.39. Similarly, for the E-W-direction, r_{max_x} and ρ are found to be 0.192 and 1.26, respectively. (However drift controls the design for this problem. Drift and deflection are not subject to the ρ factor.)

[See Sec. 5.2.3.2 for a discussion of the significant changes to the redundancy requirements in the 2003 *Provisions*.]



$$r_x = \frac{118.2}{530} = 0.223$$

Figure 5.2-18 Lateral force component in braces for N-S direction – partial elevation, Level 7 (1.0 kip = 4.45 kN).

6. Braced Frame Member Design Considerations – The design of members in the special concentrically braced frame (SCBF) needs to satisfy AISC Seismic Sec. 13 and columns also need to satisfy AISC Seismic Sec. 8. When $P_u/\phi P_n$ is greater than 0.4, as is the predominant case here, the required axial strength needs to be determined from AISC Seismic Eq. 4-1 and 4-2 [*Provisions* Eq. 4.2-3 and 4.2-4]. These equations are for load combinations that include the Ω_0, or overstrength, factors. Moments are generally small for the braced frame so load combinations with Ω_0 can control column size for strength considerations but, for this building, drift controls because of amplified accidental torsion. Note that ρ is not used where Ω_0 is used (see *Provisions* Sec. 5.2.7 [4.2.2.2]).

   Bracing members have special requirements as well, although Ω_0 factors do not apply to braces in a SCBF. Note in particular AISC Seismic Sec. 13.2c, which requires that both the compression brace and the tension brace share the force at each level (as opposed to the "tension only" braces of Example 5.1). AISC Seismic Sec. 13.2 also stipulates a *kl/r* limitation and local buckling (width-thickness) ratio limits.

   Beams in many configurations of braced frames have small moments and forces, which is the case here. V and inverted V (chevron) configurations are an exception to this. There is a panel of chevron bracing at the top story of one of the braced frames (Figure 5.2-16). The requirements of AISC Seismic Sec. 13.4 should be checked although, in this case, certain limitations of AISC Seismic do not apply because the beam is at the top story of a building. (The level above in Figure 5.2-16 is a minor penthouse that is not considered to be a story.) If the chevron bay were not at the top story, the size of the braces must be known in order to design the beam. The load combination for the beam is modified using a Q_b factor defined in AISC Seismic Sec. 13.4a. Basically, the beam must be able to

Figure 53. Design Example 5.2 from FEMA 451

carry a concentrated load equal to the difference in vertical force between the post-buckling strength of the compression brace and the yield strength of the tension brace (i.e., the compression brace has buckled, but the tension brace has not yet yielded). The prescribed load effect is to use $0.3\phi_c P_n$ for the compression brace and P_y for the tension brace in order to determine a design vertical force to be applied to the beam.

7. Connection Design – Figure 5.2-19 illustrates a typical connection design at a column per AISC Seismic Sec. 13. First, check the slenderness and width-to-thickness ratios (see above). The bracing members satisfy these checks.

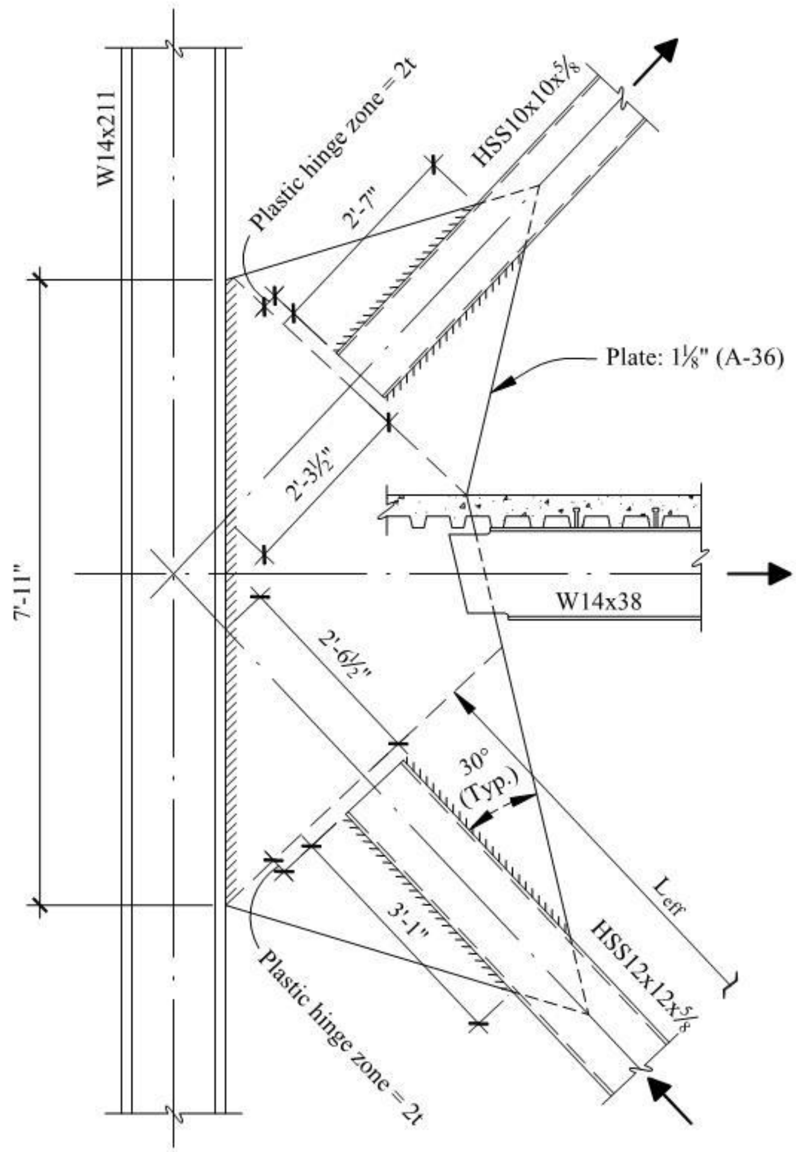


Figure 5.2-19 Bracing connection detail (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

Figure 54. Design Example 5.2 from FEMA 451

Next, design the connections. The required strength of the connection is to be the nominal axial tensile strength of the bracing member. For an HSS12×12×5/8, the nominal axial tensile strength is computed using AISC Seismic Sec. 13.3a:

$P_n = R_y F_y A_g = (1.3)(46 \text{ ksi})(27.4 \text{ in.}^2) = 1{,}639$ kips

The area of the gusset is determined using the plate thickness and the Whitmore section for effective width. See Figure 5.2-20 for the determination of this dimension.

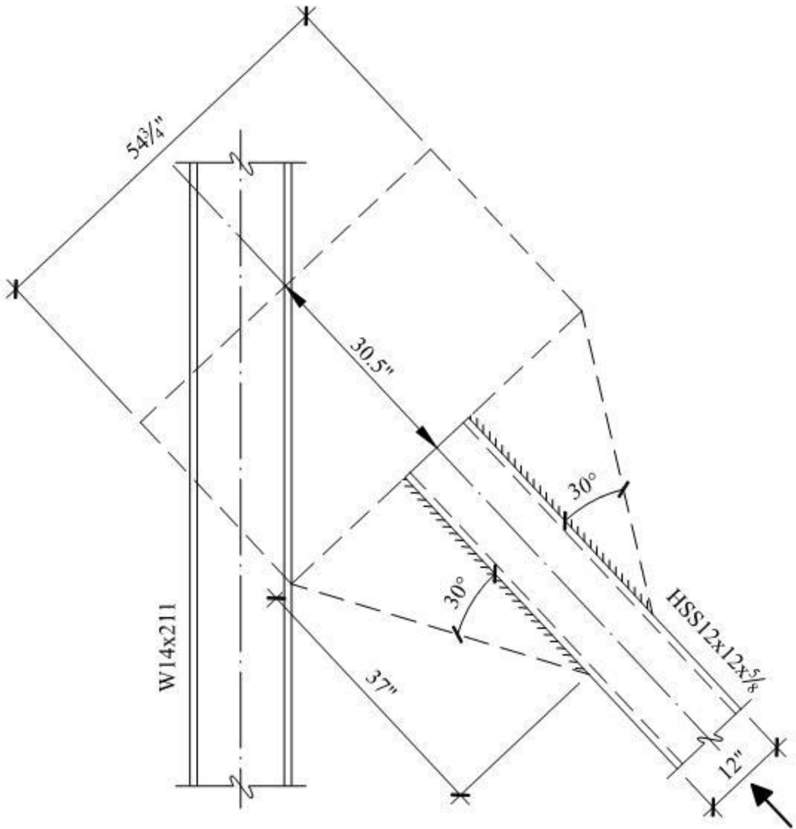


Figure 5.2-20 Whitmore section (1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m).

For tension yielding of the gusset plate:

$\phi T_n = \phi F_y A_g =$ (0.90)(36 ksi)(1.125 in. × 54.7 in.) = 1,993 kips > 1,639 kips OK

For fracture in the net section:

$\phi T_n = \phi F_u A_n =$ (0.75)(58 ksi)(1.125 in. × 54.7 in.) = 2,677 kips > 1,639 kips OK

Since 1,933 kips is less than 2,677 kips, yielding (ductile behavior) governs over fracture.

Figure 55. Design Example 5.2 from FEMA 451

For a tube slotted to fit over a connection plate, there will be four welds. The demand in each weld will be 1,639 kips/4 = 410 kips. The design strength for a fillet weld per AISC LRFD Table J2.5 is:

$\phi F_w = \phi(0.6F_{exx}) = (0.75)(0.6)(70 \text{ ksi}) = 31.5 \text{ ksi}$

For a 1/2 in. fillet weld, the required length of weld is determined to be:

$$L_w = \frac{410 \text{ kips}}{(0.707)(0.5 \text{ in.})(31.5 \text{ ksi})} = 37 \text{ in.}$$

In accordance with the exception of AISC Seismic Sec. 13.3c, the design of brace connections need not consider flexure if the connections meet the following criteria:

a. Inelastic rotation associated with brace post-buckling deformations: The gusset plate is detailed such that it can form a plastic hinge over a distance of $2t$ (where t = thickness of the gusset plate) from the end of the brace. The gusset plate must be permitted to flex about this hinge, unrestrained by any other structural member. See also AISC Seismic C13.3c. With such a plastic hinge, the compression brace may buckle out-of-plane when the tension braces are loaded. Remember that during the earthquake, there will be alternating cycles of compression to tension in a single bracing member and its connections. Proper detailing is imperative so that tears or fractures in the steel do not initiate during the cyclic loading.

b. The connection design strength must be at least equal to the nominal compressive strength of the brace.

Therefore, the connection will be designed in accordance with these criteria. First, determine the nominal compressive strength of the brace member. The effective brace length (L_{eff}) is the distance between the plastic hinges on the gusset plates at each end of the brace member. For the brace being considered, L_{eff} = 169 in. and the nominal compressive strength is determined using AISC LRFD Eq. E2-4:

$$\lambda_c = \frac{kl}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{(1)(169)}{(4.60)\pi}\sqrt{\frac{46}{29{,}000}} = 0.466$$

Since $\lambda_c < 1.5$, use AISC LRFD Eq. E2-2:

$$F_{cr} = (0.658^{\lambda_c^2})F_y = (0.658^{0.217})(46) = 42.0 \text{ ksi}$$

$P_{cr} = A_g F_{cr} = (27.4)(42.0) = 1{,}151$ kips

Now, using a design compressive load from the brace of 1,151 kips, determine the buckling capacity of the gusset plate using the Whitmore section method. By this method, illustrated by Figure 5.2-20, the compressive force per unit length of gusset plate is (1,151 kips/54.7 in.) = 21.04 kips/in.

Try a plate thickness of 1.125 in.

$f_a = P/A = 21.04 \text{ kips}/(1 \text{ in.} \times 1.25 \text{ in.}) = 18.7 \text{ ksi}$

Figure 56. Design Example 5.2 from FEMA 451

The gusset plate is modeled as a 1 in. wide by 1.125 in. deep rectangular section, pinned at one end (the plastic hinge) and fixed at the other end where welded to column (see Whitmore section diagram). The effective length factor (k) for this "column" is 0.8.

Per AISC LRFD Eq. E2-4:

$$\lambda_c = \frac{kl}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{(0.8)(30.5)}{(0.54)\pi}\sqrt{\frac{36}{29{,}000}} = 0.51$$

Since $\lambda_c < 1.5$, use AISC LRFD Eq. E2-2:

$$F_{cr} = (0.658^{\lambda_c^2})F_y = (0.658^{0.257})(36) = 32.3 \text{ ksi}$$

$\phi F_{cr} = (0.85)(32.3) = 27.4$ ksi

$\phi F_{cr} = 27.4 \text{ ksi} > 18.7$ ksi OK

Now consider the brace-to-brace connection shown in Figure 5.2-21. The gusset plate will experience the same tension force as the plate above, and the Whitmore section is the same. However, the compression length is much less, so a thinner plate may be adequate.

Try a 15/16 in. plate. Again, the effective width is shown in Figure 5.2-20. For tension yielding of the gusset plate:

$\phi T_n = \phi F_y A_g = (0.90)(36 \text{ ksi})(0.9375 \text{ in.} \times 54.7 \text{ in.}) = 1{,}662 \text{ kips} > 1{,}639$ kips OK

For fracture in the net section:

$\phi T_n = \phi F_y A_g = (0.75)(58 \text{ ksi})(0.9375 \text{ in.} \times 54.7 \text{ in.}) = 2{,}231 \text{ kips} > 1{,}639$ kips OK

Since 1,662 kips is less than 2,231 kips, yielding (ductile behavior) governs over fracture.

For compression loads, the plate must be detailed to develop a plastic hinge over a distance of $2t$ from the end of the brace. The effective length for buckling of this plate will be $k[12" + (2)(2t + \text{weld length})]$. For this case, the effective length is $0.65[12 + (2)(2 \times 15/16 + 5/16)] = 9.2$ in. Compression in the plate over this effective length is acceptable by inspection and will not be computed here.

Next, check the reduced section of the-tube, which has a 1 1/4 in. wide slot for the gusset plate (at the column). The reduction in HSS12×12×5/8 section due to the slot is $(0.581 \times 1.25 \times 2) = 1.45$ in.^{2}, and the net section, $A_{net} = (25.7 - 1.45) = 24.25$ in.^{2}

Compare yield in the gross section with fracture in the net section:

Yield = $F_y A_g$ =(46 ksi)(25.7 in.^{2}) = 1,182 kips OK

Fracture = $F_u A_n$ =(58 ksi)(24.25 in.^{2}) = 1,406 kips OK

AISC Seismic 13.3b could be used to require design fracture strength (0.75 x 1,406 = 1,055 kips) to exceed probable tensile yield (1,639 kips), but this is clearly impossible, even if the net area equaled the gross area. This design is considered satisfactory.

Figure 57. Design Example 5.2 from FEMA 451

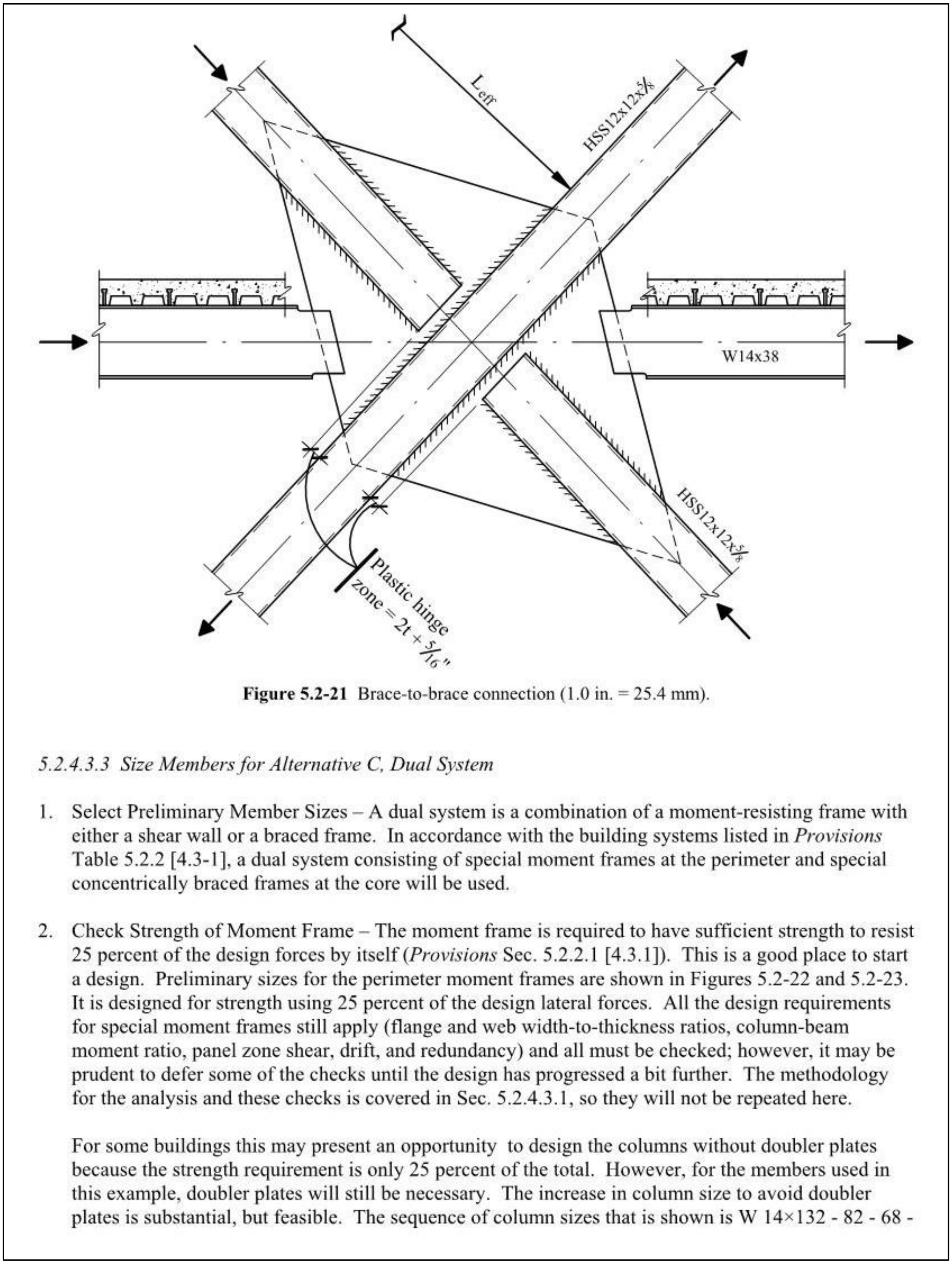


Figure 5.2-21 Brace-to-brace connection (1.0 in. = 25.4 mm).

5.2.4.3.3 Size Members for Alternative C, Dual System

1. Select Preliminary Member Sizes – A dual system is a combination of a moment-resisting frame with either a shear wall or a braced frame. In accordance with the building systems listed in *Provisions* Table 5.2.2 [4.3-1], a dual system consisting of special moment frames at the perimeter and special concentrically braced frames at the core will be used.

2. Check Strength of Moment Frame – The moment frame is required to have sufficient strength to resist 25 percent of the design forces by itself (*Provisions* Sec. 5.2.2.1 [4.3.1]). This is a good place to start a design. Preliminary sizes for the perimeter moment frames are shown in Figures 5.2-22 and 5.2-23. It is designed for strength using 25 percent of the design lateral forces. All the design requirements for special moment frames still apply (flange and web width-to-thickness ratios, column-beam moment ratio, panel zone shear, drift, and redundancy) and all must be checked; however, it may be prudent to defer some of the checks until the design has progressed a bit further. The methodology for the analysis and these checks is covered in Sec. 5.2.4.3.1, so they will not be repeated here.

   For some buildings this may present an opportunity to design the columns without doubler plates because the strength requirement is only 25 percent of the total. However, for the members used in this example, doubler plates will still be necessary. The increase in column size to avoid doubler plates is substantial, but feasible. The sequence of column sizes that is shown is W 14×132 - 82 - 68 -

Figure 58. Design Example 5.2 from FEMA 451

53 and would become W14×257 - 233 - 211 - 176 to avoid doubler plates. The beams in Figures 5.2-22 and 5.2-23 are controlled by strength because drift is not a criterion.

Note that $P_u/\phi P_n > 0.4$ for a few of the columns when analyzed without the braced frame so the overstrength requirements of AISC Seismic Sec. 8.2 [8.3] apply to these columns. Because the check using $\Omega_0 E$ is for axial capacity only and the moment frame columns are dominated by bending moment, the sizes are not controlled by the check using $\Omega_0 E$.

3. Check the Strength of the Braced Frames – The next step is to select the member sizes for the braced frame. Because of the presence of the moment frame, the accidental torsion on the building will be reduced as compared to a building with only a braced core. In combination with the larger R factor (smaller design forces), this should help to realize significant savings in the braced frame member sizes. A trial design is selected, followed by analysis of the entire dual system. All members need to be checked for width-thickness ratios and the braces need to be checked for slenderness. Note that columns in the braced frame will need to satisfy the overstrength requirements of AISC Seismic Sec. 8.2 [8.3] because $P_u/\phi P_n > 0.4$. This last requirement causes a significant increase in column sizes, except in the upper few stories.

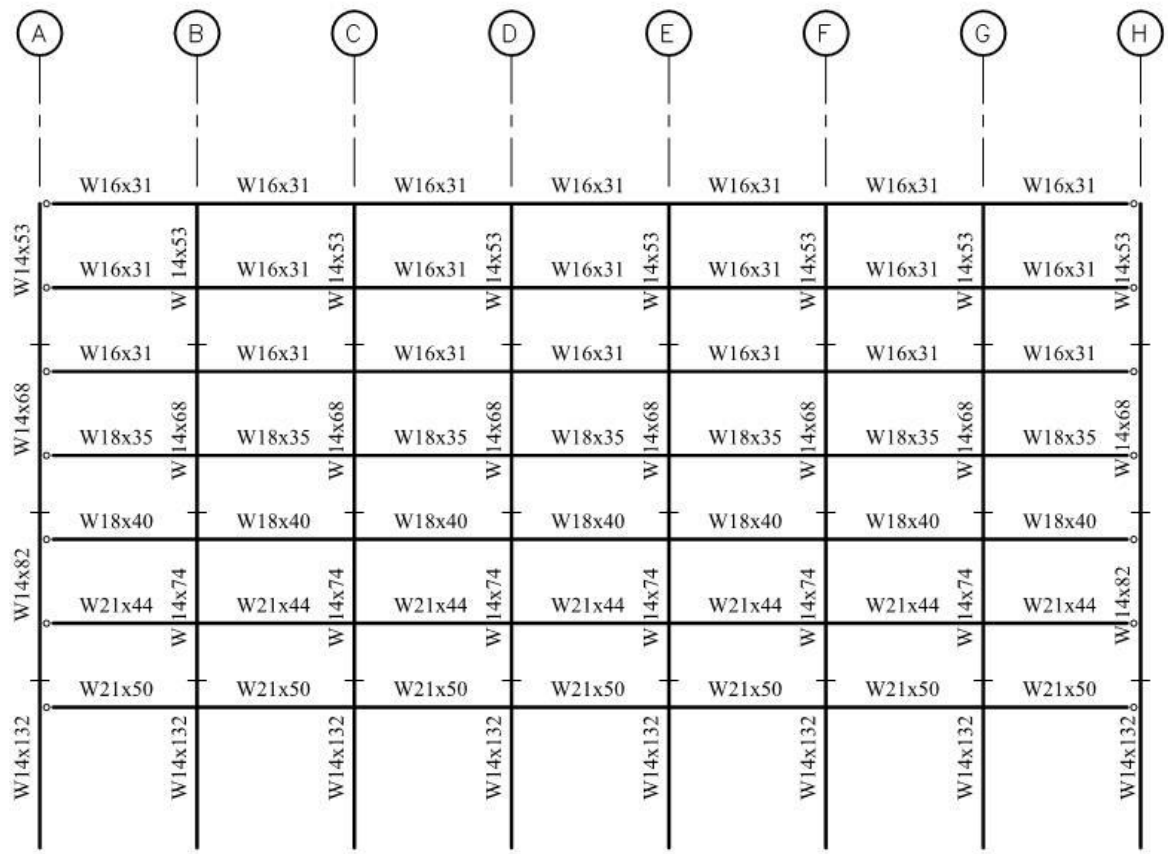


Figure 5.2-22 Moment frame of dual system in E-W direction.

Figure 59. Design Example 5.2 from FEMA 451

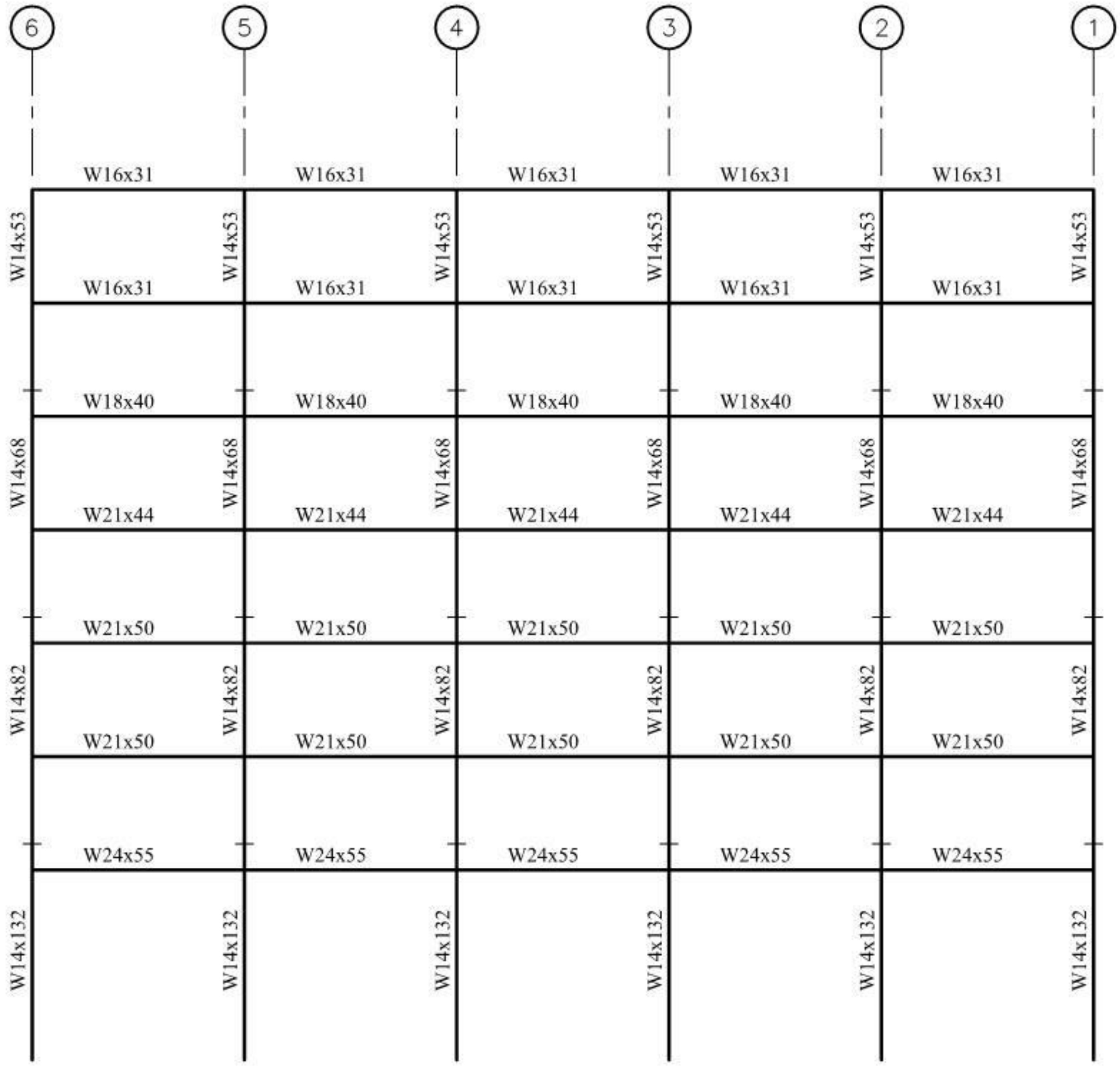


Figure 5.2-23 Moment frame of dual system in N-S direction.

4. Check Drift – Check drift in accordance with *Provisions* Sec. 5.2.8 [4.5]. The building was modeled in three dimensions using RAMFRAME. Maximum displacements at the building corners are used here because the building is torsionally irregular. Displacements at the building centroid are also calculated because these will be the average between the maximum at one corner and the minimum at the diagonally opposite corner. Use of the displacements at the centroid as the average displacements is valid for a symmetrical building.

5. Check Torsional Amplification – Calculated story drifts are used to determine A_x, the torsional amplification factor (Table 5.2-8). P-delta effects are included.

Figure 60. Design Example 5.2 from FEMA 451

Table 5.2-8 Alternative C Amplification of Accidental Torsion

	Average Elastic Displacement = Displacement at Building Centroid (in.)		Maximum Elastic Displacement at Building Corner* (in.)		$\frac{\delta \max}{\delta \text{avg}}$ **		Torsional Amplification Factor = $A_x = \left(\frac{\delta_{\max}}{1.2\delta_{\text{avg}}}\right)^2$		Amplified Eccentricity = $A_x(0.05\ L)$*** (ft.)	
	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S
R	2.77	2.69	3.57	3.37	1.29	1.25	1.15	1.09	7.19	9.54
7	2.45	2.34	3.15	3.00	1.28	1.28	1.14	1.14	7.14	10.01
6	2.05	1.91	2.63	2.50	1.28	1.31	1.13	1.20	7.07	10.46
5	1.64	1.51	2.10	2.01	1.28	1.33	1.13	1.23	7.08	10.8
4	1.22	1.11	1.56	1.50	1.28	1.35	1.14	1.27	7.13	11.15
3	0.81	0.75	1.05	1.03	1.29	1.38	1.16	1.31	7.25	11.50
2	0.43	0.41	0.57	0.57	1.32	1.40	1.20	1.37	7.52	11.98

* These values are directly from the analysis. Accidental torsion is not amplified here.

** Amplification of accidental torsion is required because this term is greater than 1.2 (*Provisions* Table 5.2.3.2, Item 1a [4.3-2, Item 1a). The building is *torsionally irregular* in plan. See discussion in footnote ** of Table 5.2.6.

*** The initial eccentricities of $0.05L$ in the E-W and N-S directions are multiplied by A_x to determine the amplified eccentricities. These will be used in the next round of analysis.

1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m.

The design that yielded the displacements shown in Table 5.2-8 does not quite satisfy the drift limits, even without amplifying the accidental torsion. That design was revised by increasing various brace and column sizes and then re-analyzing using the amplified eccentricity instead of merely $0.05L$ for accidental torsion. After a few iterations, a design that satisfied the drift limits was achieved. These member sizes are shown in Figures 5.2-24 and 5.2-25. That structure was then checked for its response using the standard $0.05L$ accidental eccentricity in order to validate the amplifiers used in design. The amplifier increased for the E-W direction but decreased for the N-S direction, which was the controlling direction for torsion. The results are summarized in Table 5.2-9.

Figure 61. Design Example 5.2 from FEMA 451

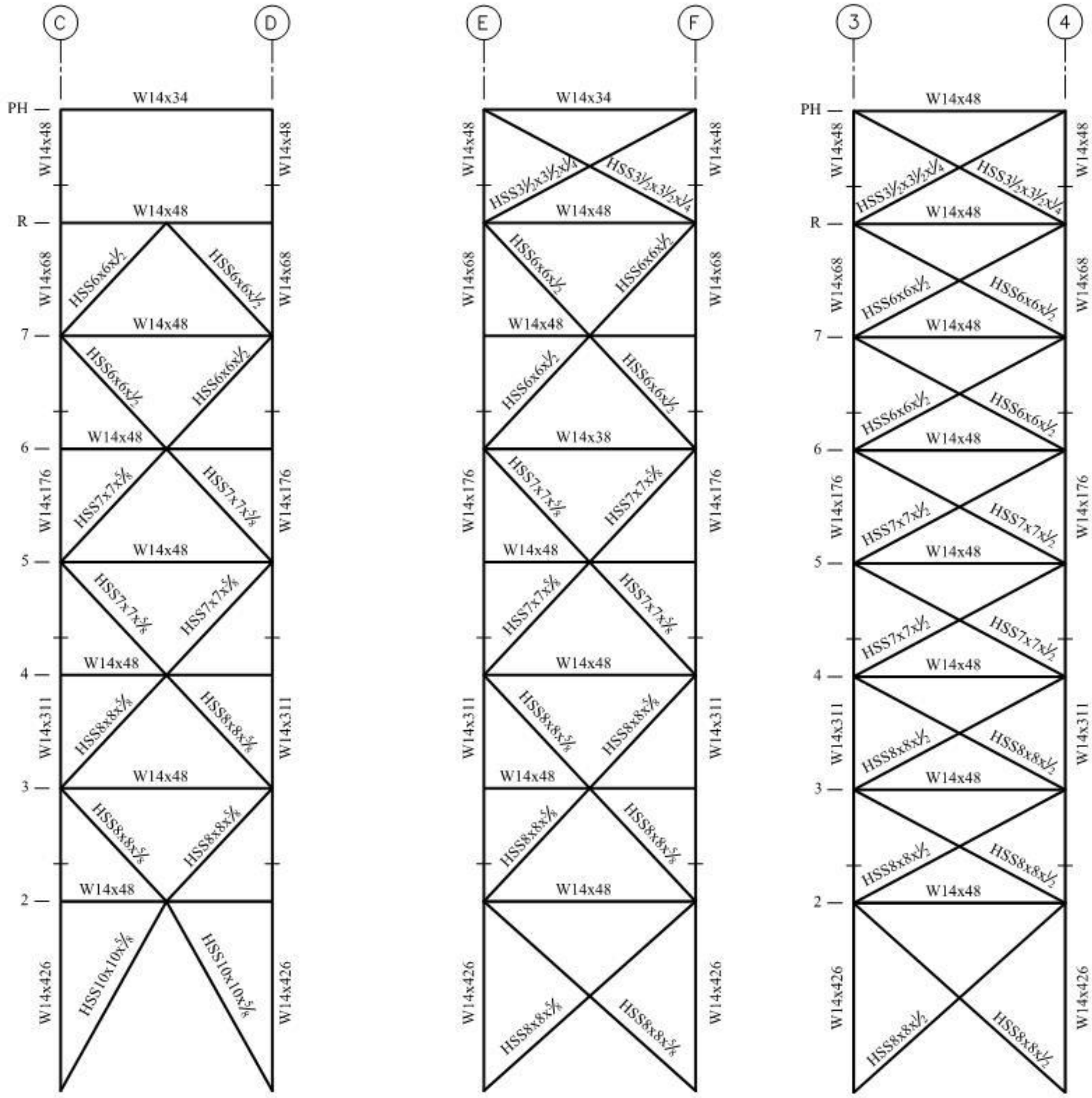


Figure 5.2-24 Braced frame of dual system in E-W-direction.

Figure 5.2-25 Braced frame of dual system in N-S direction.

Figure 62. Design Example 5.2 from FEMA 451

Table 5.2-9 Alternative C Story Drifts under Seismic Load

	Max. Elastic Displacement at Building Corners (in.)		Elastic Story Drift at Location of Max. Displacement (at corners) (in.)		C_d	(C_d) x (Elastic Story Drift) (in.)		Allowable Story Drift (in.)
	E-W	N-S	ΔE-W	ΔN-S		ΔE-W	ΔN-S	Δ
R	3.06	3.42	0.37	0.37	6.5	2.43	2.42	3.20
7	2.69	3.05	0.45	0.47	6.5	2.94	3.05	3.20
6	2.24	2.58	0.45	0.49	6.5	2.89	3.17	3.20
5	1.79	2.09	0.45	0.51	6.5	2.93	3.30	3.20
4	1.34	1.58	0.41	0.48	6.5	2.66	3.09	3.20
3	0.93	1.11	0.39	0.46	6.5	2.55	3.01	3.20
2	0.54	0.64	0.54	0.64	6.5	3.52	4.17	5.36

1.0 in. = 25.4 mm

The story drifts are within the allowable story drift limit of $0.020h_{sx}$ as per *Provisions* Sec. 5.2.8 [4.5.1]. Level 5 has a drift of 3.30 in. > 3.20 in. but the difference of only 0.1 in. is considered close enough for this example.

6. Check Redundancy – Now return to the calculation of r_x for the braced frame. In accordance with *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*], $r_{\max_x}$ for braced frames is taken as the lateral force component in the most heavily loaded brace element divided by the story shear. This is illustrated in Figure 5.2-18 for Alternative B.

For this design, r_x was determined for every brace element at every level in both directions. The lateral component carried by each brace element comes from the RAMFRAME analysis, which includes the effect of amplified accidental torsion. The maximum value was found to be 0.1762 at the base level in the N-S direction. Thus, ρ is now determined to be (see Sec. 5.2.4.2):

$$\rho = 0.8\left[2 - \frac{20}{r_{\max_x}\sqrt{A_x}}\right] = 0.8\left[2 - \frac{20}{0.1762\sqrt{21{,}875\ \text{ft.}^2}}\right] = 0.986$$

The 0.8 factor comes from *Provisions* Sec. 5.2.4.2 [not applicable in the 2003 *Provisions*]. As ρ is less than 1.0, $\rho = 1.0$ for this example.

In the E-W direction, r_{max} is less; therefore, ρ will be less, so use $\rho = 1.0$ for both directions.

[See Sec. 5.2.3.2 for a discussion of the significant changes to the redundancy requirements in the 2003 *Provisions.*]

Figure 63. Design Example 5.2 from FEMA 451

7. Connection Design – Connections for both the moment frame and braced frames may be designed in accordance with the methods illustrated in Sec. 5.2.4.3.1 and 5.2.4.3.2.

5.2.5 Cost Comparison

Material takeoffs were made for the three alternatives. In each case, the total structural steel was estimated. The takeoffs are based on all members, but do not include an allowance for plates and bolts at connections. The result of the material takeoffs are:

Alternative A, Special Steel Moment Resisting Frame	593 tons
Alternative B, Special Steel Concentrically Braced Frame	640 tons
Alternative C, Dual System	668 tons

The higher weight of the systems with bracing is primarily due to the placement of the bracing in the core, where resistance to torsion is poor. Torsional amplification and drift limitations both increased the weight of steel in the bracing. The weight of the moment-resisting frame is controlled by drift and the strong column rule.

Figure 64. Design Example 5.2 from FEMA 451

Appendix B - Permission for Use

TM Tong, Mai <Mai.Tong@fema.dhs.gov>

10:15 AM

To: Sidney Kiser Cc: Laatsch, Edward

Mr. Kiser:

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Sincerely,

Mai (Mike) Tong, Physical Scientist
Federal Emergency Management Agency
National Earthquake Hazards Reduction Program (NEHRP)
Building Science Branch, Risk Management Directorate / FIMA
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Figure 65. Permission for use