Senior Thesis Report

LIFE SCIENCES BUILDING

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I can do all things through Christ who strengthens me.
Philippians 4:13

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LIFE SCIENCES BUILDING

EAST COAST, USA

wangjae
you
structural

Perspective View from South-West Photo Credit | Ryan-Biggs Associates, P.C.

General Information

Full Height: 91 ft
Number of Stories: 5 stories
Size: 174,500 square-foot
Cost: $91.6 million
Date of Construction: September 2008 - August 2011
Project Delivery Method: Design-bid-build

Project Team

Owner:	Education Institutes
Architect:	Bohlin Cywinski Jackson
Construction	Bond Brothers, Inc.
Structural:	Ryan-Biggs Associates, P.C.
MEP/Lighting:	vanZelm Heywood & Shadford, Inc.
Sustainability:	Atelier Ten
Project Sponsor:	**Ryan-Biggs Associates, P.C.**

Architecture

The main concept of design in floor plan is to promote the interaction of idea and technique between researchers using the building. To place laboratories in the first floor provides easy accessibility to whom uses this building.

Sustainability Features

This building is certified as a LEED Platinum. Greenhouse on the roof improves building performance in energy throughout the year.

Structural Systems

Foundation: Cast-in-place concrete spread and strip footings

Framing: Structural Steel Frame with composite concrete slabs on metal deck

Lateral: Structural Steel Braced Frames

Mechanical

Hight-Performance Enthalpy Heat-Recovery Wheels
Dedicated Outdoor Air System
Chilled Beams throughout all laboratories
Active Air Quality Monitoring for Airflow Reset
Condenser Water Domestic Hot Water Heating

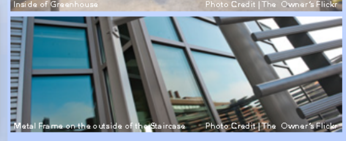
Lighting/Electrical

Two 480/277 3-Phase, 4 wire switchboards
Daylight Dimming throughout the building
High Level of Lighting Control with Occupancy Sensor

Perspective View From North-East Photo Credit | Galaxy Glass & Aluminum, Inc

Corridors Photo Credit | Bond Brothers, Inc

Inside of Greenhouse Photo Credit | The Owner's Flickr



Metal Frame on the outside of the Staircase Photo Credit | The Owner's Flickr

Mechanical Room Photo Credit | The Owner's Flickr

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EXECUTIVE SUMMARY

The Life Sciences Building is located in north east United States. The building is a five stories and 174,500 square feet. The geometry of building is L-shaped and considered a long-span structure. A greenhouse is located on the roof to serve as a research space. The foundation system consists of cast-in-place concrete spread and strip footings that support a system of wide flange steel columns. The building is designed as a composite steel floor system. The lateral system is designed as a structural steel braced frames, not seismically detailed. Hollow structural section steel (HSS) is used as braces with varying thicknesses based on the lateral loads resisting the members.

The existing structural system of the Life Sciences Building is adequate to meet both strength and serviceability requirements. Therefore, a scenario has been proposed that in which a college campus, which resides in a high seismic area, specifically in San Francisco, California, requests the design and construction of a building identical to the Life Sciences Building. San Francisco, California is classified as seismic design category D.

The structural depth consists of the redesigns of two different lateral force resisting systems: eccentrically braced frames and special moment frames. ETABS 2013 is used to design and analyze the proposed systems. To reduce the effective building weight, normal weight concrete slab is changed to lightweight concrete slab on the composite deck.

Two breadth topics are investigated: building enclosure breadth and construction breadth. In order to suggest an adequate lateral system to the owner, the cost estimate and the construction schedule will be compared between suggested lateral systems. Since the building has been relocated to San Francisco, CA, the building envelope will be reassessed to the new environment and redesigned as well. Compared to the climate in the existing location, climate in San Francisco, CA is less fluctuating and remained between 50 to 70F. The building envelope, especially wall assembly details is evaluated with WUFI 5. This analysis provides the presence of water condensation between wall assembly section.

Both eccentrically braced frames and special moment frames provide distinctive difference. Eccentrically braced frames would provide better performance over the moment frames. On the other hands, special moment frames would allow architectural freedom in designing. After investigation, the owner would choose the final design of lateral force resisting system based on the performance, architectural freedom, and constructibility.

CHAPTER 1 - BUILDING INTRODUCTION

1.1 BUILDING AND SITE OVERVIEW

The Life Sciences building is a five story laboratory building, 91 feet tall and 174,500 square feet. It is located in a college town in northeast, the United States. It was constructed between September 2008 and August 2011. The total project cost was $91.6 million, and its structural system costs $20 million. The project team's main goal was to create a building that is both aesthetically pleasing and high-functional.

Figure 1 | Building Perspective from North

The building accommodates a 4,000 square feet nuclear magnetic resonance suite, eight classroom laboratories, a 200 seat auditorium, two 80 seat and two 30 seat classrooms, and 30 teaching and research laboratories with the offices. The building is divided in to three sections: west, north, and east. Each section is clearly distinguished by its own functions. A 200 seat auditorium is placed in west side. Greenhouse and most laboratories are placed in north side. The offices and laboratories are located on the East side.

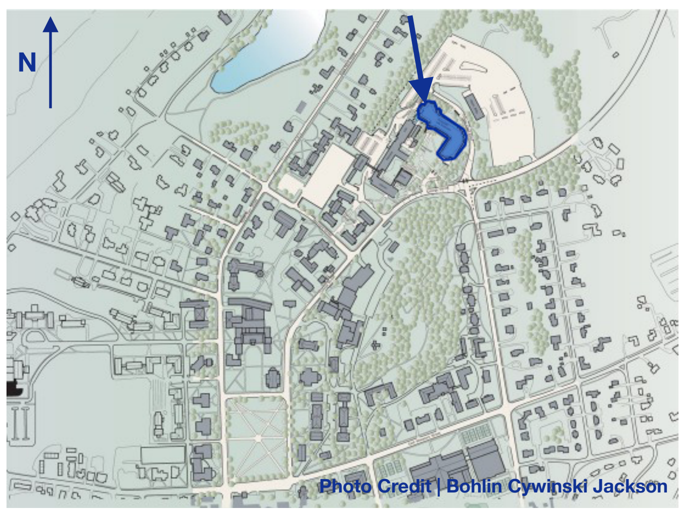


Figure 2 | Buildings Site Plan

The main concept of design in the floor plan was to create the space promoting the interaction of ideas and techniques between people using this building. Laboratories are placed in the first floor to provide better accessibility to whom uses the facilities. One of the unique feature of the project is to place greenhouse on the roof top. The greenhouse could improve building performance in energy usage in both summer and winter. However, in order to place greenhouse on the roof top, the structural engineer will have to design the roof to resist heavier loads.

With great effort and teamwork between project teams, the project was completed on schedule and within the project budget when faculty and researchers moved in on August 2011. This project was awarded a Leadership of Energy and Environmental Design (LEED) Platinum and has been considered as a national model of sustainable design for laboratories buildings.

1.2 STRUCTURAL OVERVIEW

STRUCTURAL SYSTEM SUMMARY

The Life Sciences Building is a structural steel frame with composite concrete slabs on metal deck. These structural frames are supported by cast-in-place concrete footings. Due to the activities in the laboratory, floor vibrations were strictly limited where vibration sensitive equipment was placed. Cast-in-place reinforced concrete framing was used for this building since the rigidity and mass of the concrete framing naturally limits floor vibrations. In the greenhouse on the roof, a separate concrete topping slab is placed over the structural concrete floor slab at the floor.

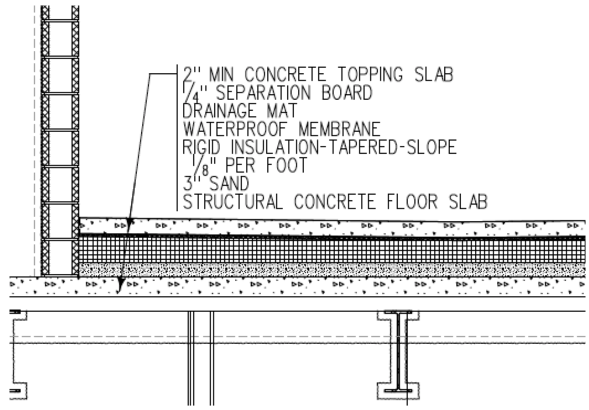


Figure 3 | Greenhouse Section | 1/A4.20

Structural steel may provide the benefits of a shorter erection time in construction schedule, especially during harsh winter weather which is common where the project is located.

Structural steel braced frames are used to resist lateral loads such as wind and seismic loads and are compliant to the International Building Code 2006 edition. Braced frames are used over moment frames due to its economy, and the location and configuration of the braced frame are determined carefully without any interference of the architectural and mechanical systems. The design of laboratory buildings typically requires better performance in mechanical, electrical, and plumbing system. Especially in the project, the layout of structural elements is important.

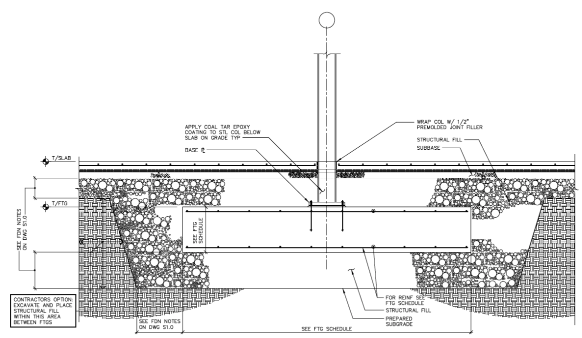


Figure 4 | Section of Typical Interior Footing | 4/S3.02

FOUNDATION SYSTEM

According to the geotechnical report prepared from Haley & Aldrich, Inc., foundation design and construction must conform to the applicable provisions of the International Building Code 2000 (IBC 2000).

The design recommends that, "Building walls and columns and other structural elements be supported on reinforced concrete spread or strip footings bearing directly on a minimum of 2 ft thickness of compacted structural fill placed above the glaciolacustrine silt deposits." The report also recommends that footings should have a least lateral dimension of 24 in or greater.

According to the geotechnical report, presumptive net soil bearing pressure = 2,500 psi on minimum 2-foot thick compacted structural fill. Concrete slab on grade varies on the rage from 5" to 1'-6" thick depend on the soil properties on geotechnical report.

BUILDING MATERIALS

Structural and Miscellaneous Steel	
Rolled Steel W Shapes	ASTM A 992
Rolled Steel C, S, M, MC, and HP Shapes	ASTM A 36
Rolled Steel Plates, Bars, and Angles	ASTM A 36
Hollow Structural Sections (HSS)	ASTM 500 - Grade B or C
Pipe	ASTM A 53 - Type E or S - Grade B
Reinforcing Steel for Concrete and Masonry	ASTM C 615 - Grade 60
** For connection, provide higher grade as required for capacity.	
Concrete	
Footings	f’c = 3,000 psi
Interior Slabs on Grade	f’c = 3,500 psi
Slabs on Deck	f’c = 3,500 psi
Foundation Walls	f'c = 4,000 psi
Retaining Walls	f'c = 4,000 psi
Piers	f'c = 4,000 psi
Grade Beams	f'c = 4,000 psi
Exterior Slabs	f'c = 4,500 psi
Exterior Equipment Pads	f'c = 4,500 psi
Miscellaneous	f’c = 3,000 psi
Piers	f'c = 4,000 psi
Grade Beams	f'c = 4,000 psi
Exterior Slabs	f'c = 4,500 psi
Masonry	
Concrete Block	ASTM C 90 Average Net Compressive Strength = 2,800 psi
Mortal	ASTM C 27 - TYPE S
Unit Masonry	ASTM C 90 CMU (2,800 psi) Types S Mortar - f’m = 2,000 psi
Grout	ASTM C 476 Compressive Strength = 2,500 psi 8 to 10 inch slump
Brick	ASTM C 216 - Type FBS - Grade SW

GRAVITY SYSTEM

Floor System Overview

The main floor system design is a structural steel framing with composite concrete slab on metal deck. Major members of the beam supporting the floor system are W18x35 and W16x26.

For a typical floor system, 7 1/2" concrete slab on 3" 20gage galvanized composite metal deck supports the floors and floor slab are reinforced with #4 rebar at 16" o.c. each way. Maximum live load deflection of composite section shall be 1/360 of clear span. In addition to composite metal deck, at greenhouse area, 4" lightweight concrete overlay slab is placed on rigid insulation on 3" cellular concrete slab, reinforced with #4 bar, epoxy coated, at 16" o.c. each way. All of main structural columns in Life Sciences Building are wide flange steel members. The size of columns is varying from W10x49 to W12x136. Most of the columns have a 12" depth vary in weight. W12x120 and W12x72 are used mostly in this building.

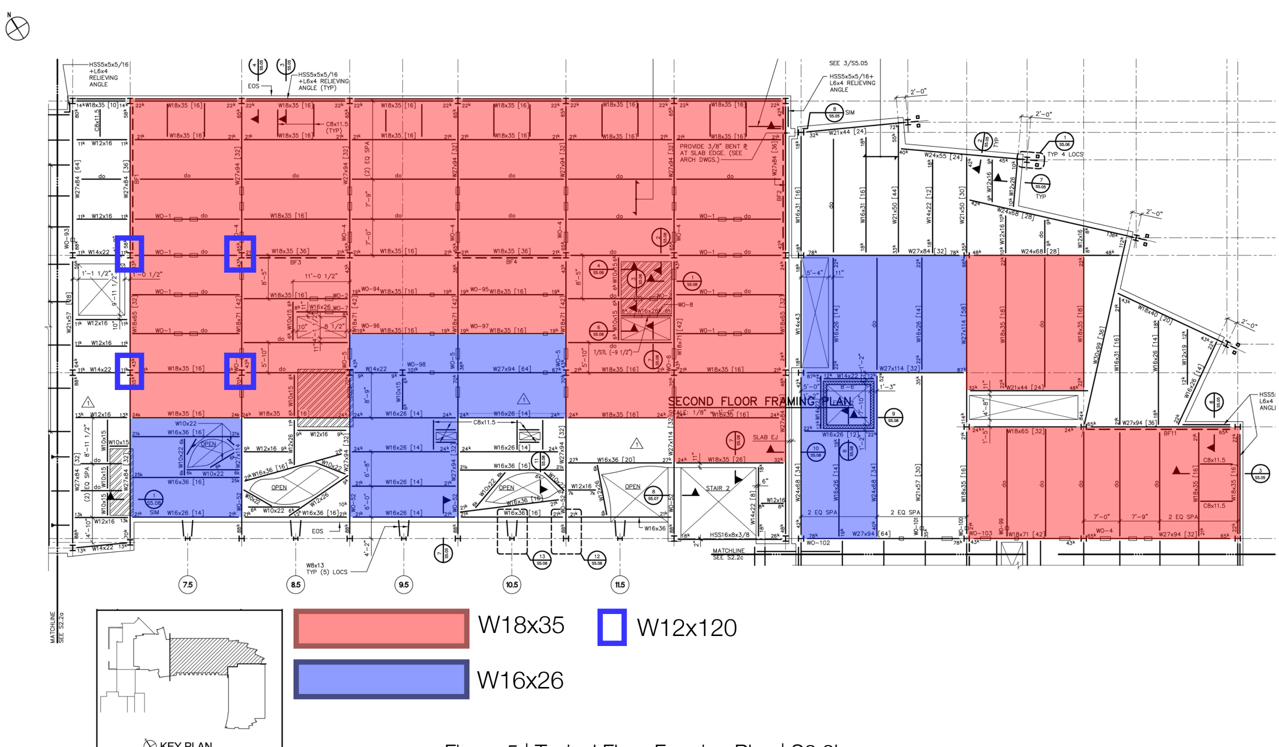


Figure 5 | Typical Floor Framing Plan | S2.2b

Laboratory Floor Vibration Design Criteria

Since this building is a laboratory building, there is a strict floor vibration design criteria. Vibrational velocity should be less than or equal to 3,000 micro-inch/second. Exciting force for vibrational velocity should be idealized footstep pulse of a 185 pound person walking at 75 step/minute, which is classified as moderate walk.

Roof System

Structural steel framing is used as the main roof framing system. A unique feature of the roof in Life Sciences Building is a 6,400 square foot greenhouse on north section and a green roof on west section. A green roof and greenhouse improve building performance in energy, especially in harsh winter in the location.

The greenhouse has metal truss framing system, Figure 6, and a green roof is supported on 6 1/2" concrete slab on 3" 20 gauge galvanized composite deck.

3" 20 gauge Type NS galvanized metal roof deck is used in north section. 3" metal deck is supported by W16x26 beams and W27x84 girders. W12x120 and W12x53 columns are supporting beams and girders

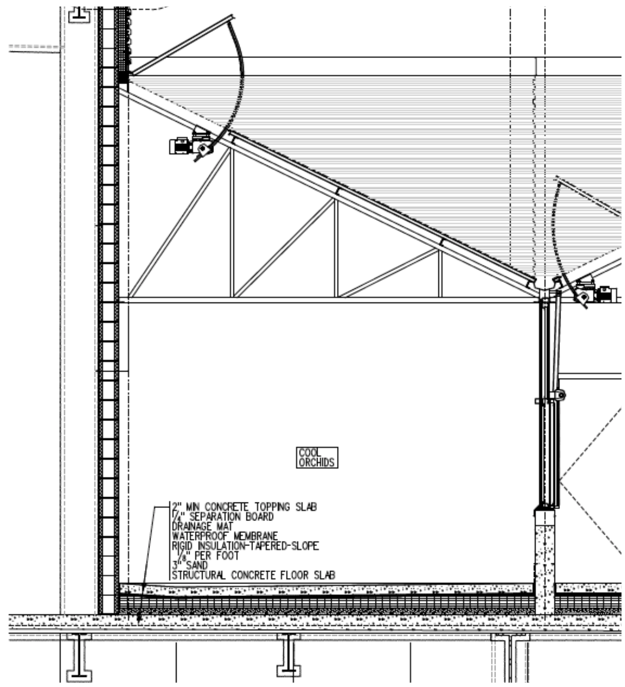


Figure 6 | Greenhouse Section | A4.20

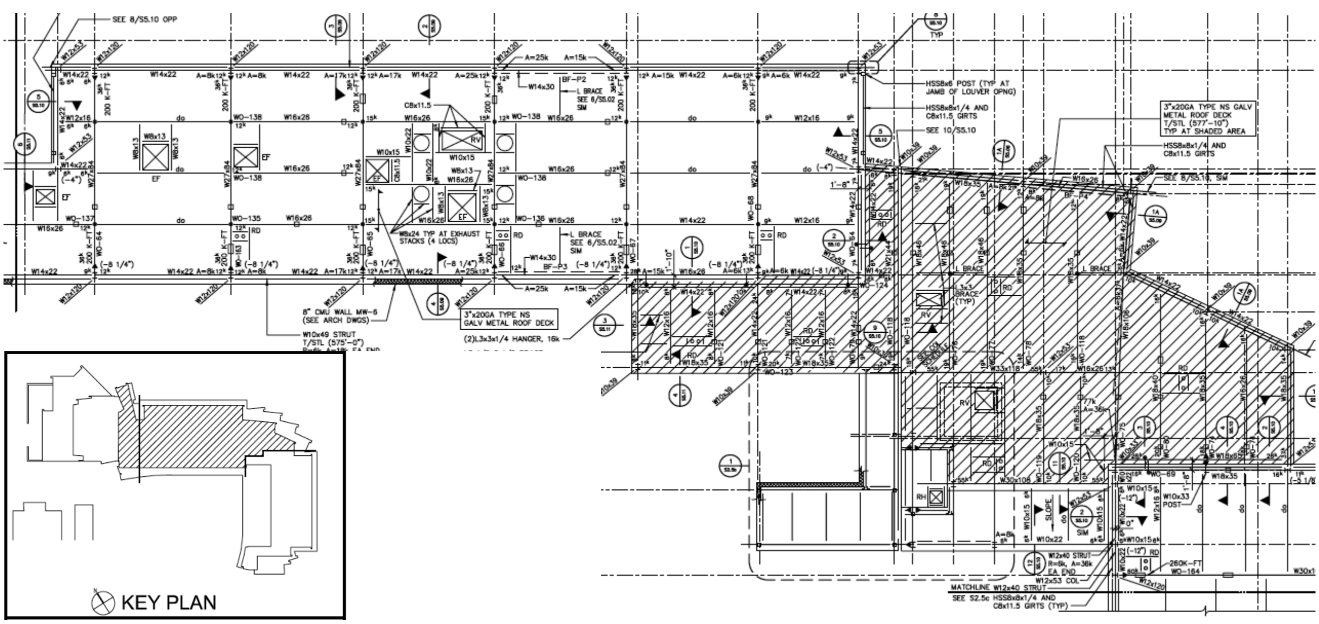


Figure 7 | Roof Framing Plan - North Section | S2.5b

LATERAL SYSTEM

The lateral force resisting system for Life Sciences Building consists of structural steel braced frames. There are sixteen braced frames of varying length and height. Majority of braces used hollow structural section (HSS) 10x10s1/2 and 10x10x3/8. The braced frames are not specially designed for seismic loads. The Figure 8 below shows the location of braced frames throughout Life Sciences Building.

Beams and braces are pin connection and the columns are continuous throughout the heights The major advantage of concentrically braced frames is high elastic stiffness. However, it reduces architectural versatility of the floor plan.

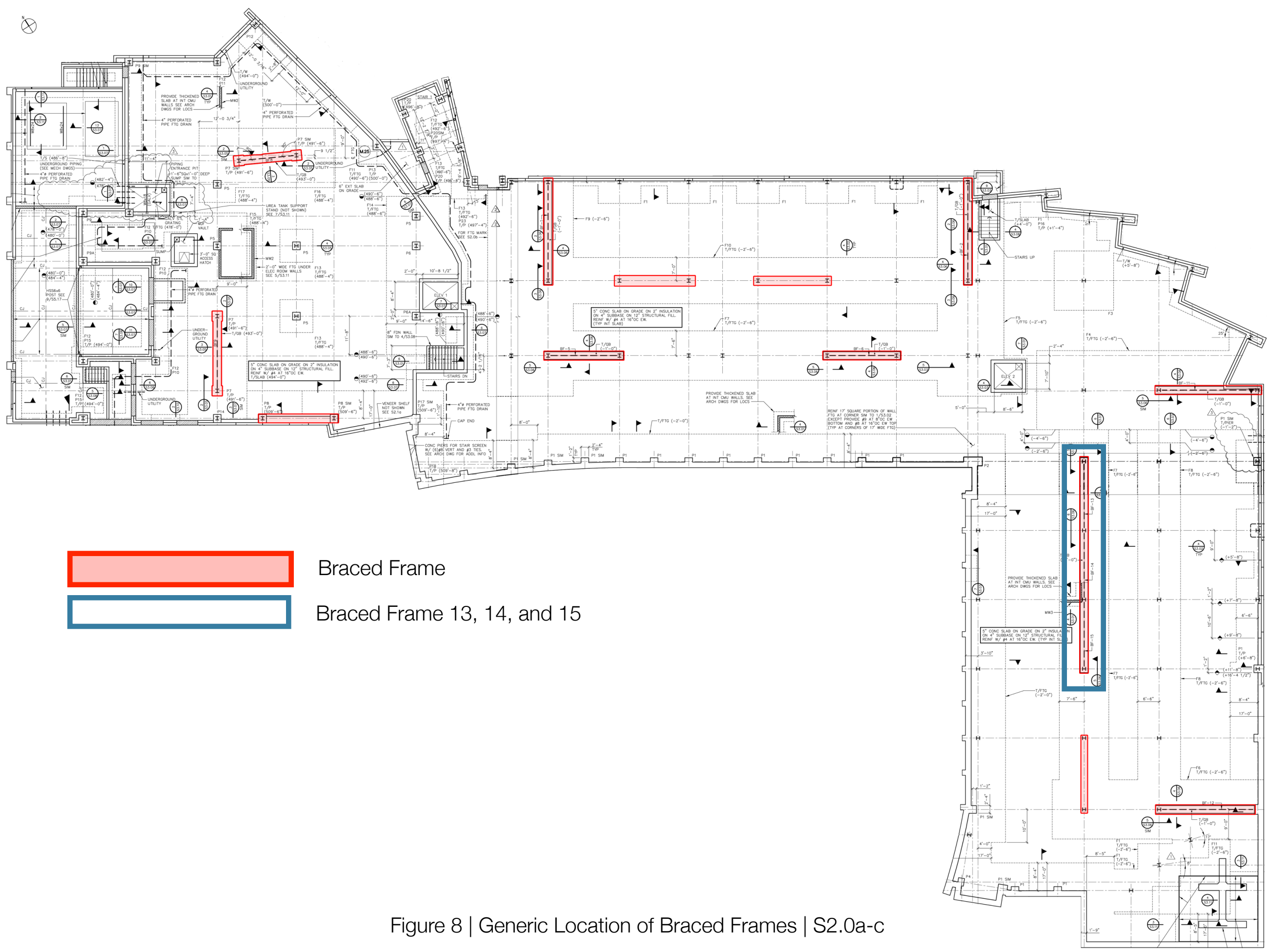


Figure 8 | Generic Location of Braced Frames | S2.0a-c

DETERMINATION OF DESIGN LOAD

National Code for Live Load and Lateral Loads

Live Load - ASCE 7-05 Chapter 4
Snow Load - ASCE 7-05 Chapter 7
Wind Load - ASCE 7-05 Chapter 6
Seismic Load - ASCE 7-05 Chapter 12 - Equivalent Lateral Force Procedure

Gravity Loads

Dead Loads

Due to the greenhouse design on the roof and its function as laboratory, dead loads are higher than a typical laboratory. The greenhouse floor load is 160 psf and other floors are at 110 psf. Roof dead loads are also higher than a typical project, 170 psf for roof gardens and terraces and 30 psf for regular roof.

Live loads

Live loads are referenced using ASCE 7-05 Chapter 4. Live loads reduction in applied when floor live loads are less than or equal to 100 psf.

Snow Loads

According to ASCE 7-05, ground snow in the location of the building is 65 psf.

Rain loads

Rain Loads is 50 psf referencing ASCE 7-05 Chapter 8.

Lateral Loads

Wind loads

Wind loads are calculated based on ASCE 7-05 Chapter 6. Basic wind speed (3 second gust) is 90 mph. Mean roof heigh is measure 80 feet.

Seismic loads

Seismic design category of the building is classified as B. Equivalent lateral force procedure is used as the analysis procedure in accordance of ASCE 7-05 Chapter 12. Seismic design base shear is calculated as 2,174 kips.

1.3 DESIGN CODES AND STANDARDS

CODES AND STANDARDS

International Code Council
- International Code Council 2006 Editions
- International Building Code 2000 Edition

American Society of Civil Engineering
- ASCE 7-05 - Minimum Design Loads of Buildings and Other Structures
- ASCE 7-10 - Minimum Design Loads of Buildings and Other Structures

American Concrete Institute
- ACI 318-11 - Building Code Requirements for Structural Concrete

American Institute of Steel Construction
- AISC Steel Construction Manual 14th Edition
- AISC Seismic Design Manual 2nd Edition

Reinforced Concrete Mechanics & Design 6th Edition by Wight and MacGregor

Vulcraft Deck Catalog

Construction Documents and Specifications of the Project

New York State Department of Transportation
- NYSDOT - Standard Specification for Construction and Materials

1.4 THESIS PROPOSAL

DESIGN SCENARIO

Problem Statement

The Life Sciences Building utilizes a composite steel framing system and the lateral system uses structural steel braced frames. Based on the previous analysis through technical reports, the existing gravity and lateral system for the Life Sciences Building are sufficient to meet both strength and serviceability requirements.

Since no significant challenges were found in the existing structural system, a scenario has been created in which a college campus, which resides in a high seismic area, specifically in San Francisco, CA, requests the design and construction of a building identical to the Life Sciences Building. The surrounding environment will be assumed to be identical to the current building site. However, in this new location, the soil characteristics, seismological characteristics, and climate conditions will differ significantly from the building's existing location.

As a result, in new building structural system, especially lateral forces resisting system will need to be checked and likely redesigned. In order to change the climate condition in the building, building envelope will be reassessed to the new environment and redesigned as well.

PROPOSED SOLUTION

Since a hypothetical scenario has been created in the problem statement, a fictitious data of the building is used for the design scenario. However, in order to get more detail analysis, it would be attempted to find the actual data related to geotechnical report.

In order to relocate the building, a building will be analyzed for new loads, and additional codes will be reviewed in new site location. The current state code, 2013 California Building Code, references *the International Building Code 2012 edition* and *American Society of Civil Engineers (ASCE) 7-10*.

The redesign of the lateral system will affect the gravity system, and a structural steel framing with composite concrete slabs on metal deck will be kept for new design. Structural braced frames will be considered as a prior design based seismic loads. However, the change of lateral systems will affect the gravity system and the configuration of lateral system will be carefully chosen due to architectural layout.

To resist the new loads, the floor system will be redesigned with the least amount of weight since the seismic load is based on the building weight. The reduction of the building weight will be benefit to design the lateral system. Compared to normal weight concrete, the lightweight concrete slab will be considered as floor system redesign, and it will affect the floor fire proofing system.

In redesign of lateral system, several designs will be suggested to the owner such as structural steel braced frames and structural steel moment frames. A high ductility system will provide the cost saving by reducing member size, but increasing extra costs in the connection details.

BREADTH STUDIES

Breadth - Construction Management

A comparative cost analysis will be performed in which the cost of the lateral system will be compared to see the advantages and disadvantages between different lateral system designs. The cost analysis will include materials and labor. The final design in gravity and lateral system will be chosen for the owner in order to achieve economical benefit and its performance between the lateral systems.

Breadth - Building Envelope

Due to the relocation of building from heating dominant to cooling dominant region, the building envelope will be investigated for the new location. The heat transfer through the envelope will be investigated based on the climate condition of the site and redesigned for new location.

MASTER OF ARCHITECTURAL ENGINEERING REQUIREMENTS

AE 530: Computer Modeling of Building Structures has provided fundamental theory of computer modeling process and the technical knowledge to model to structure of the Life Sciences Building and redesign the building in new location. Computer modeling software such as ETABS and RAM Structure will be used to analyze the existing structural system of the building and new structural system in a seismic region.

AE 534: Analysis and Design of Steel Connections has provided the foundation of understanding for the steel connections. Incorporated with the materials covered in this course, the seismic detailed connection will be designed.

AE 538: Earthquake Resistant Design of Buildings has provided a background for structural dynamics and structural behaviors in the event of earthquake. It will provide the fundamental understanding of seismic design of the building.

AE 542: Building Enclosure Science and Design has provided the understanding of the science in building envelope. It will help to evaluate the existing envelope design to see whether the existing design would be appropriate to the new environment. The redesign of envelope will also be considered.

CHAPTER 2 - STRUCTURAL DEPTH

2.1 LATERAL FORCE RESISTING SYSTEM REDESIGN OVERVIEW

For educational purpose, a scenario is developed that the project identical to the Life Sciences Building is proposed to construct in San Francisco, California. Compared to east coast where the existing building is located, structural design of the building is primarily focused on the seismic load instead of wind loads.

Figure 9 | New Project Location

The existing building has a structural steel braced frame as a lateral system and the lateral system is not seismically designed and detailed. The existing project location is considered as seismic design category B, which is a low to moderate vulnerability to the building structures.

	Existing Building Location East Coast, USA	New Location San Francisco, CA
Site Class	**D**	**D**
Seismic Design Category	**B**	**D**
Short Period Design Acceleration S_{DS}	**0.32**	**1.0**
One-Second Period Design Period, S_{D1}	**0.13**	**0.6**

Table 1 | ASCE 7-10 Table 12.2-1

However, due to the relocation of the building into a high seismic region, the structural system is required to adjust to resist a high seismic load in San Francisco, CA, especially in the lateral load resisting system. San Francisco, California, especially where the project is going to be relocated, is considered as seismic design category D and it is considered as a high vulnerability to the building structures.

According to *AISC Seismic Design Manual*, 'Seismic force resisting systems are classified in to three levels of inelastic response capability, designated as ordinary, intermediate or special, depending on the level of ductility that the system is expected to provide.' There are many types of lateral load resisting systems with seismic detailed. However, the existing structure is designed as a structural composite steel and the braced frames are already designed and placed according to the architecture. To minimize modification in architecture without change the materials of the lateral system, in this report, two alternative lateral load resisting systems are proposed: eccentrically braced frame and special moment frame.

Both eccentrically braced frame and special moment frame provide the response modification coefficient, R, of 8, compared to R = 3 for the existing braced frame system. Since the system with higher R value provide more ductile behavior to the building structure, the base shear of the building is reduced by the factor of R. This would provide the advantage to the building structure, but will require the larger member section to use its plastic behavior as well as elastic behavior.

	Existing System	Eccentrically Braced Frame	Special Moment Frame	Concrete Shear Wall
Response Modification Coefficient, R	**3**	**8**	**8**	**6**
Overstrength Factor, $\Omega 0$	**3**	**2**	**3**	**2 1/2**
Deflection Amplification, Cd	**3**	**4**	**5 1/2**	**5**

Table 2 | ASCE 7-10 Table 12.2-1

The existing floor system is a structural steel framing with normal weight concrete (NWC) slab on composite deck. The thickness of the slab is varied depend on the occupancy of the space, but mostly 7-1/2 inches thick concrete slabs. However, the building weight is critical to seismic force resisting system. According to *ASCE 7-10* 12. 8. 1, the seismic base shear force is determined proportional to the effective seismic weight of the building. According to *ASCE 7-10* 12. 7. 2, the effective seismic weight of a structure includes several factors: dead load, 25 percent of live load in a storage areas, partition loads, permanent equipment, and 20 percent of snow loads.

Seismic Base Shear, V

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

Where
C_s = the seismic response coefficient
W = the effective seismic weight

In order to reduce the weight of the building, lightweight concrete (LWC) slab is proposed with the appropriate thickness to achieve strength and fire protection requirements compared to original design. Table ## below provides the comparison of composite decks between normal weight and lightweight concrete slab. The comparison of strength in composite deck is evaluated based on *Vulcraft Steel Roof & Floor Deck.* Using lightweight concrete slab on composite deck with thinner slabs will expect the modification of gravity design with thinner steel member sizes.

	NWC Slab	LWC Slab	NWC Slab	LWC Slab
Slab Thickness	7 1/2" Slab	6 1/4" Slab	6 1/2" Slab	5 1/2" Slab
Decking	3VLI20	3VLI16	3VLI20	3VLI18
Clear Span	8 ft	8 ft	8 ft	8 ft
Strength	333 psf	374 psf	274 psf	278 psf

Table 3 | Composite Deck Comparison: NWC vs. LWC

In the existing design, braced frames in the penthouse are designed to serve the lateral forces within the penthouse only and the lateral loads were transferred by the transfer girders in the fourth floor. Due to the continuity of vertical stiffness, there are several modifications made in architectural layout of the building. The columns on the grid line L8.3 are moved to the grid line M due to the vertical continuity of lateral stiffness. This will allow braced frames to support the lateral system in the full building height.

Part of the requirements for Master of Architectural Engineering, three-dimensional structural analysis is performed. Among many different computer analysis software, ETABS 2013 is chosen to use for redesign and analysis of new project with student's capability of knowledge. Due to *ASCE 7-10* Section 12.2 structural system selection, in

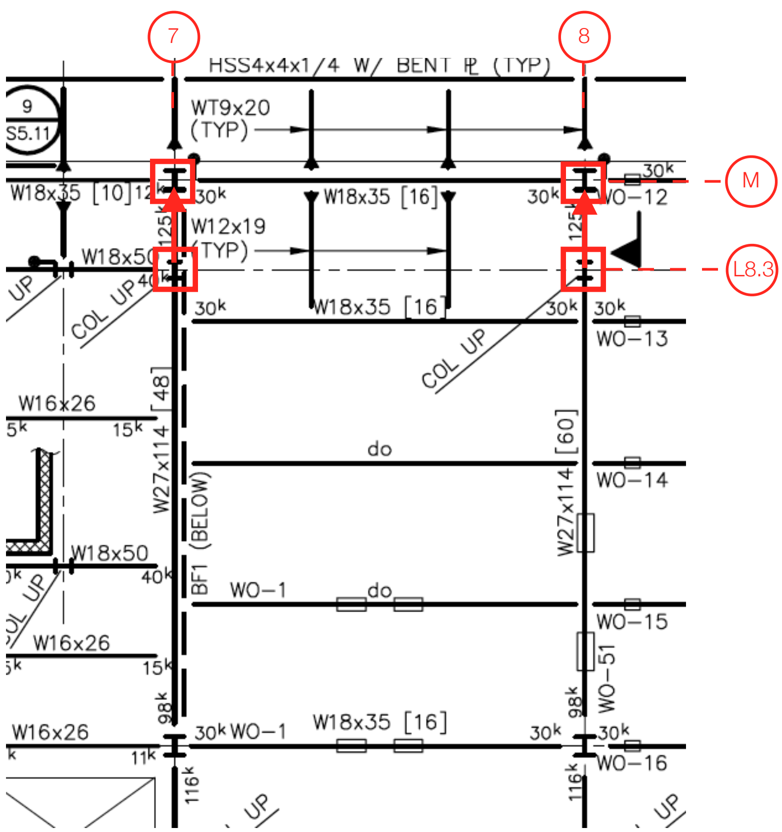


Figure 10 | Modification of the Grid Lines | S2.4b seismic design category D (SDC D), steel eccentrically braced frames is permitted where the structural height of the building is limited to 160 ft and steel special moment frames is not limited to the structural height. *ASCE 7-10* Table 12.6-1 provides permitted analytical procedures depends on its structural characteristics and seismic design category. Since the new location is classified as seismic design category D and structural heights of 91 ft, modal response spectrum analysis is appropriate to perform and it also accounts the building's structural irregularity.

For an appropriate and detailed analysis of the lateral system, the design of the diaphragm should be selected based on its behavior. There are three classifications of diaphragms: rigid, semi-rigid, flexible. Reinforced concrete slabs often treated as rigid because of the relative stiffness between beams and columns, and slabs. In most of design, the composite steel deck is also assumed as a rigid diaphragm since the stiffness of concrete slab and decking is much stiffer than the structural steel beams and columns. Shear studs between the deck and the beams and girders transfer lateral loads directly to the beams and columns. One of the most import roles of diaphragms is to transfer lateral inertial forces to vertical elements of the seismic force-resisting system.

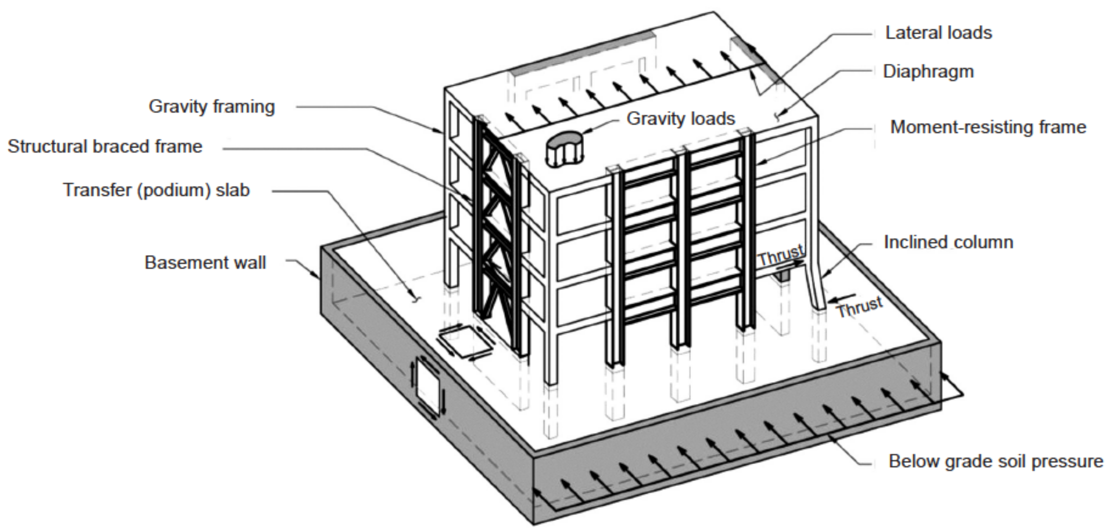


Figure 11 | The Role of Diaphragm | NEHRP Seismic Design Technical Brief No. 5

In the modeling process of seismic design, it is experienced that structure with rigid diaphragm and one with semi-rigid diaphragm provide significant difference of behaviors in diaphragm. In rigid diaphragm, the axial forces in the beams is not observed. The rigid diaphragm provides the infinite in-plane stiffness and it prevent the in-plane shear deformations. However, in seismic force resisting system, the axial forces in the beams should be considered.

To account the axial forces in the beams, the diaphragm should be modeled as a semi-rigid. According to *NEHRP Seismic Design Technical Brief No. 5*, in seismic design of the composite steel deck and concrete-filled diaphragms, diaphragms are always permitted to be treated as a semi-rigid. In ETABS 2013, the semi-rigid diaphragm stimulates in-plane stiffness. The rigid diaphragm provides similar behavior of a semi-rigid diaphragm and this will let the analysis run faster since it does not account shear deformation in diaphragm.

Since North Wing and East Wing are separated structurally by expansion joints, it is allowed to treat both wings as the complete separated structures. In this report, only North Wings is analyzed due to its structural irregularity and complexity. Compared to North Wings, the geometry of the building structure in East Wing is much simpler and architectural layout of each floor is similar through the building. Although the actual design of structure is not developed by the student, the new layout of lateral force resisting system is suggested to both two new designs.

It is recommended that for seismic design category D, E, and F, the designers may perform modal response spectrum analysis or time-history analysis to get more approximate results. For the purpose of learning the difference between linear and non-linear analysis, both equivalent lateral force analysis and modal response spectrum analysis are performed and compared.

According to *ASCE 7-10* 12. 9. 1, the analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response considered by the modal. In both designs, sufficient number of modes are provided to obtain the modal mass participation of at least 90 percent of the actual mass.

ASCE 7-10 12. 9. 2, it is required that the ground acceleration need to be scaled in order to perform the appropriate modal response spectrum analysis. The value related to story drift, support forces, and individual member forces for each mode of response shall be divided by the quantity R/I_e and the value for displacement and drift quantities shall be multiplied by the quantity C_d/I_e. In ETABS, the ground acceleration is divided by appropriate R/I_e. If the ratio of modal response spectrum to static analysis is less than 0.85, the ground acceleration for modal response spectrum analysis is multiplied by (I_e/R)*0.85(modal response/static analysis) and apply to each directions separately.

2.2 ECCENTRICALLY BRACED FRAMES DESIGN

DESIGN CONSIDERATION

Eccentrically braced frame is a hybrid system of concentrically braced frame and moment frame. It performs the lateral stiffness of concentrically braced frame and the ductility of moment frame. In *AISC Seismic Provisions*, eccentrically braced frame is described that braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace of column, forming a link that is subject to shear and flexure.

The design of a link in the eccentrically braced frame is critical in its behavior of resisting lateral loads. The link provides significant inelastic deformation capacity through shear or flexural yielding. In the graphs provided, when eccentricity is approaching toward zero, the eccentrically braced frame would behave with higher stiffness as a concentrically braced frame. On the other hands, when eccentricity is becoming a full length of the beam, it would perform as a moment frame with the ductile behavior.

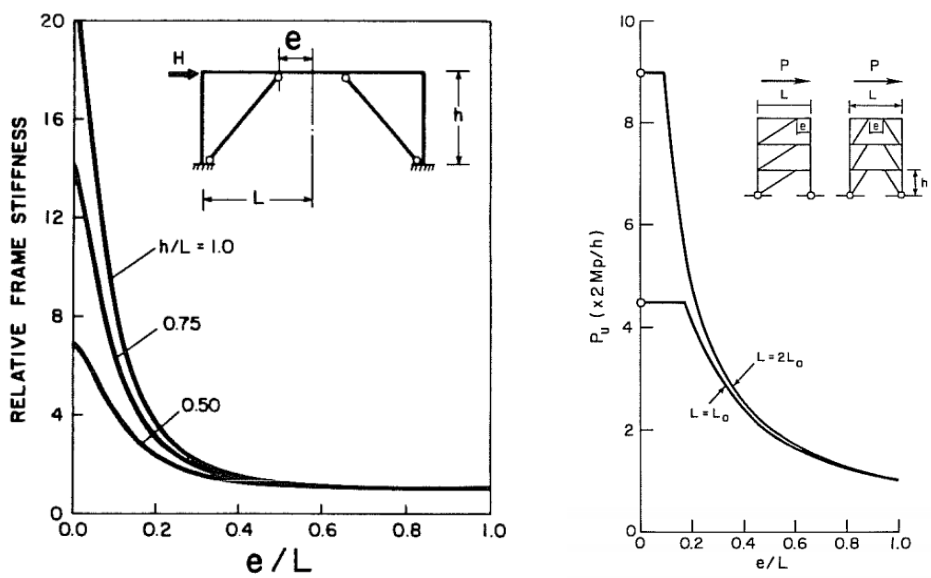


Figure 12 | Frame stiffness versus link length (Engelhardt and Popov, 1989)

The layout of lateral force resisting system is chosen carefully due to the architecture of the building. However, eccentrically braced frames often provide the advantage to architectural layout where concentrically braced frame cannot be located due to the space limitations by doors and windows. Due to a higher response modification coefficient, R=8, the project costs would be saved in construction of diaphragm and foundation by reducing the base shear force.

In preliminary designing, the existing concentrically braced frames without seismic detailed were modified their configurations of braces to eccentrically braced frames. The existing braced frames were placed carefully by the designer. It was a challenge to modify the configuration of the braces and to place additional braced frame without interfering the existing architecture. To consider a continuous load path and vertical stiffness of lateral force resisting system, single diagonal braces and double braces such as V shaped or inverted V sharped bracing were used in a few bays. Since architectural design of the building has been completed already, it is hard to manipulate architectural features by the student.

Compared to the concentrically braced frames, the stiffness of eccentrically braced frames is more complicated to analyze by hand, especially in estimating the link segment. Based on the research done by Paul W. Richards, the stiffness of eccentrically braced frames is estimated by its geometry.

The Stiffness k of an EBF story

$$k = 1.35V_{design}(E/F_y)/[0.72(1.19-0.0023L_d)(L_d^2/a)+(0.13La/h)+(1.71he/L)+(0.21eh/d)]$$

V_{design} = design story shear
F_y = beam yielding stress
E = elastic modulus of steel
d = beam depth
Ld, a, h, and e = frame dimensions

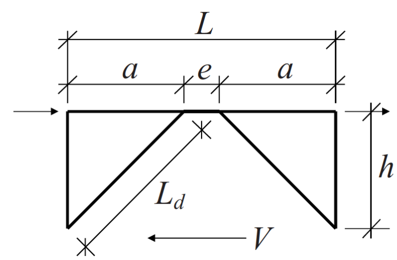


Figure 13 | Estimating the stiffness of EBF | Paul W. Richards

However, this estimating method is only valid when frame geometries are identical for all frames in a given heights and the design shear should be at least 200 kips when shear yielding links are used. In the report, the estimating method by Paul W. Richards is used to find the relative stiffness of the frames.

MODELING PROCESS AND CONSIDERATIONS

Assumptions

Modeling for the design and analysis of eccentrically braced frames is done by ETABS 2013 based on student's knowledge. To fulfill the graduation requirement of Master of Architectural Engineering, 3D modeling has been performed to analyze the lateral system redesign. The modeling of ETABS 2013 is mainly focused on the lateral force resisting system design. However, the software still provide the composite steel frame design to get the preliminary design of gravity system if necessary. The following assumptions were made during the modeling process:

- Steel frame design and composite beam design are performed to have preliminary design.
    - In steel frame design which is a built in function of ETABS 2013, the seismic detail analysis is ignored since there is a bug on ETABS 2013.
- The building base is designed as a pinned connection for both gravity and lateral frames.
- Connection details
    - Beam-to-column connection is assumed to be fully restrained and the joints are considered as fixed. Standard moment connection detail is applied in ETABS 2013.

- Brace-to-beam connection with link is assumed to be fully restrained to transferred the shear and flexural loads. The other side of braces connecting to beam and column without link is connected as a pinned.
- Design Loads
  - Self-weight factor is applied to dead load case and it is accounted as the weight of the building for seismic design.
  - Snow load shall be accounted for the effective seismic weight in seismic design. However, compared to the existing project site, snow load is neglected in San Francisco.
  - The exterior wall load is applied as a linear load on the perimeter beams to account the dead load from exterior walls.
  - Lightweight overlay concrete slab in the greenhouse is applied as a surface load in form of dead load.
- Diaphragm
  - To account the collector forces and axial forces on the beam in eccentrically braced frames, the diaphragm is modeled as a semi-rigid instead of rigid.

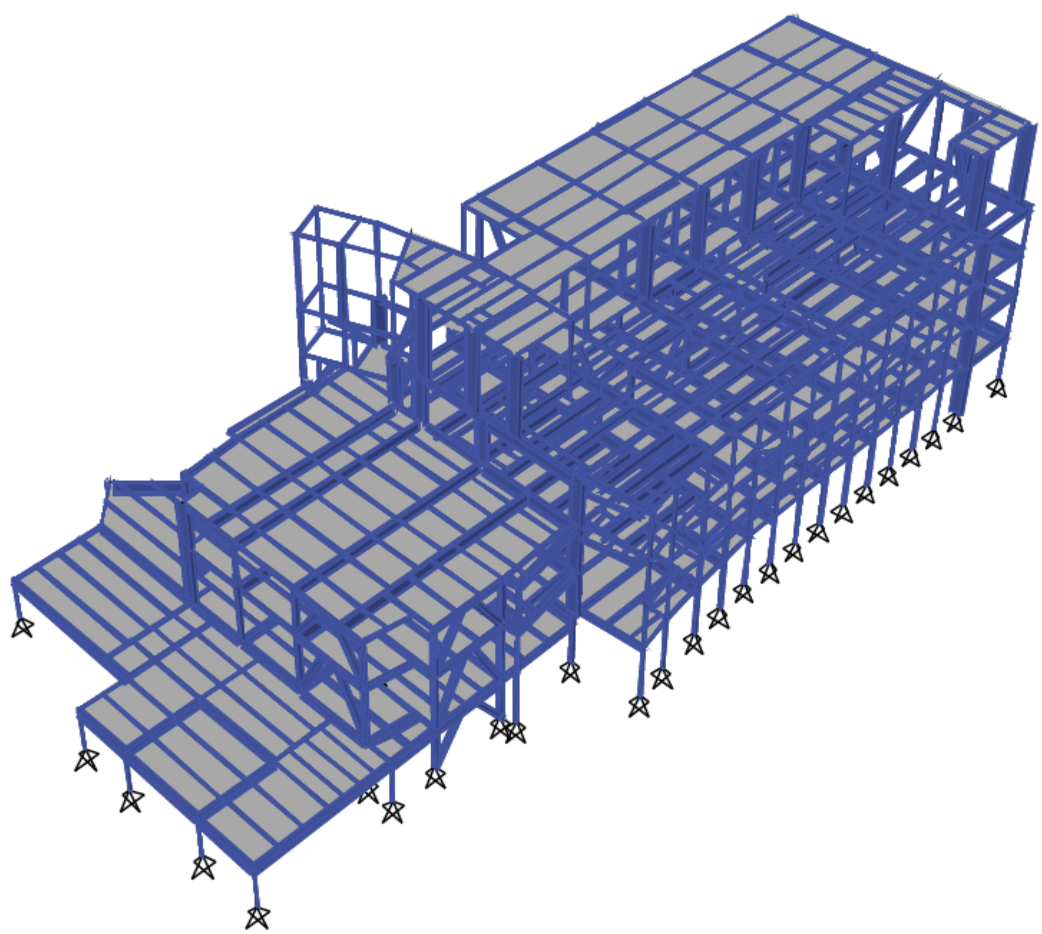


Figure 14 | ETABS 3D Model for Eccentrically Braced Frame

LAYOUT OF ECCENTRICALLY BRACED FRAMES - NORTH WING

Figure ## provides the layout of eccentrically braced frames. The original layout of concentrically braced frames was considered to be kept. To increase the lateral stiffness and strength, two additional braced frame is designed. To maintain vertical stiffness continuity, penthouse grid line is moved to match with the main grid line and this is explained on ‘2.1 Lateral Force Resisting System Redesign Overview.’ Frames highlighted in green are added in the new location whereas frames highlighted in pink are placed in the original design location of braced frames.

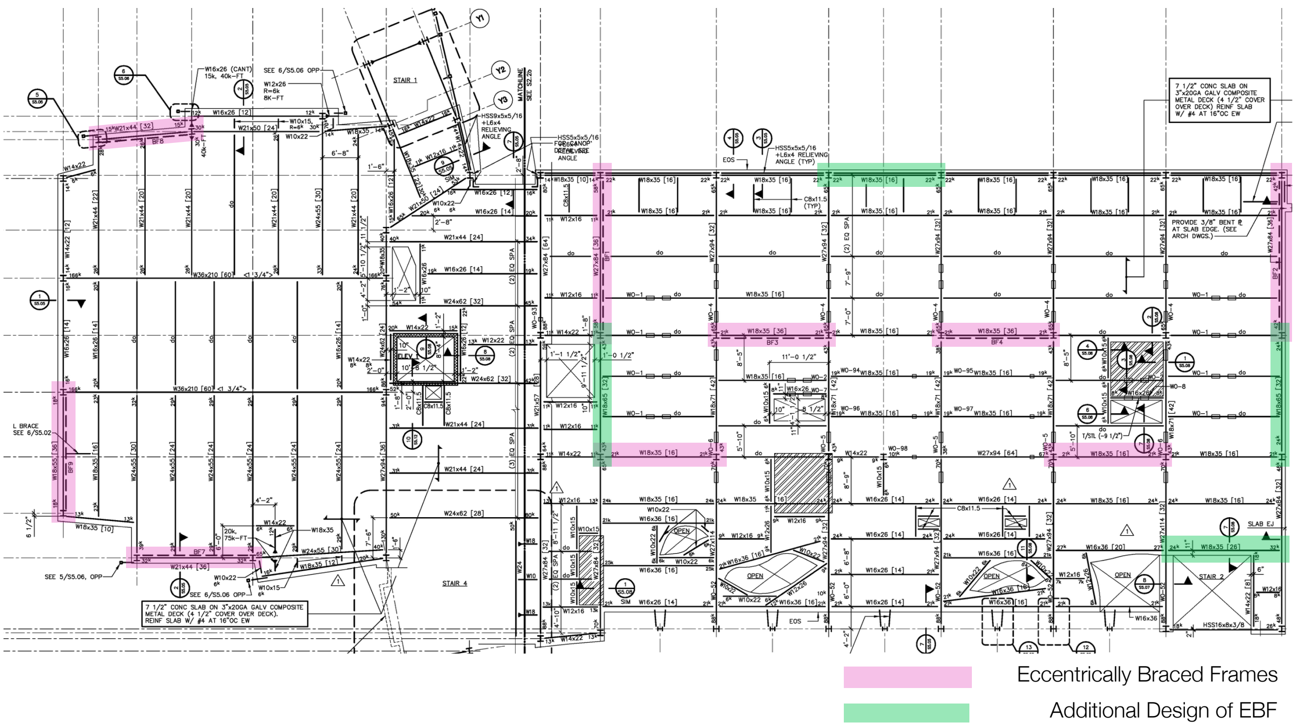
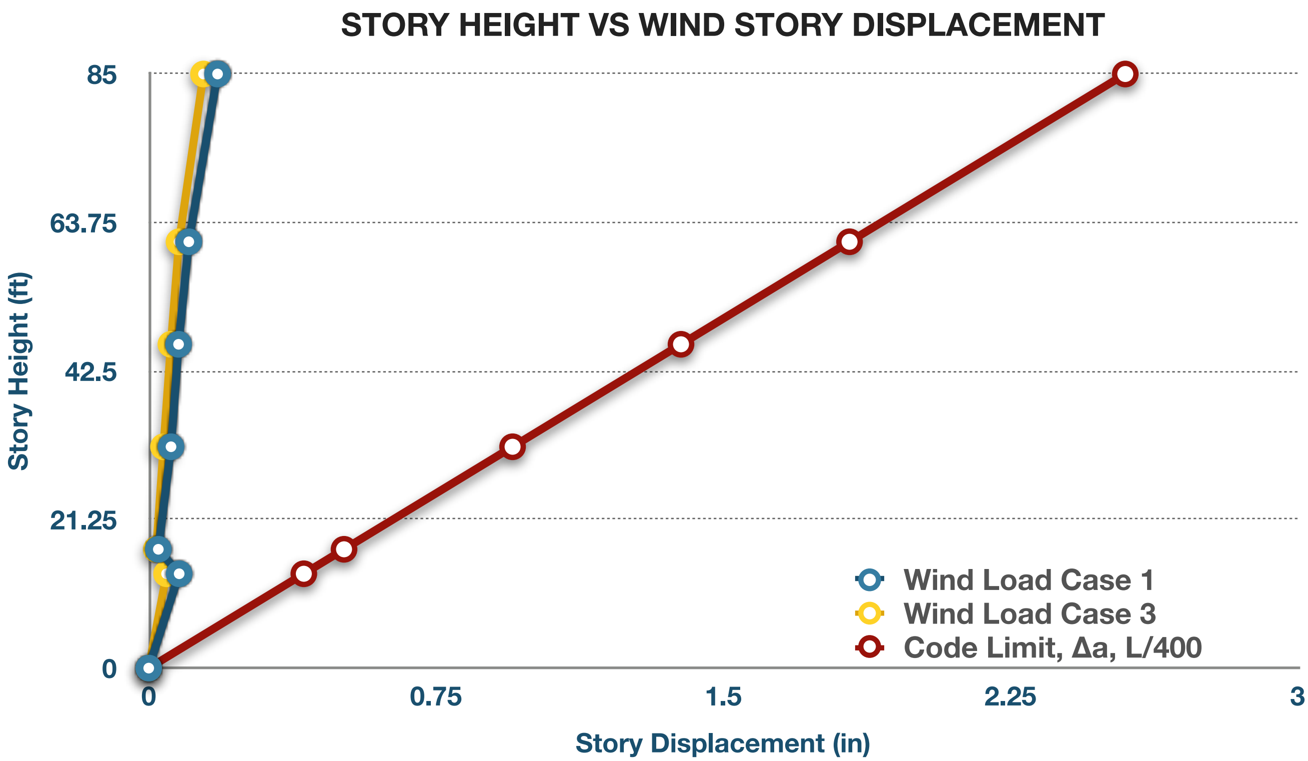


Figure 15 | Eccentrically Braced Frame Layout | S2.1a&b

WIND LOAD ANALYSIS

The specific drift value is provided with the table in the appendix. Since this report is more focused on the seismic design of lateral force resisting system in a high seismic region, the wind load is less considered and the drift comparison shows that the drift for wind load is relatively small compared to seismic load drift,



Wind Load Base Shear and Overturning

	Fx (kips)	Fy (kips)	Overturning (ft-kips)
Wind Case 1	291.741		13101.4581
		551.536	25868.3383
Wind Case 2	218.806		9826.0936
		413.652	19401.2701

Table 4 | Wind Load Base Shear and Overturning

SEISMIC LOAD ANALYSIS

Seismic Base Shear and Overturning

Level	hx (ft)	Mass (In-s^2/ft)	Weights, W (kips)	W*hx	C_{vx}	Story Forces, F_i (kips)	Story Shear, V_i (kips)
Penthouse Roof	85.00	5805.50	186.76	15874.85	0.05	63.75	63.75
4th Floor	61.00	46854.65	1507.31	91946.16	0.28	369.24	432.99
3rd Floor	46.33	67967.05	2186.50	101300.54	0.31	406.81	839.80
2nd Floor	31.67	71782.77	2309.25	73134.00	0.23	293.69	1133.49
1st Floor	17.00	49909.19	1605.58	27294.84	0.08	109.61	1243.11
Auditorium	13.50	31467.36	1012.30	13666.12	0.04	54.88	1297.99
Total		273786.52	8807.71	323216.51	**Base Shear**	1297.99	

Table 5 | Seismic Story Force Calculation - ASCE 7-10 | T= 0.597 sec

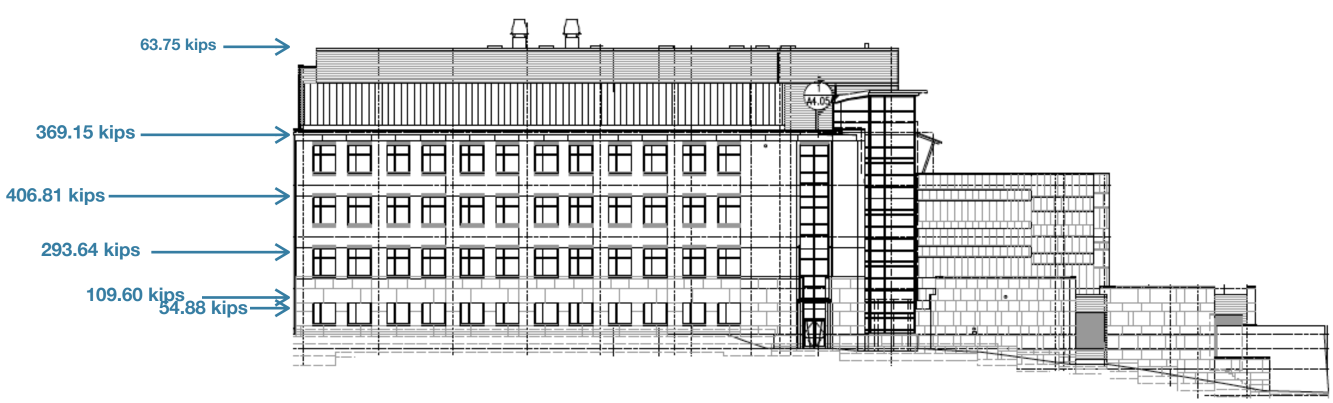


Figure 16 | Seismic Force Distribution - North Wing

	Equivalent Lateral Force Analysis	Modal Response Spectrum Analysis	Ratio of Response Spectrum to Static Base Shear
Base Shear, kips			
X - Direction	1297.986	1123.515	0.866
Y - Direction	1376.334	1154.027	0.838
Overturning Moment, ft-kip			
X - Direction	59363.549	49471.997	0.833
Y - Direction	62745.138	50824.154	0.810

Table 6 | Seismic Load Comparison

The equivalent lateral force analysis provides more conservative value than modal response spectrum analysis. In order to account collector forces and axial forces in the braced frame, modal response spectrum analysis is preferred to perform and diaphragm may be preferred to be modeled as a semi-rigid.

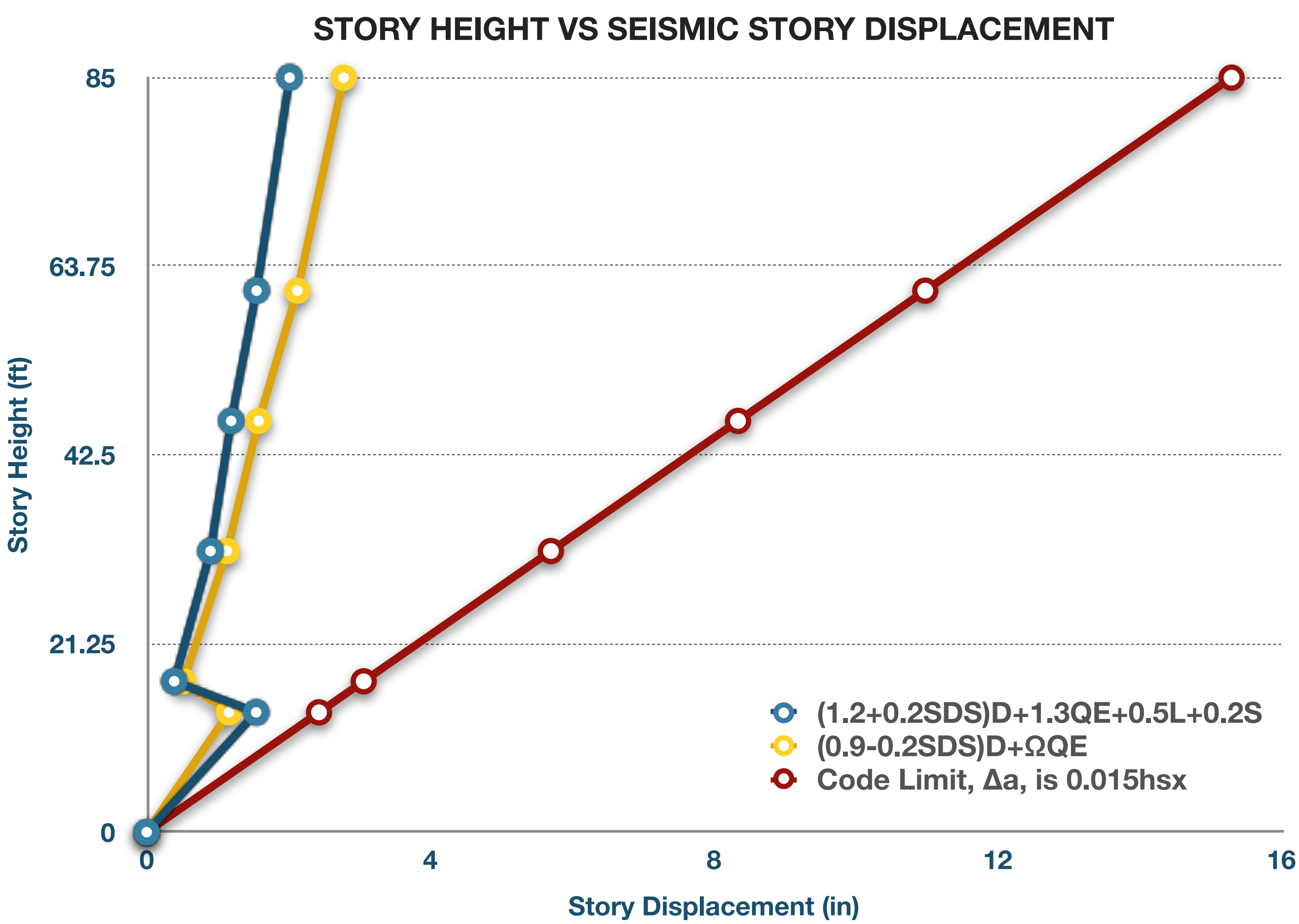
Compared to the base shear in the original design, 2174 kips, new base shear in San Francisco, CA is 1298 kips even though eccentrically braced frames provide response spectrum coefficient = 8. Due to the self-weight of the building and the building period in new location, the seismic response coefficient is higher than the original location.

STORY DRIFT COMPARISON

Floor	Story Height (ft)	(1.2+0.2SDS)D+1.3QE+0.5L+0.2S (in)	(0.9-0.2SDS)D+OmegaQE (in)	Code Limit, Δ_a, = $0.015h_{sx}$ (in)
Roof	**85.00**	2.02	2.78	15.30
4th Floor	**61.00**	1.55	2.12	10.98
3rd Floor	**46.33**	1.19	1.58	8.34
2nd Floor	**31.67**	0.90	1.13	5.70
1st Floor	**17.00**	0.39	0.52	3.06
Auditorium	**13.50**	1.54	1.16	2.43
Base	**0**	0.00	0.00	0.00

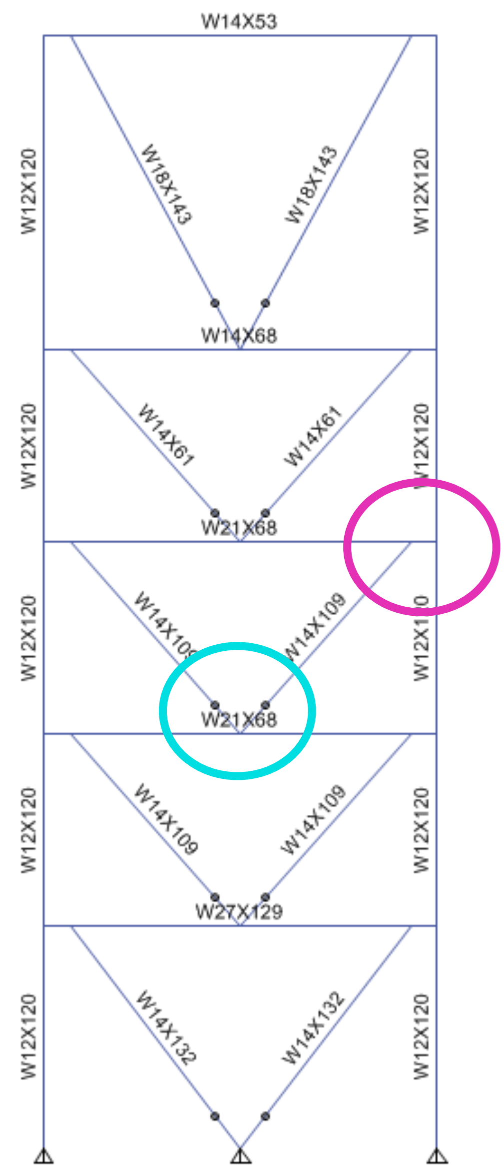
Table 7| Story Drift Comparison

Allowable story drift for seismic loads are limited by ASCE 710 Table 12.12-1. This table provides allowable story drift based on the type of lateral load resisting system and risk category. For eccentrically braced frames in risk category III, the allowable story drift, Δ_a, is $0.015h_{sx}$ where h_{sx} is the story height below Level x. The drifts of the governing load combinations are not exceed the code limits of allowable drift. Compared to wind loads, which is serviceability based, the seismic load is designed for the ultimate condition. For complexity of modeling in auditorium floor, the story drift for auditorium provides a higher value relative to other floor. There is a diaphragm discontinuity between auditorium and first floor in the ETABS model. The discontinuity may provide inappropriate drift in the auditorium and the architectural layout of auditorium space limits the additional location where the braced frames would be placed. The table also provides the actual drift value.



DESIGN PROCESS OF ECCENTRICALLY BRACED FRAME

Through the analysis of ETABS 2013, eccentrically braced frames is designed and analyzed. Due to its difficulty of estimating the actual stiffness of the frames, there were several iterations to optimize designs. Figure ## shows the final design with member sizes for typical frames. In most of frame design, the length of link is defined as 48 inches for a V shaped bracing and 30 inches for a single diagonal bracing. The Figure ## shows the fixity of the member as well. Typical connection designs are provided in detailed. Beam-to-column connection is a moment resisting connection and braces are simply support to the beam with a gusset plates. Steel WT section is used to connect the gusset plate and braces. Compared to original design of braced frames without seismical detailed, the overall member sizes become larger and heavier to dissipate more energy during the seismic event.



EBF Brace-to-Link Connection Detail | Figure 18

EBF Brace-to-Beam Connection Detail | Figure 19

Figure 17 | Typical Eccentrically Braced Frame Design

Detailed Connection Design

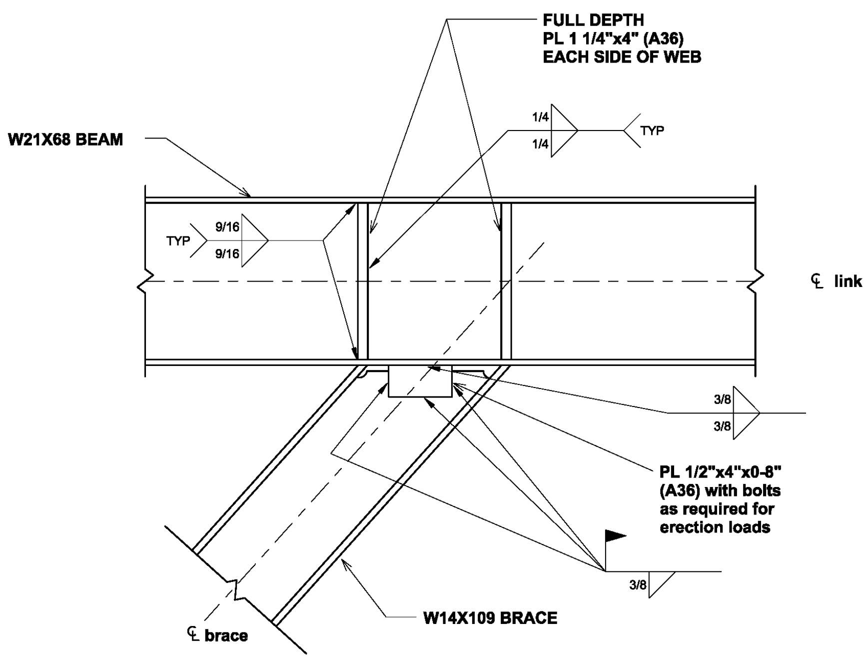


Figure 18 | Eccentrically Braced Frame Brace-to-Link Connection

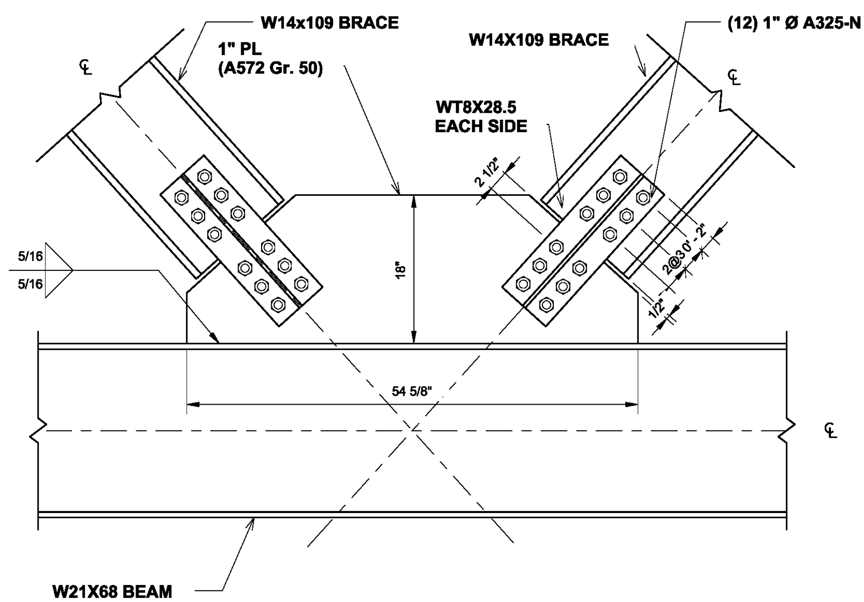


Figure 19 | Eccentrically Braced Frame Brace-to-Beam Connection

Link Design

One of the most important in designing eccentrically braced frames is a design of the link. Links are subject to shear and flexural due to eccentricity between the intersections of brace centerlines and the beam center line.

Link length is designed based on the ratio of nominal plastic flexural strength, M_p to nominal shear strength of an active link, V_p. Depend on the link length, the maximum allowable link rotation angle is limited. The major role of link is to resist the shear transferred from the braces. When Pr/Pc ≤ A 0.15, *AISC Seismic Provisions* allows to neglect the effect of axial force on the link.

For $P_u/P_y \leq 0.15$

$V_p = 0.6F_yA_{tw}$ (AISC Seismic Provision Eq. F3-2)

For $P_u/P_y > 0.15$

$V_p = 0.6F_yA_{tw}\sqrt{(1-P_u/P_y)^2)}$ (AISC Seismic Provision Eq. F3-3)

F_y = Yield Stress

A_{tw} = Link Web Area

P_u = Required axial strength

P_y = Nominal axial yield strength

In the exiting braced frame, hollow structural steel (HSS) section is used for the braces. However, the *AISC Seismic Provisions* prohibits the usages of HSS section in eccentrically braced frames as a braces. Instead, *AISC Seismic Provisions* allows I-shaped (wide flanges sections) or built-up box sections to be used as a link.

According to *AISC Seismic Provisions*, full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. The link to column connection must be a fully welded moment resisting connection with full penetration flange welds and a web connection cable of developing the shear capacity of the link.

Beam Design

The beam in eccentrically braced frames is designed in two different conditions: the link segment and the beam outside of the link. The amplified seismic load from the link is transferred to the beam outside of the link. From the effect of the overstrength factor, increasing the beam size results in increasing ultimate link force that beam must exist. According to the article by Samuel Dalton Hague, in order to avoid this complication in beam design, using shear links instead of longer links will reduce the link ultimate forces, and by selecting a brace with large flexural stiffness can reduce the demand on the beam. In order to transfer the moment and shear from the braces to beam, the brace-to-link concoction should be designed to resist the moment as a fully restrained moment connection.

Brace design

Compared to other elements in eccentrically braced frames, the braces are designed to remain elastic during the seismic event. Since the braces are fully restrained to the link in the beam, braces should be able to resist moments as well as axial. The connection between braces and column should be designed as a pinned and it would let braces to be designed as beam-columns. The braced frame that is not detailed for seismic loads does not allow to use braces as a compression member. However, the braces detailed for seismic event would be able to account the compressive strength on the braces. According to *AISC Seismic Provisions,* the seismically compact section should be used for braces and other elements.

Column Design

The columns in eccentrically braced frames are subject to the inelastic drift. The beam-to-column connection is allowed to be a fully restrained moment connection. This condition should meet the same requirements for beam-to-column connection in ordinary moment frames. The connection is also permitted to be designed as a simple connection with specific requirement in rotation of the frames.

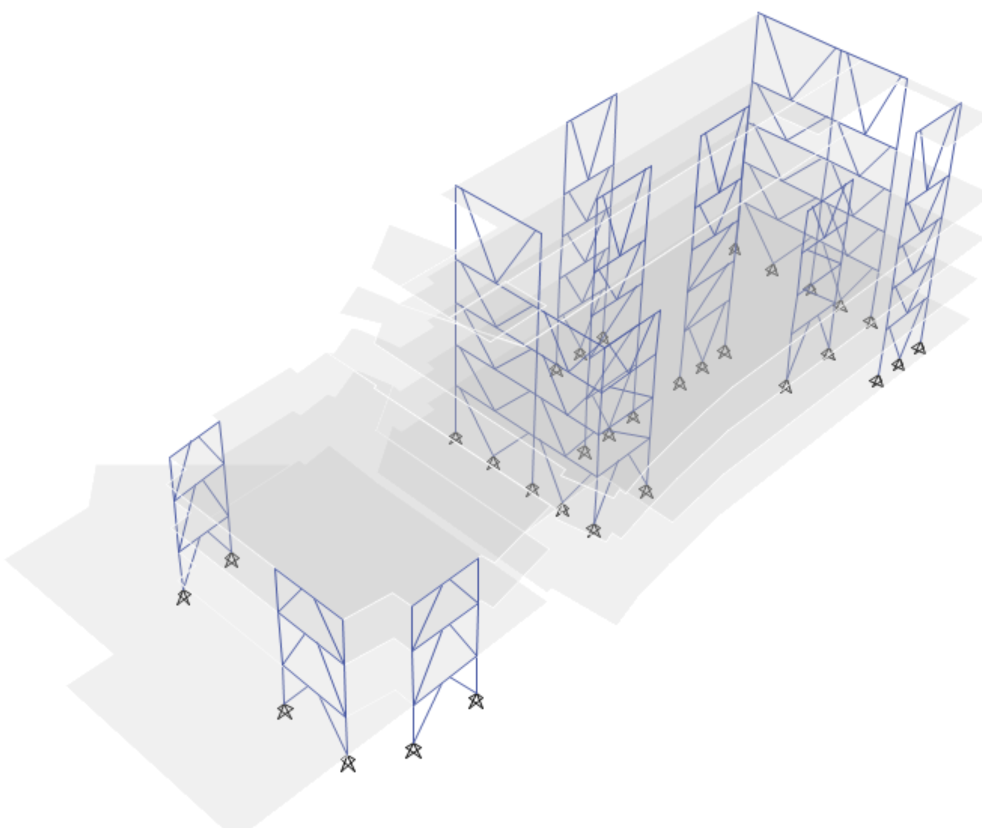


Figure 20 | Eccentrically Braced Frame Layout | ETABS 2013 3D Model

2.3 SPECIAL MOMENT FRAME DESIGN

DESIGN CONSIDERATION

According to *AISC Seismic Design Manual*, Special moment frame and intermediate moment frame systems resist lateral forces and displacement through the flexural and shear strength of the beams and columns. Compared to braced frames, SMF and IMF often have larger and heavier beam and column sizes to resist the forces and seismic drifts.Since the moment frames tend to have ductile behavior than braced frame, special moment frame and intermediate moment frame tend to have a larger and heavier members in beams and columns.

Due to the architectural freedom from using moment frames, architects may prefer to have moment frames than braced frame or reinforced concrete shear wall. However, the increased beams size may cause the problem in architectural and mechanical system layout in the building. To avoid the interference between moment frames and mechanical systems, the moment frame often placed on the perimeter of the building and it also help to control the lateral torsion in seismic and wind loads.

Similar to other seismic force resisting system, moment frames are anticipated to achieve the plastic mechanism. *AISC Seismic Design Manual* provides two primary methods to move plastic hinging of the beam away from the column: reducing beam section or special beam-to-column connection.

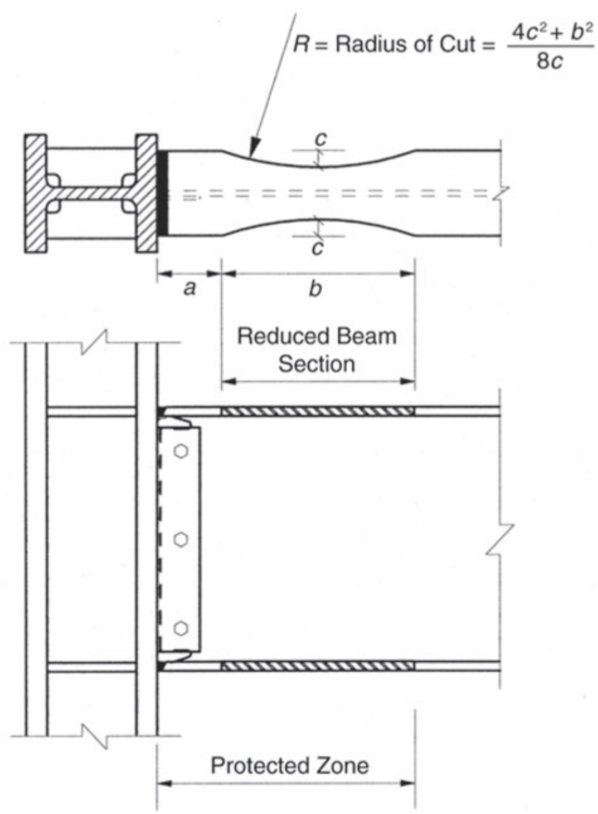


Figure 21 | Reduced Beam Section

In this report, reducing the cross sectional properties of beam at a defined location away from column is used. Reduced beam section method is to trim the beam cross sectional area at a certain point away from the column face. This method forces to place the plastic hinge in the reduced beam section before the beam-to-column connection fails during the seismic event. Special connection detailing is guided by *ANSI/AISC 468* in Part 9.2 of AISC Seismic Design Manual.

The main approach of designing moment farm is “strong column and beam connection.’, AISC Seismic Provisions the relationship of the strength between beams and columns by the Equation E3-1:

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} > 1.0 \qquad \text{(Provisions Eq. E3-1)}$$

$\sum M^*_{pc}$ = sum of the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column

$\sum M^*_{pb}$ = sum of the projection of the expected flexural strengths of the beam at the plastic hinge locations to the column centerline

MODELING PROCESS AND CONSIDERATIONS

Assumptions

Modeling for the design and analysis of special moment frame and special reinforced concrete shear wall are done by ETABS 2013 based on student's knowledge. To fulfill the graduation requirement of Master of Architectural Engineering, 3D modeling has been performed to analyze the lateral system redesign. The modeling of ETABS 2013 is mainly focused on the lateral force resisting system design. However, the software still provide the composite steel frame design to get the preliminary design of gravity system if necessary. The following assumptions were made during the modeling process:

- Steel frame design and composite beam design are performed to have preliminary design.
  - In steel frame design which is a built in function of ETABS 2013, the seismic detail analysis is ignored since there is a bug on ETABS 2013.
- The building base is designed as a pinned connection for both gravity and lateral frames.
- Connection details
  - Beam-to-column connection is assumed to be fully restrained and the joints are considered as fixed.
  - Reduced beam section is applied to all the moment connection in ETABS 2013.
- Design Loads
  - Self-weight factor is applied to dead load case and it is accounted as the weight of the building for seismic design.
  - Snow load shall be accounted for the effective seismic weight in seismic design. However, compared to the existing project site, snow load is neglected in San Francisco.
  - The exterior wall load is applied as a linear load on the perimeter beams to account the dead load from exterior walls.
  - Lightweight overlay concrete slab in the greenhouse is applied as a surface load in form of dead load.
- Diaphragm
  - To account the collector forces and axial forces on the beam in eccentrically braced frames, the diaphragm is modeled as a semi-rigid instead of rigid.

LAYOUT OF MOMENT FRAMES - NORTH WING

Compared to the layout of eccentrically braced frame, additional number of moment frame is required. The original trial system was without shear wall. After several analysis on the model, it is realized that the building with moment frame only would not be effective to resist a high seismic load in San Francisco, California. Although special moment frame provides a higher response modification coefficient, additional lateral load resisting system might be required due to the ductility of the moment frame and the effective seismic weight. 12 inch thick special reinforced concrete shear all is introduced because of its stiffness. Typical beams in moment frames was experienced around 2,000 ft-kip of seismic loads when only moment frames are placed. However, after adding reinforced shear walls are provided where the elevator shaft and staircase are, the seismic loads was reduced significantly on the lateral beam.

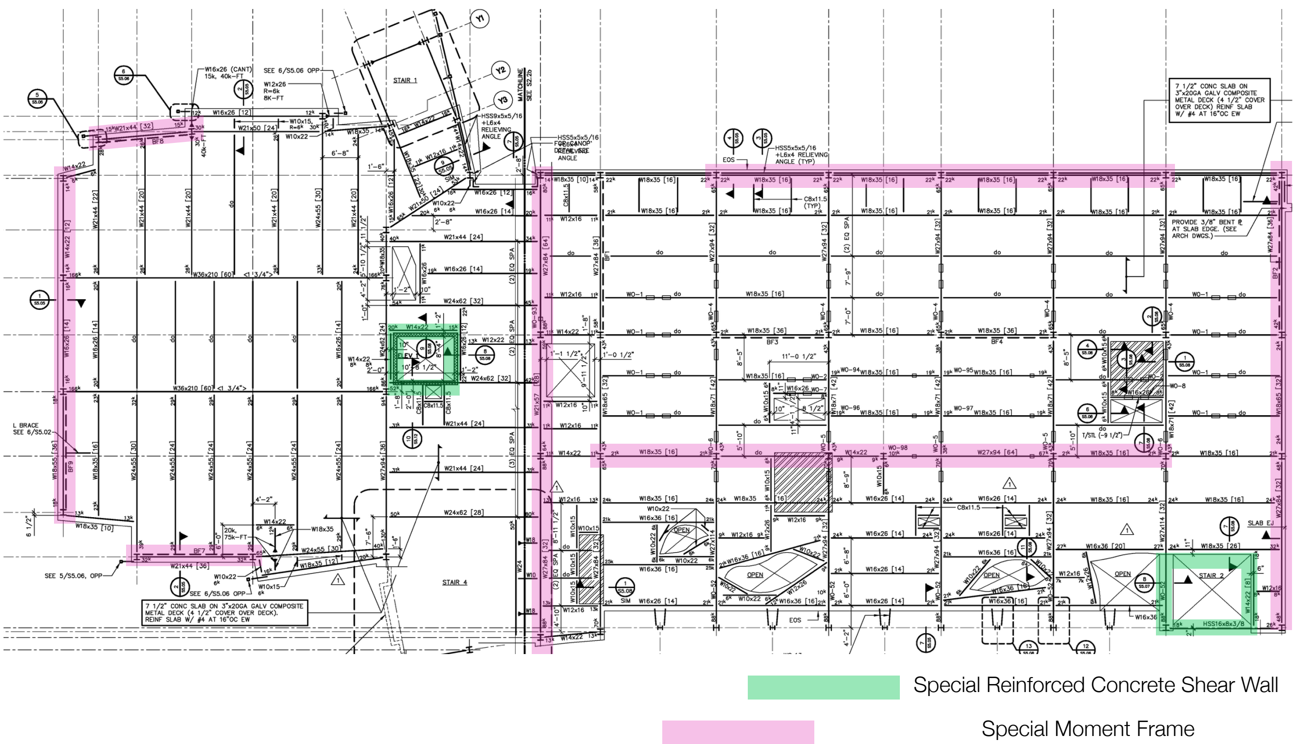
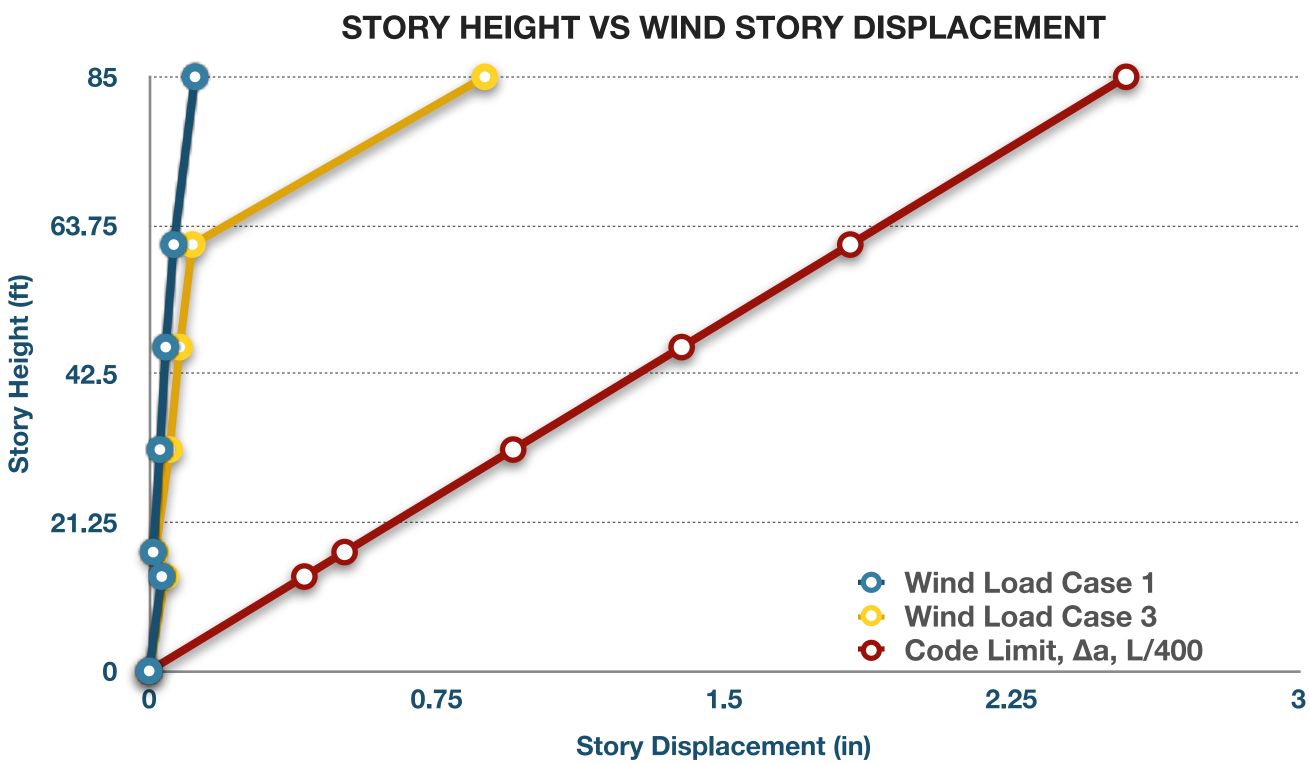


Figure 22 | Special Moment Frame Layout | S2.1a&b

WIND LOAD ANALYSIS

Wind loads are experienced in similar way as eccentrically braced frame is. Compared to seismic loads, wind loads are less considered for the structure. The story drift from wind loads are not exceed the limit specified the code. The graph shows the story drift comparison between different wind load cases.



Wind Load Base Shear and Overturning

	Fx (kips)	Fy (kips)	Overturning (ft-kips)
Wind Case 1	133.498		6055.4662
		252.922	11959.9636
Wind Case 2	100.123		4541.5995
		189.691	8969.9723

Table 8 | Wind Load Base Shear and Overturning

SEISMIC LOAD ANALYSIS

Seismic Base Shear and Overturning

The building with moment frame experiences a higher building weight than eccentrically braced frame. Since additional shear walls are placed, the weight of concrete shear walls are included to the effective seismic weight. By using the different system, the software calculates the building periods depends on the new parameter.

Level	hx (ft)	Mass (In-s^2/ft)	Weights, W (kips)	W*hx	C_{vx}	Story Forces, F_i (kips)	Story Shear, V_i (kips)
Penthouse Roof	85.00	8615.72	277.17	23559.26	0.06	128.79	128.79
4th Floor	61.00	60365.82	1941.97	118460.07	0.31	647.60	776.39
3rd Floor	46.33	79225.01	2548.67	118079.81	0.30	645.52	1421.91
2nd Floor	31.67	82151.34	2642.81	83697.75	0.22	457.56	1879.48
1st Floor	17.00	56587.12	1820.41	30946.93	0.08	169.18	2048.66
Auditorium	13.50	29013.13	933.35	12600.26	0.03	68.88	2117.54
Total		315958.14	10164.37	387344.08	**Base Shear**	2117.54	

Table 9 | Seismic Story Force Calculation - ASCE 7-10 | T= 0.484 sec

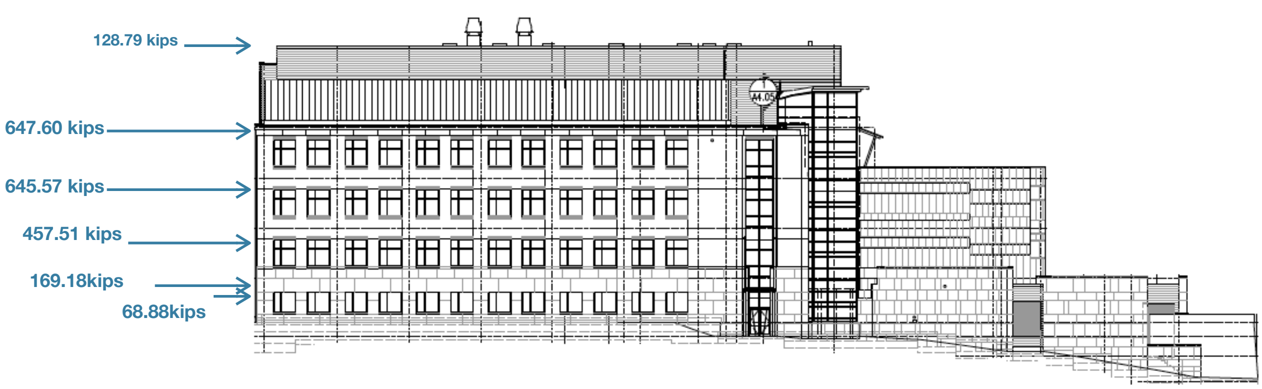


Figure 23 | Seismic Force Distribution - North Wing

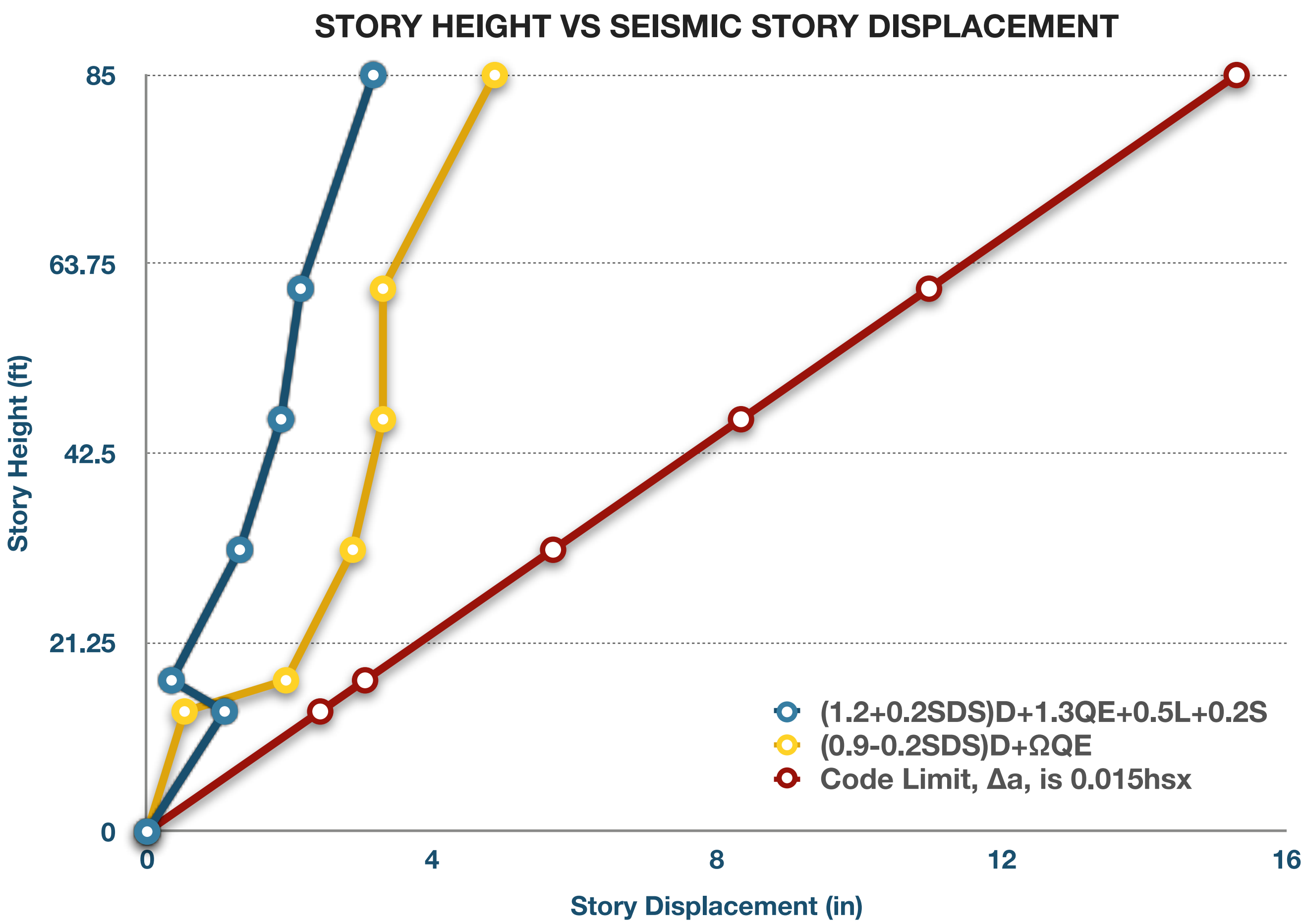
	Equivalent Lateral Force Analysis	Modal Response Spectrum Analysis	Ratio of Response Spectrum to Static Base Shear
Base Shear, kips			
X - Direction	2117.844	1878.154	0.887
Y - Direction	1913.627	1618.057	0.846
Overturning Moment, ft-kip			
X - Direction	97241.724	88518.439	0.910
Y - Direction	89012.493	206201.476	2.317

Table 10 | Seismic Load Comparison

STORY DRIFT COMPARISON

Floor	Story Height (ft)	(1.2+0.2SDS)D+1.3QE+0.5L+0.2S (in)	(0.9-0.2SDS)D+OmegaQE (in)	Code Limit, Δ_a, = $0.020h_{sx}$ (in)
Roof	**85.00**	3.18	4.88	20.40
4th Floor	**61.00**	2.15	3.31	14.64
3rd Floor	**46.33**	1.88	2.89	11.12
2nd Floor	**31.67**	1.30	1.95	7.60
1st Floor	**17.00**	0.34	0.52	4.08
Auditorium	**13.50**	1.09	1.32	3.24
Base	**0**	0.00	0.00	0.00

Table 11 | Story Drift Comparison



According to *ASCE 7-10* Table 12. 12-1, Special moment frame in risk category also defined that the allowable story drift, Δ_a, should be $0.015h_{sx}$, which is same as the eccentrically braced frames. The drifts of the governing load combinations are not exceed the code limits of allowable drift.

DESIGN PROCESS OF SPECIAL MOMENT FRAME

Through the analysis of ETABS 2013, special moment frame is designed and analyzed. In preliminary design phase, the moment frames are placed where the existing lateral systems were placed and the frames are assigned to auto-selected section, which allows the software to determine the appropriate member sizes. It is preferred to place the moment frames on the perimeter of the structure to resist lateral torsion efficiently. After first several analysis, additional moment frames were placed. However, it is realized that lateral system design with only moment frames is not effective to this building.

Instead of putting additional moment frames, special reinforced concrete shear wall is placed where the elevator shaft and stair case are. In the original design, the elevator shaft and stair case were designed masonry wall, but they are now structurally designed as part of lateral systems, except support as the elevator shaft and stair case.

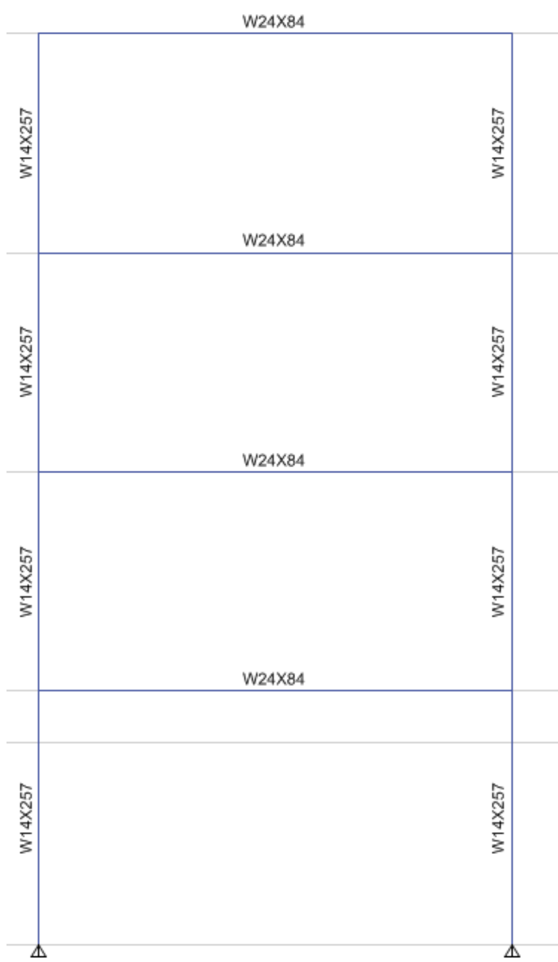


Figure 24 | Typical Special Moment Frame Design

Reduced Beam Section and Connection Design

One of the methods to place the plastic hinges to dissipate the energy is to use the reduced beam section method. The failure of the connection of structural members is one of the most critical during the seismic event. To prevent the failure of the connection, it is to force to place the plastic hinges where cross section of the beam is trimmed to fail before the connection between beam and column are failed. This reduced beam section reduces the flexural and shear capacity of the beam at the certain point.

It is recommended to use *AISC 358-10 Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* to design the connection in the special moment frames. AISC limits the section properties of beam when reduced beam section design is used.

- Beam depth is limited to W36 for rolled shaped
- Beam weight is limited to 300 lb/ft
- Beam flange thickness is limited to 1 3/4 in.
- The clear span-to-depth ratio of the beam shall be grater than 7 for special moment frames.

A typical design of special moment frame is shown above Figure ## with the appropriate member sizes.

CHAPTER 3 - BUILDING ENCLOSURE BREADTH

Due to the relocation from east coast, the United States, to San Francisco, California, the building enclosure need to reevaluate the performance in the new climate condition in San Francisco, California. In existing location of the building, building enclosure and mechanical system are controlled by the heating system. However, in new project site, the average temperature over the year is less fluctuating and staying around 50'F to 70'F.

	East Coast, USA	San Francisco, CA
Max Temperature, F	91	99
Mean Temperature, F	46.2	54.7
Min Temperature. F	-14.1	34

Table 11 | Temperature Comparison

It is predicted that the new location of the building would require a better performance in cooling process and less performance in heating around years. Since the building is classified as the biochemical laboratory building in the university, the evaluation and modification of mechanical system would be a challenge to the student in structural option.

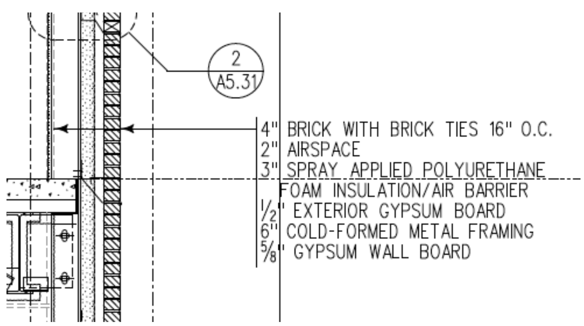


Figure 25 | Wall Assembly Section | A4.06

	Gypsum Wall Board	Metals Studs	Sprayed Polyurethane Form Insulation/Air Barirrer	Air Space	Brick
Thickness (in)	0.625	6	3	2	4
Thermal Conductivity (Btu/h*ft*F)	0.0942	0.0248	0.0144	0.1947	0.2484
Permeability (perm*in)	21.4667	106.4463	1.4483	477.037	4.3959

Table 12 | Thermal Property of Wall Assembly

Instead of analyzing the mechanical system, through this breadth, the building enclosure in both the existing and new locations will be analyzed their performances. Through *AE 542 Building Enclosure Science and Design*, WUFI 5 is introduced to perform the analysis of the moisture transportation through a building enclosure with using real weather data for the location of the building.

To evaluation the original design, a typical wall assembly is used for both locations, but the weather data will be different and chosen by the software, WUFI 5. The duration of the analysis was two years, October 2015 to October 2015. Due to lack of the climate information, the existing location is approximated to the closest city.

During the two year analysis for both locations, there were no issue of water condensation found. According to the graphs on Figure ## & ##, green circle is generated under the curve line. It explains that the interior spaces in both east coast, USA and San Francisco, CA would not experience water condensations throughout the period of the analysis, October 2015 to October 2017.

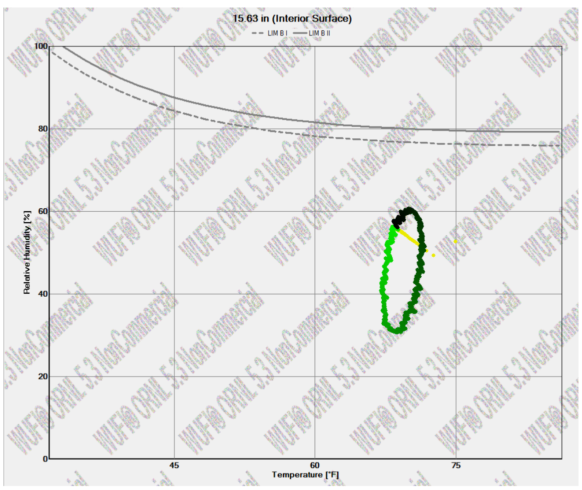


Figure 26 | Relative Humidity vs. Temperature - East Coast, USA

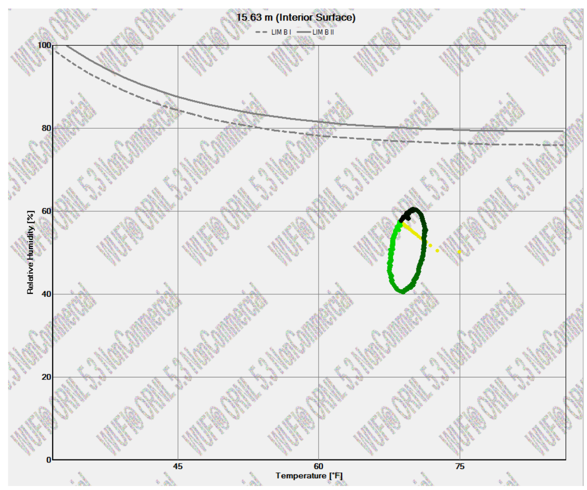


Figure 27 | Relative Humidity vs. Temperature - San Francisco, CA

Table ## - Water Content Comparison shows that less amount of wanter content value is changed at the end in San Francisco, CA changes less than East Coast, USA and it means that the existing building envelope design is performing better in the new location, San Francisco, California.

Total Water Content	East Coast, USA	San Francisco, CA
Start	0.06	0.06
End	0.28	0.09

Table 12 | Wanter Content Comparison

CHAPTER 4 - CONSTRUCTION BREADTH

The relocation of the building brings the impact on the redesign of the structural system, especially in lateral force resisting system. Due to the relocation and the effects it had on the building structure, the project cost and the project schedule for the structure would change. Since the project schedule of the existing building is not available, it is necessary to provide the approximate schedules for structural redesigns. The structural redesigns of eccentrically braced frame and special moment frames would bring different impact on the schedule.

Referencing appendix section ##, the overall sequence of the construction on the structural steel building would be similar to each other. However, in redesign of eccentrically braced frames, the project schedule would be predicted to take longer than the schedule for moment frames since the additional activities of adding braces take extra time on the construction. On the other hand, construction of the reinforced concrete shear wall would occur concurrently with the structural steel framing since the steel framing should be framed into the shear wall to resist the lateral forces. This adds an additional task to the project schedule similar to the bracing activity, however can be constructed concurrently with the steel erection whereas the bracing activity has to occur sequentially.

Beam/Girders	% of Structural Steel Elements	Quantity
W21X50	56%	3285.52
W24X84	24%	1408.08
W30X99	20%	1173.4
TOTAL	100%	5867

Table 13 | Sample Calculation of Beams/Girder Estimation

RSMeans Facilities Construction Cost Data 2015 is used to estimate the project costs and schedule.There are several assumptions made for the cost estimates and generation of the construction sequences. ETABS 2013 could generate the total amount of the structural steel members by its weights and total length of each member sizes used. The total steel for the building was calculated in ETABS 2013, which was then estimated for each floor based on its square footage. Then, it would help to estimate the production rate to generate the project schedule. Detailed calculations and project schedule are provided in appendix section ##.The construction costs including building interiors, building shells, and other factors are assumed identical to the original project. Typical structural steel member sizes, including beams, girders, braces, and columns, are selected to calculate the project costs. The calculations of roof and floor decking are also performed. The material costs and labor costs for moment connection details is difficult to estimate with RSMeans, so an additional 15 percent of costs is added to the beams, and braces if moment connection is required to both sides of members.

	Eccentrically Braced Frame	Special Moment Frame
Project Duration		
Start Date	04/07/2015	04/07/2015
Finish Date	07/15/2015	07/10/2015
Construction Cost	$2,154,381.39	$2,119,423.89

Table 14 | Project Duration and Cost Comparison

Based on the estimation of project costs on the structure and schedule, to use eccentrically braced frame would increase the project duration by five days and additional cost of $34,975.50 versus using special moment frames with special reinforced concrete shear wall. With the additional members used in eccentrically braced frames, there are more connection needed to be done on the site, increasing the schedule.

CONCLUSION

The report consisted of analysis of Life Sciences Building in east coast, the United States. After studying the existing structure in both gravity and lateral system, the design scenario was created that the identical design of Life Sciences Building is proposed to construct in San Francisco, California. San Francisco is classified as a high seismic region for structural engineers.For educational purpose, the redesign of lateral force resisting system is proposed according to the new location. Due to the redesign of the lateral system, there were several things to consider in redesigning.

To minimize the effective seismic weight, lightweight concrete slab is considered. However, since the existing building is a college laboratory building with a strict floor vibration limitation, it is suggested that the lightweight concrete slab is not effective to the building. Using lightweight concrete slab brings the reduction of building weight, however, the deduction of floor mass and shallow framing member size generate floor vibration. Therefore, using lightweight concrete requires to use deeper steel member to control the floor vibration.

The existing lateral system, steel braced frame without seismic detailed, is not appropriate to the new project site. Therefore, two new lateral force resisting system is suggested to the owner: eccentrically braced frames and special moment frames. Through the research and redesign process, each system provides its advantages and disadvantages. After the investigation of two different systems, eccentrically braced frames is more effective than special moment frames. Due to ductile behavior of special moment frames, a significant number of moment frame is required to resist the seismic loads.

In original design, the design team already laid out the lateral system carefully according to the architectural layout. With a minor modifications in architecture eccentrically braced frame would be placed and provide sufficient strength and stiffness.

In order to compare the redesign of eccentrically braced frames and special moment frames, the project cost estimates and construction schedule were generated. In the result, using eccentrically braced frames would increase the project duration by five days and addition cost $34,975.50 versus using special moment frames.

The relocation of the building suggested to evaluate the existing building enclosure system to see the existing system is sufficient enough to accommodate new climate. The analysis provides that the existing envelop system is adequate without any modification.

Although eccentrically braced frames increase the construction duration by five days and additional cost #34,975.50, it is suggested to use as lateral force resisting system to provide better performance over special moment frames.

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APPENDICES

2.2 ECCENTRICALLY BRACED FRAME

WIND LOAD STORY DRIFT COMPARISON

Floor	Story Height (ft)	Wind Load Case 1 (in)	Wind Load Case 3 (in)	Code Limit, L/400 (in)
Roof	**85.00**	0.18	0.14	2.55
4th Floor	**61.00**	0.10	0.08	1.83
3rd Floor	**46.33**	0.08	0.06	1.39
2nd Floor	**31.67**	0.06	0.04	0.95
1st Floor	**17.00**	0.03	0.02	0.51
Auditorium	**13.50**	0.08	0.05	0.41
Base	**0**	0.00	0.00	0.00

ECCENTRICALLY BRACED FRAME DESIGN

Seismological Information		
Seismic Design Category	D	
R	8.000	
Omega	2.000	
Cd	4	
I e	1.25	
S DS	1	
rho	1.3	
R M for Braced Frame Systems	1	

Story Shear		
V R	81.64	kips
V 4	527.48	kips
V 3	995.39	kips
V 2	1296.02	kips
V 1	1427.71	kips
Width of the Frame	29.9167	ft

EBF Story Drift Check						
ASCE/SEI 7 Table 12.12-1	Allowable Story Drift, delta a (in)	Story Height Below Level x, hx (ft)	Allowbla Story Drift in ASCE710	Table 12.12-1		
Roof	4.320	24.000	Delta a		0.015	hsx
4th Floor	2.640	14.667				
3rd Floor	2.64	14.67				
2nd Floor	2.64	14.67				
1st Floor	3.06	17.00				
Base						

Design Story Drift									
ASCE/SEI & Eq. 12.8-15	Story Displacement, in	Deflection at level x, dx (in)	Cd	Ie	dx = Cd*dxe/Ie	Design Story Drift (in)			
Roof	1.643524	0.4298	4	1.25	1.38		0.39	4.32	OK
4th Floor	1.213742	0.3064	4	1.25	0.98		0.23	2.64	OK
3rd Floor	0.907344	0.2355	4	1.25	0.75		-0.43	2.64	OK
2nd Floor	0.671827	0.3700	4	1.25	1.18		0.22	2.64	OK
1st Floor	0.301831	0.3018	4	1.25	0.97		0.97	3.06	OK
Base	0.0000								

EBF Link Design			Value from First-order Analysis					
P D	3.651	kips	P L	3.673	kips	P QE	84.545	kips
V D	22.867	kips	V L	28.693	kips	V QE	159.499	kips
M D	34.379	kip-ft	M L	30.303	kip-ft	M QE	122.867	kip-ft
Link Length	48	in						
Beam Span	29.9167	ft						

Beam			**Brace**			**Plate**			**Column**		
Size	w21x68		Size	w14x109		Size	wt8x28.5		Size	w12x120	
Fy	50	ksi	Fy	50	ksi	Fy	50	ksi	Fy	50	ksi
Fu	65	ksi	Fu	65	ksi	Fu	65	ksi	Fu	65	ksi
Ry	1.1	AISC Seismic Provisions Table A3.1	Ry			Ry			Ry		AISC Seismic Provisions Table A3.1
E	29000	ksi	E	29000	ksi	E	29000	ksi	E	29000	ksi
T beam	18.375		T brace	10	in						
A	20	in^2	A	32	in^2	A	8.39	in^2	A	35.2	in^2
d	21.1	in	d	14.3	in	d	8.22	in	d	13.1	in
t_w	0.43	in	t_w	0.525	in	t_w	0.43	in	t_w	0.71	in
b_f	8.27	in	b_f	14.6	in	b_f	7.12	in	b_f	12.3	in
t_f	0.685	in	t_f	0.86	in	t_f	0.715	in	t_f	1.11	in
k_{det}	1.375	in	k_{det}	2.1875	in	k_{det}		in	k_{det}	2	in
k_{des}	1.19	in	k_{des}	1.46	in	k_{des}		in	k_{des}	1.7	
k_1	0.875	in	k_1	1.5	in	k_1		in	k_1	1.1875	in
$b_f/2t_f$	6.04		$b_f/2t_f$	8.49		$b_f/2t_f$			$b_f/2t_f$	5.57	
h/t_w	43.6		h/t_w	21.7		h/t_w			h/t_w	13.7	
I_x	1480	in^4	I_x	1240	in^4	I_x		in^4	I_x	1070	in^4
Z_x	160	in^3	Z_x	192	in^3	Z_x		in^3	Z_x	186	in^4
h_o	20.4	in	h_o	13.4	in	h_o		in	h_o	12	in^3
r_y	1.8	in	r_y	3.73		r_y	1.6	in	r_y	3.13	in
phi b Mn from Table 3-10	270		p	-2.11E+00	kips^-1	y bar	1.94	in	p	8.04E-04	kips^-1
p	8.85E-03	kips^-1	bx	1.90E-03	kip-ft^-1	x bar	1.94	in	bx	1.32E-03	kip-ft^-1
			by	2.56E-03	kip-ft^-1	Gage	3.5	in	by	2.78E-03	kip-ft^-1
			Lb	19.57	ft				Lb	14.67	ft

LRFD Load Combination
(1.2+0.2*S DS)D+rho*QE+0.5L+0.2S

Required Shear Strength of the Link

Vu, kips	V D	rho	V QE	V L	V S
253.71	22.867	1.3	159.499	28.693	0

P story

P story	Pe story, kips	Rm	H (Total Story Sh	L, in	delta H, in
3680 kips	964954.33	1	1427.711425	204	0.301831

B2

B2	alpha	P story	Pe story	
1.00	1	3680	964954.33	AISCE Specification Equation A-8-6

Required Axial Strength of the Link including Second-Order Effect

Pu, kips	P D	B2	rho	P QE	P L	P S
117.28	3.651	1.00	1.3	84.545	3.673	0

B1

B1	Cm	alpha	Pr	Pel, kip	E	I	K1	Link Length, L, in
1.00	1	1	117.28	240138	29000	1480	0.875	48

Required Flexural Strength of the Link including Second-Order Effect

Mu, kip-ft	M D	B2	rho	M QE	M L	M S
223.62	34.3794	1.00	1.3	122.8673	30.303	0

Py, kips	Fy	Ag
1000	50	20

Pc = Py	Pr/Pc	
1000	0.117	AISC Seismic Provisions Section F3.5b(2)

Pr/Pc =< 0.15

Vp	Fy	A lw	d	t f	t w
254.52	50	8.48	21.1	0.685	0.43

Mp, kip-in	Fy	Z
8000	50	160

Link Length, e

Link Length, e		1.6Mp/Vp			
48	<	50.29	OK	w21x68	satisfies the requirements for moderately ductile link beam flanges. **Check AISC Seismic Design Manual Table 1-3

Available Shear Strength
Shear Yielding - AISC Seismic Provisions Eq. F3-1

Vn	Vp
254.52	254.52

Flexural Yielding - AISC Seismic Provisions Eq. F3-7

Mn	Mp	e
333.33	8000	48

phi Vn

phi Vn	phi	Vn		Vu	
229.07	0.9	254.52	>	253.71	OK

Link Rotation Angle - AISC Seismic Provisions Section F3.4a

		X	Vp	Mp	e
1.6	>	1.53	254.52	8000	48

Limit, rad		γ p, rad	θp	delta p	h, ft	
0.08	>	0.024345	0.00325504	0.664	17.00	OK

Available Compressive Strength - AISC Manual Table 6-1

Pu, kips		phi c Pn, kip	p*10^3
117.28	<	112.99	8.85

Available Flexural Strength - AUSC Manual Table 3-2 **Check Lb<Lp

Mu, kip-ft		phi b Mp, kip-ft	
223.62	<	563	OK

Combined Loading

		Pr/Pc	Pr	Pc	
0.2	>	1.038	117.28	112.99	NG

FALSE

Spec. Eq. H1-1b

		Pr/2Pc+(Mr:	Pr	Pc	Mrx	Mcx	Mry	Mcy	
1	>	0.916	117.28	112.99	223.62	563	0	563	OK

w21x68 is adequate to resist the loadas given for the link segment.

Lateral Bracing Requirments - AISC Seismic Provisions F3.4b

Pu	Ry	Fy	Z	ho
25.88	1.1	50	160	20.4

Mr, kip-in	Ry	Fy	Z
8800	1.1	50	160

β br, kip/in	phi	Mr	Cd	Lb	ho	
119.83	0.75	8800	1	48	20.4	AISC Manual Specification Appendix 6

Stiffener Requirements
Minimum Width of Each Stiffener

Stiffener Requirements

Minimum Width of Each Stiffener

		bf	tw
w min, in	3.71	8.27	0.43

Minimum Required Thickness

		tw	0.75tw	3/8 in	
t min	0.38	0.43	0.3225		0.38

Required Spacing for a Link Rotation Angle

Link Rotation Angle = 0.08 rad

		tw	d
Spacing	8.68	0.43	21.1

Link Rotation Angle = 0.02 rad or Less

		tw	d
Spacing	18.14	0.43	21.1

By interpolation

		Rotation Angle
Spacing	17.15	0.0263

AISC Seismic Provisions F3.5b(4)

Link Depth less than 25in = intermediate stiffners required on one side of the web only

Minimum Required Thickness

		tw	3/8 in
t min	0.43	0.43	0.375

Minimum Required Width of Intermediate Stiffeners on one side only

		bf	tw
w min	3.71	8.27	0.43

AISC Seismic Provisions pg 5-351-352 must be checked

EBF Beam Outside of the Link Design

P D	9.452	kips	P L	5.362	kips	P QE	84.545	kips
V D	11.138	kips	V L	12.760	kips	V QE	8.607	kips
M D	32.770	kip-ft	M L	28.588	kip-ft	M QE	69.921	kip-ft

Link Length	48	in
Beam Span	29.9167	ft

Beam

Size	w21x68		
Fy		50	ksi
Fu		65	ksi
Ry		1.1	AISC Seismic Provisions Table A3.1
E		29000	ksi

Geometric Property

A	20	in^2
d	21.1	in
t_w	0.43	in
b_f	8.27	in
t_f	0.685	in
k_{det}	1.375	in
k_{des}	1.19	in
k_1	0.875	in
$b_f/2t_f$	6.04	
h/t_w	43.6	
I_x	1480	in^4
Z_x	160	in^3
h_o	20.4	in
r_y	1.8	in

Brace

Size	w14x109	
Fy	50	ksi
Fu	65	ksi
Ry		
E	29000	ksi
T brace	10	in
A	32	in^2
d	14.3	in
t_w	0.525	in
b_f	14.6	in
t_f	0.86	in
k_{det}	2.1875	in
k_{des}	1.46	in
k_1	1.5	in
$b_f/2t_f$	8.49	
h/t_w	21.7	
I_x	1240	in^4
Z_x	192	in^3
h_o	13.4	in
r_y	3.73	
p	-2.11E+00	kips^-1
bx	1.90E-03	kip-ft^-1
by	2.56E-03	kip-ft^-1
Lb	19.571176	ft

Plate

Size	wt8x28.5		
Fy	50	ksi	
Fu	65	ksi	
Ry	0		0
E	29000	ksi	
	0		0
	0		0
A	8.39	in^2	
d	8.22	in	
t_w	0.43	in	
b_f	7.12	in	
t_f	0.715	in	
k_{det}	0	in	
k_{des}	0	in	
k_1	0	in	
$b_f/2t_f$	0		0
h/t_w	0		0
I_x	0	in^4	
Z_x	0	in^3	
h_o	0	in	
r_y	1.6	in	
y bar	1.94	in	
x bar	1.94	in	
		in	

Column

Size	w12x120	
Fy	50	ksi
Fu	65	ksi
Ry		AISC Seismic Provisions Table A3.1
E	29000	ksi
A	35.2	in^2
d	13.1	in
t_w	0.71	in
b_f	12.3	in
t_f	1.11	in
k_{det}	2	in
k_{des}	1.7	
k_1	1.1875	in
$b_f/2t_f$	5.57	
h/t_w	13.7	
I_x	1070	in^4
Z_x	186	in^4
h_o	12	in^3
r_y	3.13	in
p	9.78E-04	kips^-1
bx	1.67E-03	kip-ft^-1
by	3.51E-03	kip-ft^-1
Lb	14	ft

LRFD Load Combination
(1.2+0.2*S DS)D+rho*QE+0.5L+0.2S

Required Strength **AISC Seismic Provisions F3.3**
Adjusted link shear strength

V	Ry	Vn
307.97	1.1	254.52

Axial force in the beam outside of the link

P Emh, kips	Ry	Vn, kip	L, ft	H, ft
270.98	1.1	254.52	29.9167	17.00

The Resulting Link End Moment

M link, kip-in	Ry	Vn, kip	e, in
7391.17	1.1	254.52	48

The Portion of the Moment taken by the Beam Outside of the Link (bol)

Ratio	I bol, in^4	L bol, ft	I br, in^4	L br, ft	M link
0.663	1480	12.95835	1240	21.38	7391.17

Moment in the Beam Outside of the Link
408.47 kip-ft

Resulting Overstrength Factor

Factor	Ry	Vn	V QE
1.93	1.1	254.52	159.499

Moment in the beam outside of the Link due to the link mechanism based on the expected shear strength of the link

M Emh, kip-ft	Resulting Overstrength Factor	M QE, kip-ft
135.00	1.93	69.9206

Axial Force in the beam the outside of the link due to the link mechanism based on the expected shear strength of the link

P Emh	Resulting Overstrength Factor	P QE
163.24	1.93	84.545

Shear in the beam the outside of the link due to the link mechanism based on the expected shear strength of the link

V Emh	Resulting Overstrength Factor	V QE
16.6	1.93	8.607

Amplified Seismic Loads
LRFD Load Combination
(1.2+0.2*S DS)D+Emh+0.5L+0.2S

Required Axial Strength of the Beam outside the link

Pu, kip	P D	P Emh	P L	P S
179.16	9.452	163.24	5.362	0

Required Flexural Strength of the Beam outside the link

Mu, kip-ft	M D	M Emh	M L	M S
195.18	32.7704	135.00	28.5883	0

Required Shear Strength of the Beam outside the link

Vu, kip	V D	V Emh	V L	V S
38.59	11.138	16.6	12.76	0

Width-to-Thickness Limitations
Unbraced Length

Lb, ft	L, ft	e	dc
12.41251667	29.9167	48	13.1

Second-Order Effects

Pr		Pnt	B2	Plt	
342.40		179.16	1	163.24	
B1	Cm	alpha	Pr	Pe1	**AISC Specification Eq. A-8-3**
1.01	1	1	179.16	19093	
Pe1, kips	E	I	K1	L	
19093	29000	1480	1	148.9502	
Mu					
195.18					

Available Compressive Strength of the Beam

phi Pn	phi c	Fcr, ksi	Ag
458.87	0.9	25.49	20
Fe, ksi	E, ksi	Lb, ft	ry, in
29.94	29000	14.67	1.8
Rr*Fy/Fe	Ry	Fy, ksi	Fe, ksi
1.837	1.1	50	29.94
Fcr, ksi	Ry*Fy/Fe	Ry	Fy, ksi
25.49	1.837	1.1	50

Available Flexural Strength of the Beam — **AISC Manual Table 3-2 and 3-10**

phi b Mn, kip-ft	Ry	phi b Mn from Table 3-10
924	1.1	840

Check combined flexure and compression of the Beam — **AISC Specification Eq. H1-1a**

Pr/Pc	Pr	phi c Pn = Pc	Pr/Pc > 0.2?			
0.746	342.40	458.87	**OK**			
Pr/Pc+8/9*(Mrx/phi,b,Mnx+Mry/phi,b,Mny)	Pr/Pc	Mrx	Mcx	Mry	Mcy	Pr/Pc+8/9*(Mrx/phi,b,Mnx+Mry/phi,b,Mny) < 1.0?
0.934	0.746	195.18	924	0	924.000	**OK**
Pr/2Pc+(Mrx/phi,b,Mnx+Mry/phi,b,Mny)	Pr/Pc	Mrx	Mcx	Mry	Mcy	Pr/2Pc+(Mrx/phi,b,Mnx+Mry/phi,b,Mny) < 1.0?
0.934	0.746	195.18	924	0	924.000	**OK**

Available Shear Strength — **AISC Manual Table 3-6**

Phi,v Vn, kip	Vu, kip	
147	38.59	**OK**

EBF Brace Design

P D	15.452	kips	P L	22.134	kips	P QE	226.672	kips
V D	1.634	kips	V L	0.987	kips	V QE	2.713	kips
M D	15.230	kip-ft	M L	19.233	kip-ft	M QE	53.103	kip-ft

Link Length	48	in
Beam Span	29.9167	ft

Required Strength

V, kip	Ry	Vn, kip
349.96	1.1	254.52

Overstrength Factor

Factor	Ry	Vn, kip	V QE, kip
2.19	1.1	254.52	159.499

Moment in the brace due to the link mechanism

M Emh	Overstrength Factor	M QE
116.51	2.19	53.1028

Axial forece in the brace due to the link mechanism

P Emh	Overstrength Factor	P QE
497.35	2.19	226.672

Shear in the brace due to the link mechanism

V Emh	Overstrength Factor	V QE
5.95	2.19	2.713

Amplified Seismic Loads

LRFD Load Combination

(1.2+0.2*S DS)D+Emh+0.5L+0.2S

Required Axial Strength of the Beam outside the link

Pu, kip	P D	P Emh	P L	P S
530.05	15.452	497.35	22.134	0

Required Flexural Strength of the Beam outside the link

Mu, kip-ft	M D	M Emh	M L	M S
147.45	15.2302	116.51	19.2333	0

Required Shear Strength of the Beam outside the link

Vu, kip	V D	V Emh	V L	V S
8.73	1.634	6.0	0.987	0

Width-to-Thickness Limitations

Unbraced Length

Lb, ft
21.38

Second-Order Effects

Pr, kips	Pnt	B2	Plt		
530.05		1			
B1	**Cm**	**alpha**	**Pr**	**Pe1, kips**	**AISC Specification Eq. A-8-4**
0.665	0.6	1	530.05	5394	****Check B1<1**
Pe1, kips	**E**	**I**	**K1**	**L, ft**	
5394	29000	1240	1	21	
Mu, kip-ft					
147.45					

Combined Loading - AISC Manual Table 6-1

Size	w14x109							
Pr/Pc	p	bx	Pr, kips					
-1.12E+03	-2.11E+00	1.90E-03	530.05					
pPr+bxMrx+byMry	p	Pr, kips	bx	Mrx, kip-ft	by	Mry	**pPr+bxMrx+byMry=<1?**	**AISC Manual Eq. 6-1**
-1119.708	-2.11E+00	530.05	1.90E-03	147.45	2.56E-03	0	**OK**	

Available Shear Strength - AISC Manual Table 3-6

phi,v Vn, kip	Vu, kip	**phi,v Vn>Vu?**
258	8.73	**OK**

EBF Column Design

P D	108.707	kips	P L	72.421	kips	P QE	343.792	kips
M Dx	5.700	kip-ft	M Lx	6.051	kip-ft	M Emhx	153.978	kips
M Dy	1.246	kip-ft	M Ly	0.358	kip-ft	M Emhy	0.242	kip-ft

Link Length	48	in
Beam Span	29.9167	ft

Required Strength

Sum of the adjusted link yield strength of the links at the 3rd, 4th, and roof

V	Ry	Sum Vn	Vn for 3rd	Vn for 4th and Roof
692.75	1.1	572.52	254.52	318

Governing Load Combination for the Column in Compression

(1.2+0.2*S DS)D+Emh+0.5L+0.2S

Required Axial Compressive Strength of the Column

Pu, kip	P D	P Emh	P L	P S
881.15	108.707	692.75	72.421	0

Required Flexural Strength of the Column simultaneous with the Axial Compression

Mux, kip-ft	M Dx	M Emhx	M Lx	M Sx
164.98	5.6995	153.98	6.051	0

Required Flexural Strength of the Column simultaneous with the Axial Compression

Muy, kip-ft	M Dy	M Emhy	M Ly	M Sy
2.17	1.2461	0.24	0.3578	0

Governing Load Combination for the Column in Tension

(0.9-0.2*S DS)D+Emh+1.6H

Required Axial Tensile Strength of the Column

Pu, kip	P D	P Emh	P H
-616.65	108.707	-692.75	0

Required Flexural Strength of the Column simultaneous with the Axial Tension

Mux, kip-ft	M Dx	M Emhx	M Hx
157.97	5.6995	153.98	0

Required Flexural Strength of the Column simultaneous with the Axial Tension

Muy, kip-ft	M Dy	M Emhy	M Hy
1.11	1.2461	0.24	0

Trial Size **w12x50**

Width-to-Thickness Limitation

****AISC Seismic Provisions Table 1-3 must be checked****

Second-Order Effects

Pr, kips		Pnt	B2	Plt	
76.09		-616.65	1	692.75	
B1x	Cmx	alpha	Pr	Pe1x, kips	**AISC Specification Eq. A-8-4**
0.613	0.6	1	76.09	3692	****Check B1<1**
Pe1x, kips	E	I x	K1	L, ft	
3692	29000	1070	1	24	
B1y	Cmy	alpha	Pr	Pe1y, kips	**AISC Specification Eq. A-8-4**
0.681	0.6	1	76.09	642	****Check B1<1**
Pe1y, kips	I y	I x			
642	186	1070			

Combined Loading - AISC Manual Table 6-1

Size	w12x50						
Pr/Pc	p	bx	Pr, kips				
0.07	9.78E-04	1.67E-03	76.09				
pPr+bxMrx+byMry	p	Pr, kips	bx	Mrx, kip-ft	by	Mry	**pPr+bxMrx+byMry=<1?**
0.358	9.78E-04	76.09	1.67E-03	164.98	3.51E-03	2.17	**OK**

EBF Brace-to-Link Connection Design

P D	kips	P L	kips	P QE	kips	P S	kips	P H	0.000	kips
M Dx	kip-ft	M Lx	kip-ft	M Emhx	kips	M Sx	kip-ft	M Hx	0.000	kip-ft
M Dy	kip-ft	M Ly	kip-ft	M Emhy	kip-ft	M Sy	kip-ft	M Hy	0.000	kip-ft

Link Length 48 in
Beam Span 29.9167 ft

Brace Connection Forces

Pu, kip	Vu, kip	Mu, kip-ft
530.05	8.73	147.45

Brace Flange Force

Pfa, kips	265.02	Force in each flange due to axial load
Pff, kips	131.65	Force in each flange due to the moment
Puf, kips	396.68	Maximum resultant force in each flange

Brace Web Force

Vw 8.73

Brace Flange Connection

phi Rn, kips	phi	Fu, ksi	Ae, in^2	b f, in	t f, in	phi Rn>Puf?
612.105	0.75	65	12.556	14.6	0.86	OK

Concentrated Forces at Brace Flange Connection

Vuf	Puf	H, ft	L, ft
297.27	396.68	14.67	19.57118

Phi Rn	phi	Rn	Fyw	t w	k des	l b	phi Rn>Puf?
170.93	1	170.93	50	0.43	1.19	2	NG

If the design fails,

Beam Web Local Crippling Strength at the Brace Flange Connection

Phi Rn	phi	Rn	t w, beam	l b	d, beam	t f, beam	E	Fyw	phi Rn>Puf?
192.46	0.75	256.61	0.43	2	21.1	0.685	29000	50	NG

Flange Local Bending Strength

Phi Rn	phi	Rn	Fyf	t f	phi Rn>Puf?
131.97	0.9	146.63	50	0.685	NG

phi Rn - Limit

131.97

Beam Web Stiffners

Ps, kip	Vuf, kip	phi Rn, kip
82.65	297.27	131.97

Try	Width, in	Cornor Clips		w st, in	L st, in
	4.00	1	2.375	3	14.98

t min, in	Ps, kip	phi	Fy, ksi	w st, in
0.850	82.65	0.9	36	3

Use **0.900** x **4.00**

Minimum double-sided fillet weld size required to transfer the required stiffner load from the beam flange to the siffner

D min, sixteenths	Ps, kip	Weld Strength, kip/in	w st, in
6.60	82.65	1.392	3

Use 7.00 siteenths

Minimum double-sided fillet weld size required to transfer the stiffener force to the web

D min, sixteenths	Ps, kip	Weld Strength, kip/in	L st, in
1.98	82.65	1.392	14.98

Use 2.00 siteenths

EBF Brace-to-Beam/Column Connection Design

Brace - Axial			Brace - Shear			Beam - Axial			Beam - Shear			Collector	
P D	15.452	kips	V D	1.634	kips	P D	9.452	kips	V D	11.138	kips		
P L	22.134	kips	V L	0.987	kips	P L	5.362	kips	V L	12.760	kips		
P QE	226.672	kips	V QE	-2.713	kips	P QE	84.545	kips	V QE	-8.607	kips	P QE	kips

Link Length 48 in
Beam Span 29.9167 ft

Beam

Size	w21x68	
Fy	50	ksi
Fu	65	ksi
		AISC Seismic Provisions Table
Ry	1.1	A3.1
E	29000	ksi
T brace	18.375	
A	20	in^2
d	21.1	in
t_w	0.43	in
b_f	8.27	in
t_f	0.685	in
k_{det}	1.375	in
k_{des}	1.19	in
k_1	0.875	in
$b_f/2t_f$	6.04	
h/t_w	43.6	
I_x	1480	in^4
Z_x	160	in^3
h_o	20.4	in
r_y	1.8	in

Brace

Size	w14x109	
Fy	50	ksi
Fu	65	ksi
Ry		
E	29000	ksi
T brace	10	
A	32	in^2
d	14.3	in
t_w	0.525	in
b_f	14.6	in
t_f	0.86	in
k_{det}	2.1875	in
k_{des}	1.46	in
k_1	1.5	in
$b_f/2t_f$	8.49	
h/t_w	21.7	
I_x	1240	in^4
Z_x	192	in^3
h_o	13.4	in
r_y	3.73	
p	-2.11E+00	kips^-1
bx	1.90E-03	kip-ft^-1
by	2.56E-03	kip-ft^-1
Lb	19.57117584	ft

Plate

Size	wt8x28.5			
Fy		50	ksi	
Fu		65	ksi	
Ry		0		0
E		29000	ksi	
	0	0		0
	0	0		0
A		8.39	in^2	
d		8.22	in	
t_w		0.43	in	
b_f		7.12	in	
t_f		0.715	in	
k_{det}		0	in	
k_{des}		0	in	
k_1		0	in	
$b_f/2t_f$		0		0
h/t_w		0		0
I_x		0	in^4	
Z_x		0	in^3	
h_o		0	in	
r_y		1.6	in	
y bar		1.94	in	
x bar		1.94	in	
Gage		3.5	in	

Column

Size	w12x120	
Fy	50	ksi
Fu	65	ksi
		AISC Seismic Provisions
Ry	0	Table A3.1
E	29000	ksi
A	35.2	in^2
d	13.1	in
t_w	0.71	in
b_f	12.3	in
t_f	1.11	in
k_{det}	2	in
k_{des}		
k_1	1.1875	in
$b_f/2t_f$	5.57	
h/t_w	13.7	
I_x	1070	in^4
Z_x	186	in^4
h_o	12	in^3
r_y	3.13	in
p	9.78E-04	kips^-1
bx	1.67E-03	kip-ft^-1
by	3.51E-03	kip-ft^-1
Lb	14	ft

Brace-to-Beam/Column Connection Design

Connection Design

n min	8.33
# of bolts	10.00
Spacing, in	3
Edge distance, in	2

Initial gusset plate

Thickness, in	1
# of rows	2
# of interior bolts in one row	4
# of edge bolts in one row	1
Gage	3.5

Force from the brace to achieve equilibrium

304.20

Required Strength of the brace-to-gusset connection (Condition 1)

Ru	Pu	Vu
530.06	530.05	-3.17

****Draw the FBD of Condition 1 & 2****

Connection Design
Required number of bolts

n min	Ru	phi rn	d bolts	**ASTM A325-N in double sehar
8.33	530.06	63.6	1	

Try	# of bolts	Spacing, in	Edge distance, in
	10.00	3	2

Bearing strength of gusset plate
Initial gusset plate

Thickness, in	# of rows	# of interior bolts in one row	# of edge bolts in one row	Gage
1	2	4	1	3.5

Available Bearing Strength of the Plate at each of the interior bolts - AISC Manual Table 7-4

phi rn, kips	Bearing Strength, kip/in	thickness, in
113.0	113	1

Available Bearing Strength of the plate at each of the edge bolts - AISC Manual Table 7-5

phi rn, kips	Bearing stregnth, kip/in	thickness, in
85.9	85.9	1

Total available bearing strength of the gusset plate

phi rn, kips	Edge bolts, kips	Interior bolts, kips	Ru	**phi rn > Ru?**
1075.8	171.8	904	530.06	**OK**

Block shear strength of gusset plate

phi Rn, kips	phi	U bs	A gv, in^2
704.4	0.75	1	28

A nv, in^2	A gt, in^2	A nt, in^2	Fu*A nt, kips	0.60*Fu*A nv, kips	0.60*Fy*A gv, kips	**phi rn > Ru?**
20.1	3.50	2.38	154.38	784.88	840	**OK**

Compression buckling strength of the gusset							
Width of the Whitmore section							
Lw, in							
17.4							
Average unbraced length of the gusset plate							
****Draw the detail section**							
L, in							
6.04							
Radius of gyration of the gusset plate							
r, in	t, in						
0.289	1						
KL/r	K	L, in	r, in	**KL/r =< 25 ?**	**AISC Specification Commentary Table C-A-7.1**		
13.60	0.65	6.04	0.289	**OK**			
Design strength of the gusset							
Pn, kips	F cr	A g	Lw	t plate, in	**AISC Specification J4.4(a)**		
867.82	50	17.36	17.4	1	F cr = F y		in
phi Pn	phi	Pn	Ru	**phi Pn > Ru?**			
781.04	0.9	867.82	530.06	**OK**	**USE**	1	
Trial connection between gusset and brace							
Size	# of members						
wt8x28.5	2						
bf	T brace	bf < T brace?					
7.12	10	**OK**					
Tensile yielding stregth of WT sections							
phi Rn, kips	phi	Rn, kips	Fy, ksi	Ag, in^2	**phi Rn > Ru?**		
755.1	0.9	839	50	16.78	**OK**		
Tensile rupture strength of WT sections							
phi Rn, kips	phi	Rn, kips	Fu	Ae	U	x bar	
554.28	0.75	739.04	65	11.37	0.838	1.94	
l	An, in^2	Ag, in^2	d h, in	t f, in	**phi Rn > Ru?**		
12	13.56	8.39	1.125	0.715	**OK**		
Compressive strength of WT sections							
KL/r	K	L, in	r, in	**KL/r =< 25 ?**	**AISC Specification Commentary Table C-A-7.1**		
2.23	0.65	5.5	1.6	**OK**	**L = last bolt on brace to first bolt on gusset plate**		
					AISC Specification J4.4(a)		
phi Pn	phi	Pn	Fy	Ag	**phi Pn > Ru?**		
755.1	0.9	839	50	16.78	**OK**		
Bearing strength of the WT							
Block shear rupture strength of the WT sections							
Tensile rupture strength of the brace							
An, in^2	Ag, in^2	dh, in	tw, in				
30.82	32	1.0625	0.525				
x bar, in	t f, in	b f, in	d, in	t w, in			
2.92	0.86	14.6	14.3	0.525			
U	x bar, in	l					
0.757	2.92	12					
phi Pn, kips	phi	Pn, kips	Fu, ksi	Ae, in^2	**phi Pn > Ru?**		
1137.17	0.75	1516.22	65	23.33	**OK**		
Gusset-to-beam and column connection interface forces							
e b, in	e c, in	thetha, degree					
10.55	6.55	46.1					
****must check this from the diagram**							
****alpha and alpha bar must be same to avoid the moment at the beam or column interface**							
****beta and beta bar must be same to avoid the moment at the beam or column interface**							

alpha, in	beta, in									
11.8	9.25									
r, in	alpha, in	e c, in	beta, in	e b, in						
27.0	11.8	6.55	9.25	10.55						
V ub, kips	e b, in	r, in	Pu, kips							
207.15	10.55	27.0	530.06							
V uc, kips	beta, in	r, in	Pu, kips							
181.62	9.25	27.0	530.06							
H ub, kips	alpha, in	r, in	Pu, kips							
231.69	11.8	27.0	530.06							
H uc, kips	e c, in	r, in	Pu, kips							
128.61	6.55	27.0	530.06							
****Draw the force diagram**										
Design the weld at the gusset-to-beam interface										
l w, in	width of the gusset plate, in	thickness of the end-plate, in	corner clip, in							
20.38	22	0.625	1							
Stresses at the gusset-to-beam interface										
f uv, kip/in	H ub, kips	lw								
11.37	231.69	20.38								
f ua, kip/in	V ub, kips	lw								
10.17	207.15	20.38								
f ur, kip/in	f uv, kip/in	f ua, kip/in								
15.25	11.37	10.17								
Required strength per inch of weld										
f ur, kip/in - adjusted	weld ductility factor	f ur, kip/in								
19.07	1.25	15.25								
Required fillet weld size for two lines of weld										
D min, sixteenths	f ur, kip/in	weld strength, kip/in								
5.42	19.07	1.392								
Use double-sided	**6**	**/16**	**fillet welds to connect the gusset plate to the beam**							
Gusset rupture at weld										
Shear rupture strength of the gusset plate										
phi Rn, kips/in	phi	Rn, kips	Fu, ksi	t gusset, in						
39	0.6	65	65	1						
Yielding of the gusset										
Shear yielding strength of the gusset plate										
phi Rn, kip/in	phi	0.6*Rn, ksi	l w, in	**phi Rn > f ur?**			**phi Rn > V ub?**			
30	1	30	1	**OK**			**OK**			
						l b, in				
Beam web local yielding						20.38				
phi Rn, kips	phi	Rn, kips	Fyw, ksi	t w, in	k des, in		E, ksi	Fyw, ksi	t f, in	**phi Rn > V ub?**
502.03	1	502.03	50	0.43	1.19		29000	50	0.685	**OK**
						t f, in				
Beam web local crippling						0.685				
phi Rn, kip	phi	Rn	t w, in	l b, in	d, in					
411.54	0.75	548.72	0.43	20.38	21.1					
Weld between the gusset and the end plate										
l w, in	height of the gusset plate, in	corner clip, in								
16.5	17.5	1								
f uv, kip/in	V uc, kip	l w, in								
11.00744896	181.62	16.5								
f ua, kip/in	H uc, kip	l w, in								
7.794463859	128.61	16.5								
f ur, kip/in	fuv, kip/in	f ua, kip/in								
13.49	11.01	7.79								
Required strength per inch of weld										
f ur, kip/in - adjusted	weld ductility factor	f ur, kip/in								
16.86	1.25	13.49								
Load angle with respect to the longitudinal axis of the weld group										
theta, degree	H uc, kip/in	V uc, kip/in								
34.92	128.61	181.62								
Required fillet weld size for two lines of weld										
D min, sixteenths	f ur, kip/in	weld strength, kip/in								
4.98	16.86	1.392								
Use double-sided	**5**	**/16**	**fillet welds to connect the gusset plate to the beam**							

Check gusset rupture at gusset-to-end plate weld

phi Rn, kip/in	Ru, kip/in	phi Rn > Ru ?
30	13.49	OK

Design the weld between the beam and the end plate

Vertical force component at the beam-to-end plate itnerface

Vub + Vubeam, kip	Vub, kip	Vubeam, kip
210.24	207.15	3.09

minimum double-sided fillet weld size	D, sixteenths
	4.11

Check Beam web rupture sterngth at weld

phi Rn	Rn, kip	Fu, ksi	Anv	phi Rn > Ru ?
231.11	308.14875	65	7.90125	OK

Design the weld between the beam flanges and ten end plate

Amplified collector force, Hu, kip	Axial force in the beam outside the link, Hu	Horizontal component at the gusset-to-column	Ruf	D, sixteenth
-149.54	-30.28	128.61	-15.14	-0.88

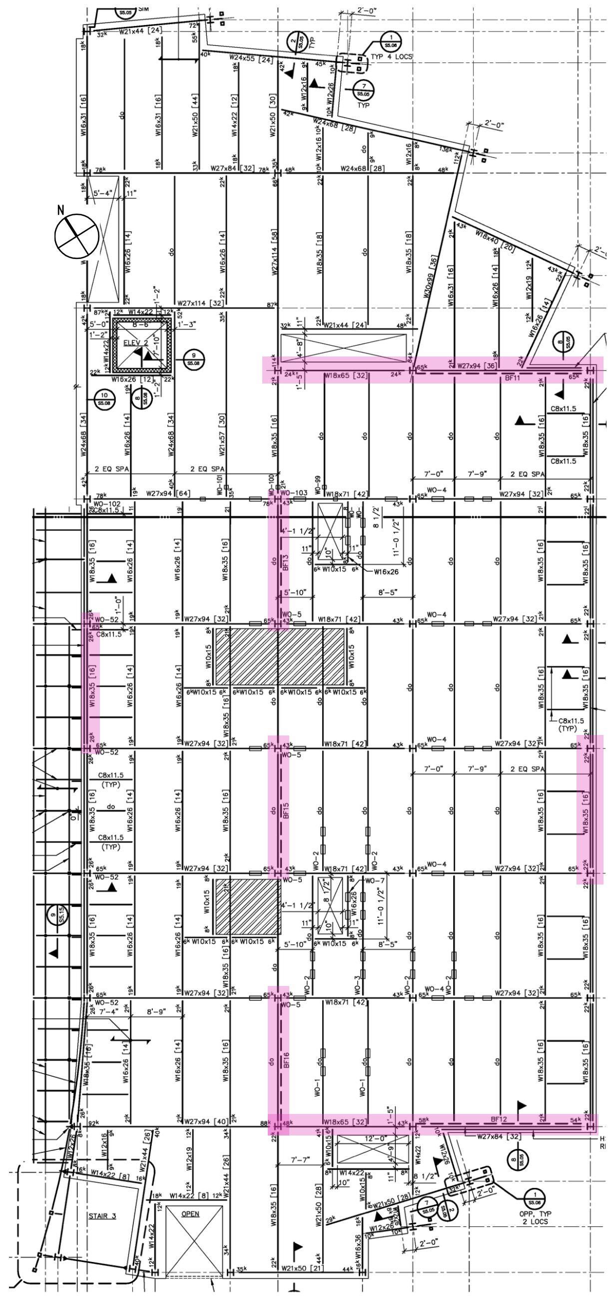
Check beam flange rupture at weld

phi Rn, kip	Rn, kip	Fu, ksi	Ae, in^2	bf, in	tf, in
276.17	368.22	65.00	5.66	8.27	0.69

Design end-plate bolts

n b	r uv, kips/bolts	Vuc, kips	Vbu, kp	Vubeam, kip
10	38.57	181.62	207.15	3.09
frv	ruv	A bolts		
49.13157176	38.57	0.785		
F'nt	Fnt, ksi	Fnv		
7.82	90	54		
phi rnt, kips	phi F'nt	Ab		
4.60	5.864047061	0.785		
Rut				
-3.78				

SUGGESTED LAYOUT OF ECCENTRICALLY BRACED FRAMES - EAST WING



Eccentrically Braced Frames

2.3 SPECIAL MOMENT FARME

SPECIAL MOMENT DESIGN

Seismological Information

Seismic Design Category	D	I e	1.25
R	8	S DS	1
Omega	3	rho	1.3
Cd	5.5	R M for Special Moment Frame	1

Beam			**Column**		
Size	**w24x84**		Size	**w14x257**	
Fy	50	ksi	Fy	50	ksi
Fu	65	ksi	Fu	65	ksi
Ry	1.1	AISC Seismic Provisions Table A3.1	Ry		
E	29000	ksi	E	29000	ksi

Geometric Property

A	24.7	in^2	A	75.6	in^2
d	24.1	in	d	16.4	in
t_w	0.47	in	t_w	1.18	in
b_f	9.02	in	b_f	16	in
t_f	0.77	in	t_f	1.89	in
k_{det}	1.6875	in	k_{det}	3.1875	in
k_{des}	1.27	in	k_{des}	2.49	
k_1	1.0625	in	k_1	1.8125	in
$b_f/2t_f$	5.86		$b_f/2t_f$	4.23	
h/t_w	45.9		h/t_w	9.71	
I_x	2370	in^4	I_x	3400	in^4
Z_x	224	in^3	Z_x	487	in^4
h_o	23.3	in	h_o	14.5	in^3
r_x	9.79	in	r_x	6.71	in
r_y	1.95	in	r_y	4.13	in
AISC Manual Table 3-2			k_1	1.8125	in

Lp	6.89	ft	p		kips^-1
Lr	20.3	ft	bx		kip-ft^-1
phi Vn	340	kips	by		kip-ft^-1
RBS Dimensions			Lb		ft
a	5.5	in	phi Pn	2010	kips
b	18	in	phi Mpx	1200	kip-ft
c	2	in	phi Vn	378	kips
R	21.25	in	d z/ 90	0.251	in
Z RBS	152.14	in^3	w z/ 90	0.14	in
Total Cut, in	4	in	(dz+wz)/90	0.39	
			t > (dz+wz)/90?	OK	
Single Angle Kicker			**Plate**		
Size	**L5x5x5/16**	AISC Steel Manual Table 4-12			
phi Pn	21.7	kips	Fy	50	ksi
A	3.07	in^2	Fu	65	ksi
E	29000	ksi	Thickness	1	in
Beam Span	31.75	ft	Width	6	in

Beam		
Beam Depth	w24x84	< W36
Beam Weight	w24x84	< 300 plf
Beam Flange Thickness	0.77	< 1.75 in
Clear Spand-to-Depth Ratio of the Beam	15.13	> 7 for SMF
Gravity Loads on the Beam		
w D	0.840	kip/ft
w L	0.600	kip/ft

ASCE/SEI & Eq. 12.8-15	**Story Heights, ft**	**Story Displacement, in**	**Deflection at level x, dx (in)**	**Allowable Story Drift, in, 0.020*hx**
Roof	24	0.703	0.139	5.76
4th Floor	14.67	0.563	0.482	3.5208
3rd Floor	14.67	0.469	0.365	3.5208
2nd Floor	14.67	0.416	-0.066	3.5208
1st Floor	17	0.482	0.482	4.08

Story Shear		
V R	128.79	kips
V 4	776.40	kips
V 3	1421.96	kips
V 2	1879.48	kips
V 1	2048.66	kips

Design Story Drift

delta design < allowable story drift? OK

Frame Stability

Px, kips	Ax, ft^2
2820	9000

Check the maximum permitted theta, theta < theta max? OK

SMF Column Strength Check

Axial Strength with amplified seismic load

Pu, kip 138.642

Axial and Flexural Strength with seismic effects		
Pu, kip	138.642	
Vu, kip	15.615	
Mu, top, ft-kip	61.8187	
Mu, bottom, ft-kip	123.3063	
Column Element Slenderness	AISC Seismic Provisions	Table D1.1
Available Compressive Strength, kips, Pu < phi Pn ?	OK	
Combined Loading Check	OK	
Available Shear Strength, Vu < phi Vn ?	OK	**AISC Table 3-2**
<u>SMF Beam Strength Check</u>		
Governing load at the face of the column		
Vu, kip	67.687	
Mu, ft-kip	563.5435	
Governing load at the centerline of the RBS		
Mu, ft-kip	563.5435	
RBS Dimensions Check		
0.5b bf < a < 0.75 b bf ?	**0.65d < b < 0.85 d?**	**0.1 b bf < c < 0.25 b bf**
OK	OK	OK
lamda f < lamda hd?	**lamda w < lamda hd ?**	
OK	OK	
Spacing of Lateral Bracing, Lb < Lb max ?	OK	
Available Flexural Strength, Lp < Lb < Lr ?	OK	
Available and Required Flexural Strength at Centerline of RBS, Mu@RBS < phi Mn@ RBS ?	OK	
Available and Required Flexural Strength at the Face of the Column, Mu < phi Mn ?	OK	
Available Shear Strength, Vu < phi Vn ?	OK	
Available Axial Strength of the Single Angle, Purb < phi Pn ?	OK	**AISC Steel Manual Table 4-12**
Probable Maximum Moment at the Center of RBS, C pr < 1.2?	OK	
<u>SMF Beam-Column Connection Design</u>		

Probable Maximum Moment at the Center of RBS, C pr < 1.2?	OK		
Free Body Diagram of Portion of Beam between RBS Cuts			
Plastic Moment of the Beam based on the expected yield stress	**or AISC Seismic Manual Table 4-2**		
Check moment at the face of the column, M f < phi M pe ?	OK		
Required Shear, Vu, of the Beam and Beam Web-to-Column Connection, Vu < phi Vn ?	OK		
Beam Web-to-Column Connection			
Required Minimum Remaining Web Depth, d min < d beam ?	OK		
d min, in		5.48	
Continuity Plate Requirments			
min t cf > t cf ?	OK		
min t cf > t cf ?	OK		
Continuity Plate?	**Not Required**		
Trial Thickness of Contiunity Plate, in		1	
Minimum Continuity Plate Thickness, in		0.77	**CJP groove welds**
Trial > min t ?	OK		
Min Contiuity Plate Width, in		3.92	
Trial Width, in		6	**OK**
Max Contiuity Plate Width, in		7.41	
Double-sided Fillet Weld, D		10.78	sixteenths
Column-Beam Relationship, sum M*pc / sum M*pb > 1.0 ?	OK		
h t, in		7.335	ft
h b, in		7.335	ft
Required Strength of the Panel Zone, Ru		806.12	kips
Pr < 0.75*Pc ?	OK		
Doubler Plate Required?	Required		
Minimum Thickness of Each Component of the Panel Zone, t		0.391	in thick Doubler Plate
Thickness of the Plate, t p, in		0.025	
Use		**0**	**in thick Doubler Plate**
Use	**1 in x 1 in**		**Clip**
Extend the doubler plate 6 in above and below the beams			

The maximum cut, in	Flange Reduction	b_f	
4.51	0.500	9.02	
% of the maximum cut	Total cut, in	Maximum Flage Cut	
89%	4	4.51	

Drift Check

Design Story Drift, in	delta xe RBS, in	delta xe, in	Allowable Story Drift, in		**delta design < allowable story drift?**
2.31	0.525	0.482	4.08	rho-ASCE7-12.3.4.2	OK
				1	

Frame Stability

Stability Coefficient, theta	Px, kips	Ax, ft^2	delta, in	Ie	Vx, kips	hsx, ft
0.0082	2820	9000	1.75	1.25	776.3960314	15

Theta/(1+theta)	theta
0.0081	0.0082

Check the maximum permitted theta

Theta max	beta		**theta < theta max?**
0.091	1	1.0 = conservative	OK

SMF Column Strength Check

Load Combination
(1.2+0.2SDS)*D+rho*QE-0.5L+0.2S

Axial Strength with amplified seismic load

Pu, kip
138.642

Axial and Flexural Strength with seismic effects

Pu, kip	138.642
Vu, kip	15.615
Mu, top, ft-kip	61.8187
Mu, bottom, ft-kip	123.3063

Column Element Slenderness	AISC Seismic Provisions	Table D1.1		
Effective Length Factor				
Kx*Lx/rx	Kx	Lx, ft	rx	
42.9	1	24	6.71	
Ky*Ly/ry	ky	Ly, ft	ry	
69.7	1	24	4.13	
Available Compressive Strength, kips	Pu, kips	Size	Effective Length	**Pu < phi Pn ?**
2010	138.642	w14x257	24	OK

Available Flexural Strength, kip-ft	AISC Manual Table 3-2
1200	

Combined Loading Check	Interaction of Compression and Flexure	Pr/Pc	Pr/Pc < 0.2 ?	Pr/2Pc+(Mrx/phi,b,Mnx+Mry/phi,b,Mny)	Pr/Pc+8/9*(Mrx/phi,b,Mnx+Mry/phi,b,Mny)	Pr	Pc	Mrx	Mcx	Mry	Mcy
OK	0.14	0.07	OK	0.14	0.16	139	2010	123.31	1200	0	1000

Available Shear Strength	Vu	**Vu < phi Vn ?**	**AISC Table 3-2**
378	15.615	OK	

SMF Beam Strength Check

Load Combination
(1.2+0.2SDS)*D+rho*QE-0.5L+0.2S

Governing load at the face of the column		**Governing load at the centerline of the RBS**		
Vu, kip		Mu, ft-kip		
67.687		563.5435		
Mu, ft-kip				
563.5435				
RBS Dimensions			**Dimension Check**	
0.5b bf	a	0.75b bf	**0.5b bf < a < 0.75 b bf ?**	
4.51	5.5	6.77	OK	
0.65d	b	0.85d	**0.65d < b < 0.85 d?**	
15.67	18.0	20.49	OK	
0.1b bf	c	0.25b bf	**0.1 b bf < c < 0.25 b bf**	
0.90	2.0	2.26	OK	
R, in	21.25			
The reduced flange width, in				
bf, RBS, in	bf	R	b	c
6.75	9.02	21.25	18	2
lamda f	bf, RBS	t f		
4.38	6.75	0.77		
lamda hd	E	Fy	**lamda f < lamda hd?**	
7.22	29000	50	OK	
lamda hd	lamda w = h/tw	**lamda w < lamda hd ?**		
59.0	45.9	OK		
Spacing of Lateral Bracing				
D1.2b Requirement				
0.086*ry*E/Fy, ft	L b	**Lb < Lb max ?**		
8.11	7.94	OK		
Available Flexural Strength				
Lb	Lp	Lr	**Lp < Lb < Lr ?**	
7.9375	6.89	20.3	OK	
Plastic Section Modulus at the Center of RBS				
Z RBS, in^3	Z x	c	t bf	d
152.14	224	2	0.77	24.1
Available and Required Flexural Strength at Centerline of RBS				
phi Mn @ RBS, ft-kip	Mn @ RBS, ft-kip	Fy	Z RBS, in^3	**Mu@RBS < phi Mn@ RBS ?**
570.5385	633.93	50	152.14	OK
Available and Required Flexural Strength at the Face of the Column				
phi Mn	Mp, ft-kip	Zx, in^3	**Mu < phi Mn ?**	
840	933.33	224	OK	
Available Shear Strength				
phi Vn, kips	**Vu < phi Vn ?**			

340	OK

Lateral Bracing

Required Brace Force

Prb, kips	Cd	h0	Mr, in-kips	Ry
10.58	1	23.3	12320	1.1

Length of the Brace

L, ft
14.80

Available Axial Strength of the Single Angle AISC Steel Manual Table 4-12

phi Pn	Purb	**Purb < phi Pn ?**
21.7	10.58	OK

Required Brace Stiffness

beta br, kip/in	phi	Mr, in-kips	Cd	Lb, in		Fy	Z
						50	224
74.0	0.75	12320	1	95.25			

Stiffness of the Angle

k, kips/in	A, in^2	E, ksi	L, ft	cos theat ^2
492.87	3.07	29000	14.80	7.47

<u>SMF Beam-Column Connection Design</u>

Gravity Loads on the Beam

w D, kip/ft	w L, kip/ft					h0
0.840	0.600					23.3

Beam Requirements

Beam Depth	w24x84	< W36		beta br < k ?
Beam Weight	w24x84	< 300 plf		OK
Beam Flange Thickness		0.77 < 1.75 in		
Clear Spand-to-Depth Ratio of the Beam		15.13 > 7 for SMF		

Probable Maximum Moment at the Center of RBS

C pr	Fy	Fu	**C pr < 1.2?**
1.15	50	65	OK
M pr, ft-kips	Z RBS	Ry	
801.92	152.14	1.1	

Shear at the Center of the RBS at Each End of the Beam

Gravity Load on the Beam

w u, kip/ft	w D, kip/ft	w L, kip/ft
1.31	0.840	0.600

Distance from the Column Face to the Center of the RBS cut

S h, in	a, in	b, in
14.5	5.5	18

Distance between Center of RBS cuts

L h, in	L, ft	d col, in	S h, in
336	31.75	16.4	14.5

V RBS, kip	Mpr, ft-kip	L h, in	w u, kip/ft
75.64	801.92	336	1.31
V' RBS, kip	Mpr, ft-kip	L h, in	w u, kip/ft

39.06	801.92	336	1.31

Free Body Diagram of Portion of Beam between RBS Cuts

Probable Maximum Moment at the Face of the Column

M f, in-kip	M pr, ft-kip	V RBS, kip	S h, in
10719.84	801.92	75.64	14.5
M' f, in-kip	M pr, ft-kip	V' RBS, kip	S h, in
10189.43	801.92	39.06	14.5

Plastic Moment of the Beam based on the expected yield stress	**or AISC Seismic Manual Table 4-2**		
M pe, in-kip	Ry	Fy, ksi	Zx, in^3
12320	1.1	50	224

Check moment at the face of the column

M f, in-kip	phi M pe	**M f < phi M pe ?**
10719.84	12320	OK

Required Shear, Vu, of the Beam and Beam Web-to-Column Connection

phi Vn	Vu, kips	V RBS, kips	w u, kip/ft	S h, in	**Vu < phi Vn ?**
340	77.22	75.64	1.31	14.5	OK

Beam Web-to-Column Connection

Required Minimum Remaining Web Depth

d min, in	Vu, kips	Fy, ksi	tw, in	Cv	**d min < d beam ?**
5.48	77.22	50	0.47	1	OK

Continuity Plate Requirments

min t cf, in - Provisions EQ. E3-8	R yb	R yc	b bf, in	t bf, in	F yb, ksi	F yc, ksi	**min t cf > t cf ?**
1.41	1.1	1.1	9.02	0.77	50	50	OK

min t cf, in - Provisions EQ. E3-9	b bf, in	**min t cf > t cf ?**
1.50	9.02	OK

Continuity Plate?

Not Required

Trial Thickness of Contiunity Plate, in	**Minimum Continuity Plate Thickness, in**		**Trial > min t ?**
1	0.77	**CJP groove welds**	OK

Minimum Contiuity Plate Width, in

2.42

Length of the Contact between Each Continuity Plate and the Column Flange, in	k1	Cornor Clip
0.69	1.8125	0.5

Min Contiuity Plate Width, in	**Trial Width, in**	**Max Contiuity Plate Width, in**	
3.92	6	7.41	**OK**

Contact width, in	k1	Cornor Clip
4.2775	1.8125	0.5

Design Tensile Strength, kips							
phi t Tn	phi t	Fy	Contact Area				
384.98	0.9	50	4.2775				
Contact width with the Web, in	Cornor Clip, in						
7.025	4.69						
Design Shear Strength of the Contiuity Plate, phi Vn, kips	Fy	Contact Area, in^2					
210.75	50	7.025					
Design Strength of the Panel Zone, phi Rn, kips	Fy, ksi	d c, in	t w, in	b cf, in	t cf, in	d b, in	
794.00	50	16.4	1.18	16	1.89	24	
Tn, kips	Ry	Fy, ksi	b f, in	t f, in			
763.99	1.1	50	9.02	0.77			
Design Load for Continuity Plate to Column Web Weld							
210.75							
Double-sided Fillet Weld, D							
10.78	sixteenths						
Column-Beam Relationship							
sum M*pc, in-kip	Z xt, in^3	Fy, ksi	P uc, kips	Ag, in^2	h t, in	d b, in	h b, in
49661.71	487	50	138.642	75.6	288	24	176.04
sum M*pb, in-kip	Mpr, in-kip	Muv, in-kip	V RBS, kips	V' RBS, kips	a, in	b, in	dc, in
21849.79	9623.08	2603.62	75.64	39.06	5.5	18	16.4
sum M*pc / sum M*pb	**sum M*pc / sum M*pb > 1.0 ?**						
2.27	OK						
Panel Zone Check							
Vc, kips	M f, in-kip	M' f, in-kip	h b, ft	h t, ft			
90.12	10719.84	10189.43	24	14.67			
Required Strength of the Panel Zone							
Ru, kips	sum M f, in-kip	d b, in	t f, in	Vc, kips			
806.12	20909.27	24.1	0.77	90.12			
Pr < 0.75*Pc							
Pr, kips	0.75*Pc, kips	Fy, ksi	Ag, in^2	**Pr < 0.75*Pc ?**			
138.642	2835	50	75.6	OK			
phi Rn, or AISC Seismic Manual Table 4-2	Fy, ksi	d c, in	t w, in	b cf, in	t cf, in	d b, in	**Ru < phi Rn ?**
799.33	50	16.4	1.18	16	1.89	24	NG
Doubler Plate Required?							
Required							
Minimum Thickness of Each Component of the Panel Zone, t, in	d z/90, in	w z/90, in					
0.391	0.251	0.14					
Thickness of the Plate, t p, in	Ru, kips	Fy, ksi	d c, in	t w, in	b cf, in	t cf, in	d b, in
0.025	806.12	50	16.4	1.18	16	1.9	24.1
Use	0	**in thick Doubler Plate**					
Use	**1 in x 1 in**	**Clip**					
Extend the doubler plate 6 in above and below the beams							

3.0 BUILDING ENCLOSURE BREADTH

EXTERIOR TEMPERATURE AND RELATIVE HUMIDITY GRAPH

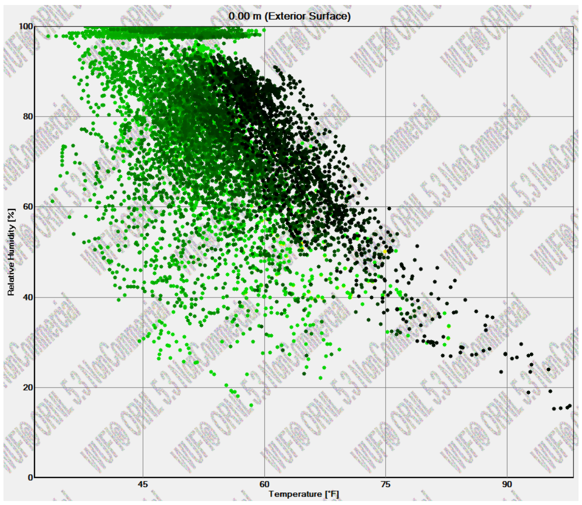


Figure ## | Exterior Temperature and Relative Humidity in East Coast USA

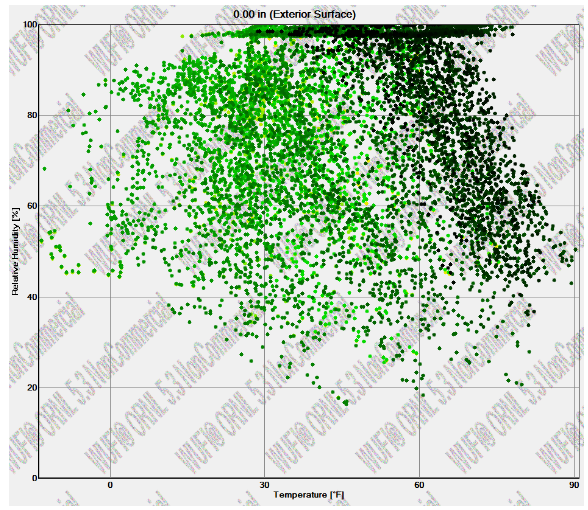
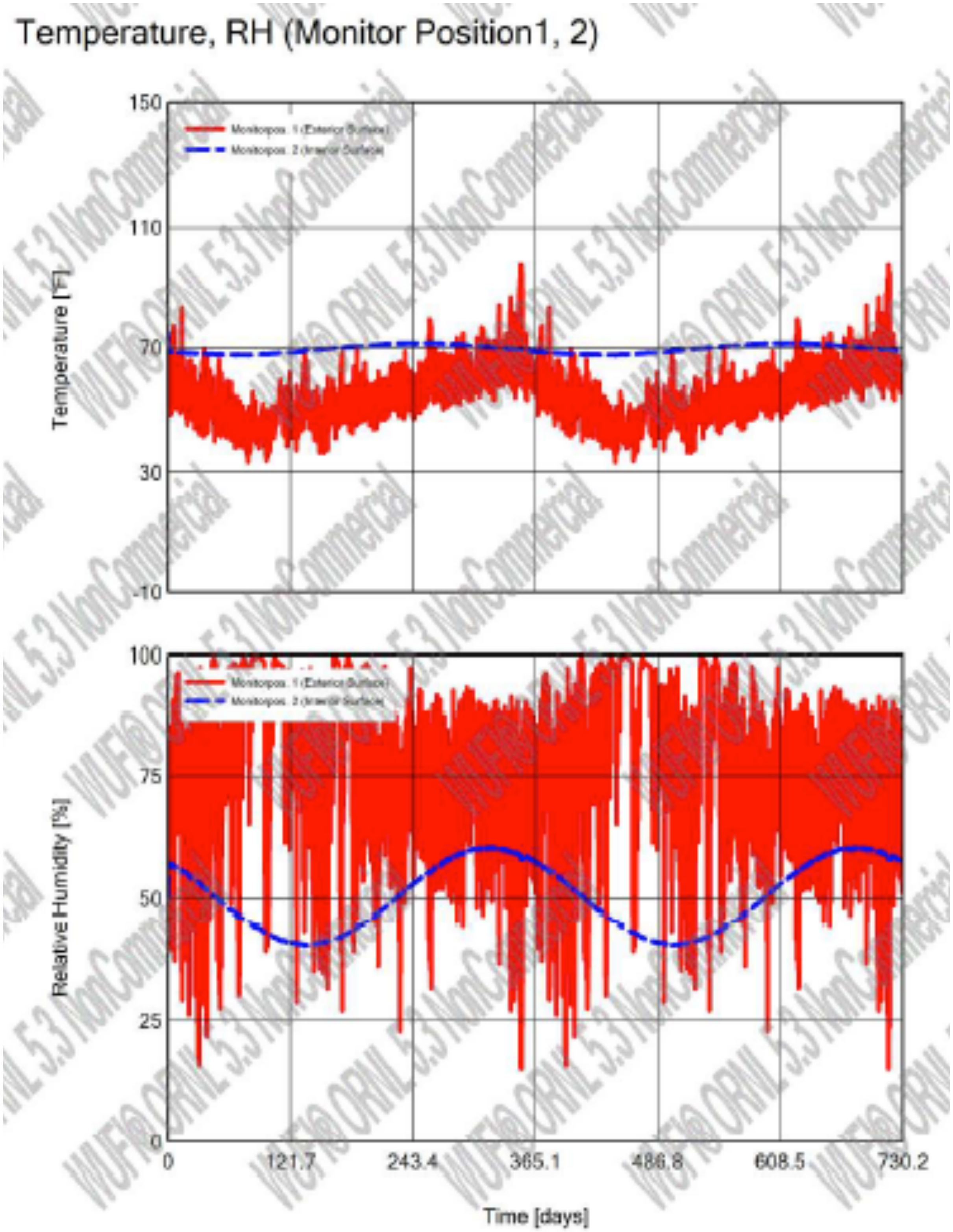


Figure ## | Exterior Temperature and Relative Humidity in East Coast

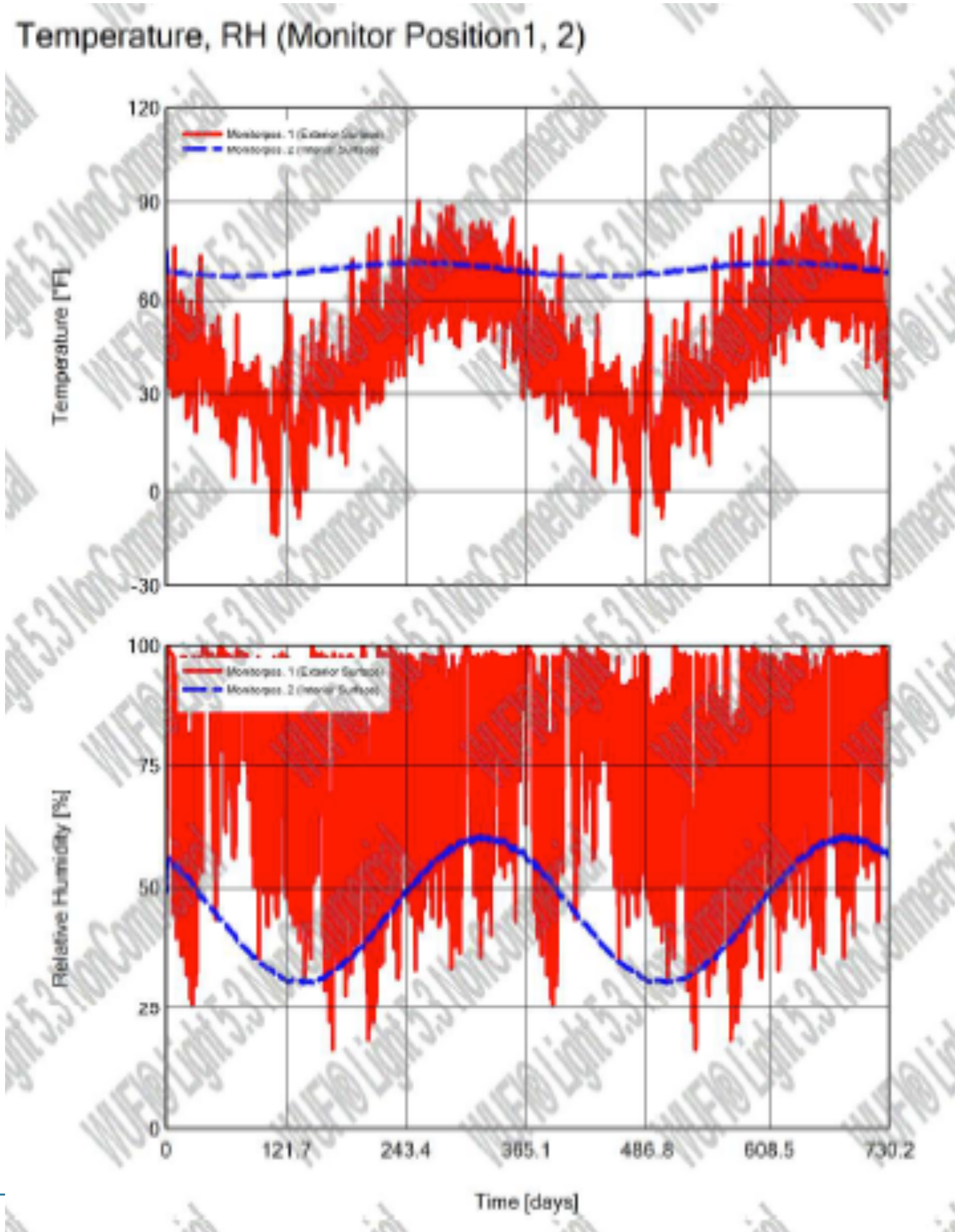
WATER CONTENTS FOR INDIVIDUAL MATERIALS

	East Coast, USA		San Francisco, CA	
Material	**Start**	**End**	**Start**	**End**
Gypsum Board	0.19	3.32	0.19	0.34
Batt Insulation	0.00	0.08	0.00	0.02
Sprayed Polyurethane Foam	0.02	0.05	0.02	0.04
Air Space	0.03	0.05	0.03	0.04
Bick	0.13	0.13	0.13	0.13

San Francisco



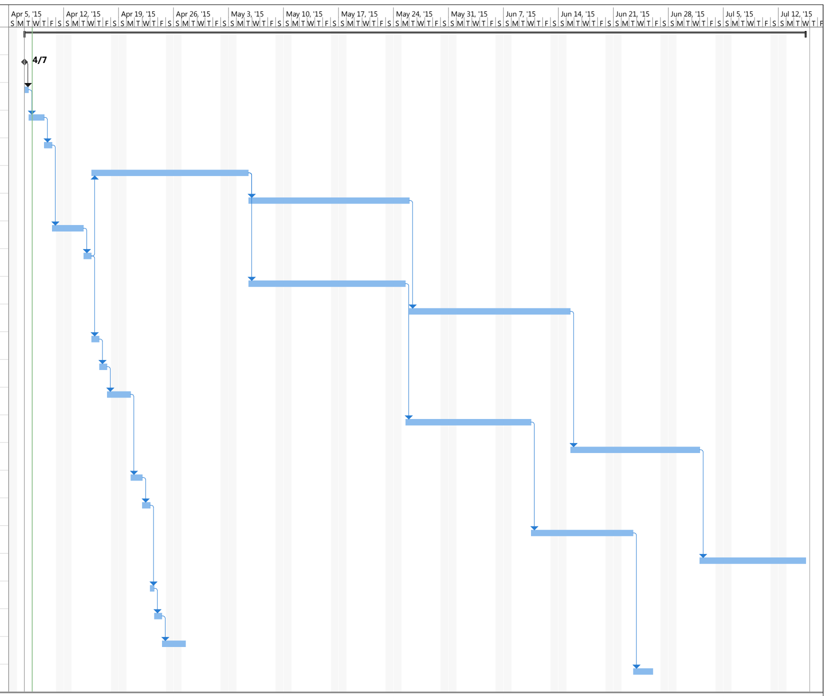
East Coast, USA



4.0 CONSTRUCTION BREADTH

ECCENTRICALLY BRACED FRAME

ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors
1			**Segment A Structure**	**71.5 days**	**Tue 4/7/15**	**Wed 7/15/15**	
2			Segement A Structure Starts	0 days	Tue 4/7/15	Tue 4/7/15	
3			Lvl LL & 1 Columns	0.5 days	Tue 4/7/15	Tue 4/7/15	2
4			Lvl 1 Beams/Girders	2 days	Tue 4/7/15	Thu 4/9/15	3
5			Lvl 1 Bracing	1 day	Thu 4/9/15	Fri 4/10/15	4
6			Lvl 1 Metal Deck	14 days	Wed 4/15/15	Tue 5/5/15	9
7			Lvl 1 FRP Slab	14.5 days	Tue 5/5/15	Mon 5/25/15	6
8			Lvl 2 Beams/Girders	2 days	Fri 4/10/15	Tue 4/14/15	5
9			Lvl 2 Bracing	1 day	Tue 4/14/15	Wed 4/15/15	8
10			Lvl 2 Metal Deck	14 days	Tue 5/5/15	Mon 5/25/15	6
11			Lvl 2 FRP Slab	14.5 days	Tue 5/26/15	Mon 6/15/15	7
12			Lvl 2 & 3 Columns	1 day	Wed 4/15/15	Thu 4/16/15	9
13			Lvl 3 Beams/Girders	1 day	Thu 4/16/15	Fri 4/17/15	12
14			Lvl 3 Bracing	1 day	Fri 4/17/15	Mon 4/20/15	13
15			Lvl 3 Metal Deck	12 days	Mon 5/25/15	Wed 6/10/15	10
16			Lvl 3 FRP Slab	12.5 days	Mon 6/15/15	Wed 7/1/15	11
17			Lvl 4 Beams/Girders	1.5 days	Mon 4/20/15	Tue 4/21/15	14
18			Lvl 4 Bracing	1 day	Wed 4/22/15	Wed 4/22/15	17
19			Lvl 4 Metal Deck	9 days	Wed 6/10/15	Tue 6/23/15	15
20			Lvl 4 FRP Slab	9.5 days	Thu 7/2/15	Wed 7/15/15	16
21			Lvl 4 Columns	0.5 days	Thu 4/23/15	Thu 4/23/15	18
22			Roof Lvl Beams/Girders	1 day	Thu 4/23/15	Fri 4/24/15	21
23			Roof Lvl Bracing	1 day	Fri 4/24/15	Mon 4/27/15	22
24			Roof Lvl Roof Deck	2.5 days	Tue 6/23/15	Thu 6/25/15	19



Project: Brace Frame with mom
Date: Wed 4/8/15

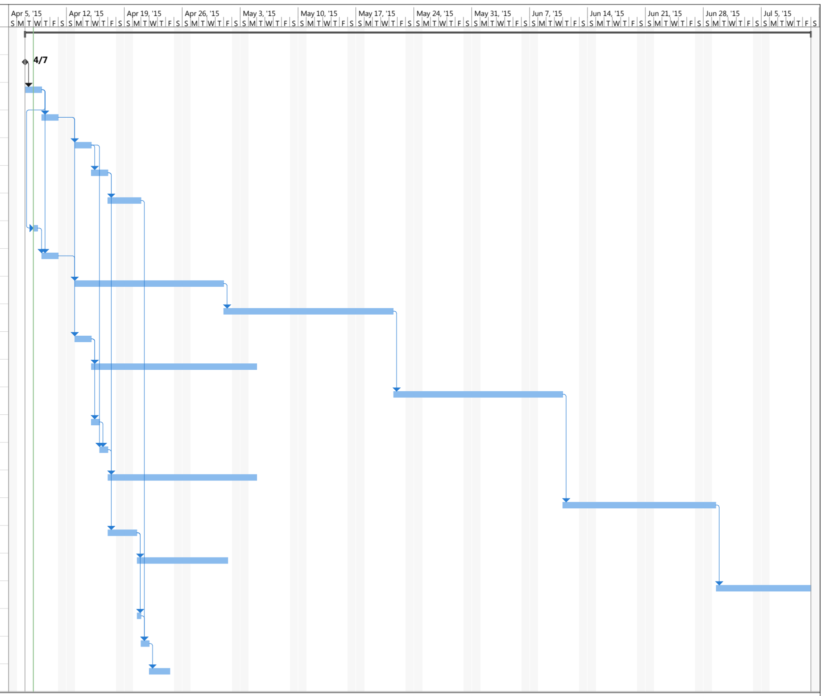
Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | External Tasks | External Milestone | Deadline | Progress | Manual Progress

Page 1

	Description	% of Structure	Unit	Quantity	Daily Output	Labor Hours	Material Cost/Unit	Material Cost ($)	Labor Cost/Unit	Labor Cost ($)	Equipment Cost/Unit	Equipment Cost ($)	Total Cost/Unit	Total Cost ($)	Total Incl O&P Cost/Unit	Total Incl O&P Cost ($)
12 23.75	Columns							$ -		$ -		$ -		$ -		$ -
	W12X50	65%	L.F	499.2	1032	0.054	73	$ 36,441.60	2.81	$ 1,402.75	1.46	$ 728.83	77.27	$ 38,573.18	86.5	$ 43,180.80
	W14X120	35%	L.F	268.8	960	0.058	175	$ 47,040.00	3.02	$ 811.78	1.57	$ 422.02	179.59	$ 48,273.79	199	$ 53,491.20
12 23.75	Beams / Girders							$ -		$ -		$ -		$ -		$ -
	W21X50	56%	L.F	3285.52	1064	0.075	73	$ 239,842.96	3.93	$ 12,912.09	1.56	$ 5,125.41	78.49	$ 257,880.46	88.5	$ 290,768.52
	W24X84	24%	L.F	1408.08	1080	0.074	122	$ 171,785.76	3.87	$ 5,449.27	1.53	$ 2,154.36	127.4	$ 179,389.39	144	$ 202,763.52
	W30X99	20%	L.F	1173.4	1200	0.067	144	$ 168,969.60	3.48	$ 4,083.43	1.38	$ 1,619.29	148.86	$ 174,672.32	167	$ 195,957.80
12 23.75	Bracing							$ -		$ -		$ -		$ -		$ -
	W14X120	100%	L.F	837.5	720	0.078	175	$ 146,562.50	4.02	$ 3,366.75	2.1	$ 1,758.75	181.12	$ 151,688.00	201	$ 168,337.50
03 30 53.40	Concrete Topping							$ -		$ -		$ -		$ -		$ -
	Lightweight, 110# per C.F., 2-1/2" thick floor fill		S.F	131650	2585	0.022	1.46	$ 192,209.00	0.91	$ 119,801.50	0.28	$ 36,862.00	2.65	$ 348,872.50	3.38	$ 444,977.00
								$ -		$ -		$ -		$ -		$ -
05 31 13.50	Floor Decking							$ -		$ -		$ -		$ -		$ -
	3" - 16 ga		S.F	131650	2700	0.012	3.87	$ 509,485.50	3.87	$ 509,485.50	0.6	$ 78,990.00	0.05	$ 6,582.50	5.45	$ 717,492.50
05 31 23.50	Roof Decking							$ -		$ -	0.63	$ -	0.05	$ -		$ -
	3" - N - 16 ga - over 500 squares		S.F	6993	3400	0.009	4	$ 27,972.00	0.5	$ 3,496.50	0.04	$ 279.72	4.54	$ 31,748.22	5.35	$ 37,412.55
															Total	**$ 2,154,381.39**

SPECIAL MOMENT FRAME

ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
1		**Segment A Structure**	**69 days**	**Tue 4/7/15**	**Fri 7/10/15**	
2		Segement A Structure Starts	0 days	Tue 4/7/15	Tue 4/7/15	
3		LL Elevator & Stair Shear Walls	2 days	Tue 4/7/15	Wed 4/8/15	2
4		Lvl 1 Elevator & Stair Shear Walls	2 days	Thu 4/9/15	Fri 4/10/15	3
5		Lvl 2 Elevator & Stair Shear Walls	2 days	Mon 4/13/15	Tue 4/14/15	4
6		Lvl 3 Elevator & Stair Shear Walls	2 days	Wed 4/15/15	Thu 4/16/15	5
7		Lvl 4 Elevator & Stair Shear Walls	2 days	Fri 4/17/15	Mon 4/20/15	6
8		Lvl LL & 1 Columns	0.5 days	Wed 4/8/15	Wed 4/8/15	3FS-1 day
9		Lvl 1 Beams/Girders	2 days	Thu 4/9/15	Fri 4/10/15	8,3
10		Lvl 1 Metal Deck	14 days	Mon 4/13/15	Thu 4/30/15	9
11		Lvl 1 FRP Slab	14.5 days	Fri 5/1/15	Thu 5/21/15	10
12		Lvl 2 Beams/Girders	2 days	Mon 4/13/15	Tue 4/14/15	4
13		Lvl 2 Metal Deck	14 days	Wed 4/15/15	Mon 5/4/15	12
14		Lvl 2 FRP Slab	14.5 days	Thu 5/21/15	Wed 6/10/15	11
15		Lvl 2 & 3 Columns	1 day	Wed 4/15/15	Wed 4/15/15	12
16		Lvl 3 Beams/Girders	1 day	Thu 4/16/15	Thu 4/16/15	15,5
17		Lvl 3 Metal Deck	12 days	Fri 4/17/15	Mon 5/4/15	16
18		Lvl 3 FRP Slab	12.5 days	Thu 6/11/15	Mon 6/29/15	14
19		Lvl 4 Beams/Girders	1.5 days	Fri 4/17/15	Mon 4/20/15	6
20		Lvl 4 Metal Deck	9 days	Mon 4/20/15	Fri 5/1/15	19
21		Lvl 4 FRP Slab	9.5 days	Mon 6/29/15	Fri 7/10/15	18
22		Lvl 4 Columns	0.5 days	Mon 4/20/15	Mon 4/20/15	19
23		Roof Lvl Beams/Girders	1 day	Tue 4/21/15	Tue 4/21/15	22,7
24		Roof Lvl Roof Deck	2.5 days	Wed 4/22/15	Fri 4/24/15	23



Project: Moment Frame with Sh
Date: Wed 4/8/15

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | External Tasks | External Milestone | Deadline | Progress | Manual Progress

Page 1

							Material		Labor		Equipment		Total		Total Incl O&P	
	Description	% of Structure	Unit	Quantity	Daily Output	Labor Hours	Cost/Unit	Cost ($)	Cost/Unit	Cost ($)	Cost/Unit	Cost ($)	Cost/Unit	Cost ($)	Cost/Unit	Cost ($)
12 23.75	Columns							$ -		$ -		$ -		$ -		$ -
	W12X50	65%	L.F	499.2	1032	0.054	73	$ 36,441.60	2.81	$ 1,402.75	1.46	$ 728.83	77.27	$ 38,573.18	86.5	$ 43,180.80
	W14X120	35%	L.F	268.8	960	0.058	175	$ 47,040.00	3.02	$ 811.78	1.57	$ 422.02	179.59	$ 48,273.79	199	$ 53,491.20
12 23.75	Beams / Girders							$ -		$ -		$ -		$ -		$ -
	W21X50	56%	L.F	3285.52	1064	0.075	73	$ 239,842.96	3.93	$ 12,912.09	1.56	$ 5,125.41	78.49	$ 257,880.46	88.5	$ 290,768.52
	W24X84	24%	L.F	1408.08	1080	0.074	122	$ 171,785.76	3.87	$ 5,449.27	1.53	$ 2,154.36	127.4	$ 179,389.39	144	$ 202,763.52
	W30X99	20%	L.F	1173.4	1200	0.067	144	$ 168,969.60	3.48	$ 4,083.43	1.38	$ 1,619.29	148.86	$ 174,672.32	167	$ 195,957.80
	W14X120	100%	L.F	837.5	720	0.078	175	$ 146,562.50	4.02	$ 3,366.75	2.1	$ 1,758.75	181.12	$ 151,688.00	201	$ 168,337.50
03 30 53.40	Concrete Shear Wall - Elevator tower							$ -		$ -		$ -		$ -		$ -
	12" thick		C.Y.	96.5	40	5	154	$ 14,861.00	234	$ 22,581.00	18.8	$ 1,814.20	406.8	$ 39,256.20	570	$ 55,005.00
03 30 53.40	Concrete Shear Wal - Stair tower							$ -		$ -		$ -		$ -		$ -
	12" thick		C.Y.	137.5	40	5	154	$ 21,175.00	234	$ 32,175.00	18.8	$ 2,585.00	406.8	$ 55,935.00	570	$ 78,375.00
03 30 53.40	Concrete Topping							$ -		$ -		$ -		$ -		$ -
	Lightweight, 110# per C.F., 2-1/2" thick floor fill		S.F	131650	2585	0.022	1.46	$ 192,209.00	0.91	$ 119,801.50	0.28	$ 36,862.00	2.65	$ 348,872.50	3.38	$ 444,977.00
								$ -		$ -		$ -		$ -		$ -
05 31 13.50	Floor Decking							$ -		$ -		$ -		$ -		$ -
	3" - 16 ga		S.F	131650	2700	0.012	3.87	$ 509,485.50	3.87	$ 509,485.50	0.6	$ 78,990.00	0.05	$ 6,582.50	5.45	$ 717,492.50
05 31 23.50	Roof Decking							$ -		$ -	0.63	$ -	0.05	$ -		$ -
	3" - N - 16 ga - over 500 squares		S.F	6993	3400	0.009	4	$ 27,972.00	0.5	$ 3,496.50	0.04	$ 279.72	4.54	$ 31,748.22	5.35	$ 37,412.55
															Total	**$ 2,287,761.39**

PROJECT DURATION CALCULATION

							Material		Labor		Equipment		Total		Total Incl O&P	
	Description	% of Structure	Unit	Quantity	Daily Output	Labor Hours	Cost/ Unit	Cost ($)	Cost/Unit	Cost ($)	Cost/Unit	Cost ($)	Cost/ Unit	Cost ($)	Cost/ Unit	Cost ($)
12 23.75	**Columns**							$ -		$ -		$ -		$ -		$ -
	W12X50	65%	L.F	499.2	1032	0.054	73	$ 36,441.60	2.81	$ 1,402.75	1.46	$ 728.83	77.27	$ 38,573.18	86.5	$ 43,180.80
	W14X120	35%	L.F	268.8	960	0.058	175	$ 47,040.00	3.02	$ 811.78	1.57	$ 422.02	179.6	$ 48,273.79	199	$ 53,491.20
12 23.75	**Beams / Girders**							$ -		$ -		$ -		$ -		$ -
	W21X50	56%	L.F	3285.52	1064	0.075	73	$ 239,842.96	3.93	$ 12,912.09	1.56	$ 5,125.41	78.49	$ 257,880.46	88.5	$ 290,768.52
	W24X84	24%	L.F	1408.08	1080	0.074	122	$ 171,785.76	3.87	$ 5,449.27	1.53	$ 2,154.36	127.4	$ 179,389.39	144	$ 202,763.52
	W30X99	20%	L.F	1173.4	1200	0.067	144	$ 168,969.60	3.48	$ 4,083.43	1.38	$ 1,619.29	148.9	$ 174,672.32	167	$ 195,957.80
03 30 53.40	**Concrete Shear Wall - Elevator tower**							$ -		$ -		$ -		$ -		$ -
	12" thick		C.Y.	96.5	40	5	154	$ 14,861.00	234	$ 22,581.00	18.8	$ 1,814.20	406.8	$ 39,256.20	570	$ 55,005.00
03 30 53.40	**Concrete Shear Wal - Stair tower**							$ -		$ -		$ -		$ -		$ -
	12" thick		C.Y.	137.5	40	5	154	$ 21,175.00	234	$ 32,175.00	18.8	$ 2,585.00	406.8	$ 55,935.00	570	$ 78,375.00
03 30 53.40	**Concrete Topping**							$ -		$ -		$ -		$ -		$ -
	Lightweight, 110# per C.F., 2-1/2" thick floor fill		S.F	131650	2585	0.022	1.46	$ 192,209.00	0.91	$ 119,801.50	0.28	$ 36,862.00	2.65	$ 348,872.50	3.38	$ 444,977.00
								$ -		$ -		$ -		$ -		$ -
05 31 13.50	**Floor Decking**							$ -		$ -		$ -		$ -		$ -
	3" - 16 ga		S.F	131650	2700	0.012	3.87	$ 509,485.50	3.87	$ 509,485.50	0.6	$ 78,990.00	0.05	$ 6,582.50	5.45	$ 717,492.50
05 31 23.50	**Roof Decking**							$ -		$ -	0.63	$ -	0.05	$ -		$ -
	3" - N - 16 ga - over 500 squares		S.F	6993	3400	0.009	4	$ 27,972.00	0.5	$ 3,496.50	0.04	$ 279.72	4.54	$ 31,748.22	5.35	$ 37,412.55
															Total	**$ 2,119,423.89**