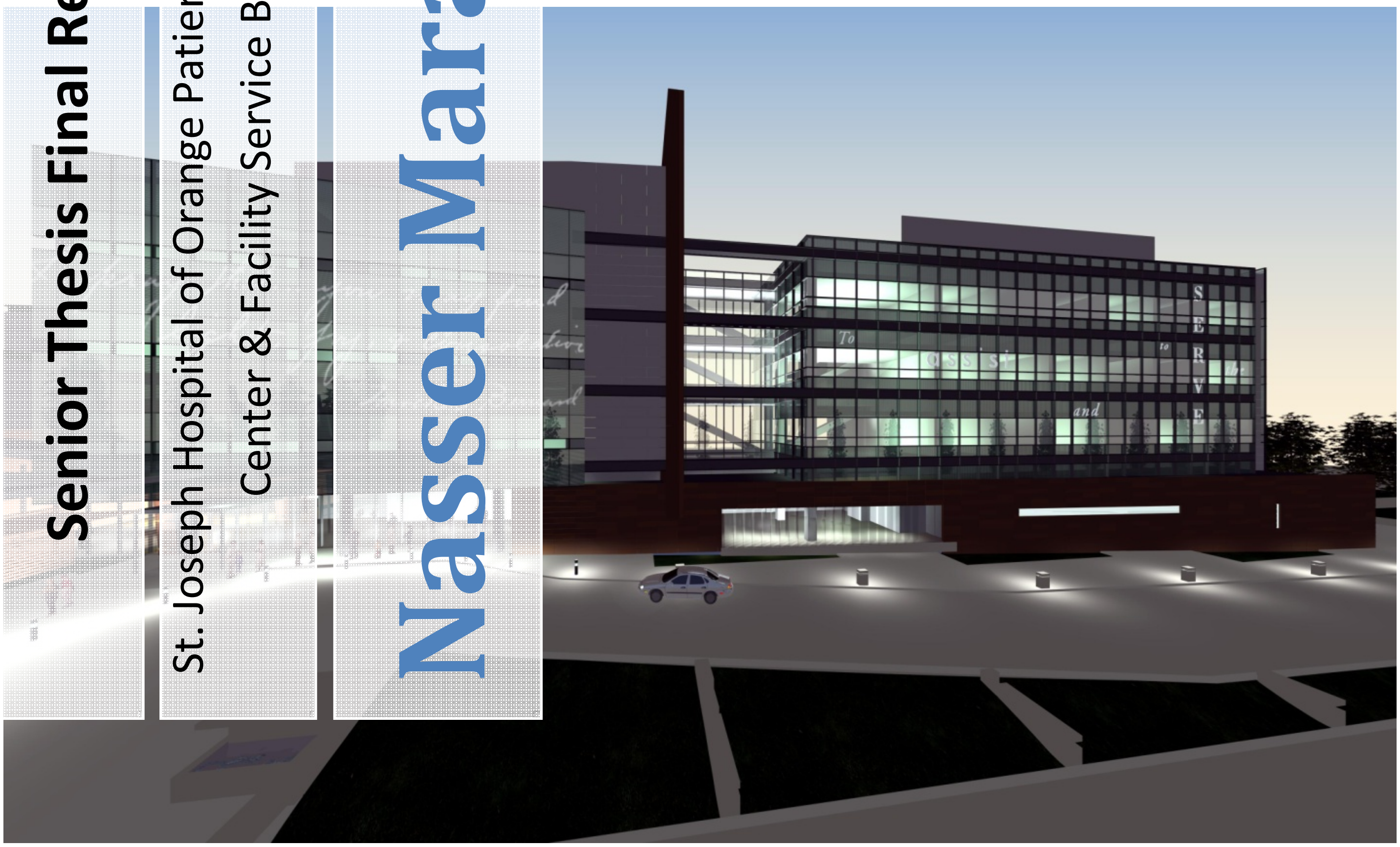
Senior Thesis Final Report

St. Joseph Hospital of Orange Patient Care Center & Facility Service Building

Nasser Marafi



Final Report

Advisor: Professor Andres Lepage

STRUCTURAL OPTION

April 9th

2008

St. Joseph Hospital of Orange

Orange, CA

Patient Care Center & Facility Service Building

design team

• Owner • St. Joseph Health System •

• Architect • NBBJ •

• Civil • KPFF Consulting Engineers •

• Construction
• Services McCarthy Building Companies INC. •

• Structural • KPFF Consulting Engineers •

• Equipment
• Technology Foundations INC. •
• NBBJ •

• Interior Design + Graphics
• Mechanical + Electrical •

• Syska & Hennessy INC.
• Davis Design •

• Low Voltage Consultant
• Acoustical Consultant
• Martin Newson and Associates •

• Security Consultant
• Diversified Security Systems •

architectural

• etched inspirational message on exterior glazing •

• braced frames accentuated through the exterior glazing •

• 19'-0" tall canopy located at the main entry •

• patient rooms overlooking central courtyard •

• exterior facade consists of precast concrete panels
+ stucco facade facing the central courtyard
+ metal equipment screening @ roof•

statistics

• 252,712 sq. ft •

• 4 stories
+ basement •

• 82'-0" tall •

• costs
$130 million •

• guaranteed
maximum price •

structural

• floor system consists of composite steel framing •

• typical floor areas designed for 80 psf •

• lateral system consist of a series of concentrically steel braced frames on levels above ground while shear walls are used @ basement level •

• pile foundation system is used to support the main entry canopy , while continuous deep footings support the shear walls •

electrical

• 4 substations feed
(2 emergency) •

• substations feed @
480/277V
, 3 Phase, 4 Wire •

• 45-300kVA transformers
located on every floor •

lighting

• typical lamps
32W compact
fluorescent •

• photocells
control
hospital
dimming
system •

mechanical

• 8 air handling units supplying 36,000 - 70,000 CFM •

• 2 unfired clean steam generator
supplying @ 4000 lbs\hr •

• each room is equipped with a
terminal air unit meeting patient's comfort •

• campus chiller water and steam
used to condition water supply •

| nasser marafi | structural option |
http://www.arche.psu.edu/thesis/portfolios/2008/nam202/

Executive Summary

This report evaluates the existing main lateral force resisting system of the St. Joseph Hospital of Orange Patient Care Center & Facility Service Building. Special centrically braced frames were originally designed as the main lateral force resisting system. The system was evaluated and it was determined that a redesign to an eccentrically braced frame system with moment connection away from links would be beneficial.

The redesign was able to reduce the seismic base shear coefficient hence reduce the amount of steel required in the lateral force resisting system. 2 bays were eliminated from each bay set, while the overall system used smaller steel shapes sections compared to the original design. A built up section was also introduced into the design of a link, which was customized to the appropriate loading criteria to ensure efficiency of the overall EBF system. Typical connections were taken into consideration and designed for according to the new AISC 341-05 seismic provisions.

The overall EBF design was then comparable to the original and a cost and time estimate was performed. The EBF system was able to reduce the structural steel density by 2.4 pound per square foot from 10 to 7.6 pounds per square foot. The redesign would save approximately $1,562,500 which is approximately 1% of the total project cost.

Breadth Study

The report then does a lighting study and redesign of the central courtyard. That creates a place of refuge for patients and even hospital workers looking to escape the high stress, hectic and uncomfortable environment of the hospital.

Acknowledgments

The author of this final report would like to acknowledge the following individuals, design professionals, and firms for their help making this thesis study possible.

KPFF Consulting Engineers – New York

Jeff Albert

KPFF Consulting Engineers – Los Angeles

Aaron Reynolds

The Pennsylvania State University

Dr. Andres Lepage

Dr. Linda Hanagan

Dr. Louis F. Geschwindner

Dr. Richard Mistrik

Colleagues

Landon Roberts

Kirk Stauffer

Tom Yost

A special thanks to family and friends for their much needed support.

Table of Contents

Introduction

St. Joseph Hospital of Orange Patient Care Center & Facility Service Building is built within the Saint Joseph Hospital Campus serving the healthcare needs of the Orange county community in Orange, CA. The Patient Care Center is linked to the main hospital through an underground tunnel and through a lobby to further serve the patients' needs. The building consists of four stories with basement that gives 252,712 square foot of additional hospital space. The buildings is approximately 285'-0" by 198'-0" on Level 1 and 2 and then the floor plan is reduced to 240'-0" by 198'-0" on Level 3, 4 and the roof. The highest point of the building is about 82'-0" from ground level.

The main entrance to the lobby is connected to the adjacent hospital reception area. The Patient Care Center consists of operating rooms to expand the surgical capacity of the main hospital. Operating rooms are equipped with latest innovative technology and medical equipment. To help further serve the main hospital, the Patient Care Center also has additional room for incoming patients and rooms for patients requiring intensive care.

The Patient Care Center has a central sterile plant located on the basement level with MEP equipment. The first level of the hospital consists of surgical rooms, administrative rooms and the lobby. The upper floors are separated by the central courtyard located on level 2. The west side consists of patient rooms and the east side consists of intensive care units. The remaining mechanical equipment is located on the roof level.

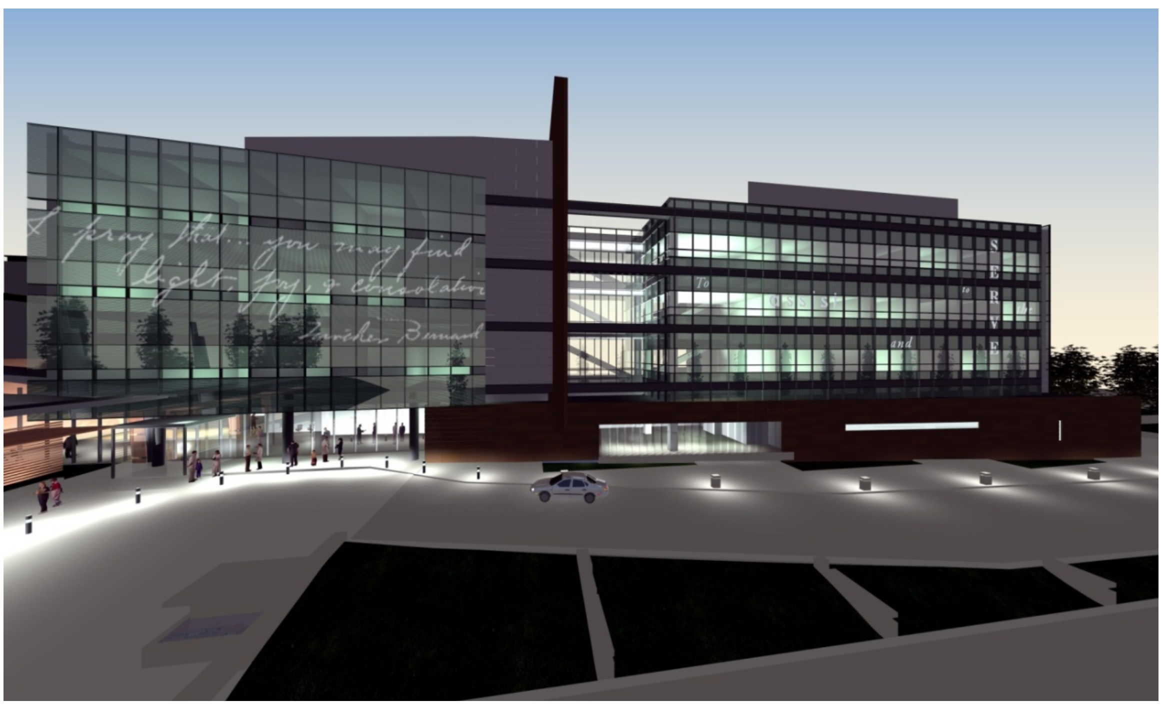


Figure 1. Computer rendering of the Patient Care Center's North elevation. Courtesy of KPFF-LA

Existing Structural System

Structural Elements

Floor Framing

There are minor variations to the floor framing through the Patient Care Center. The typical floor system is a composite steel framing using lightweight concrete and a total thickness of 6¼", 3" composite deck is used with 5" long, ¾"diameter shear studs for composite action. The typical infill beam is a W16x31, 30'-0" long spaced at 10'-0" on center, which frame onto a W24x68 30'-0" long. Variations from the typical floor system are based on the use of the space. Light weight concrete was used in the typical steel deck configuration to reduce shear and overturning moment during seismic events.

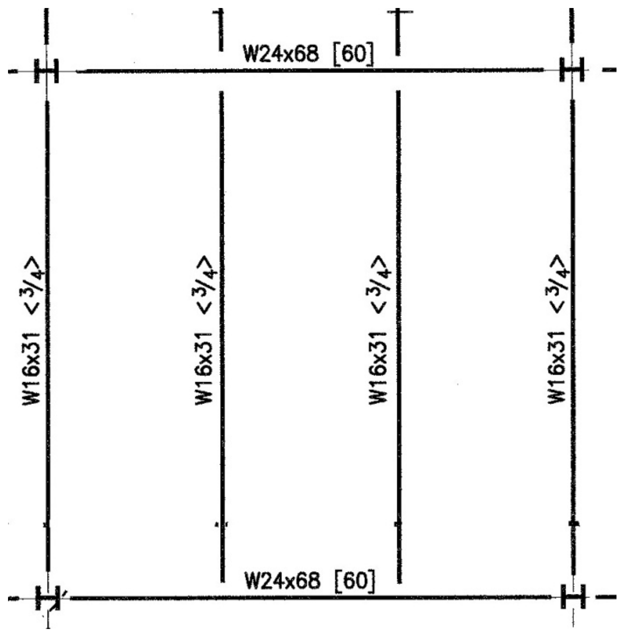


Figure 2. Typical 30'-0"x30'-0" bay located on Levels 2, 3 and 4

First floor

The floor framing plan on the first floor differs from the rest due to different loading criterion used. Typical infill members used are W18x35 framing into W24x68 girders. Composite steel framing is used with normal weight concrete and a total thickness of 7½", 3" composite deck with 5" long, ¾" diameter shear studs.

Second floor

There is a central courtyard which is supported by the second floor framing system. Due to the high loading W21x111 infill beams are used which frame into W30x148. A composite steel framing system is also used with normal weight concrete and a total thickness of 9", 3" composite deck with 5" long, ¾" diameter shear studs.

Roof

Due to the location of air handling units on the roof, members with a higher loading capacity are required. Therefore the member sizes change to a W18x40 for beams and W24x84 for girders. A 9" composite steel system exists similar to the second floor courtyard but covered with rigid insulation.

Lateral Resisting System

The lateral system consists of 6 sets of special concentrically braced frames located both along the N-S and E-W planes. It ranges from 2 bays to as long as 6 bays framing vertically to the roof of the structure. These braces are supported by shear walls at basement level. The braced frames are typically X-bracing while a whole set running E-W is diagonally braced. Both configurations are considered concentrically braced frames. X-Braced frames are typically HSS shaped which are slotted and slipped in with a gusset which is then welded. The gusset plate is then welded onto the column and beam, allowing the brace to buckle out of plane to dissipate energy at time of an earthquake. While diagonally braced member consists of a W Shape section which have splice plates welded onto the web and gusset plate. While the flanges being slotted and welded to the gusset plate. All beams connected to the bracing system are designed as welded drag connections, consisting of a shear plate which is bolted onto the attached beam during construction. The flanges are then welded top and bottom with a complete joint penetration weld. The following two figures are details of brace connections.

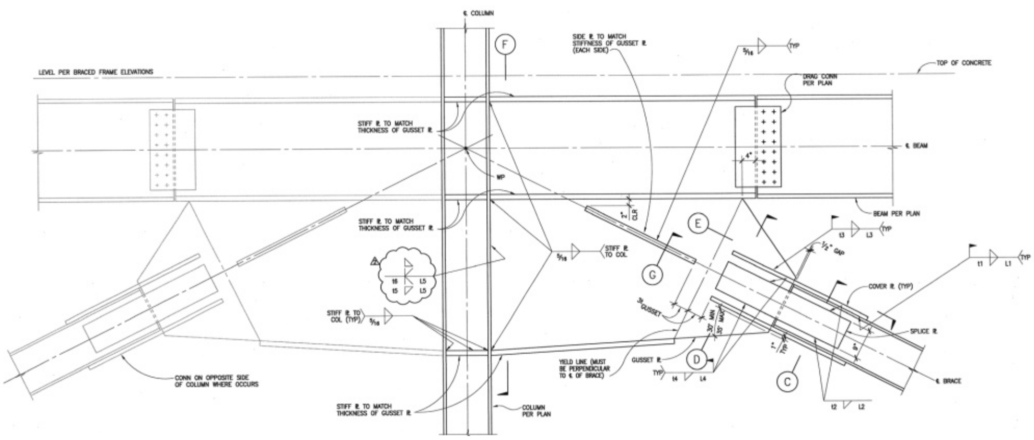


Figure 3. Diagonal Brace Connection Detail

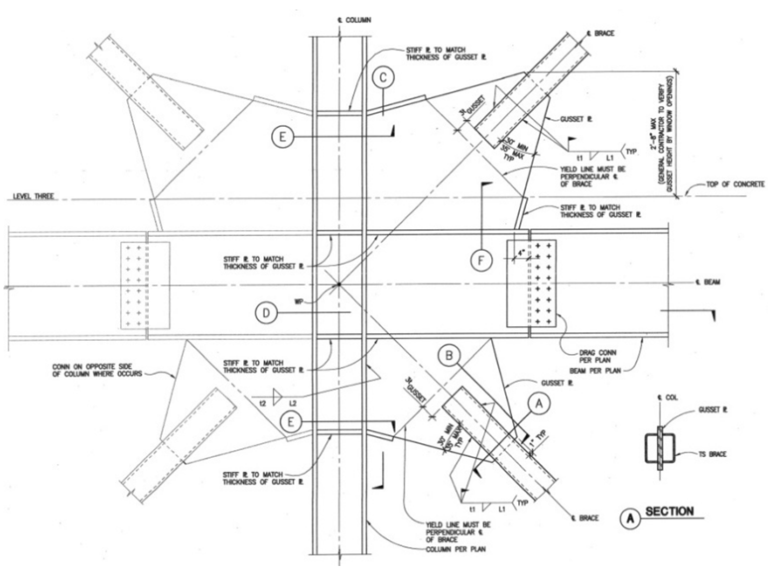


Figure 4. X Brace Connection Detail

Foundation system

Gravity columns at the basement level are supported by concrete footings. These footings range depth from 1'-6" to 3'-6" and their size ranges from 2'-0"x2'-0" to 16'-0"x16'-0". While the shear walls are supported by continuous deep footings typically 5'-0" deep and 7'-0" wide from each face of the wall. The majority of the foundation is considered shallow as advised by the geotechnical engineer. While the main entrance canopy is supported by piles capes each connected to 4 piles.

Columns

There are two columns sets per gridline intersection which are usually spliced at 5'-0" from the Level 2. Typical columns sizes are W14x99 on the upper levels (Level 2 to Roof); while the lower columns are W14x145 or W14x132 depending on location and there loadings. Columns existing in the brace frame are usually W14x145 except end columns which are W14x211 on the top levels and W14x311 at the bottom levels. These columns have greater strength capacities due to the excess tension and compression they carry from the bracing system.

Connections

Beams and Girders are typically connected to each other using bolted connections on the beams with steel plates and welded on the girder. The gravity girders have similar connections to the columns, where a shear plate is welded on the column flange.

Framing Plans and Elevations

1st Floor plans

The following figure represents the first floor plan labeled with occupant use. The designer assumed a live load of 80 psf throughout the first floor.

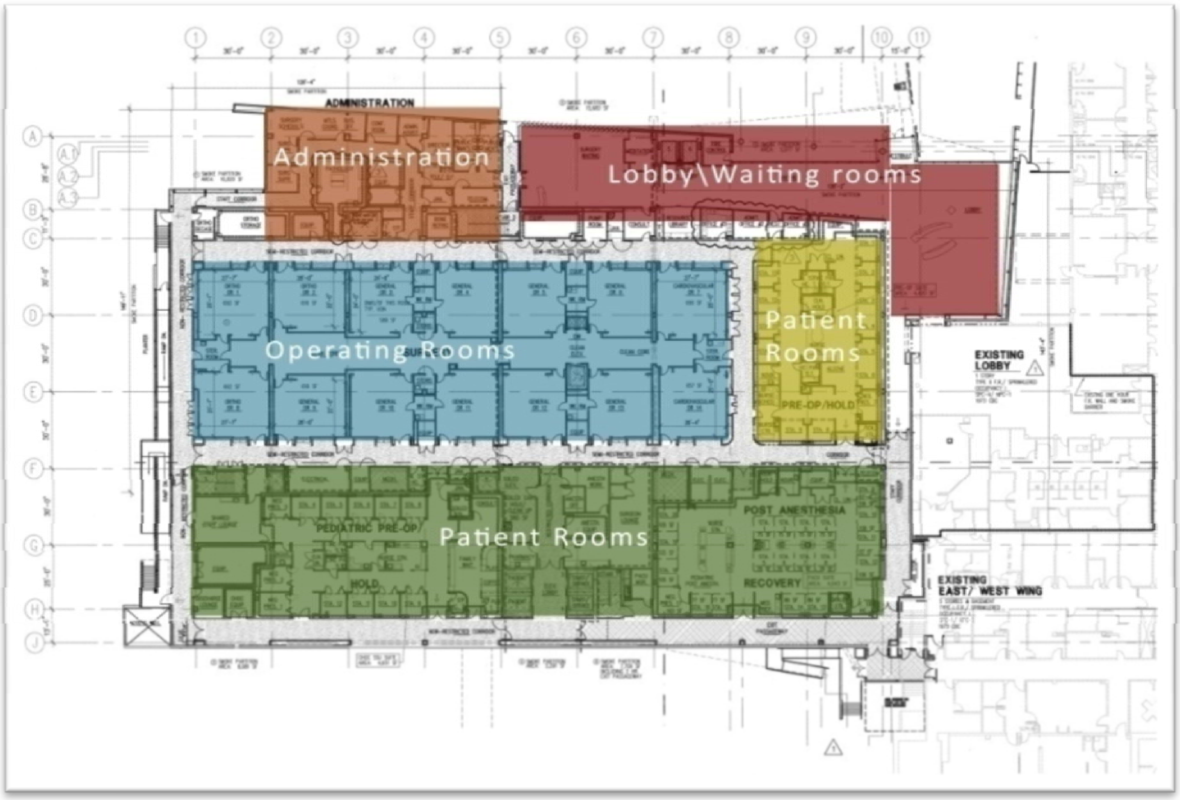


Figure 5. First floor plan showing occupant use.

The figure below represents the framing system on the first floor. A typical 30'-0"x30'x0" bay was maintained through the center part of the structure for simplicity. While a 10'-0" span joist was maintained throughout the structure.

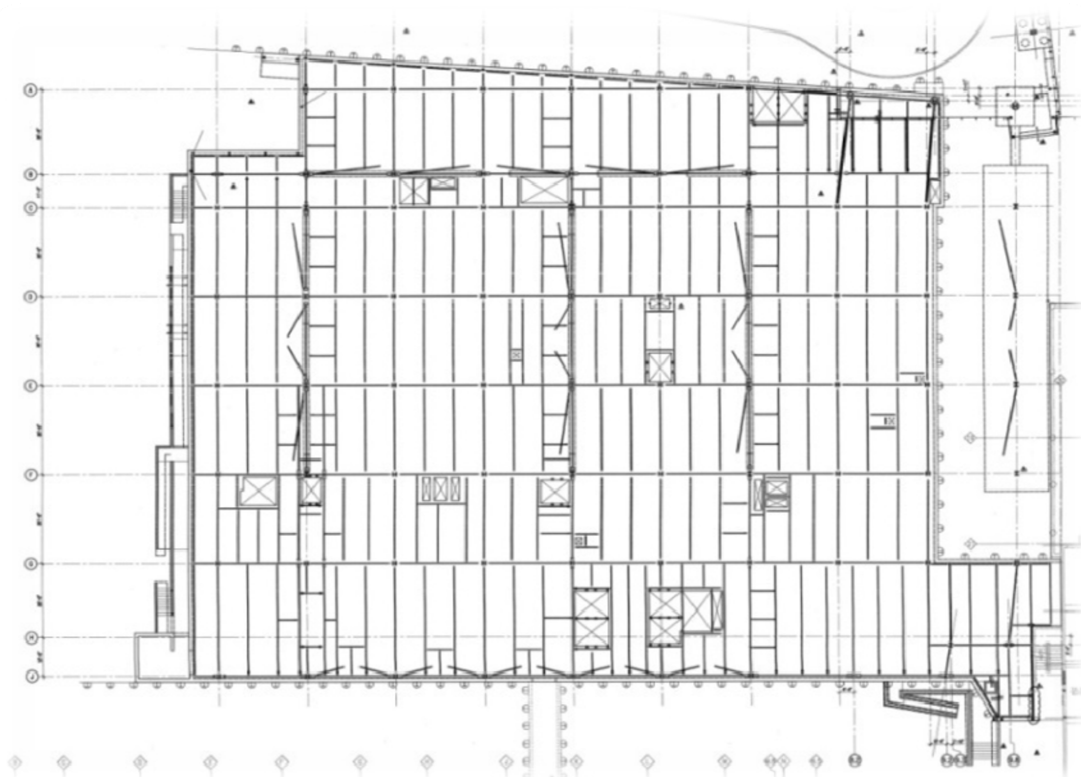


Figure 6. First floor framing plan.

2nd Floor Plan

The figure below represents the 2nd floor plan occupant use. Loadings here are assumed to be 80 psf where patient rooms and the Intensive care units exist. While at the court yard a super imposed dead load is added counting for pavements, planters and trees. The roof on the west side is designed for future planters.

Figure 7. 2nd floor plan showing occupant use.

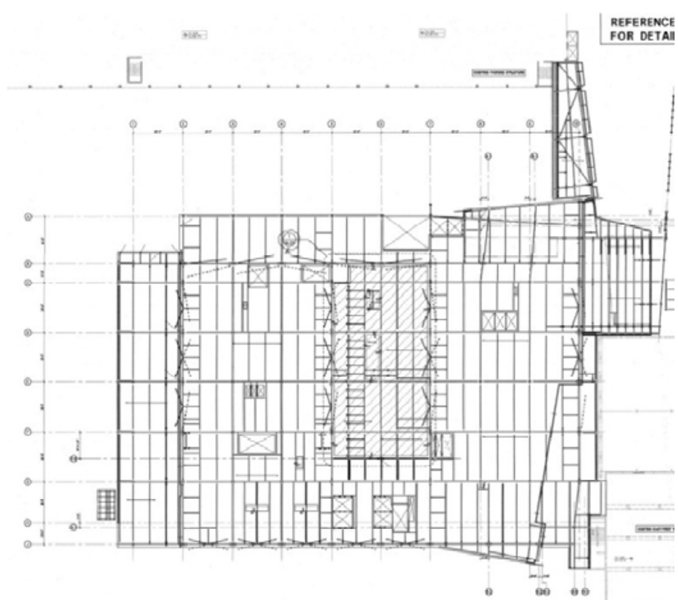


Figure 8. 2nd floor plan showing framing plan.

3rd Floor plan.

On the third floor of the patient care center, the occupant usage is similar to the second floor without the courtyard. The live loads here are assumed to be 80 psf by the designer. The fourth and third floor is similar in loading and occupant layout.

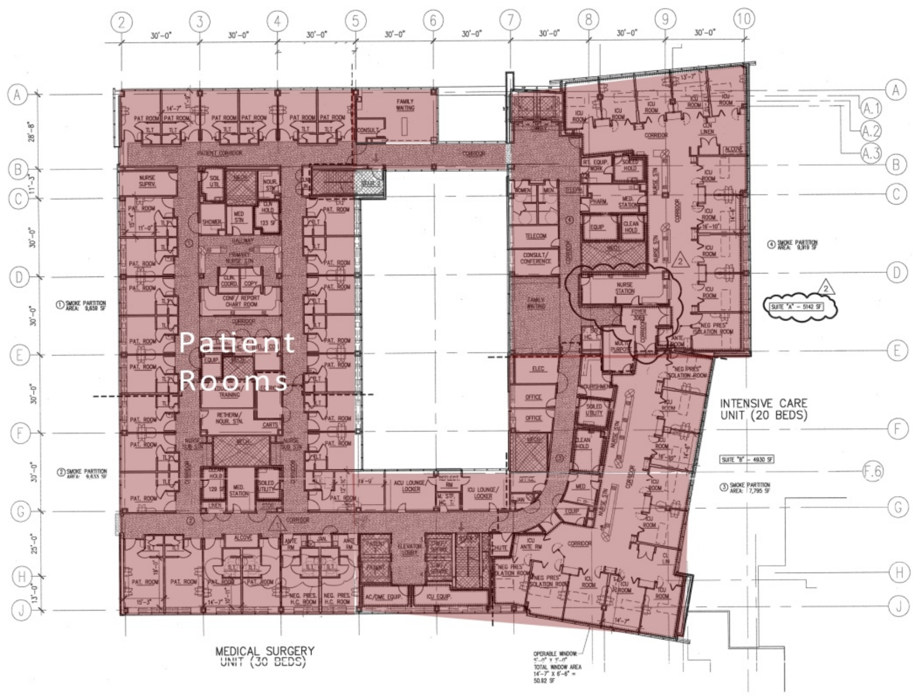


Figure 9. 3rd Floor Plan showing occupant use.

Building Sections

The figures below represent the building sections through the building taken at the courtyard. This report does not take into consideration wind pressures that might arise inside the courtyard space.

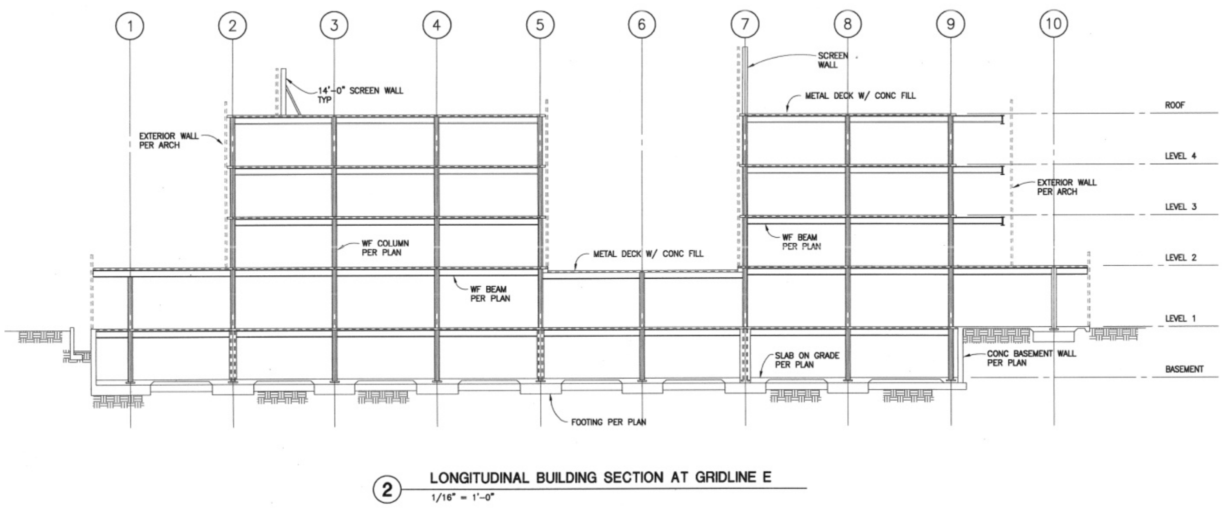


Figure 10. Longitudinal Section at gridline E.

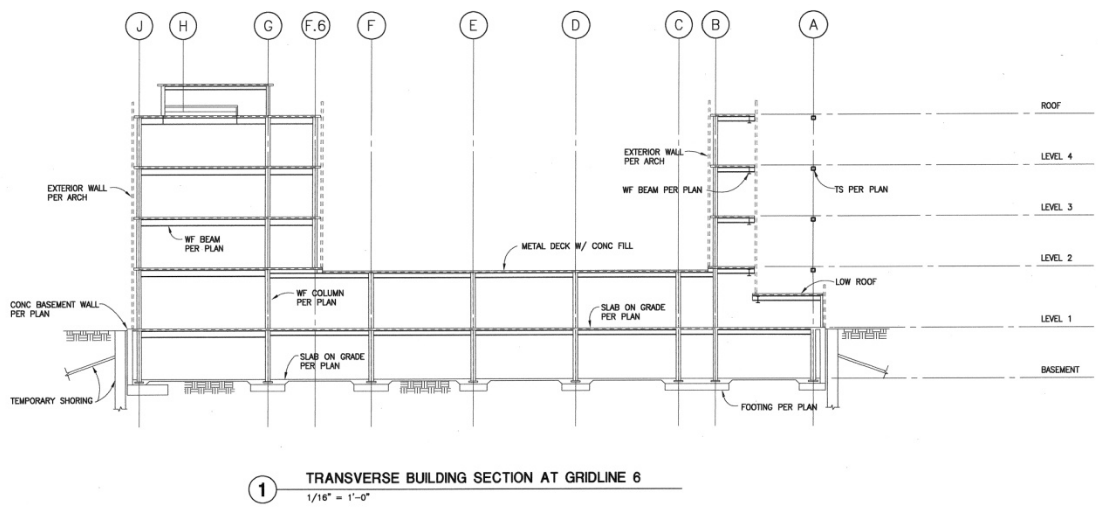


Figure 11. Transverse Building Section at gridline 6.

Lateral Resisting system

The following figure represents the lateral system labeled on level 1. The lateral system consists of special concentrically braced frames. There are 6 groups of braced frames altogether, and two types, one consists of diagonal bracing while the others are all X braced frames.

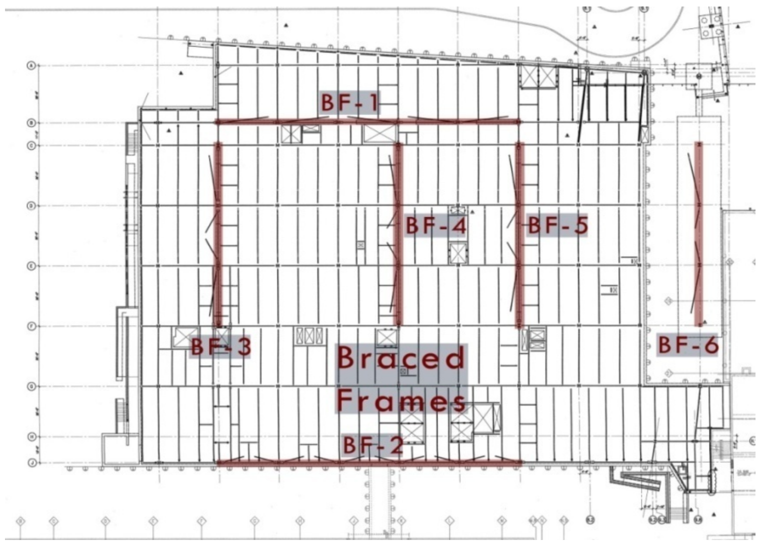


Figure 12. 1st Floor plan labeling all braced frames

BF-1

Consist of diagonal wide flange members, wide flange size's range from W14x90 on level 4, to W14x211 on level 1. Braced frames are supported by shear walls located on the basement floor and tied into a 5'-0" continuous footing. The entire brace frame is 150'-0" wide.

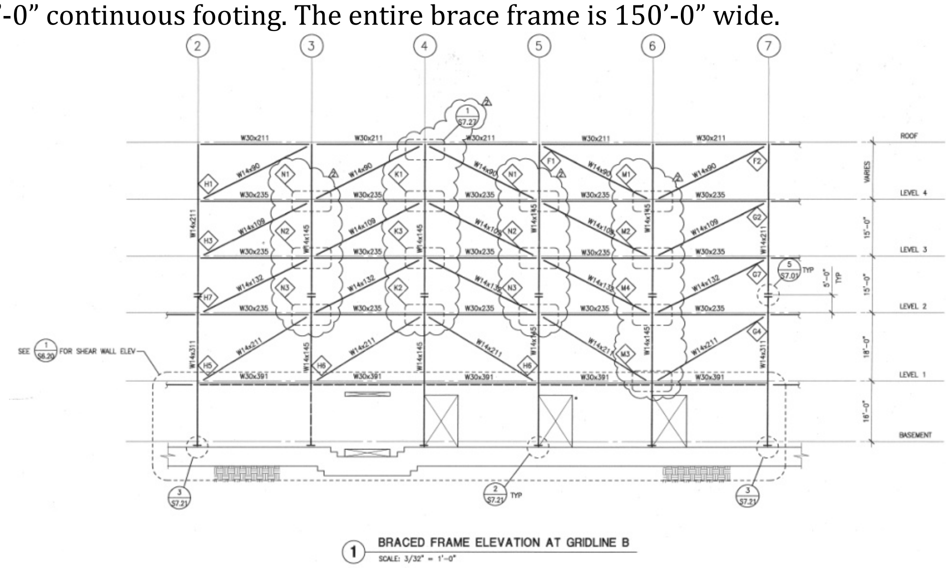


Figure 13. BF-1 Braced Frame Elevation

BF-3 X-Braced Members

Consist of X-braced members; members are usually steel tubes. There are two W14x145 running as diagonal members on each end bay on level 1. Braced frames are supported by shear walls located on the basement floor and tied into a 5'-0" continuous footing. The entire braced frame is 90'-0" wide.

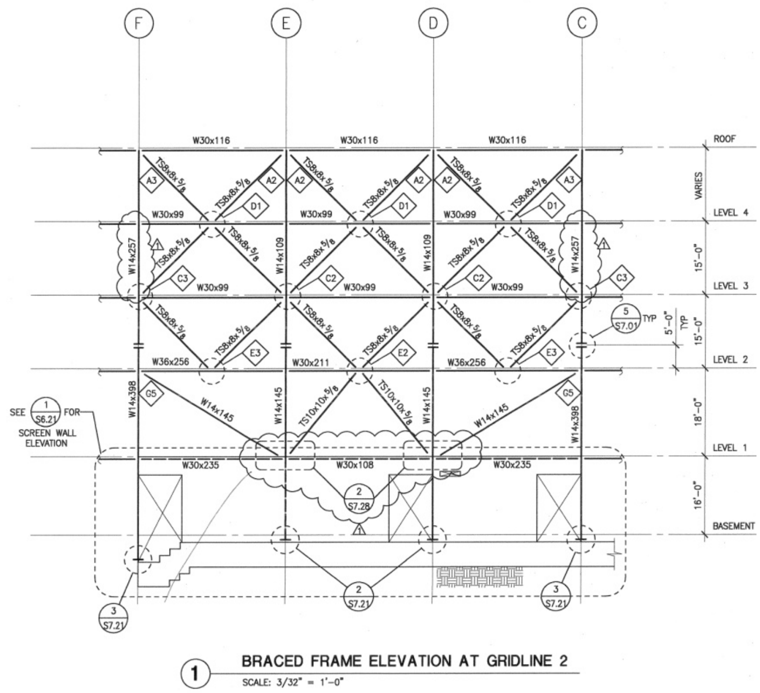


Figure 14. BF-3 Braced Frame Elevation

Codes and Material Properties

Codes and Referenced Standards

The following table shows the codes that were adopted in this report and codes that were implemented by the designer. Refer to Page XX for further design codes adopted in this report.

Codes adopted by this report	Codes adopted by the designer
2007 California Building Code	Title 24, Part 1 2001 California Building Code
American Society Civil Engineers, Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)	1997 Uniform Building Code with California amendments

Material Strength Requirements

These requirements correspond to the general structural notes on the plans.

Concrete	Strength	Density
Footings	4000 psi	150 pcf
Basement Walls	4000 psi	150 pcf
Composite Concrete Light Weight	3000 psi	110 pcf
Composite Concrete Normal Weight	4000 psi	150 pcf
Slab on Grade	4000 psi	150 pcf
Drilled Concrete Piles	4000 psi	150 pcf
Reinforcing (Steel)	ASTM706 Grade 60	

Steel Deck	I (in^4)	S (in^3)
3" x 18 GA Deck	1.203	.767

Structural Steel	ASTM	Fu (ksi)	Fy (ksi)
Wide-Flange Shapes (WF Shapes, W14 and larger)	A992	65	50
WF Shapes, W12x14, W10x12, W8x12 and smaller	A992	65	50
Plates	A572, Gr50	65	50
Connection Plates	A36	58	36
Pipe Columns	A53 Grade B	80	40
Tube Sections	A500 Grade B	58	46
Bolts	A325N, A490SC	$F_{nt} = 90$	$F_{nu} = 48$
Bolts in Concrete	A307, A3548C	$F_{nt} = 45$	$F_{nu} = 24$
Angles, Channel and WT Shapes	A36	58	36

Foundation	
Allowable Bearing (Gravity Loads)	4000 psf (Basement Footings) 2500 psf (Ground Floor Footings)
Equivalent Fluid Pressure	30 pcf (unrestrained walls) 23 pcf (unrestrained walls)
Passive Earth Pressure	300 pcf

Building Loads

Live Loads

Live loads are determined in accordance with ASCE 7-05.

Occupancy	Designer's Uniform Live load (psf)	2007 CBC Uniform Live loads (psf)
Roof	20	20
Patient Rooms	80[1]	40
Operating Rooms, Laboratories	80[1]	60
Corridors	80[1]	100
Storage	120	125
Computer Rooms	100	100
Elevator Machines Rooms	125[1]	
Public Areas, Assemblies	100	100
Mechanical Rooms	150[1]	50
Roof Gardens	100	100
Office	80[1]	50

[1] Designer's value used for simplicity reasons.

Dead Loads

Refer to Appendix for dead load calculations. Material weights are taken from the ASCE 7-05 Chapter C3.

	LVL1	LVL2	LVL3	LVL4	ROOF
Concrete Topping	75	44	44	44	94
Steel Deck (18 Gage)	3	3	3	3	3
Super Imposed	12	12	12	12	25
Partitions	20	20	20	20	
Total Dead Load	**110**	**79**	**79**	**79**	**122**

*Units in pounds per square foot

Level 2 Courtyard	PAVER	PLANTER W/ TREES	PLANTER
Concrete Topping	94	94	94
Steel Deck (18 Gage)	3	3	3
Super Imposed	22	552	342
Topping	80		
Total Dead Load	**200**	**649**	**439**

*Units in pounds per square foot

Structural Depth Study

Existing Structure System Check

We have concluded from previous reports that the Patient Care Center is in a high seismic region and that wind loading does not control over seismic. The following is design check with seismic loading according to ASCE 7-05 Chapter 11 and 12.

The mapped acceleration parameters are determined according to the Earthquake Ground Motion Parameter Java Application available on the USGS website.

Mapped Spectral Response Accelerations	S_s = 1.378 S_1 = .497

The ground soil properties were classified to be as site class D according to the geotechnical report. The site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters was determined in accordance with ASCE 7-05 11.4.3.

Site Class	D
Site Class Factors	F_a = 1 F_v = 1.5
$S_{MS} = F_a(S_s)$	1.378
$S_{M1} = F_v(S_1)$	.7455

The design spectral acceleration parameters was determined in accordance with ASCE 7-05 11.4.4. And the following parameters were used.

$S_{DS} = 2/3(S_{MS})$	.919
$S_{D1} = 2/3(S_{M1})$	.497

The patient care center is considered a health care facility therefore the occupancy category is considered to be type IV in accordance with ASCE 7-05 Table 1-1. And the importance factor used when designing the structure for seismic loads is 1.5 according to ASCE 7-05 table 11.5-1.

Occupancy Category	IV
Importance Factor (I_E)	1.5

The seismic design category was determined to be SDC 'D' in accordance with ASCE 7-05 11.6.

Seismic Design Category	D

The designer used special concentric braced frames to be the main lateral force resisting system. Chapter 12 of ASCE 7-05 is used to determine the base shear design force of the lateral system. ASCE 7-05 Table 12.2-1 determines the response modification coefficient, system over strength factor and the deflection amplification factor. ASCE 7-05 Table 12.2-1 also determines the structural system limitation of building height to be 160' which is less than the building height of the patient care center at 60'.

Response Modification Factor (R)	6 (Special Steel Concentrically Braced Frames)
Deflection Amplification Factor (C_d)	4
Over Strength Factor (Ω_o)	2
Building Height Limitation	160'

Section 12.3 of the ASCE 7-05 code determines all diaphragm flexibilities, configuration irregularities and redundancy. A rigid diaphragm condition exists since the floor system is considered to be a concrete filled metal deck with a span to depth ratio of 3 or less.

Span to Depth Ratio	30'/30' = 1
Diaphragm type	Concrete filled metal deck
Diaphragm flexibility	RIGID

Horizontal structural irregularities are checked according to ASCE 7-05 section 12.3.2.1. The following table represents each irregularity type and its check.

Horizontal Structural Irregularities

	Irregularity Type	Comment	Status
1a.	Torsional	After Modeling structure in ETABS, it has been concluded that this irregularity does not exist. Check appendix for reference.	OK
2.	Reentrant Corner	Irregularity does not exist by inspection of the floor plans	OK
3.	Diaphragm Discontinuity	Irregularity does not exist by inspection of the floor plans	OK
4.	Out of plane Offsets	No vertical element out of plane offsets exists by inspection of plans	OK
5.	Non Parallel System	All lateral force resisting systems are parallel to the orthogonal axes	OK

Vertical structural irregularities are checked according to ASCE 7-05 section 12.3.2.2. The following table represents each irregularity type and its check.

Vertical Structural Irregularities

	Irregularity Type	Comment	Status
1a.	Stiffness-Soft Story	Members are upsized going down the building, therefore higher stiffness	OK
2.	Weight Mass	Roof Weight/Adjacent Story Weight = 117psf/96psf < 150%. Refer to Appendix for story weights.	OK
3.	Vertical Geometric	X Direction: 285' (2nd Story)/240' (3rd Story) < 130% Y Direction: 198' (2nd Story)/198 (3rd Story) <130%	OK
4.	In-Plane discontinuity of vertical lateral force resisting element	No discontinuity exists by inspection of plans	
5.	Discontinuity in Lateral Strength	Members are upsized going down the building, therefore higher strength	OK

Therefore after previous inspections there are no Vertical and Horizontal Irregularities existing in the structure.

Redundancy is checked in accordance with ASCE 7-05 Section 12.3.4. After inspection of the lateral force resisting system, there is a total of 15 braces in the X direction, and a total of 22 braces in the Y direction. Therefore as a preliminary check it can be concluded that any removal of an individual brace will not result in a reduction of 33% of the story strength. Further checks on redundancy are preformed after modeling in ETABS; refer to appendix for calculations.

Seismic load effects and combination are applied in accordance with ASCE 7-05 section 12.4. The following load combination can be concluded when using strength design.

5. $(1.2 + 0.2S_{DS})D + \rho Qe + L + 0.2S$
7. $(.9 - .2S_{DS})D + \rho Qe + 1.5H$

Where $S_{DS} = .92$, $\rho = 1$, and Qe = Horizontal earthquake load. Note that live load may be reduced by 50% where live load is less than 100psf.

ASCE 7-05 Section 12.5 refers to the direction of the loading to be used when designing the lateral force resisting system. Since horizontal structural irregularity type 5 does not exist and by inspection of the plans there is no individual column that takes seismic forces from each orthogonal direction. Therefore ASCE 7-05 code permits the design seismic forces to be applied independently in each of two orthogonal directions and orthogonal interaction effects are also permitted to be neglected.

The approximate fundamental period of the structure is determined according to ASCE 7-05 section 12.8.2.1. The following table represents the results.

Seismic Response Coefficient (C_t)	.02 (Table 12.8-2 Concentric Braced Frame)
Period Coefficient (x)	.75 (Table 12.8-2 Concentric Braced Frame)
Building Height (h_x)	63'-0"
Coefficient for upper limit (C_u)	1.4 (Table 12.8-1)
Approx. Period $T = (C_u)(C_t)(h_x)^x$	.626
Period T_b from ETABS Model	.48 (See page 30 for reference)

$T_s = S_{D1}/S_{DS}$	.54
Approx. Period T	.626

The permitted analysis procedure is determined by ASCE 7-05 section 12.6. Since the structure is considered regular with no irregularities and $T<3.5T_s$ it is permitted to used any analytical procedure. Therefore for simplicity purposes, the Equivalent Lateral Force Analysis will be used in accordance with ASCE 7-05 section 12.8.

The calculated seismic response coefficients will be determined in accordance with ASCE 7-05 section 12.8.1.1. The following table represents the results refer to appendix for calculations.

Seismic Response Coefficients (min C_s)	**=.23 (Controls)** =.3 =5.83

The Vertical Distribution of forces was determined in accordance with ASCE 7-05 section 12.8.3. Accidental torsion was also taken into account according to ASCE 7-05 section 12.8.4.2. The following table represents the results.

Vertical Force Distribution in the X Direction

i	hi	h	w	w*hk	Cvx	fi	Vi	By	5% By	Ax	Mz
	ft	ft	kips			kips	kips	ft	ft		k-ft
Roof	15	63	4317	353078	0.40	762	762	198	9.9	1.0	7539
4	15	48	3566	218444	0.25	1095	1856	198	9.9	1.0	10836
3	15	33	3566	146677	0.16	316	2172	198	9.9	1.0	3132
2	18	18	7927	171189	0.19	858	3030	198	9.9	1.0	8492
		Sum	**19376**	**889388**		**3030**					**29999**

Vertical Force Distribution in the Y Direction

i	hi	h	w	w*hk	Cvx	fi	Vi	Bx	5% By	Ax	Mz
	ft	ft	kips			kips	kips	ft	ft		k-ft
Roof	15	63	4317	353078	0.40	1769	1769	240	12.0	1.0	21230
4	15	48	3566	218444	0.25	1095	2864	240	12.0	1.0	13135
3	15	33	3566	146677	0.16	735	3599	240	12.0	1.0	8820
2	18	18	7927	171189	0.19	858	4457	285	14.3	1.0	12223
		Sum	**19376**	**889388**		**4457**					**55408**

ETABS Modeling

After all necessary seismic provisions were taken into account and seismic loads were calculated. A lateral analysis was done using the finite element analysis software ETABS. The following modeling assumptions were taken into account.

- The Main Lateral Resisting System was only modeled consisting of all the braced frame bays in the structure. The material properties and frame sections in those bays were modeled according to the structural drawings.
- A Rigid Diaphragm was modeled at every floor with the lateral load being assigned to the diaphragm.
- Lateral forces were applied to the center of mass with a calculated moment due to torsion.
- The mass of the structure was assigned to a Null Shell Property at each floor. This gives us an approximate period from the modal analysis. Please see appendix for the assigned mass at each level to the ETABS model.
- Tributary Dead and Live Loads were assigned to each member refer to appendix for loadings.
- Proper Load Combinations were assigned, See appendix for reference.
- All shear walls were modeled at basement level with their assigned properties according to the structural plans. Shear wall openings were also modeled according to plan. An infinite lateral stiffness was then assigned to level to ensure 0% drift at ground level.
- Shear walls were meshed at 24”x24” rectangles to ensure accuracy of the model.
- Braces were assumed to be pinned at each end.
- Structure was assigned to fixed support.
- P-Delta effects are automatically taken into consideration in the model

The following figure represents a 3-D view of the ETABS model, and lateral force resisting frames shown.

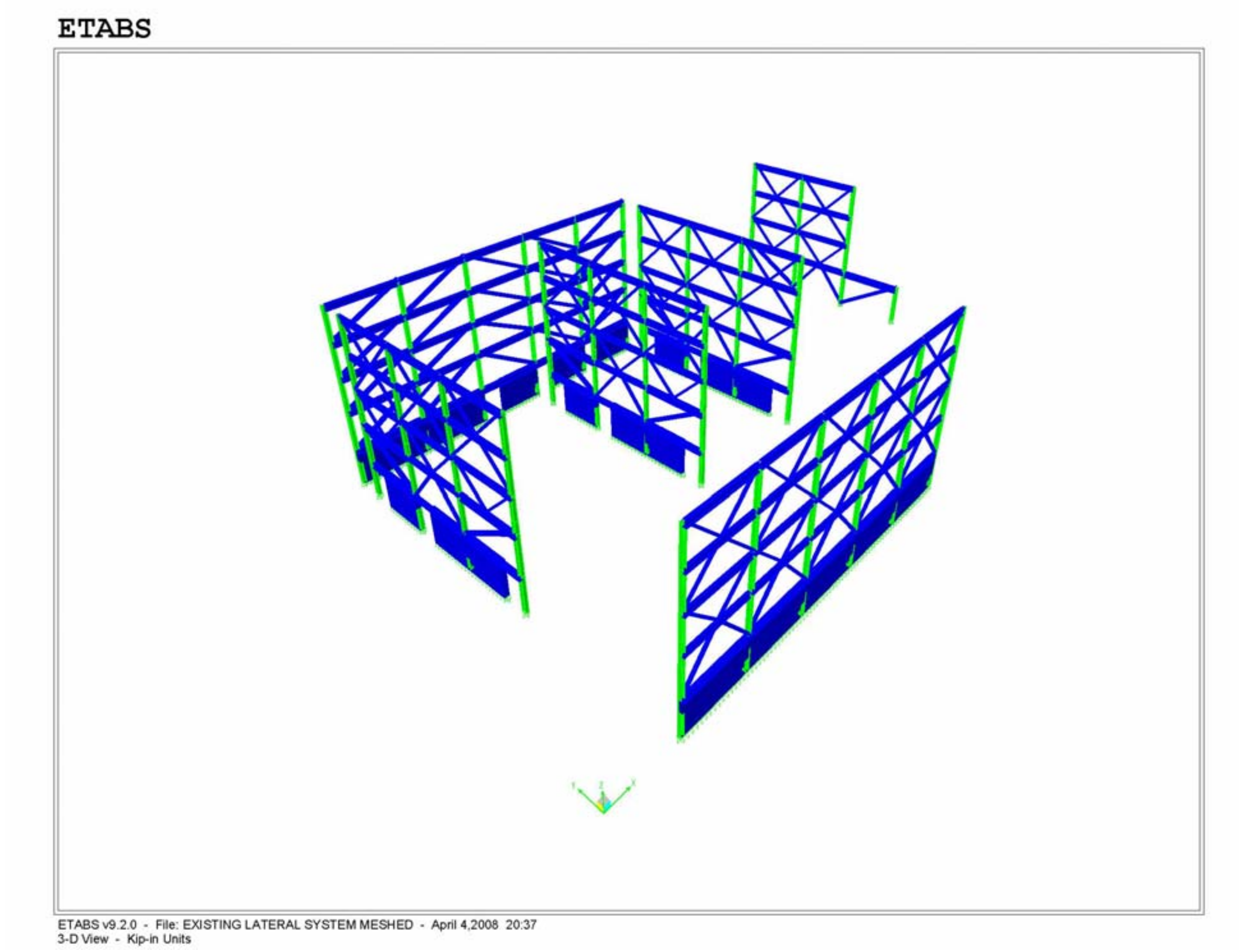


Figure 15. 3-D ETABS Model

Existing Design Check

A series of checks were done to the ETABS model to conclude the adequacy of the existing lateral system, the following table represents a summary of the series of checks performed and observations made.

Check	Comment	Observation
Story Drifts	Allowable story drifts for each level are met in each of the two orthogonal directions. Although the computed story drifts is at most 30% of the allowable.	OK
Torsion	Accidental Torsion = 5%, Inherent torsion is assumed by applying loads at the center of mass and being resisted by the center of rigidity of the structure.	OK
Redundancy	Adequate amount of braces in each direction resisting less than 33% of the total story shear.	OK
Modal Period	ASCE 7-05 Approximate Period: 0.626 seconds ETABS Model Period: 0.4217 seconds ETABS Model period is less than the conservative period approximation of the ASCE 7-05 code. Since the period of the structure is proportionally related to the inverse of stiffness. This concludes that there is more stiffness then needed for the mass of the building.	Overdesigned System
Member Spot Checks	Columns and braces are approximately at 40% to 70% of their total design strength. Refer to appendix for further calculations	Overdesigned System

Story Drifts

The following tables represent the story displacements based on the strength level applied seismic loads in the ETABS model. A deflection amplification factor equal to 5 was used to amplify the drift. The story drift limit is 1.5% of the story height, according to ASCE 7-05 provisions.

X Direction					
Story	**hx (ft)**	**Drift Ratio**	**Allowable Drift (in)**	**Story Drift (in)**	**Check**
Roof	15	0.000712	2.7	0.4272	OK
4	15	0.000949	2.7	0.5694	OK
3	15	0.000917	2.7	0.5502	OK
2	18	0.000818	3.24	0.58896	OK

Y Direction					
Story	**hx (ft)**	**Drift Ratio**	**Allowable Drift (in)**	**Story Drift (in)**	**Check**
Roof	15	0.000748	2.7	0.4488	OK
4	15	0.001044	2.7	0.6264	OK
3	15	0.001364	2.7	0.8184	OK
2	18	0.00092	3.24	0.6624	OK

Torsion

According to ASCE 7-05 section 12.8.4.2, diaphragms that are non flexible are required to account for Inherent torsion M_t and Accidental Torsion M_{ta}.

Inherent Torsion

Since the Lateral forces are applied to the center of mass and the center of rigidity is calculated in the ETABS model, this will account for inherent torsion. A hand calculation for the center of rigidity and center of mass was done to verify the accuracy of the ETABS model.

Accidental Torsion

The analysis was run with strength level seismic loads running in the x and y assigned to the center of mass with 5 percent accidental torsion. The worst case in deflections were found and the amplification of accidental torsional moment was determined according to ASCE 7-05 section 12.8.4.2. The amplification factor was determined to be both equal to 1 in the x and y directions. Refer to appendix for calculations.

Redundancy

After the ETABS analysis was performed, the lateral forces taken by each braced frame were calculated. The figure below represents the percentage contribution of the total applied seismic forces in the x and y direction of each braced frame at level 1. Please refer to appendix for the hand calculations of relative stiffness for comparison with the ETABS model.

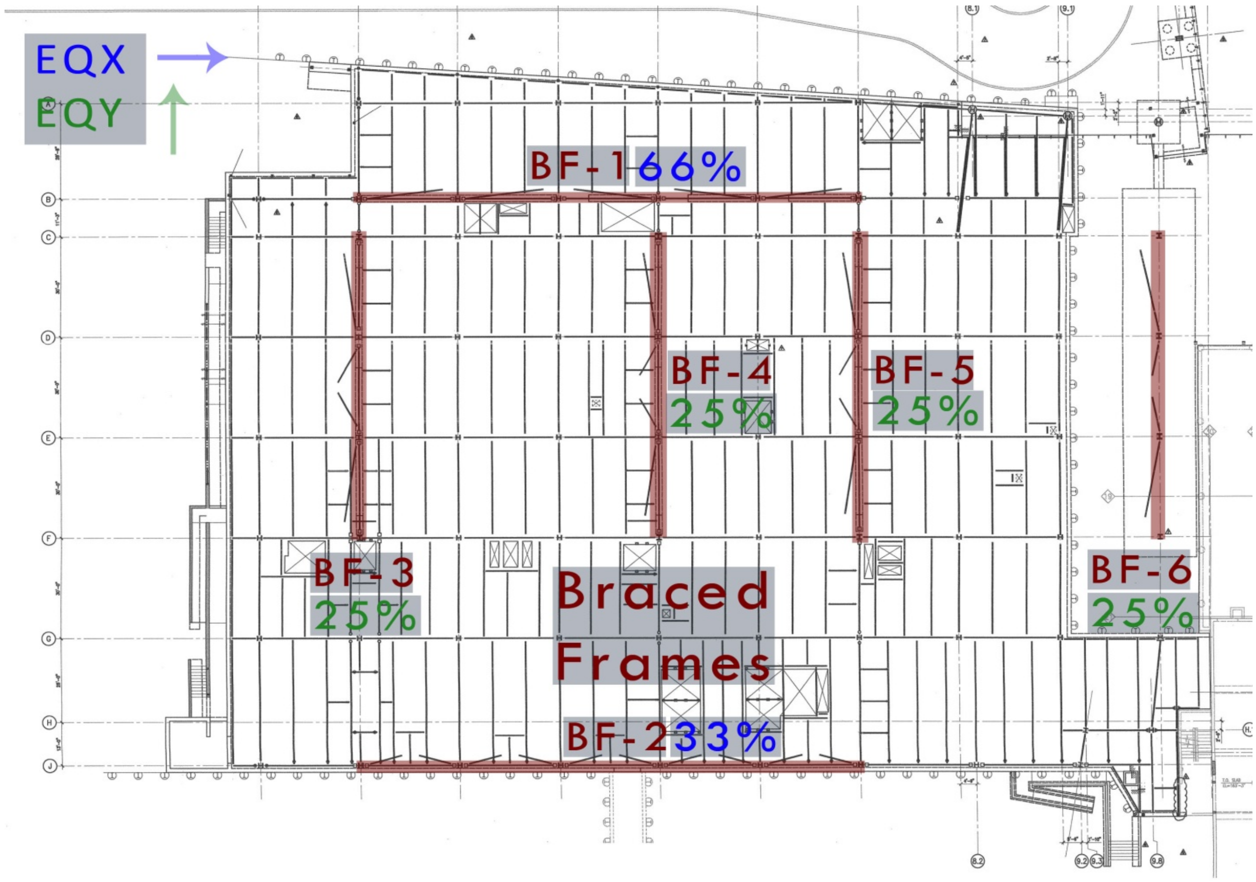


Figure 16. Distribution of Lateral Forces among frames plan.

Since BF-1 and 2 are resisting more than 33% of the base shear, requirements per ASCE 7-05 section 12.3.4.2 must be met or else the redundancy factor must be assumed as 1.3 instead of 1.0. ASCE 7-05 section 12.3.4.2.a states that a removal of any brace in the frame shall not result in a 33% reduction in story strength. A quick spot check is done as follows:

BF-1 consists of 5 braces; removal of one would result in about 20% decrease. Since BF-1 is taking 66%, 20% of 66% = 13%. Therefore a 13% decrease would result in a removal of one braced frame which is less than 33%. This complies with Section 12.3.4.2.a therefore a redundancy factor of 1.0 can be used.

Modal Period

The ETABS model's first period for the structure is at 0.4217 seconds. Compared to the calculated period based on ASCE 7-05 section 12.8.2.1 which is at .626 seconds; the ETABS model tells us that the building is stiffer than approximation by the ASCE code. This might be because the designer used the UBC code to design the building. The following figure is a 3-D view of the first mode, which shows that the building is excited in the y direction.

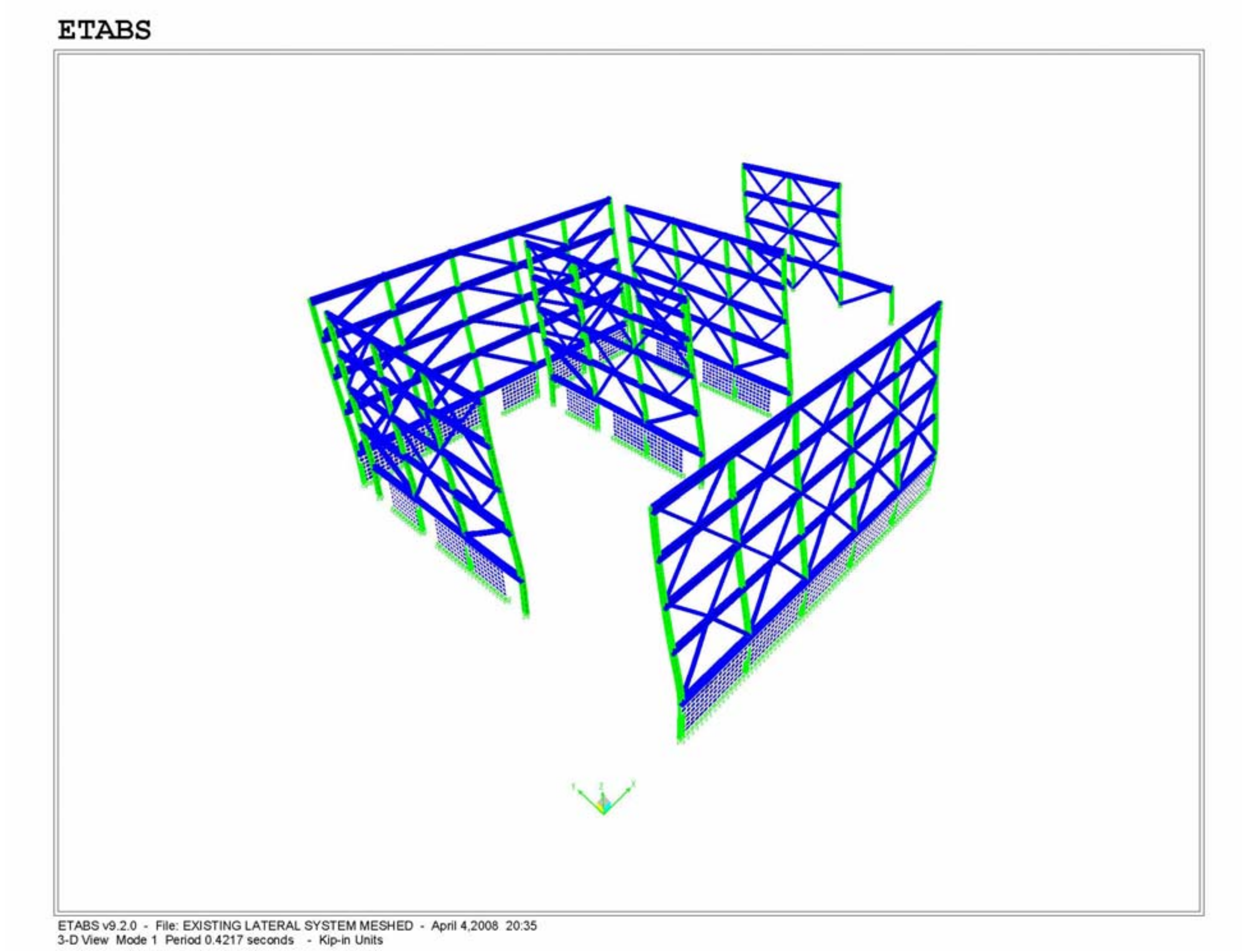


Figure 17. 3-D Modal Period

Existing Lateral System's Problem Statement

After analysis of the lateral force resisting frames with code provisions from ASCE 7-05, it was determined that the lateral resisting frames have been over designed. This may have been due to different assumptions taken by the designer when using the UBC 1997. Most structural members were well below their demand capacity ratio. If the design of the structure was less rigid, the fundamental period of the structure would have increased, which may reduce the base shear of the structure. Reducing the base shear reduces the amount of steel required by the lateral force resisting system which essentially reduces construction cost and time.

Lateral Force Resisting System Redesign

Introduction

Different lateral force resisting systems were evaluated. Based on previous technical reports, it was concluded that a steel structure was most beneficial to the patient care center due to the long spans required by the architectural layout. The ease of renovating and installing new equipment with a steel structure is advantageous over a concrete structure.

A lateral force resisting system was selected based on the following criteria, a higher R value to reduce the base shear furthermore while also having little impact on the architecture as possible. Two options were considered when redesigning the main lateral force resisting system. Moment frames, and eccentrically braced frame systems both fit the new design criteria. Moment frames offer additional architectural advantage due to the elimination of braces in the lateral system. On the other hand, eccentrically braced frames (EBF) do offer slight architectural advantage over centrically braced frames (CBF) as they do not require framing from one corner of the bay to the opposite. These two systems essentially provide greater architectural flexibility than a concentrically braced frame. Both systems offer further ductility than a CBF which essentially will increase the fundamental period of the structure, hence reducing base shear. While the response modification factors are similar according to ASCE 7-05, there lateral stiffness of both systems differs. EBF's provide further more lateral stiffness than moment frames do, therefore designing with an EBF will essentially reduce the number of bays required to be a part of the lateral system. This essentially will reduce the cost of using an EBF over a Moment Frame.

Therefore looking at an eccentrically braced frame lateral system with moment connections away from links as an alternative steel lateral force resisting system would be a good option for redesign. Due to its higher response modification coefficient and higher period approximation, there could be a potential reduction in base shear. Reduction in base shear would result in a reduction in the amount of braced frames required, potentially saving construction cost and time. Refer to the table below for a summary of the results, calculations are provided in the appendix.

System	Special Concentrically Braced Frames (Existing System)	Eccentrically Braced Frames with Moment Connections (New System)
Response Modification Coefficient (R)	6	8
Approximate Period (CuTa or Tb)	Tb = .4217 CuTa = .626	CuTa = .939
Seismic Response Coefficient (Cs)	.23	.099

Therefore using an EBF with moment connections would reduce approximately **57%** of the base shear.

EBF Design Goals

The following is a list of goals that the EBF system shall accomplish:

- Redesign of Lateral System so that the structure is less stiff, and therefore increase the modal period hence decreasing the spectral response acceleration. The following figure represents where the existing system's response acceleration is, and where the goal for the new system will be.

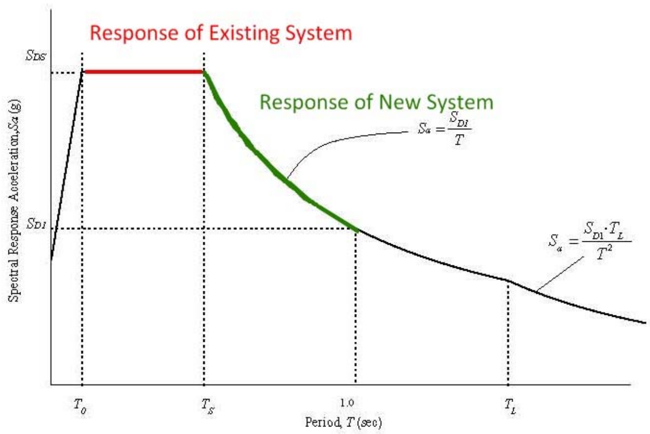


FIGURE 11.4-1 DESIGN RESPONSE SPECTRUM

Figure 18. ASCE 7-05 Design Response Spectrum Chart

- A reduction in total base shear. Which ultimately reduces:
  - Tonnage of steel used in the lateral system.
  - Required braces therefore also complex connections.
  - Construction cost.
  - Construction time

EBF Design Codes

The following is a list of design codes the EBF system shall comply with.

- American Institute of Steel Construction, Steel Manual 13th Edition
- American Institute of Steel Construction, Specification for Structural Steel Buildings (AISC 360-05)
- American Institute of Steel Construction , Seismic Provision for Structural Steel Building (AISC 341-05)
- American Institute of Steel Construction, Prequalification Connections for Special and Intermediate Steel Moment Frames for Seismic Applications (AISC 358-05)
- Federal Emergency Management Agency, Recommended Seismic Design Criteria for New Steel-Moment Frame Buildings (FEMA-350)

EBF System Design Criteria

When designing an EBF system the following measures were taken into consideration before any analysis was done.

- All wall openings were located from the plans, and minimal changes to any architectural layout were also taken into consideration. The following figures represents the location of any wall openings in the braced frame bays running in the X direction and Y direction.

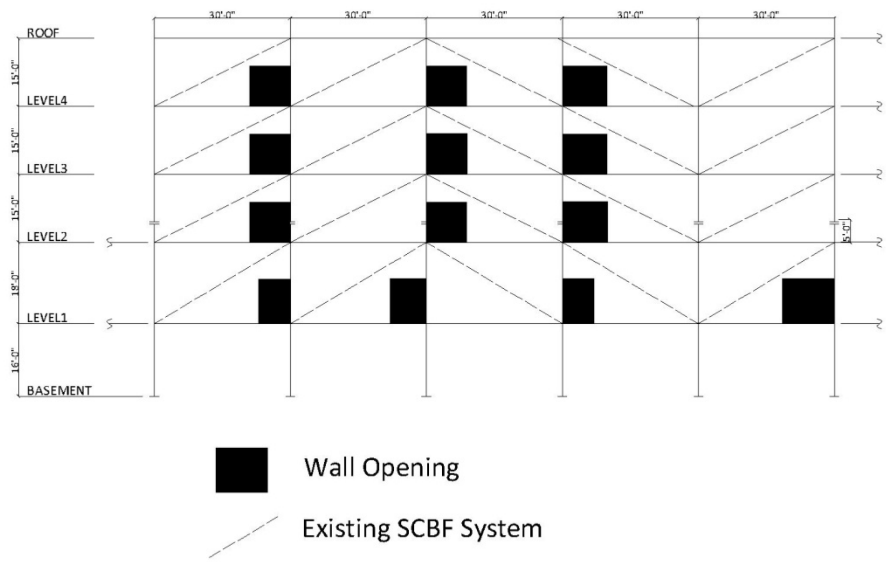


Figure 19. Existing Lateral Frame in the X Direction

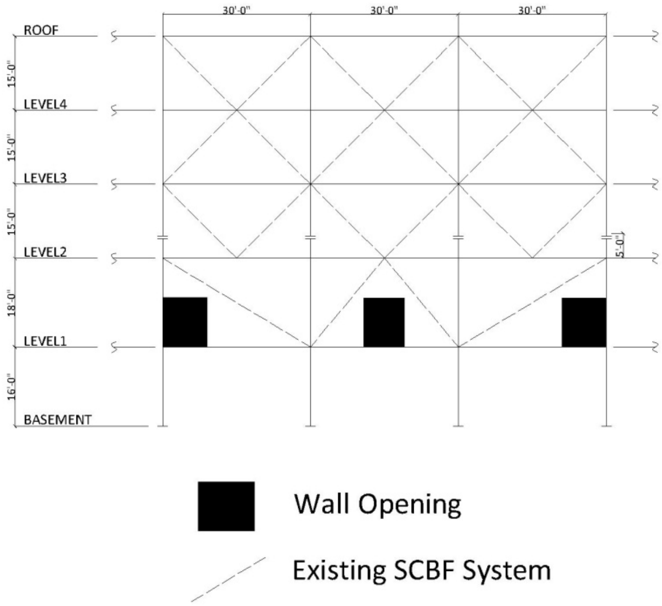


Figure 20. Existing Lateral Frame in the Y Direction

- Structural frame symmetry between the frames running in the same direction. And also symmetry within the frame itself to eliminate any torsion and to ensure the use of repetitive members throughout the lateral system.
- Using preliminary design link lengths to be .15(Bay Length) for chevron configuration and .2(Bay Length) diagonal configuration. While also trying to maintain an angle for the brace between 35 to 60 degrees. This common design practice was obtained from a publication of Steel Tips, "Seismic design practice for eccentrically braced frames by Roy Becker and Michael Ishler". To check this assumption the following calculation was done. If drift is limited to 1.5%, and link rotation to 8%, then it may be concluded that e (Link length) shall equal to 18.75% of the bay length.
- When choosing link sections, the influence of shear forces on inelastic behavior is recommended. Shear yielding is uniform and ductile therefore causing concentrated structural damage in the link. Therefore link lengths of less than 1.6(Moment Capacity/Shear Capacity) shall be designed for.
- When looking at bracing configurations, the X brace was incorporated as much as possible. X bracing minimizes the number of links in the overall system, while also isolating the link to brace connections. X bracing also minimize the axial force in the link beam which may lead to a smaller section.

EBF Design Configuration

After all the above measures were taken into account, the following bracing configuration was determined. Note that the base shear was reduced by almost 57%, therefore approximately two thirds of the bays were removed. The link lengths connected to the diagonal braces shall equal 72", while the link lengths connected to the X braces shall equal 54". 2D ETABS models were performed on different design configurations, where in figure 21, braces in the middle bays were oriented in different ways to achieve the most effective solution. The following design configurations have been determined to be the most effective solution due to the symmetry of the whole braced frame. The forces on the diagonal members are similar which leads to repetition of members with similar demand capacity ratios. Although symmetry was established an architectural impact analysis shall need to be done due to the blockage of a wall opening. Refer to next section for analysis.

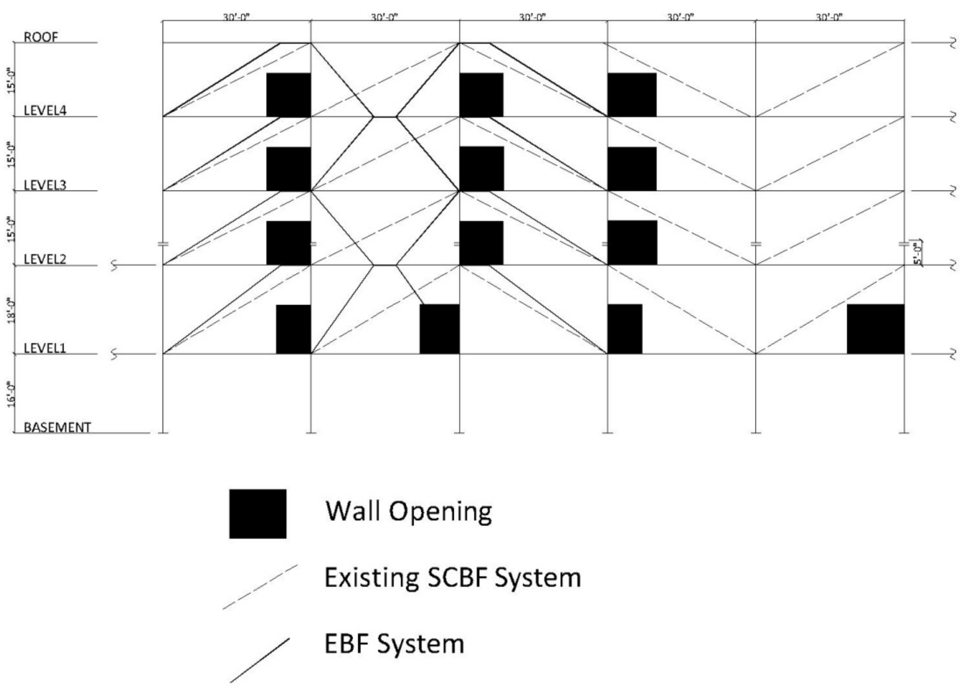


Figure 21. Proposed Lateral Frame in the X Direction

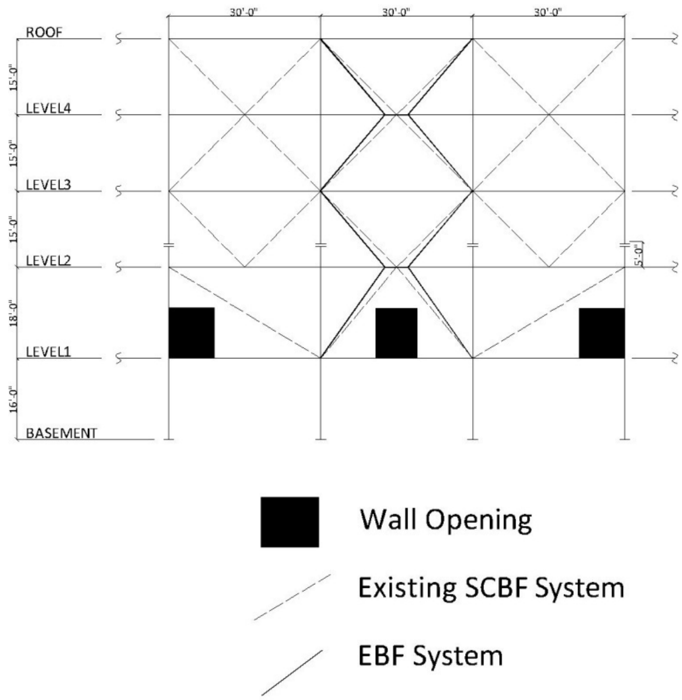


Figure 22. Proposed Lateral Frame in the Y Direction

Architectural Impact

Since one of the braces intersects a wall opening on the 1st floor next to gridline B and gridline 4. Minor architectural layout rearrangements will be required. After analyzing the space, the corridor to the administration area can be shifted, and will result in minor square footage reductions. When also looking at the mechanical duct work, the new architectural layout will need to have the duct work reconfigured to the new space location. Since there is no duct work running vertical where the impact exists, the change in duct work layout is minimal. The following are plans of the locating where the architectural impact occurs while also showing the architectural layout before and after the change.

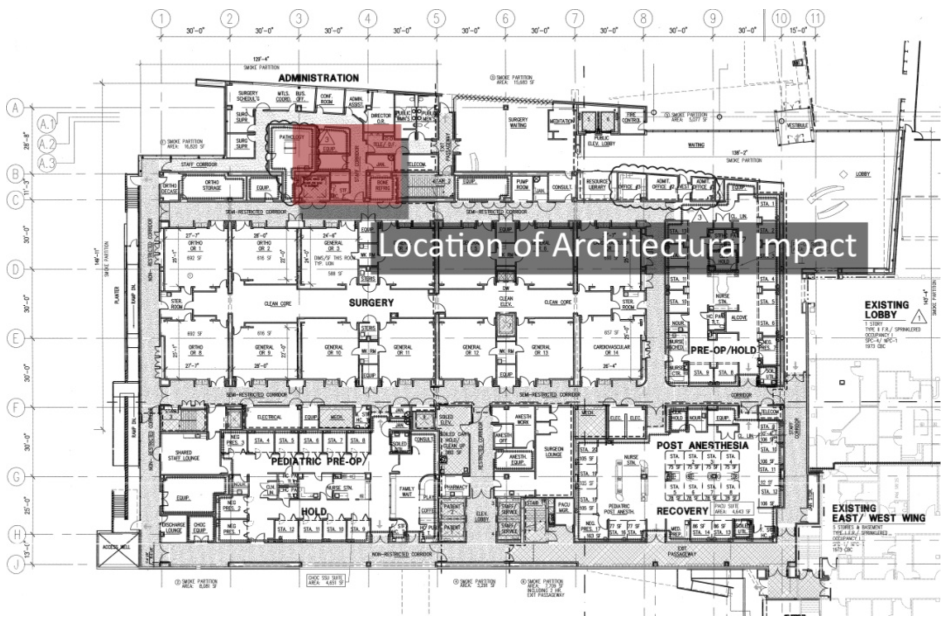


Figure 23. Location of Architectural Impact

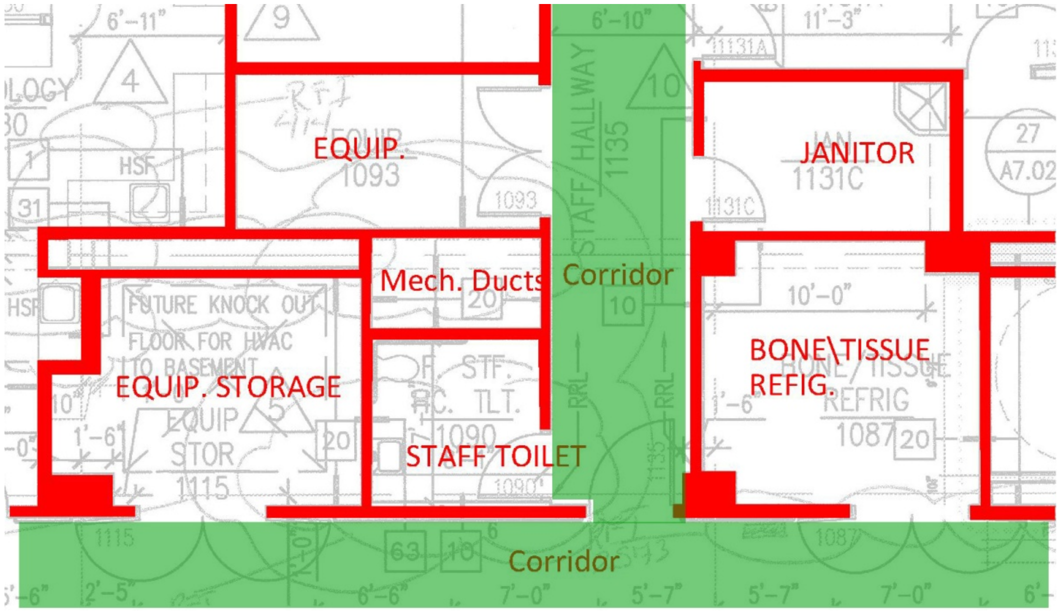


Figure 24. Original Architectural Layout

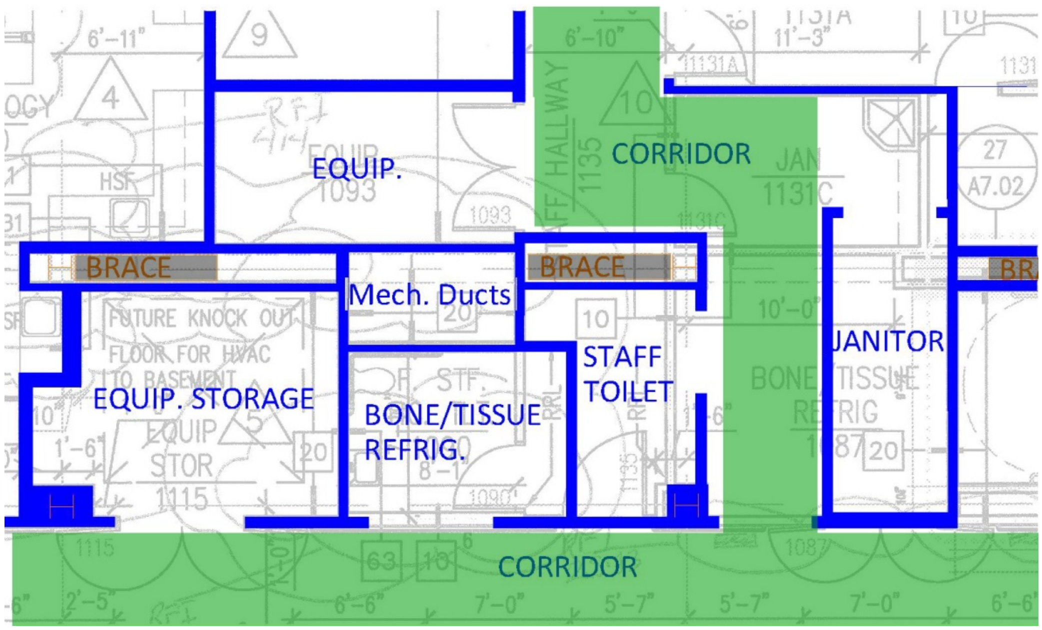


Figure 25. Architectural Layout Change

The following table represents the spaces affected by the rearrangements of space and how their square footages have changed. There is significant reduction in square footage of certain spaces, meanwhile the advantages of the structural changes proposed are numerous and extremely beneficial and shall still be considered. Further detailed architectural impact analysis would be required to ensure minimal space square footage changes.

Spaces	Area Before (SF)	Area After (SF)	Percentage Change
Staff Toilet	60	70	+17%
Bone Tissue Refrigerator	120	83	-31%
Janitor	80	78	-3%

The following is a line diagram of all the members in the braced frame with the revised wall openings.

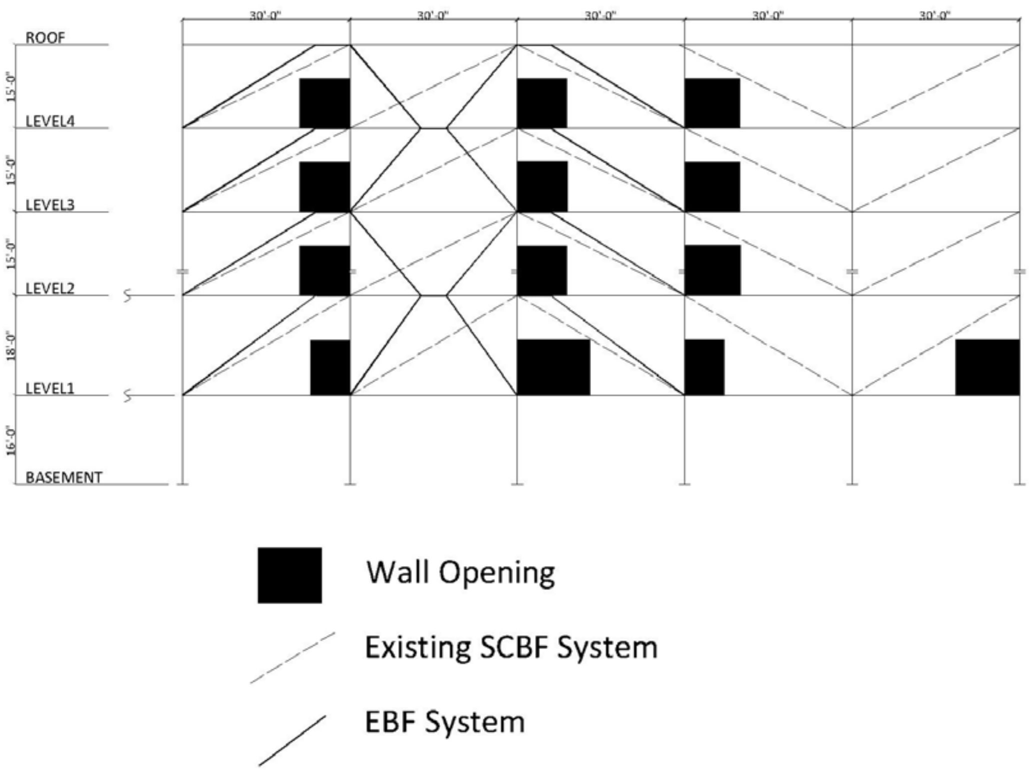


Figure 26. BF-1 Revised Wall Openings

The removal of 2 bays from each brace frame set lead to a decrease of braces blocking windows in the patient rooms looking out into the courtyard and outside. The following is a plan showing where a clear view to the outside now exists and where the new braces are located.

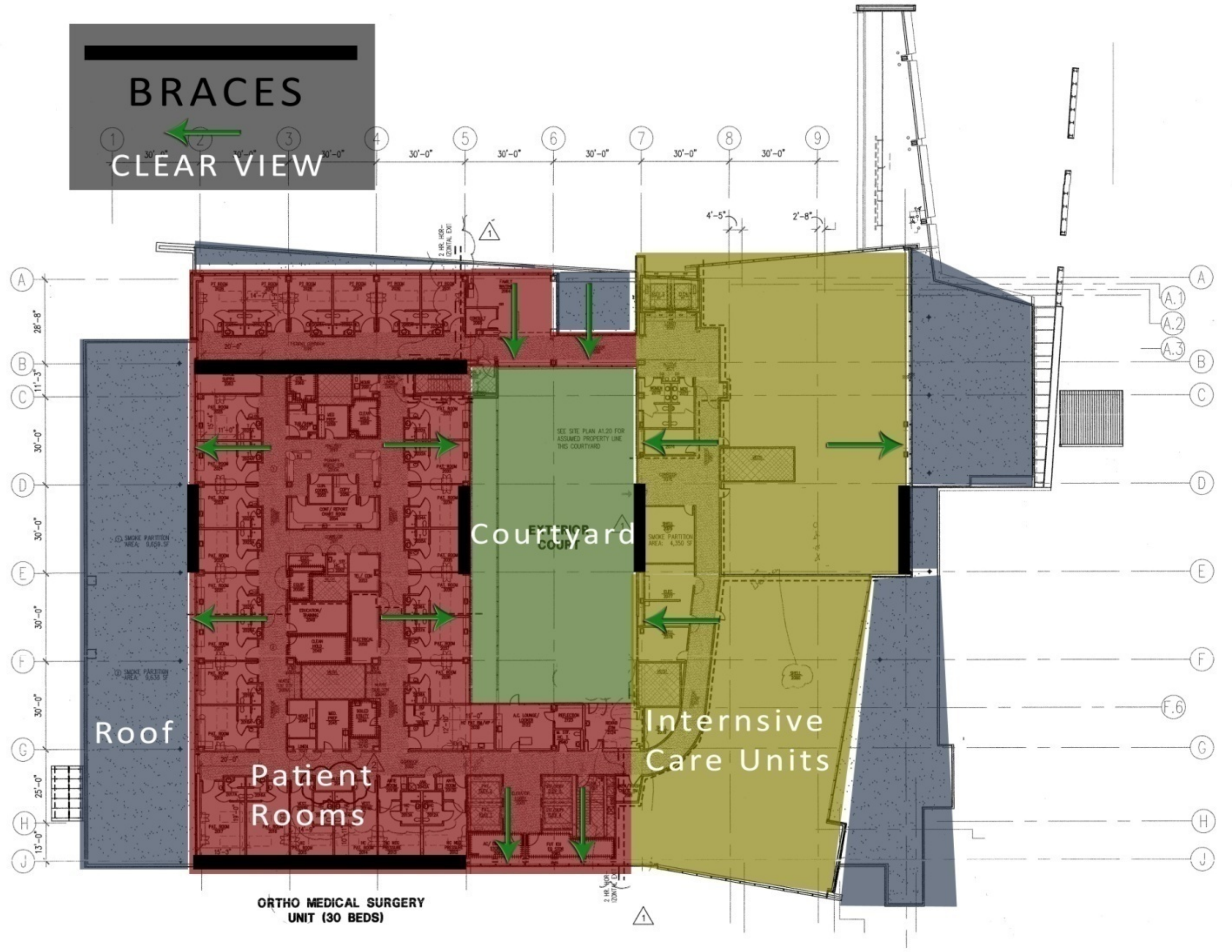


Figure 27. Architectural plan of level 2 showing where the new clear views to the outside exist.

Design Process

The following represents a flow chart of the design process undertaken of the EBF system. Due to the time constraint of the student semester there are other structural issues that would need to be considered for the design to be fully complete. Refer to Page 68 for more information.

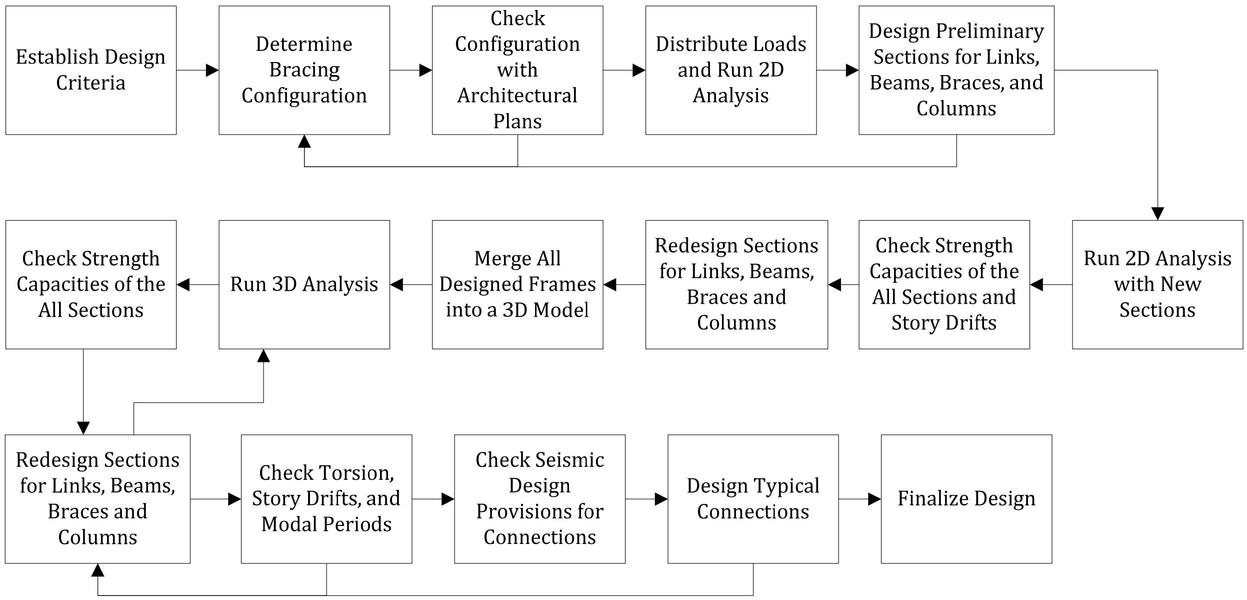


Figure 28. EBF Design Process Flow Chart

Member Design

In an EBF system the design of the links controls the design of all other structural elements (i.e., beams, braces and columns). Once the links sections are determined, the over-strength factor is calculated and used to design the beams, braces and columns that are all connected to the link. The over strength factor is a ratio of the shear the link is resisting due to earthquake loads to the overall shear capacity of the link. The following flow chart represents the steps taken when designing the members on an EBF.

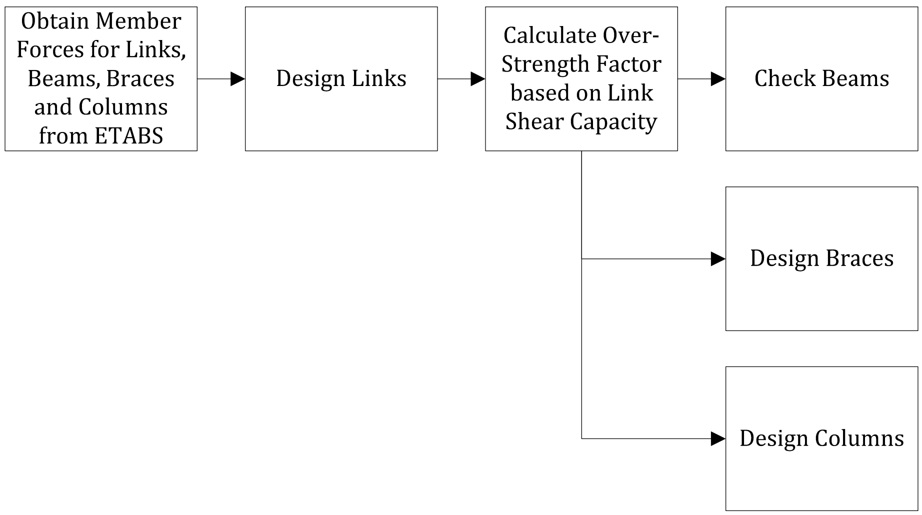


Figure 29. Member Design Flow Chart

Designing the Links

The following figure is flow chart design process of a link. Refer to next section to conclude the design iterations of the link.

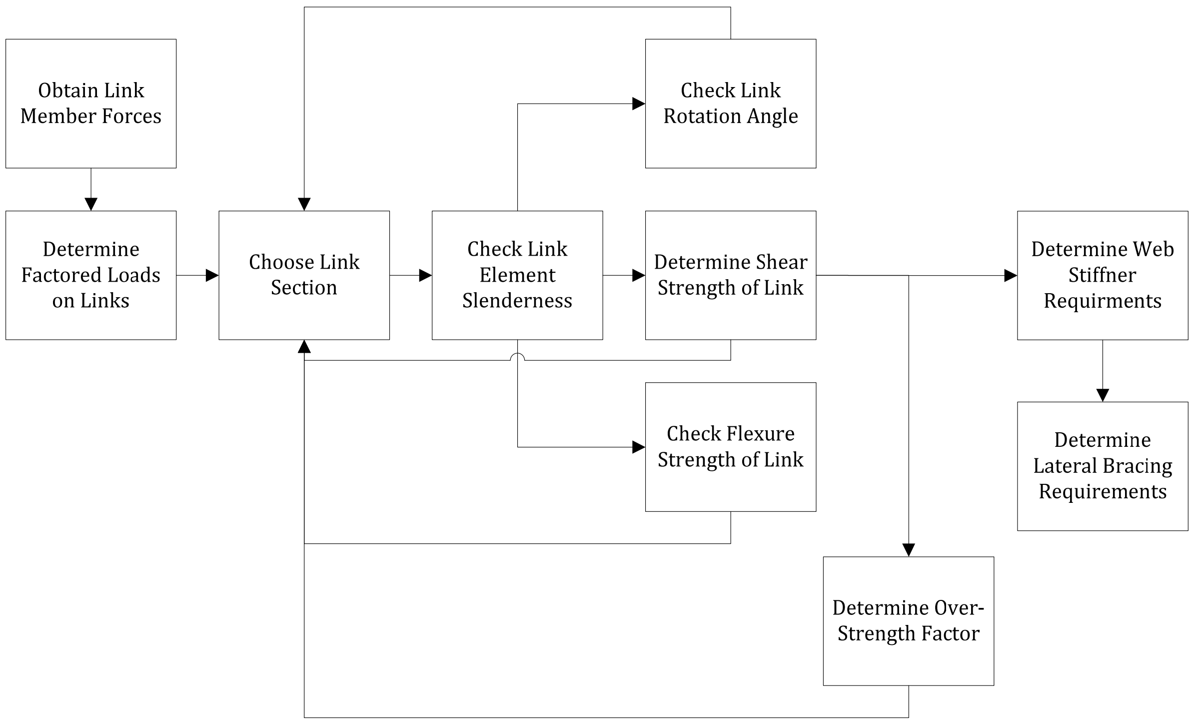
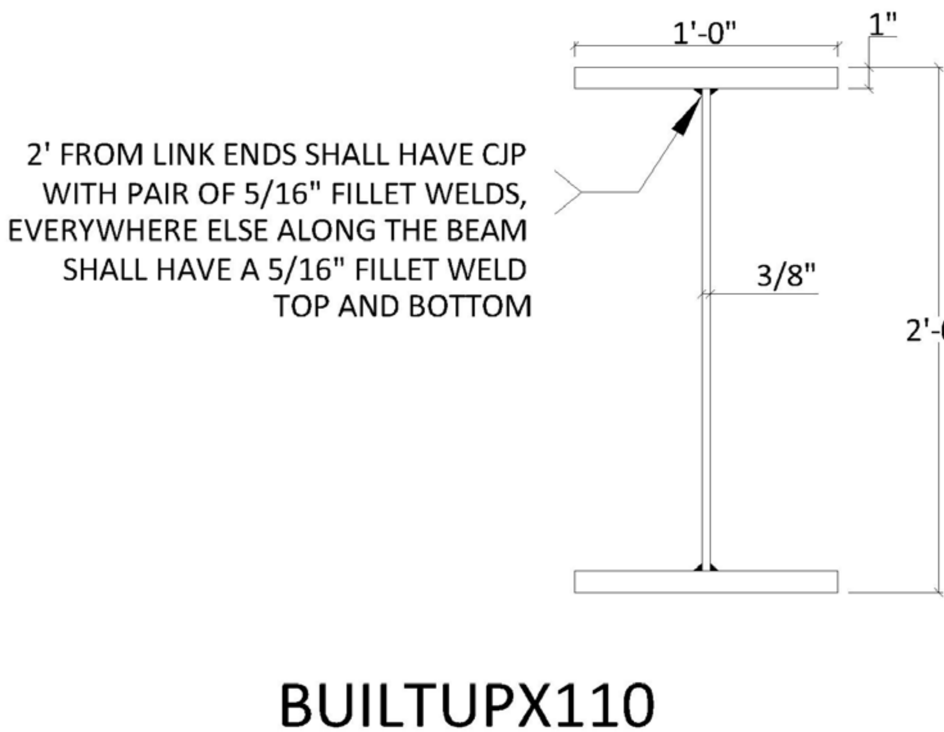


Figure 30. Link Design Flow Chart

Built Up Section

After a couple of design iterations, it has been concluded that a built up section would be the most sufficient for the links connected to the diagonal braces in BF-1 and BF-2. Due to the high requirement of flexural capacity in the beams outside the links and the links themselves; there was an insufficient amount of rolled up sections that maintained the right amount of shear capacity while having a high flexural capacity. Therefore to avoid a high over-strength factor which will essentially increase the sizes of all other structural components connected to the link. The following is a detail of a customized built up section with its properties geared towards the applicable loading scenario present in BF-1 and BF-2. A flange thickness of 1" was mandated in the requirements for a prequalified stiffened extended end plate moment connection in AISC 358-05. Flange to web connection was determined according to AISC 358-05 section 2.3.2.a.



d=	24"	$b_{f=}$	12"
t_w=	3/8"	A_g=	32.25 in^2
J=	7.96	C_w=	38,088
I_x=	3509 in^4	I_y	288 in^4
S_x=	292 in^3	S_y=	38 in^3
Z_x=	321 in^3	Z_y=	72.8 in^3
r_x=	10.4 in	r_y=	2.99 in
h/w=	61.33	w_{self}=	110 plf

Figure 31. Built Up Section with Properties

Check Beams Outside of Links

The following figure is flow chart design check for the beams outside of links. If the beams are inadequate then another link section is selected, most likely one with higher flexural capacity.

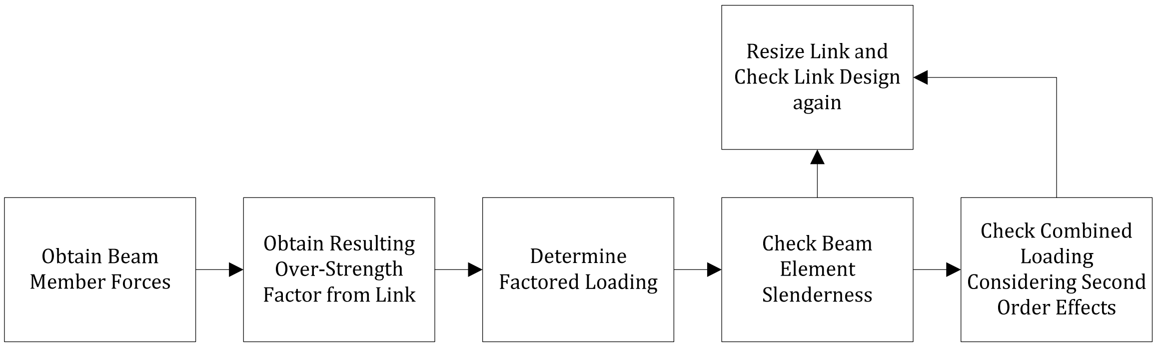


Figure 32. Beam Design Flow Chart

Designing Braces

HSS shapes were chosen specifically for the braces; the connections are much simpler and easier than W shapes and tend to save construction time. Refer to the Typical Connection sections for connection details. The following is flow chart design process of a brace.

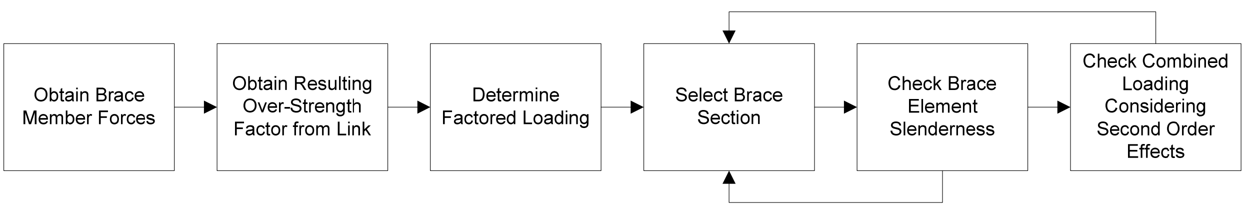


Figure 33. Brace Design Flow Chart

Designing Columns

W14 Shapes are typically used in building in high seismic regions and were also used when designing the EBF system. The following is flow chart design process of a column.

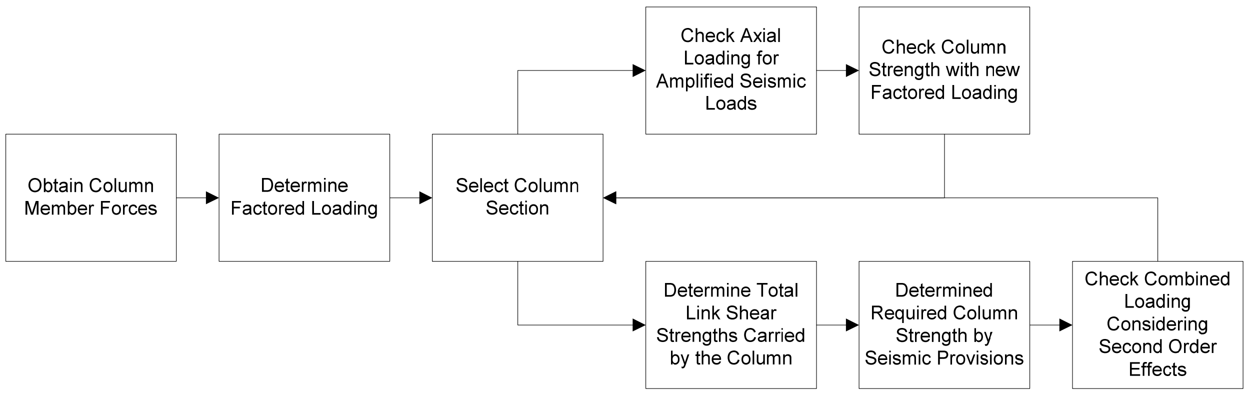


Figure 34. Column Design Flow Chart

ETABS Modeling

The following modeling assumptions were taken into account when performing the preliminary 2D model for the sizing of members, and then the final 3D model of the whole lateral system for the iterative design process.

2D Model Assumptions

- Seismic forces were increased by 10% to account for torsion and divided by the number of EBF sets to be design for in that direction.
- Semi rigid diaphragm was assigned to each story level so that axial loads in the beams and links can be considered.
- Refer to the rest of assumptions on page 25.

3D Model Assumptions

- Seismic forces were assigned to the center of mass of the diaphragm.
- Rigid diaphragms were assigned to each story level.
- Refer to the rest of assumptions on page 25.

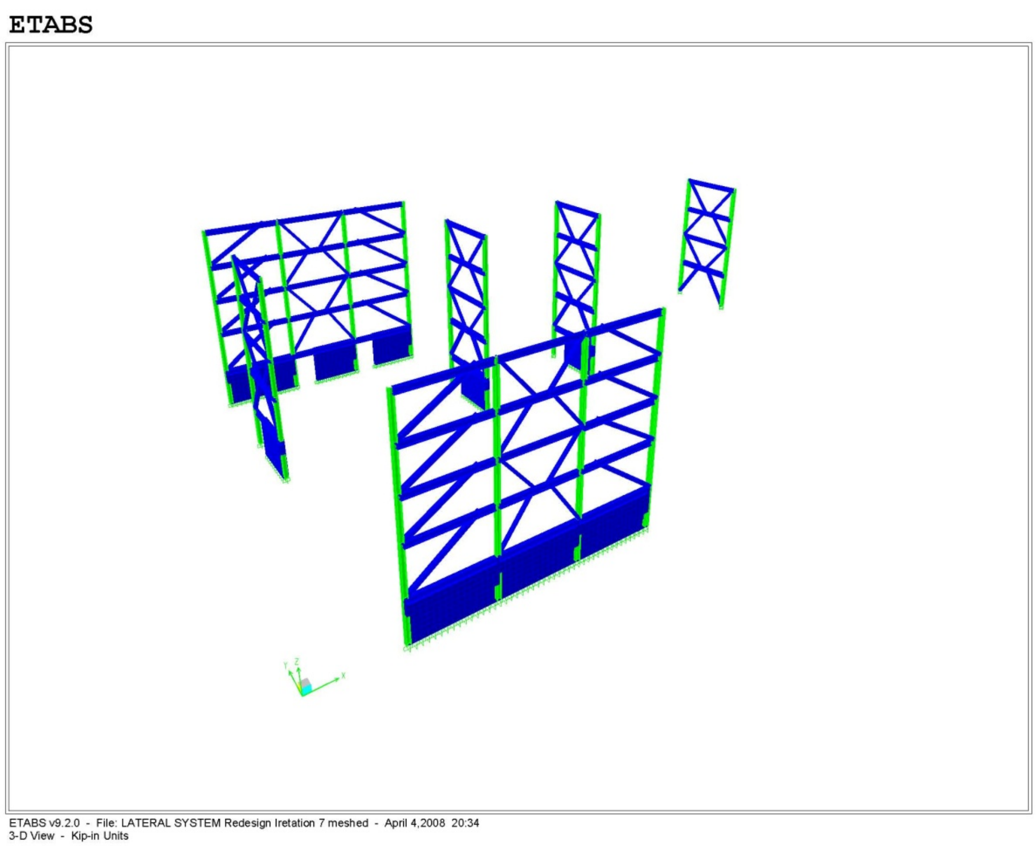


Figure 35. 3D ETABS Model of the EBF system

Demand Capacity Ratios of the EBF system.

The following are spread sheet summaries with the required to available strength capacity ratios of all the designed structural members in the system. Note that a 5% standard of care was taken into consideration, which means if the beam was under designed by 5%, it is still considered adequate.

Links Design Spread Sheet Summary

The built up section was used entirely for the links connected to the diagonal braces. Repetition was ensured as much as possible. Taking into consideration that overdesigning the links may lead to overdesigning of all connected structural elements. The following is a summary of the demand capacity ratios of all designed links in the EBF.

Link Design Summary

Link Location	Link Shape	e (ft)	e/(Mp/Vp)	Vu/ΦVn	Checks Shear	Flexural	Rotation
BF-1 BAY 2-3							
BF-1 Roof B1	BUILTUP4	6	1.110	0.534	OK	OK	OK
BF-1 4th B1	BUILTUP4	6	1.110	0.602	OK	OK	OK
BF-1 3rd B1	BUILTUP4	6	1.110	0.732	OK	OK	OK
BF-1 2nd B1	BUILTUP4	6	1.110	0.983	OK	OK	OK
BF-1 BAY 3-4							
BF-1 4th B2	W24X146	4.5	1.135	0.554	OK	OK	OK
BF-1 2nd B2	W24X146	4.5	1.135	0.800	OK	OK	OK
BF-3							
BF-3 4th Floor	W24X103	4.5	1.435	0.780	OK	OK	OK
BF-3 2th Floor	W30X148	4.5	1.194	0.941	OK	OK	OK
BF-6							
BF-6 4th Floor	W24X103	4.5	1.435	0.831	OK	OK	OK
BF-6 2th Floor	W30X148	4.5	1.194	1.013	Not OK	OK	OK

Link Web Stiffeners and Lateral Bracing Design Spread Sheet Summary

The following is a design summary of the link web stiffeners required with their spacing. To ensure repetition of stiffener plates, intermediate and link end web stiffeners were designed to the same thickness. The lateral link brace was also all designed to a section that was compatible to all links. Minor variation in the required strength of the lateral braces made repetition possible.

Link Design Summary						
Link	**Link**	**Link Stiffeners**			**Link Lateral Brace Design**	
Location	**Shape**	**Width (in)**	**t (in)**	**Spac. (in)**	**Section**	**Pb (kips)**
BF-1 BAY 2-3						
BF-1 Roof B1	BUILTUP	5.75	0.375	14.7	L6X6X5/8	46.1
BF-1 4th B1	BUILTUP	5.75	0.375	14.5	L6X6X5/8	46.1
BF-1 3rd B1	BUILTUP	5.75	0.375	13.6	L6X6X5/8	46.1
BF-1 2nd B1	BUILTUP	5.75	0.375	13.1	L6X6X5/8	46.1
BF-1 BAY 3-4						
BF-1 4th B2	W24X146	6	0.75	26.9	L6X6X5/8	58.4
BF-1 2nd B2	W24X146	6	0.75	23.6	L6X6X5/8	58.4
BF-3						
BF-3 4th Floor	W24X103	4	0.625	16.1	L6X6X5/8	39.3
BF-3 2th Floor	W30X148	4.75	0.75	21.5	L6X6X5/8	55.9
BF-6						
BF-6 4th Floor	W24X103	4	0.625	16.1	L6X6X5/8	39.3
BF-6 2th Floor	W30X148	4.75	0.75	21.5	L6X6X5/8	55.9

Beams Outside of Link Design Spread Sheet Summary

The following is a design check summary of the beams outside of the links.

Beam Outside of Links Design Summary					
Location	Shape	Over-Strength Factor	Lb (ft)	H1-1	Check
BF-1 BAY 1					
BF-1 Roof B1	BUILTUP	5.08	10	0.762	OK
BF-1 4th B1	BUILTUP	2.83	10	0.753	OK
BF-1 3rd B1	BUILTUP	2.22	10	0.829	OK
BF-1 2nd B1	BUILTUP	1.52	10	0.647	OK
BF-1 BAY 2					
BF-1 4th B2	W24X146	2.60	10	0.905	OK
BF-1 2nd B2	W24X146	1.74	10	0.840	OK
BF-3					
BF-3 4th Floor	W24X103	1.76	10	1.021	Not OK
BF-3 2th Floor	W30X148	1.46	10	0.825	OK

Beams Design Spread Sheet Summary

The following is a design summary of the beams outside the EBF (not connected to any links).

Beam Design Summary				
Location	**Shape**	**Lb (ft)**	**H1-1**	**Check**
BF-1				
BF-1 ROOF	W24X55	10	0.646	OK
BF-1 LVL3	W24X55	10	0.164	OK
BF-1 LVL 1	W30X116	10	0.978	OK
BF-3				
BF-3 ROOF	W24X55	10	0.648	OK
BF-3 LVL3	W24X55	10	0.373	OK
BF-3 LVL 1	W30X116	10	0.634	OK

Braces Design Spread Sheet Summary

The following is a design summary of the braces connected to the links.

Brace Design Summary						
	Connected	**Over-Strength**	**Brace**	**Unbraced Length**	**Combined Loading**	
Location	**Link**	**Factor**	**Shape**	**Lb (ft)**	**H1-1**	**Check**
BF-1 Bay 1						
BF-1 Roof B1	BUILTUP4	5.77	HSS14X14X5/8	28.3	0.87	OK
BF-1 4th B1	BUILTUP4	3.21	HSS14X14X5/8	28.3	0.82	OK
BF-1 3rd B1	BUILTUP4	2.52	HSS14X14X5/8	28.3	0.81	OK
BF-1 2nd B1	BUILTUP4	1.73	HSS14X14X5/8	30	0.74	OK
BF-1 Bay 2						
BF-1 4th B2	W24X146	2.96	HSS10X10X5/8	19.69	0.64	OK
BF-1 4th B2	W24X146	2.96	HSS10X10X5/8	19.69	0.96	OK
BF-1 2nd B2	W24X146	1.98	HSS10X10X5/8	19.69	0.76	OK
BF-1 2nd B2	W24X146	1.98	HSS10X10X5/8	22.06	0.95	OK
BF-3						
BF-3 4th Floor	W24X103	2.00	HSS10X10X5/8	19.69	0.50	OK
BF-3 4th Floor	W24X103	2.00	HSS10X10X5/8	19.69	0.82	OK
BF-3 2th Floor	W30X148	1.66	HSS10X10X5/8	19.69	0.79	OK
BF-3 2th Floor	W30X148	1.66	HSS14X14X5/8	22.06	0.70	OK

Columns Design Spread Sheet Summary

The following is a design summary of the columns. The governing total nominal shear strength capacity the column would take from the links was determined, which was then used to size the column.

Column Design Summary				
	Column	Link	Combined Loading	
Location	Shape	Σ1.1RyVn	H1-1	Check
BF-1 Grid 2				
BF-1 LOWER	W14X233	1200.32	0.870	OK
BF-1 UPPER	W14X176	900.24	0.766	OK
BF-1 Grid 3				
BF-1 LOWER	W14X233	1200.32	0.863	OK
BF-1 UPPER	W14X176	900.24	0.840	OK
BF-3				
BF-3 LOWER	W14X233	1119.25	0.829	OK
BF-3 UPPER	W14X176	784.685	0.742	OK

Story Drifts

The following tables represent the story displacements based on the strength level applied seismic loads in the ETABS model. A deflection amplification factor equal to 4 was used to amplify the drift. The story drift limit is 1.5% of the story height, according to ASCE 7-05 provisions.

X Direction					
Story	hx (ft)	Drift Ratio	Allowable Drift (in)	Story Drift (in)	Check?
Roof	15	0.001007	2.7	0.48336	OK
4	15	0.001271	2.7	0.61008	OK
3	15	0.001667	2.7	0.80016	OK
2	18	0.001901	3.24	1.094976	OK

Y Direction					
Story	hx (ft)	Drift Ratio	Allowable Drift (in)	Story Drift (in)	Check?
Roof	15	0.002433	2.7	1.16784	OK
4	15	0.002599	2.7	1.24752	OK
3	15	0.002789	2.7	1.33872	OK
2	18	0.002071	3.24	1.192896	OK

Modal Period

The following figure represents a 3D model in ETABS of the structure's first modal excitation in the y direction. The first modal period is at 1.1453 seconds, which is more than the ASCE 7-05 approximation therefore it is not required to recompute the base shear coefficient.

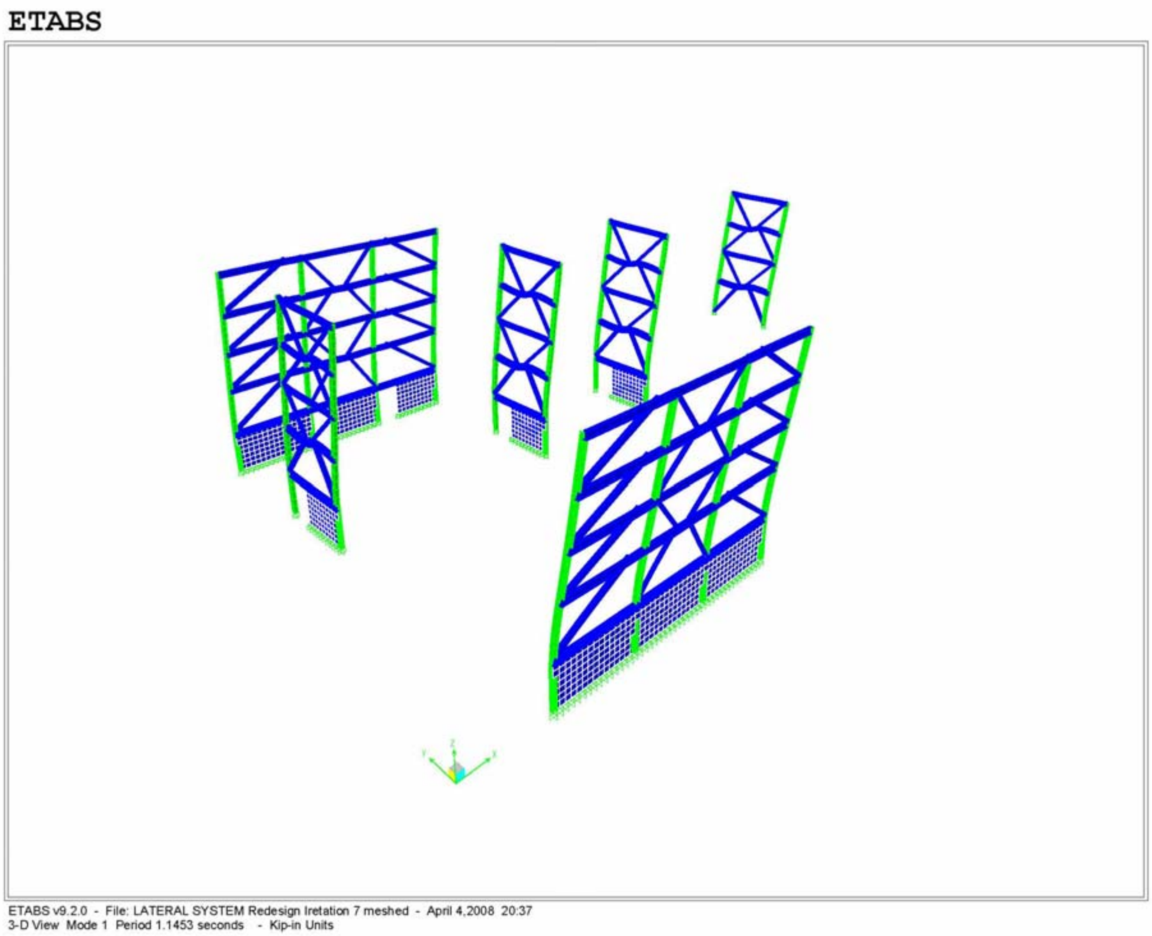


Figure 36. 3D Model in ETABS of the structure's first modal excitation.

Design Check Summary

The table below is a design check that was performed after the final iteration. Refer to appendix for calculations

Check	Comment	Observation
Story Drifts	Allowable story drifts for each level are met in each of the two orthogonal directions. Although the computed story drifts is at most 50% of the allowable.	OK
Torsion	Accidental Torsion = 5%	OK
Redundancy	Adequate amount of braces in each direction resisting less than 33% of the total story shear.	OK
Modal Period	ASCE 7-05 Approximate Period: 1.1453 seconds ETABS Model Period: .939 seconds The structure's first modal period is excited in the y direction. Refer to figure 54 for a 3D model in ETABS.	OK
Required-Available Strength Capacity Ratio	Most structural members are within 75% to 100% of their total strength capacity, members who are below that are oversized, are due to the repetition of similar sections in other bays.	OK

EBF Final Design

The following are plans of the designed EBF system.

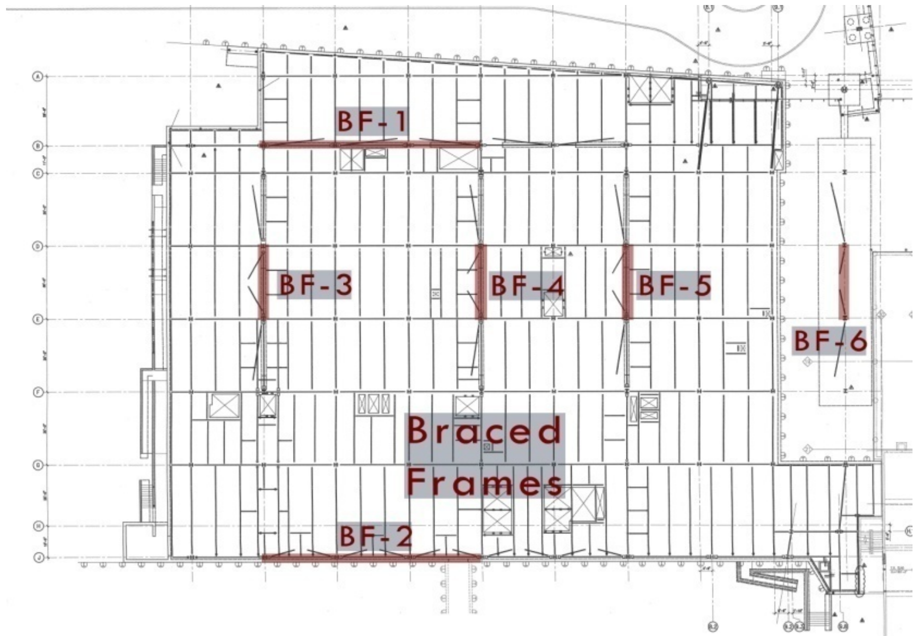


Figure 37. Level 1 plan with EBF system location

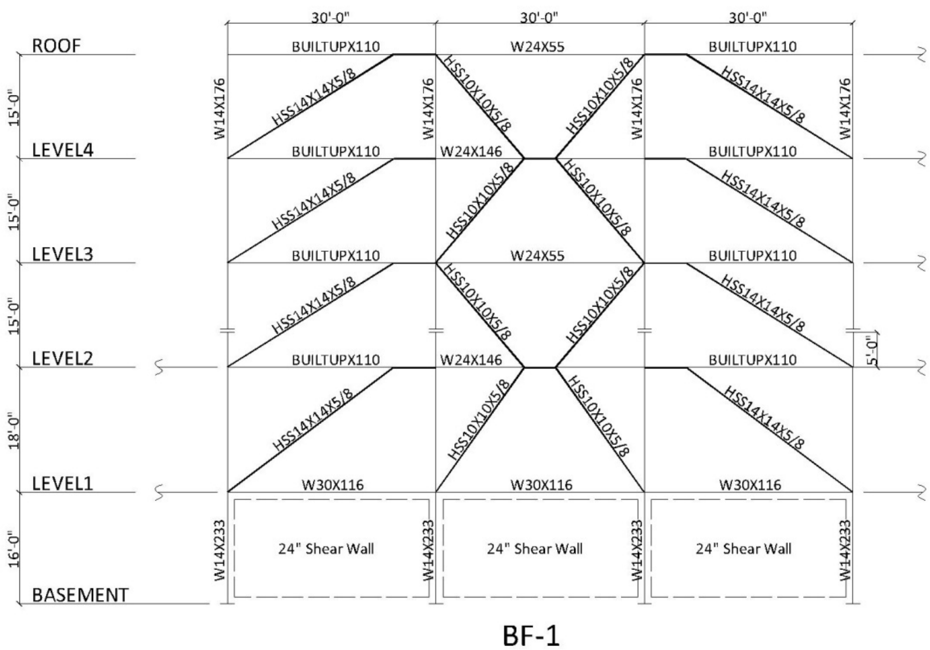


Figure 38. Elevation of BF-1

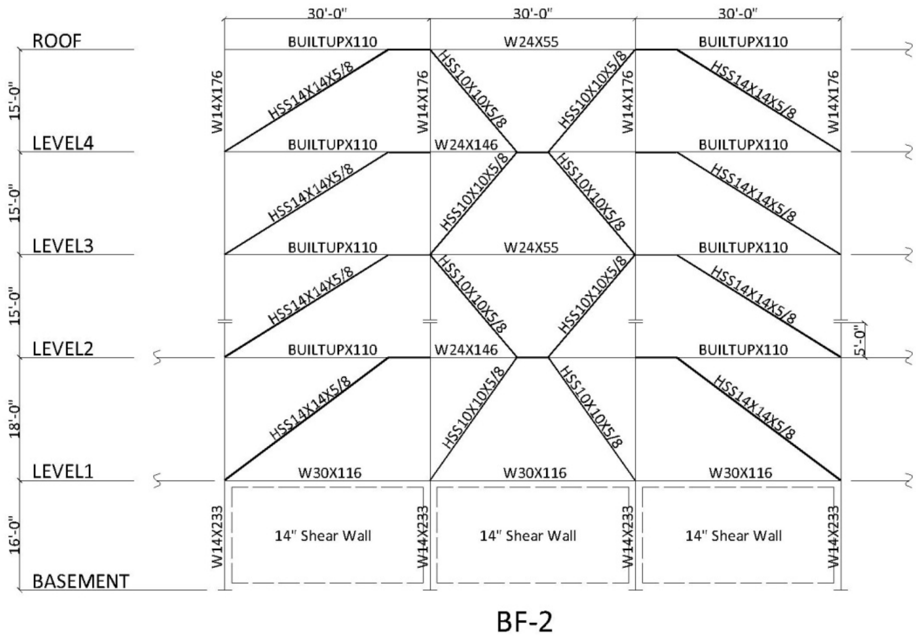


Figure 39. Elevation of BF-2

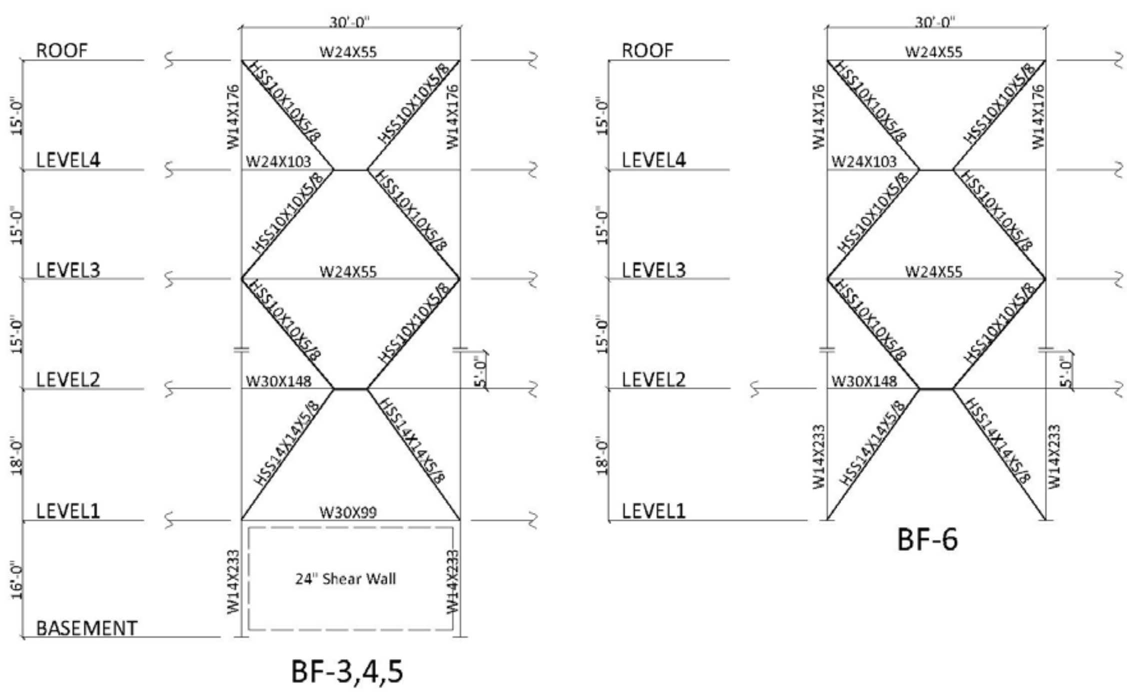


Figure 40. Elevations of BF-3,4,5,6

Typical Connections

The following section represents details and designs of typical connections in the EBF system.

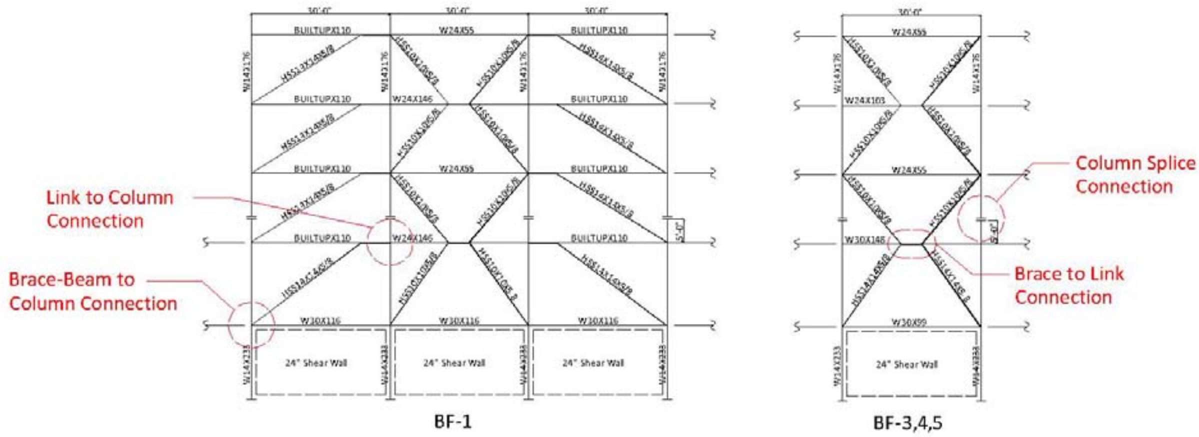


Figure 41. Location of Typical Connections in Elevation view.

Link-Column Connection

The link to column connection is designed according to AISC 341-05 section 15.4.a. The seismic provisions state that the connection shall go through cyclic testing, or that the connection is to be designed according to one of the prequalified connections in AISC 358-05. After checking the prequalified connections in AISC 358-05, a bolted stiffened end plate moment connection with a total of 8 at each flange can be used. Although after further investigation of AISC 358-05 Section 15.4, the following exception shall also be taken into consideration for a different alternative connection:

"Exception: Where reinforcement at the beam-to-column connection at the link end precludes yielding of the beam over the reinforced length, the link is permitted to be the beam segment from the end of the reinforcement to the brace connection. Where such links are used and the link length does not exceed 1.6Mp /Vp, cyclic testing of the reinforced connection is not required if the available strength of the reinforced section and the connection equals or exceeds the required strength calculated based upon the strain-hardened link as described in Section 15.6. Full depth stiffeners as required in Section 15.3 shall be placed at the link-to-reinforcement interface."[1]

Since all link to column connections are designed with the Builtupx110 section. Builtupx110 has a 1" thick flange which meets the maximum allowed flange slenderness ratio. The built up section is also 24" deep and has a greater flange thickness than most W24 rolled shapes. Therefore it may be considered that the built up section is similar to a W24 rolled section but with a reinforced flange. And since the web of the link is connected to the column with a shear tab which is welded to the

[1]*American Institute of Steel Construction, Seismic Provisions for Structural Steel, (AISC 341-05 Section 15.4)*

And since the web of the link is connected to the column with a shear tab which is welded to the column with a complete joint penetration, then the web can also be considered to be reinforced. Due to the reinforcement at the link end to column face; it ensures that any inelastic shear yielding will occur at the link and not at the connection. All link lengths at the columns are also less than 1.6Mp/Vp. Therefore the exception's criteria have been met, and a non-qualified link to column connection can also be designed.

Option 1: Bolted Stiffened Extended End-Plate Moment Connection

The following figure is a detail of the link to column connection.

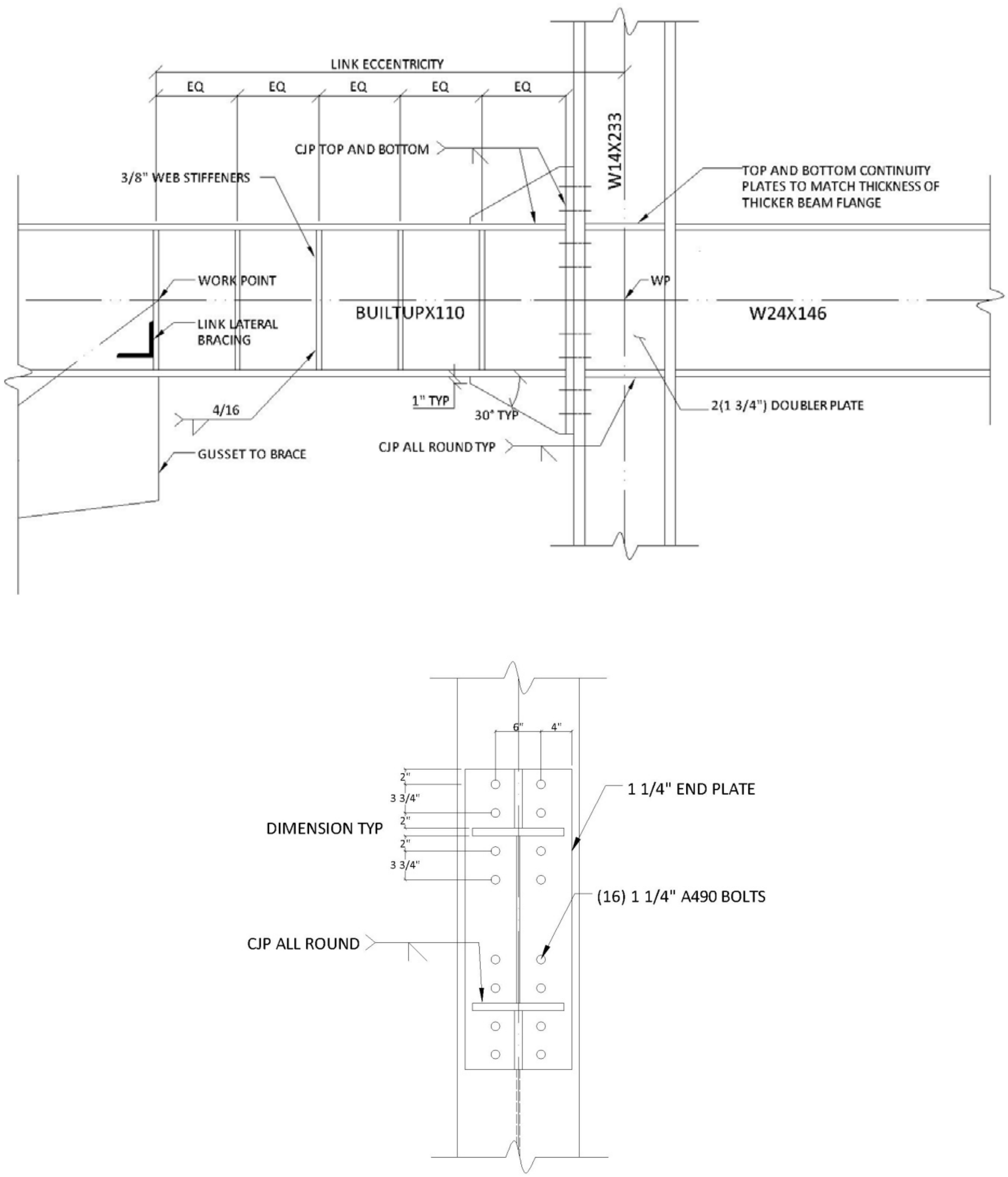


Figure 42. Link to Column Connection Option 1

The following is a list of limit states taken into consideration when designing the connection. Refer to appendix for calculations.

- For bolts:
  - Bolt shear yield and rupture
  - Bolt tension yield
  - Bolt bearing and tear out failure
- For end plate:
  - Shear yield
- For column:
  - Flange shear yielding
  - Flange flexural yielding
  - Web yielding
  - Web buckling
  - Web crippling
  - Panel zone shear yielding

Option 2: Welded Flange, Welded Web Moment Connection.

The following figure is a detail of the link to column connection.

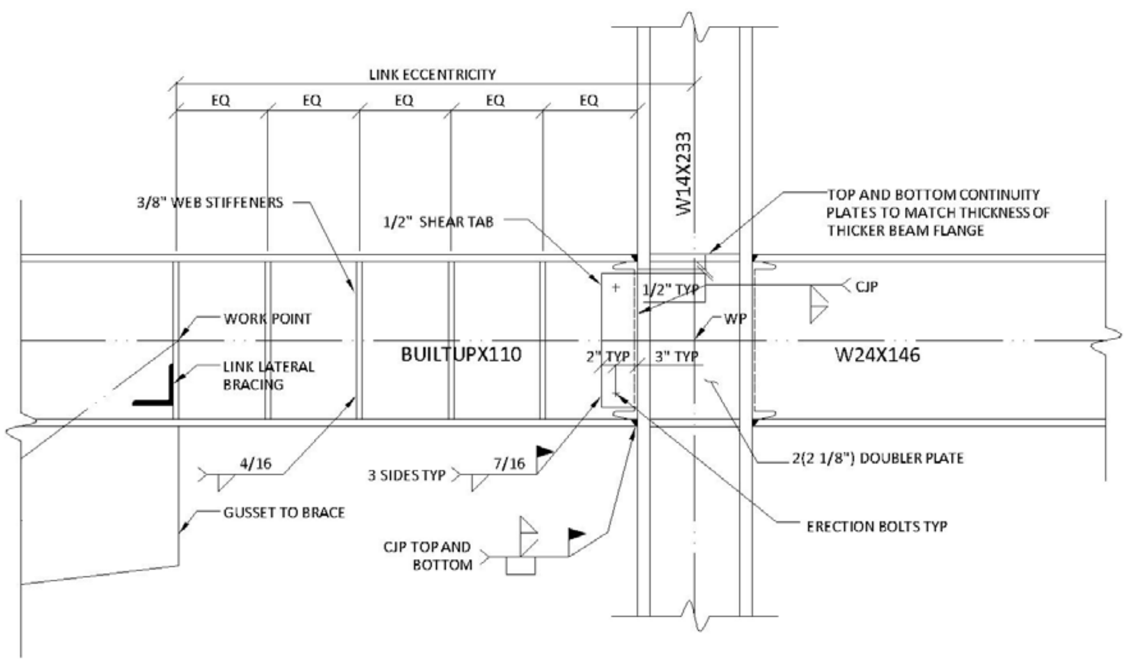


Figure 43. Link to Column Connection Option 2.

The following is a list of limit states taken into consideration when designing the connection. Refer to appendix for calculations.

- For column:
  - Column Flange flexural yielding

  - Column Web local yielding
  - Column Web buckling
  - Column Web crippling
  - Column Panel zone shear yielding
  - Column rupture at welds
  - Column shear yield at welds
- For beam:
  - Beam shear yield at shear tab
  - Beam Rupture at welds
  - Beam Shear yield at welds
- For beam web shear tab:
  - Shear tab shear yield
  - Weld rupture due to eccentric loadings

When further looking into the constructability of both options. Option 1 would be most economical, as it eliminates all CJP welds onsite during erection. This insures easier construction, while also increasing the rate of erection. Meanwhile if you look at the actual eccentricity length (unreinforced length) of both link connections, option 1 offers greater actual eccentricity length which decreases the rotation of the link. Therefore before choosing which option to pursue, a final link rotation check must be performed to insure the link is adequate. Refer to page 69 for final link rotation check. However since link rotations are well below 50% of their allowable rotation. It is permissible to use any of the 2 options.

Brace-Link Connection

The following connection detail has been designed in accordance with AISC 341-05 Section 15.6c. The following detailed also shows a typical detail of the link stiffeners which have been designed in accordance with AISC 341-05 Section 15.3.

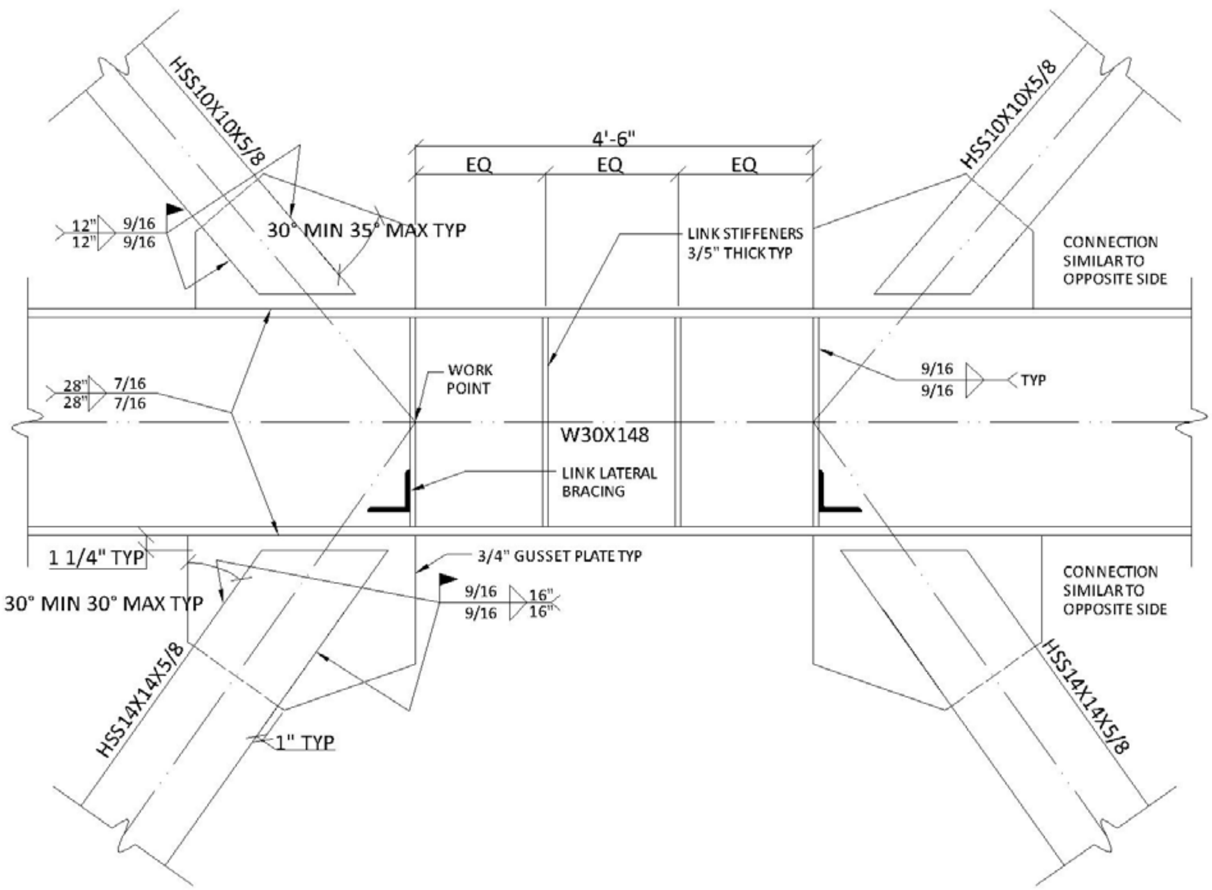


Figure 44. Brace to Link Connection

The following is a list of limit states that have been taken into consideration when designing the connection. Refer to appendix for calculations.

- For Braces:
  - Brace rupture at welds
  - Shear yield strength of brace at welds
  - Shear lag tension rupture of brace
- For Gusset:
  - Gusset plate rupture at welds

- Shear yield of gusset plate at welds
- Compression buckling of gusset plate

- For beam:
  - Beam web local yielding
  - Beam web crippling
  - Beam flange rupture at welds
  - Beam shear yield at web
- For beam web shear tab:
  - Shear tab shear yield
  - Weld rupture due to eccentric loadings

Beam-Column Connection

AISC 341-05 Section 15.7 states that the design of a beam to column connection shall meet the requirements of an OMF (ordinary moment frame) connection. After investigating the requirements of an OMF in AISC 341-05 Section 11.2 and 11.5, it can be concluded that the connection will be designed similar to the Link to Column Connection. The connection will therefore be considered a fully restrained moment connection as required by ASCE 7-05 when designing an Eccentric Brace System with Moment Connection Away from Links.

Brace-Beam to Column Connection

The brace–beam to column connection has been designed in accordance with AISC 3341-05 Section 15.6c and 15.7. The beam to gusset connection shall be done in the shop while the gusset and beam to column connection and brace to gusset connection shall be done on site. The following figure is a detail of the connection.

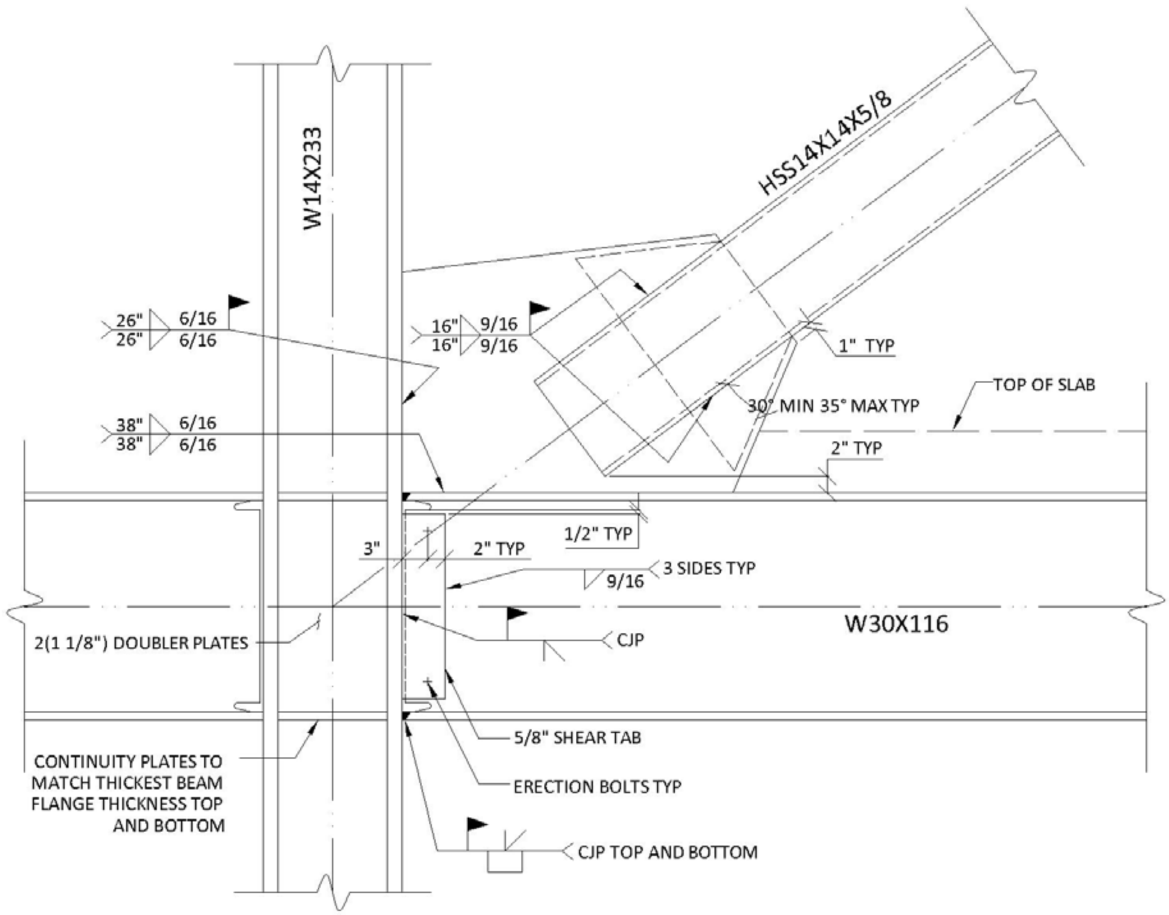


Figure 45. Brace-Beam to Column Connection

The following is a list of limit states taken into consideration:

- For brace:
  - Brace rupture at welds
  - Brace shear yield at welds
  - Brace shear lag tension rupture
- For gusset:
  - Compression buckling of gusset
  - Rupture at welds
  - Shear yield at welds

- For beam:
  - Beam web local yielding
  - Beam web crippling
  - Beam rupture at welds
  - Beam shear yield at welds
  - Beam shear yield at shear tab
- For column:
  - Continuity plate requirements per seismic provisions
  - Panel zone shear yielding
  - Column rupture at welds
  - Column shear yield at welds

Column Splice

Column splices shall be designed in accordance with AISC 341-05 Section 8.4. The following figure is a detail of the column splice. Note that erection aids are not shown.

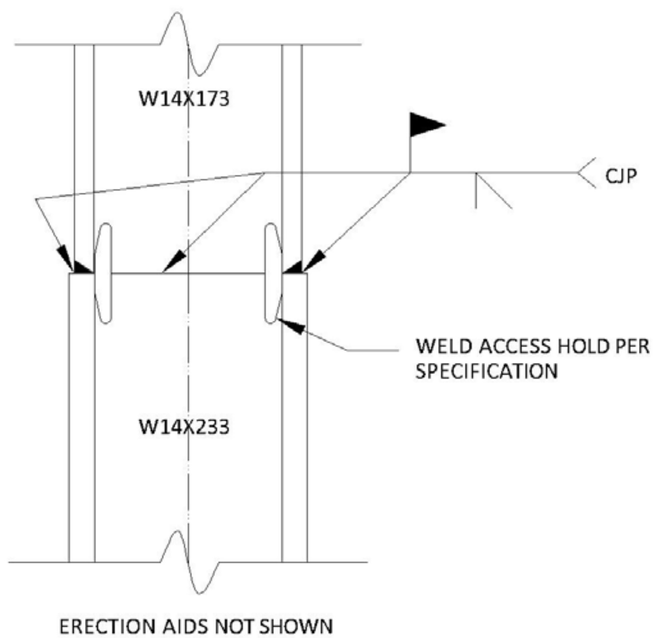


Figure 46. Column Splice Connection

The following is a list of limit states taken into consideration:

- Shear yield of column web
- Tension yield of column flange

Link Lateral Bracing Connection

The connection below is a typical lateral link bracing connection with L6X6X5/8 Angle bolted to the full depth stiffener at each end of the link

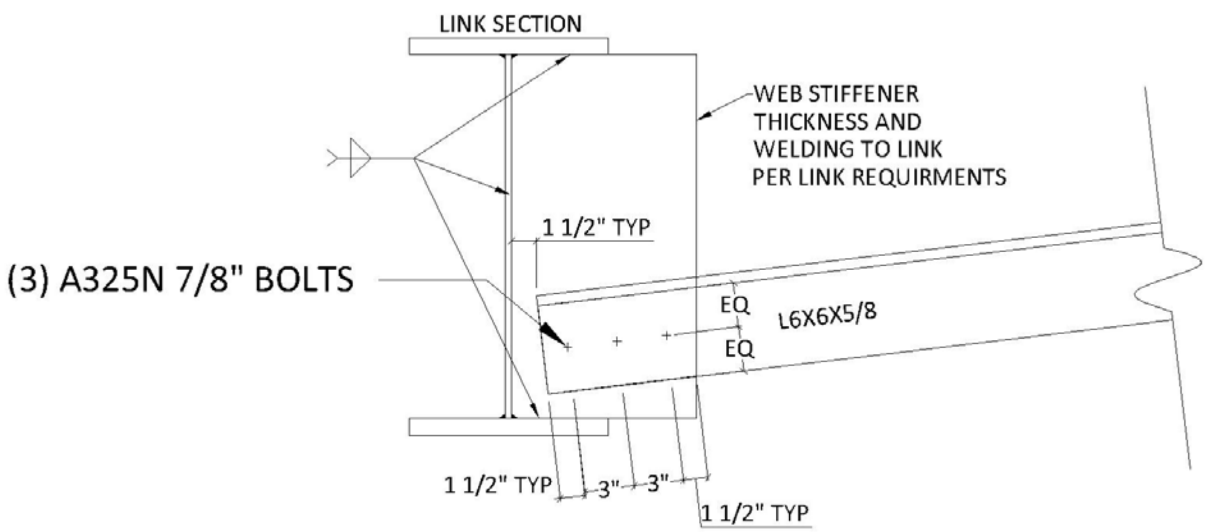


Figure 47. Link Lateral Brace Connection

The following is a list of limit states taken into consideration:

- For Bolts:
  - Bolt Shear
  - Bolt Bearing on Angle and Stiffener Plate
  - Bolt Tear-out
- For Angles Brace
  - Tension Yield
  - Tension Rupture
  - Block Shear
- Web Stiffener
  - Tension Rupture
  - Tension Yield

Other Structural Impacts

Foundation and shear wall design was beyond the scope of this report, but the reduction in number of bays in the lateral system essentially will reduce the length of the shear walls and strip foundation at the basement. This reduction will be taken into consideration when estimating the cost of the lateral system.

Diaphragm and Collector Elements

Diaphragms and collector elements were not considered in this report, but were looked out briefly. The reduction in seismic story shears will most likely reduce the section sizes of the diaphragm and collector elements. Since the lateral system was reduced the following check needs to be done to the diaphragm to insure its adequacy. The following plan is of level 3 with the EBF braces shown. The green highlights represent areas in the diaphragm where shear due to seismic loads needs to be checked.

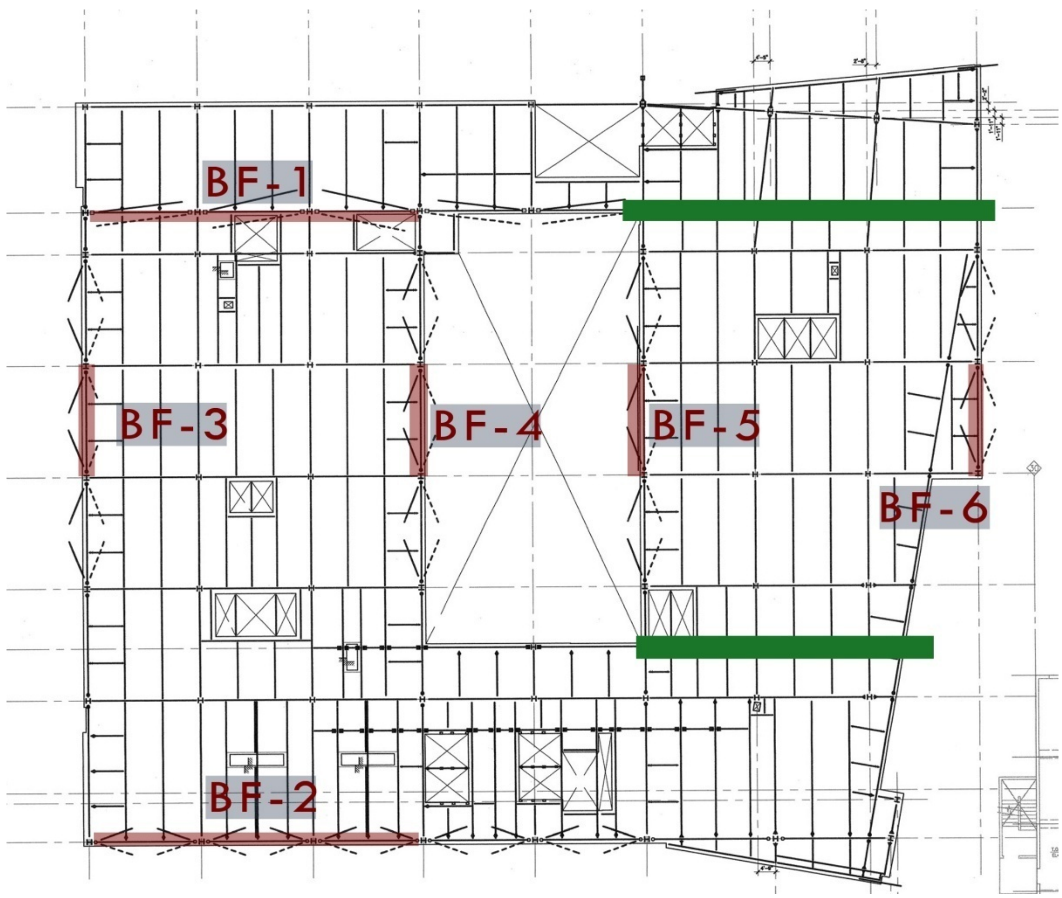


Figure 48. Plan of Level 3 showing EBF system and critical diaphragm locations

The beams that were part of the lateral system in the existing structure will need to be designed as collector elements as they are now part of the gravity system. (When referring to the figure above, the beams that are not shaded in red but are in the same grid line as the braced frame will now have to be designed as collectors.)

Final Link Rotation Checks

After the connections have been designed, the exact link lengths can be determined and the link rotations can be checked again to see if they do not exceed 0.08 radians. The following is a table with the final link rotation checks.

Link Rotation Checks							
	Link	e	Story Drift	Story Height	Bay Length	Link Rotation	Check
Location	Shape	(in)	(in)	(ft)	(ft)	(radians)	?
BF-1 BAY 1							
BF-1 Roof B1	BUILTUP4	49	0.6042	15	30	0.0247	OK
BF-1 4th B1	BUILTUP4	49	0.7626	15	30	0.0311	OK
BF-1 3rd B1	BUILTUP4	49	1.0002	15	30	0.0408	OK
BF-1 2nd B1	BUILTUP4	49	1.36872	18	30	0.0466	OK
BF-1 BAY 2							
BF-1 4th B2	W24X146	54	0.7626	15	30	0.0282	OK
BF-1 2nd B2	W24X146	54	1.36872	18	30	0.0422	OK
BF-3							
BF-3 4th Floor	W24X103	54	1.5594	15	30	0.0578	OK
BF-3 2th Floor	W30X148	54	1.49112	18	30	0.0460	OK
BF-6							
BF-6 4th Floor	W24X103	54	1.5594	15	30	0.0578	OK
BF-6 2th Floor	W30X148	54	1.49112	18	30	0.0460	OK

The drifts are below 50% of the limit therefore it would require the link sizes to reduce by 50% for them not to meet 8% rotation.

Fine Tuning the EBF

The following is a list of items that should be pursued to finalize the design on an EBF.

- Adjusting link lengths and sizes for further ductility but also meeting code drift requirements. Although further ductility would mean added cost in the design of windows mullions and other non structural components. Therefore knowing where the line of cost equilibrium must be determined.
- In an EBF it is highly unlikely that one link will undergo strain hardening while the other is intact. Therefore it is beneficial to perform a redistribution of shear forces between the links and design for the least allowable shear force in the link. This would insure a highly efficient design.
- Redesign of all Collector Elements, and check diaphragm adequacy.
- Redesign strip foundation, and Shear Walls at basement level.

EBF Design Conclusion

The following table shows the amount of steel the existing SCBF system has in compared to the new EBF system. The ratio of the tonnage of steel used is about 40% of the existing system. And since the reduction in base shear using an EBF is around 43% we can conclude that the reduction in base shear is proportional to the reduction of steel.

Lateral System	Tonnage of Steel
Existing (SCBF System)	637
New (EBF System + Gravity System)	330
New (EBF System Only)	252

The following table puts the comparison in pounds per square foot of space of the whole structure.

Structural Steel System	Structural Steel Density
Gravity System	5 psf
Existing (SCBF System)	5 psf
New (EBF System)	2.6 psf

The number of braces is equivalent to the amount of connections required. To avoid complex connection that involves wide flange braces, all wide flange braces were eliminated in the EBF system. HSS to gusset connections require less time to complete and do not require any additional splice plates like the wide flange brace connection.

Lateral System	# of Braces
Existing (SCBF)	(28) Wide Flange Braces (162) HSS Braces
New (EBF)	(66) HSS Braces

For cost and construction time comparison refer to the construction management breadth study. For a further conclusive argument refer to the Lateral System Redesign conclusion.

Construction Management Breadth Study

As we concluded before, the design of the EBF system lead to a decrease of the total amount of steel used, and a total number of moment restrained connections with brace to gusset connections. The following is a study of what the cost and construction time impact it will have on the total project.

Cost Analysis

The reduction in the amount of steel used in the EBF system is significant to the amount used in the existing system. Due to the decrease in the number of bays in lateral system, there will be other structural changes to the existing structure. The reduction in bays will reduce the length of the continuous footings while also reducing the length of the shear walls under the lateral system. This impact will have effect on the construction cost. The table below represents a summary of an RS Means cost estimate of the existing lateral system compared to the new EBF lateral system.

Existing (SCBF System) Cost Estimate Summary	
Component	**Construction Cost**
Structural Steel	$2,312,000
Continuous Footings	$276,000
Shear Walls	$232,500
Total	**$2,820,500**

New (EBF System) Cost Estimate Summary	
Component	**Construction Cost**
Structural Steel	$1,195,000
Gravity Footings	$62,400
Total	**$1,257,000**

The total construction cost saved of using an EBF system is $1,563,500, and the total cost of the project is $130 million. Therefore using an EBF system will result in approximately a 1.3% reduction in total construction cost.

Scheduling Analysis

The following is an RS means estimate of the construction time the existing and EBF system will take to construct. There is an insufficient amount of detail regarding estimating the construction time of member with fully restrained moment connections compared to normal shear connections. Therefore a comparison of the connections in the existing structure compared to the new EBF system was done. It has been found since both systems use complete joint penetration flanges with shear tab connections. It can therefore be concluded that the time to erect both system is relatively similar. The following is a table showing how many days it would take **1 crew** to build both systems. Actual days may differ to the number of crews working on the building.

Existing (SCBF System)	Schedule
Component	**Days**
Structural Steel	45
Continuous Footings	29
Shear Walls	37
Total	**111**

New (EBF System) Schedule	
Component	**Days**
Structural Steel	23
Gravity Footings	5
Total	**28**

Comparing the man hours the steel erectors will take is as follows.

Structural Steel System	Man Hours
Gravity	3550
SCBF (Existing)	3550
EBF (New)	1850

The redesigned structure will take the steel erectors 76% of the total time it took to erect the original structure.

Lateral System Redesign Conclusion

A lateral system redesign has certainly proven to be effective. The reduction in base shear reduced the amount of tonnage in the lateral system which essentially cut construction cost. The reduction in amount of lateral resisting frames reduced the amount of braces which essentially reduced the amount of complex connections. The design was able to eliminate the need of wide flange braces and use only HSS braces that use much simpler brace to gusset connections. This saves erection time and cost.

EBF was not only advantageous from a cost stand point but also performance. During a major earthquake inelastic shear deformations provide ductility but also stability unlike SCBFs which mainly provides stability or SMFs which only provides ductility. Damage is also isolated in on area (near the link) therefore limits the cost of repair after a major earthquake. The original design used a SCBF that utilizes out of plane buckling; one might argue that is it safer for the brace to buckle out of plane than for a link to rotate and deform the floor.

Had the design of the EBF been brought earlier in the project, the architectural layout may be entire configured so that it is not affected by the lateral system as much as it did. Meanwhile the EBF system designed in this report eliminated most braces blocking views into the central courtyard and outside. Therefore this insures that the EBF system is aesthetically friendly compared to the SCBF.

The lateral system redesign is still incomplete and requires further design with other affected structural components like foundation, diaphragms, and collector elements. Non structural component would also need to be looked at to insure if they are still adequate with the new drifts. Once everything is designed a detailed cost analysis could be performed to determine the exact cost savings. Currently the cost saving are around 1% of the total project cost. This percentage would go up with the new added cost savings of the other redesigned structural elements. Think how happy this would make the owner and you the structural designer would probably be the one working on his next project!

Lighting Breadth Study

Problem Statement

The central courtyard located on the second floor is an interesting space in the patient care center. This central courtyard is surrounded by various patient rooms and family waiting rooms that have direct views into the courtyard. The courtyard has small planters, benches and a water feature which provide a comfortable environment for the patients. Hospital patients generally tend to dislike long stays in the hospitals so this space will aim to provide a relaxing environment for patients to get fresh air and spend time outside while still being in the hospital. This design will provide an opportunity for people to explore or relax while still maintain a comfortable lighting environment after sundown.

Appropriate light fixtures will be selected and placed at areas to accentuate architectural features and landscaping in the courtyard space. The lighting fixtures will also provide necessary footcandles in the outdoor space for the patients while not distracting adjacent patient rooms that are facing the court yard space. The space will be modeled using AGI-32, a lighting design software for calculations and visualizations.

Central Courtyard Lighting Design

Goals

- Design a comfortable lighting environment that welcomes patients into the courtyard after sunset.
- Illuminate planters, trees and the water fountain so that the patients feel comfortable and spend time there after sunset as if they were spending time in their back yard garden.
- Adhere to the California Energy Commission, 2005 Building Energy Efficiency Standards.

Design Criteria

The following are design criteria specified in the California Energy Commission, 2005 Building Energy Efficiency Standards.

- Section 6.2.2 mandates that any lamp over 100 watts must have an efficacy of 60 lumens per watt or it shall be controlled by motion sensors.
- Section 6.2.4 mandates that there shall be automatic shutoff controls so that the light fixture automatically shuts off when certain daylight levels are present.
- Section 6.2.5 requires there to be multi-level switches for façade lighting so that the owners may control up to 50% of the lighting power.
- Section 6.3 classifies the central courtyard to be designed for urban areas, due to its location in Orange, California, which is classified under Lighting Zone 3.
- Section 6.4 specifies the outdoor lighting power allowance to be 0.17 watts per square foot for spaces in Lighting Zone 3. Section 6.5.3 allows for an adjustment for security for spaces within 100 feet of the building's entrance. Since the space is inside the building and within

100 feet of the main building entrance then a multiplier of 1.25 shall be used, which brings the power allowance to 0.213 watts per SF.

- Section 6.6 addresses specific lighting applications such as the lighting of building facades. Specific lighting applications shall be taken into account for the total power density of the space and shall be addressed separately. For façade lighting fixtures Table 147-B specifies an allowance for 0.35 watts per square feet for facades in lighting zone 3.
- The power density of the entire space is less than the existing so panelboards and wire sizes did not change.

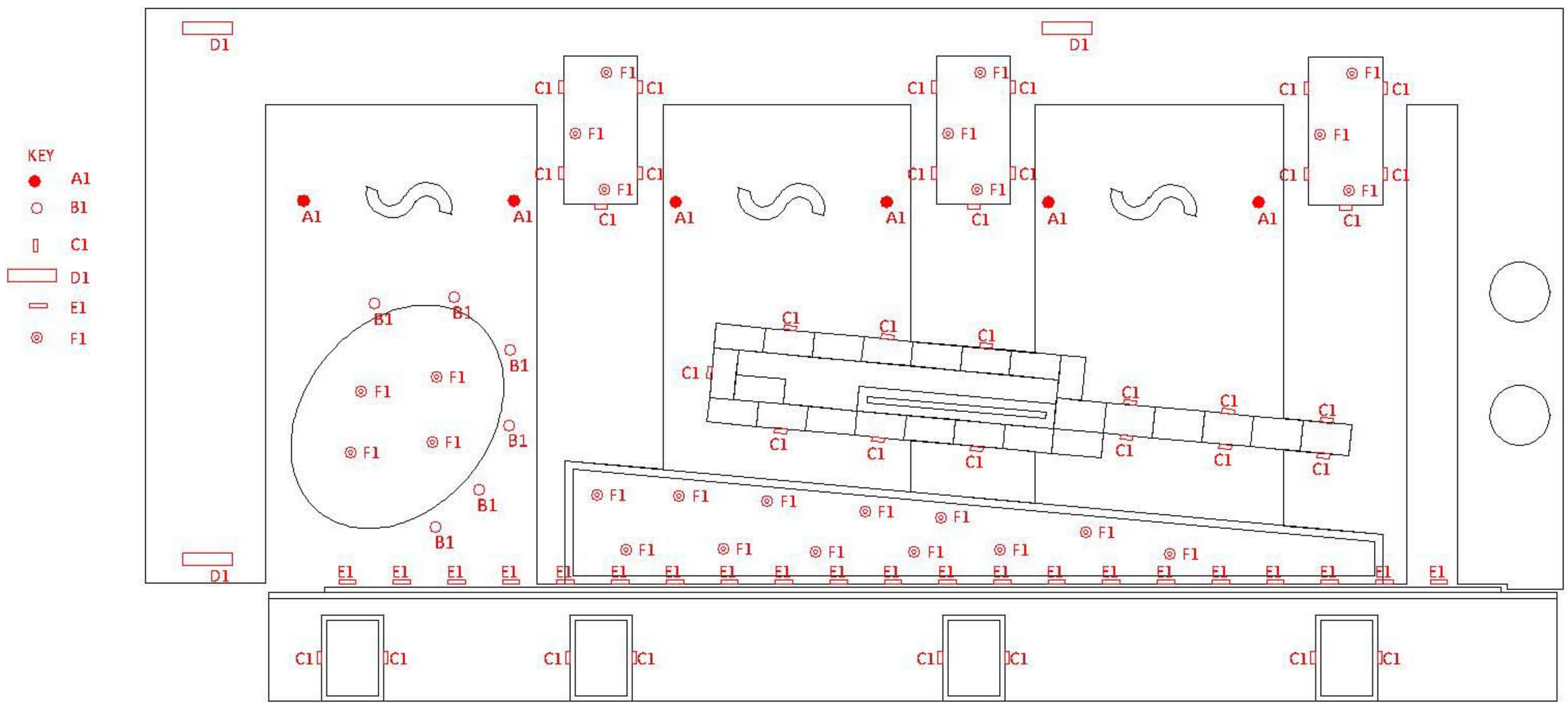
Lighting Hardware

Light Fixture	Lighting Comment
Compact Fluorescent Bollard	Comfortable lighting, general illumination next to seating
Compact Fluorescent In grade Circular up light	Visual interest, highlight tree planter
Compact Fluorescent Step Light	Compliments architecture, directional flow
LED Wall Washers	Creates points of interest, highlights wall materials
LED Linear Fixture	Highlights material of wood wall
LED Spot Light	Highlights plants and tree leaves

Luminaire Schedule

LUMINAIRE SCHEDULE																
Fixture Tag	Fixture Image	Description	Volt.	Watt.	Manufac.	Catalog #	Lamps				Ballast Type	Mounting Type	Maximum Fixture Depth/Height	General Location	Remarks	
							Quant.	Type	CCT	CRI						
A1		EXTRUDED ALUMINUM COMPACT FLUORESCENT BOLLARD, SINGLE DIE CAST ALUMINUM TOP HOUSING AND BASE, LOUVER/GUARD SECURED HOUSING.	277	14	BEGA	8429P	1	PL-C-13W-830-4P-ALTO	3000	82	FDB-T418-277-1	RECESSED	NOMINAL 40 INCH HEIGHT X 6 INCH DIAMETER	COURTYARD	MOUNTED ON FLOOR	
B1		COMPACT FLUORESCENT INGRADE CIRCULAR UPLIGHT, DIE CAST ALUMINUM OUTER HOUSING, STAINLESS STEEL FACEPLATE.	277	10	BEGA	8703P	1	PL-C-9W-830-4P-ALTO	3000	82	FDB-T418-277-1	INGRADE	NOMINAL 6-1/2 INCH APERTURE X 7-1/2 INCH DEPTH	COURTYARD	LOCATED INGRADE ALONG CIRCULAR LARGE FLOWERBED	
C1		COMPACT FLUORESCENT STEP LIGHT WITH DIE CAST ALUMINUM FACEPLATE, TRANSLUCENT WHITE CERAMIC COATING FLUSH TO FACEPLATE SURFACE	277	10	BEGA	2289P	1	PL-S-9W-830-4P-ALTO	3000	82	FDB-T418-277-1	WALL MOUNTED	NOMINAL 10 INCH WIDTH X 3 INCH HEIGHT X 4-1/2 INCH DEPTH	COURTYARD	LOCATED AROUND FLOWERBEDS AND WATER FOUNTAIN	
D1		LED LINEAR FIXTURE, EXTRUDED ALUMINUM HOUSING WITH CUSTOM WEATHERPROOF INCASING, 8 CIRCUIT BOARDS WITH 18 LEDS EACH.	277	280	COLOR KINETICS	COLORBLAZE48	144	LEDS	3000	82	-	FLOOR MOUNTED	NOMINAL 48 INCH LENGTH X 8 INCH HEIGHT X 8 INCH WIDTH	COURTYARD	LOCATED ALONG WALL	
E1		LED LINEAR FIXTURE, DIE CAST ALUMINUM, POWDER COATED HOUSING AND CUSTOM WEATHERPROOF HOUSING.	277	15	COLOR KINETICS	IW COVE POWERCORE	10	LEDS	3000	82	IW DATA ENABLER	FLOOR MOUNTED	NOMINAL 12 INCH LENGTH X 2 INCH HEIGHT X 2 INCH DEPTH	COURTYARD	LOCATED ALONG WOOD WALL	
F1		LED SPOTLIGHT, PAINTED SILVER HOUSING, CUSTOM WEATHER AND WATER PROOF HOUSING.	277	4	COLOR KINETICS	CAR702-WT12F	4	LEDS	3000	83	-	FLOOR MOUNTED	NOMINAL 2 INCH WIDTH X 2 INCH DIAMETER	COURTYARD	LOCATED BELOW PLANTS AND IN WATER FOUNTAIN	

Lighting Plan



COURTYARD LIGHTING PLAN

Power Densities

The existing lighting is around 3040 watts while after the redesign only 1759 watts is needed. Therefore no electrical resizing of panel board and wiring will be required. The power density for the new lighting meets code requirements. The power density of the existing lighting does not meet the 2005 code this may be due to different assumptions taken with a different code by the original designer.

Redesigned Courtyard's Power Density

Light Fixture	Wattage	#	Total Wattage	Sq. Footage	Watt per SF
Lighting Zone 3: Courtyard Lighting					
A1	14	6	84	6347	0.013
B1	10	6	60	6347	0.009
C1	10	36	360	6347	0.057
E1	15	21	315	6347	0.050
F1	4	25	100	6347	0.016
		SUM	**919**		**0.145**
Specific Lighting Application: Façade Lighting					
D1	280	3	840	2520	0.33
		SUM	**840**		**0.333**

Existing Power Density

Light Fixture	Wattage	#	Total Wattage	Sq. Footage	Watt per SF
Courtyard Lighting					
EHH	90	6	540	6347	0.085
EHG	40	3	120	6347	0.019
EHJ	70	17	1190	6347	0.187
ELA	70	17	1190	6347	0.187
		SUM	**3040**		**0.479**

AGI32 Modeling assumptions

The following is a list of modeling assumptions that were considered in AGI.

- A light loss factor of 70% was taken into account in the model. The courtyard is closed by four vertical walls and is open to the sky. Therefore Leafs, rain, dust and dirt may have a large contribution to light loss for bollards, in grade fixtures and step lights.
- AGI default material reflectances were taken into account in the model.
- Lighting for the worst case scenario was taken into account, and no light from any interior space was taken into account.

AGI32 Modeling Renderings

The following are renderings of the model produced in AGI with all the new light fixtures.

Figure 49. AGI Rendering of the Central Courtyard facing south.

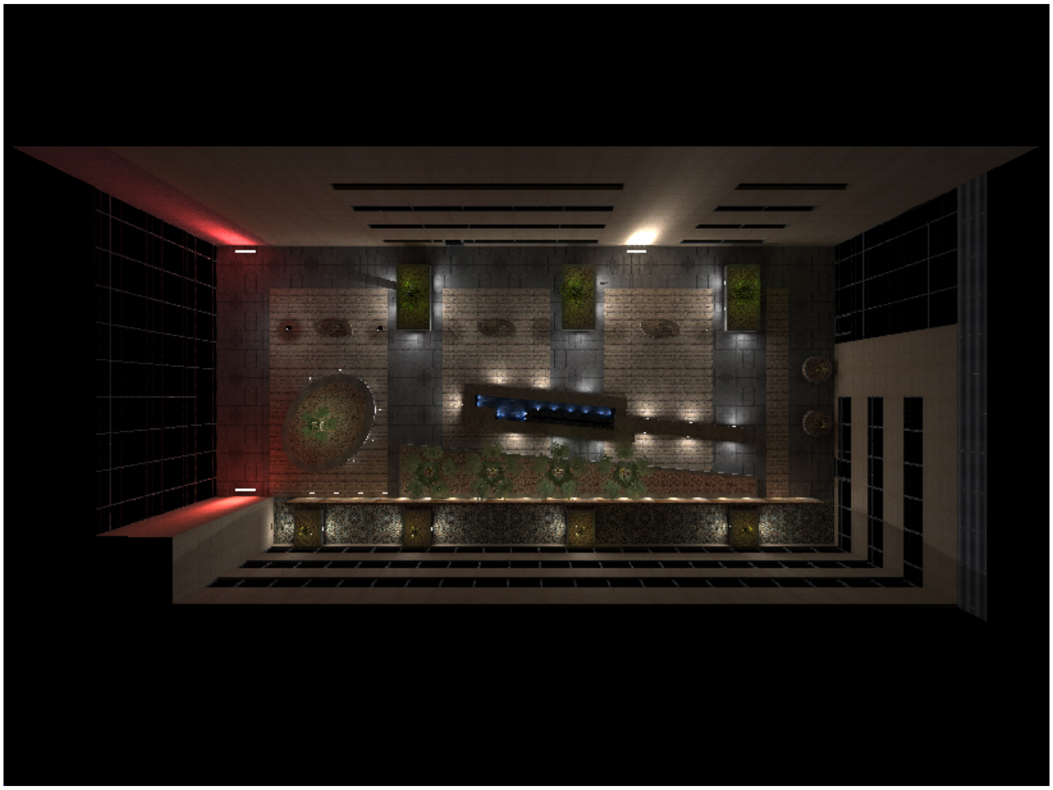


Figure 50. AGI Rendering of the Central Courtyard in plan.

Figure 51. AGI Rendering of the Central Courtyard from viewer's perspective.

Figure 52. AGI Rendering of the Central Courtyard facing north from a viewer's perspective.

Figure 53. AGI Rendering of the Central Courtyard from a patient room.

Conclusion

By creating visual points of interest in the space, the courtyard lighting design successfully provides a place of refuge for patients and even hospital workers looking to escape the high stress, hectic and uncomfortable environment of the hospital while still maintaining stringent California lighting and energy codes. Non-uniform peripheral lighting and highlighted architectural elements in the courtyard provide a calm and interesting space that juxtapose the high intensity, direct and uniform hospital lighting. Visual points of interest along the periphery such as the color changing LED wallwasher and LED spotlights create sporadic points of interest that draw the attention of the viewer around the space. The step lights and bollards help to direct the viewer around the space.