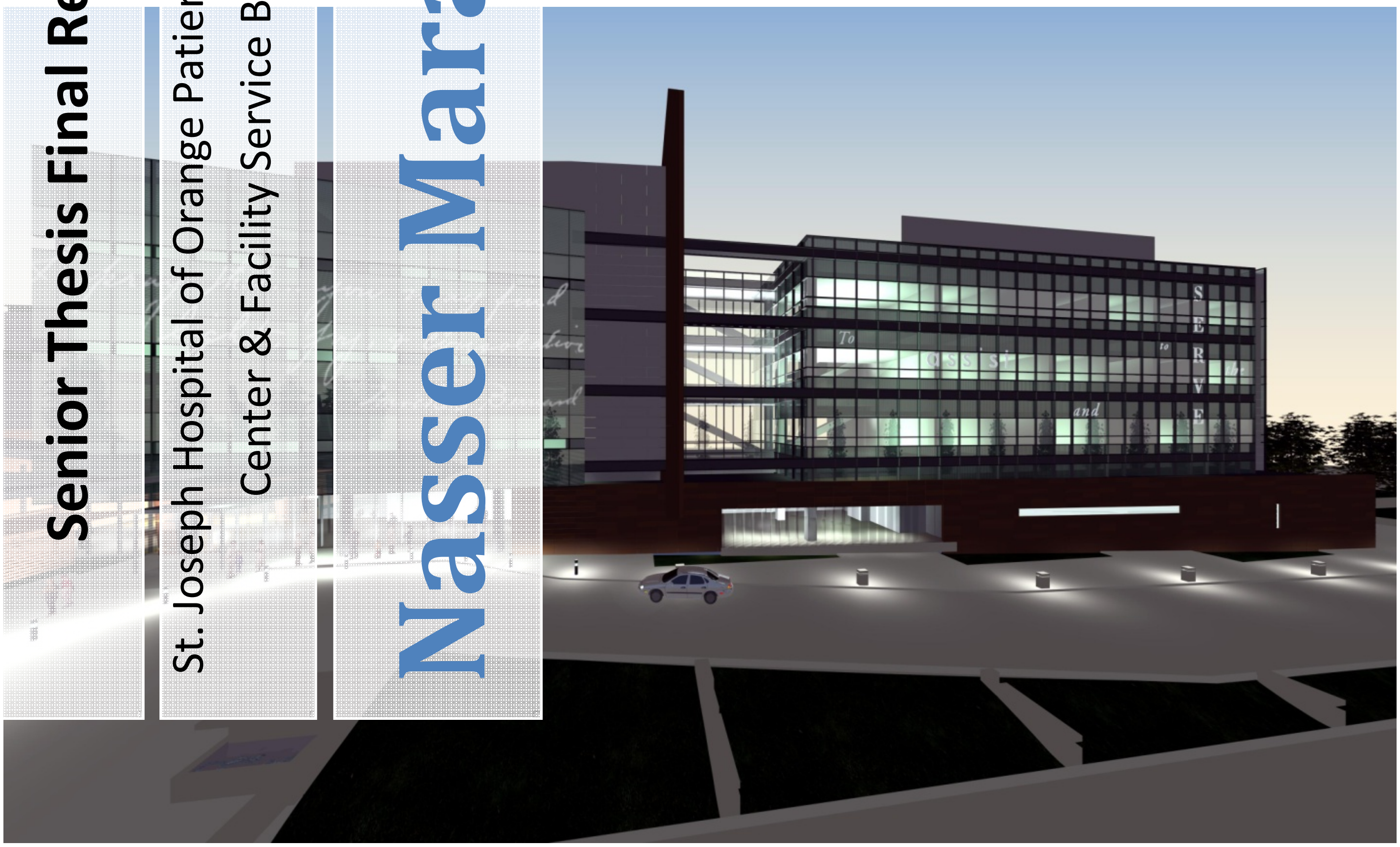
Senior Thesis Final Report

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 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 in a construction cost stand point.

The redesign was able to reduce the seismic base shear coefficient hence reduce the amount of steel required in the main lateral force resisting system. 2 bays were eliminated from each bay set, while the overall system used smaller steel shape 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

A lighting study and redesign of the central courtyard located on level 2 was performed. 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.

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Dr. Richard Mistrik

Colleagues

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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.

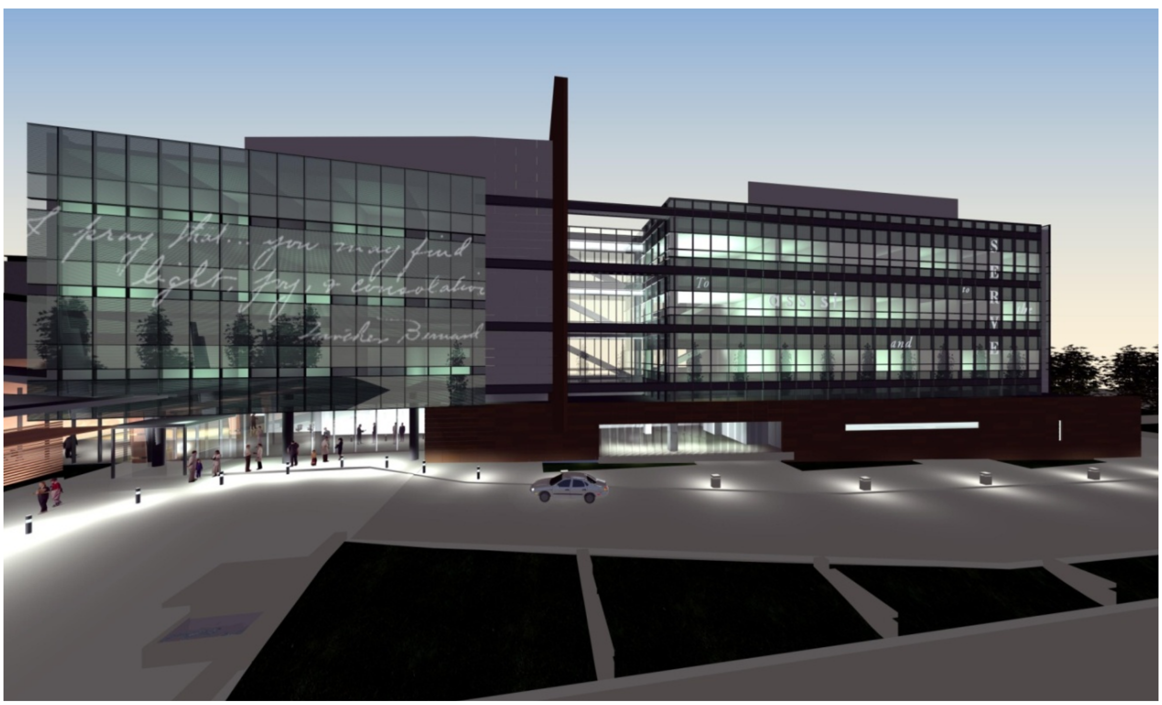


Figure 1. Computer rendering of the Patient Care Center's North elevation. Courtesy of KPFF-LA

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.

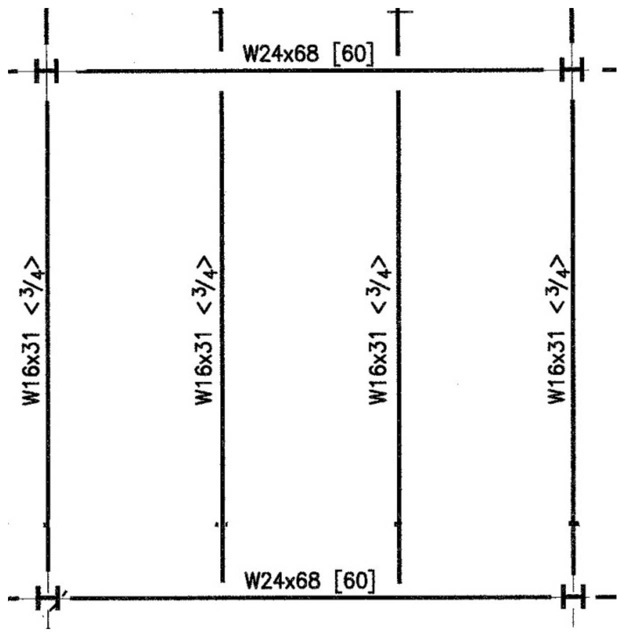


Figure 2. Typical 30'-0"x30'-0" bay located on Levels 2, 3 and 4

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.

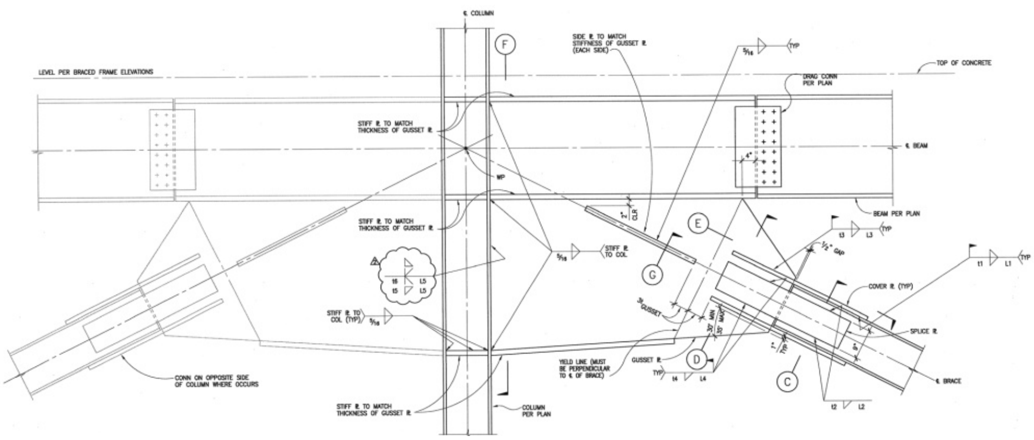


Figure 3. Diagonal Brace Connection Detail

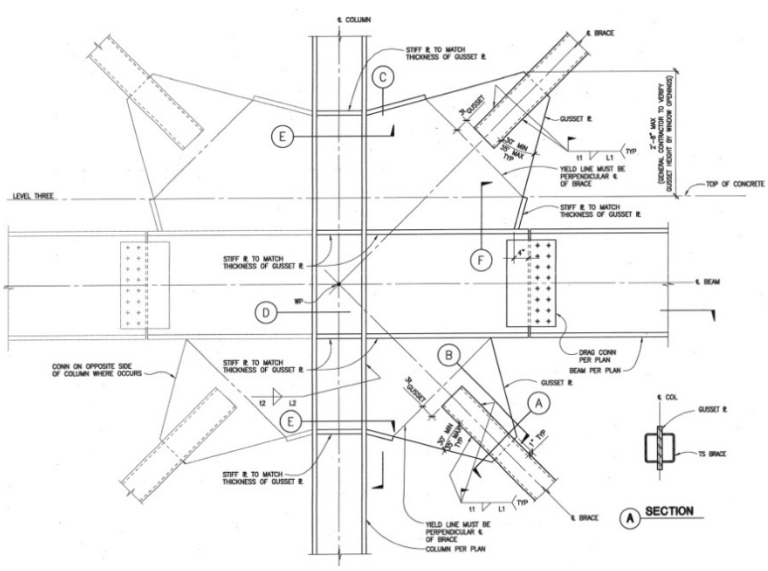


Figure 4. X Brace Connection Detail

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.

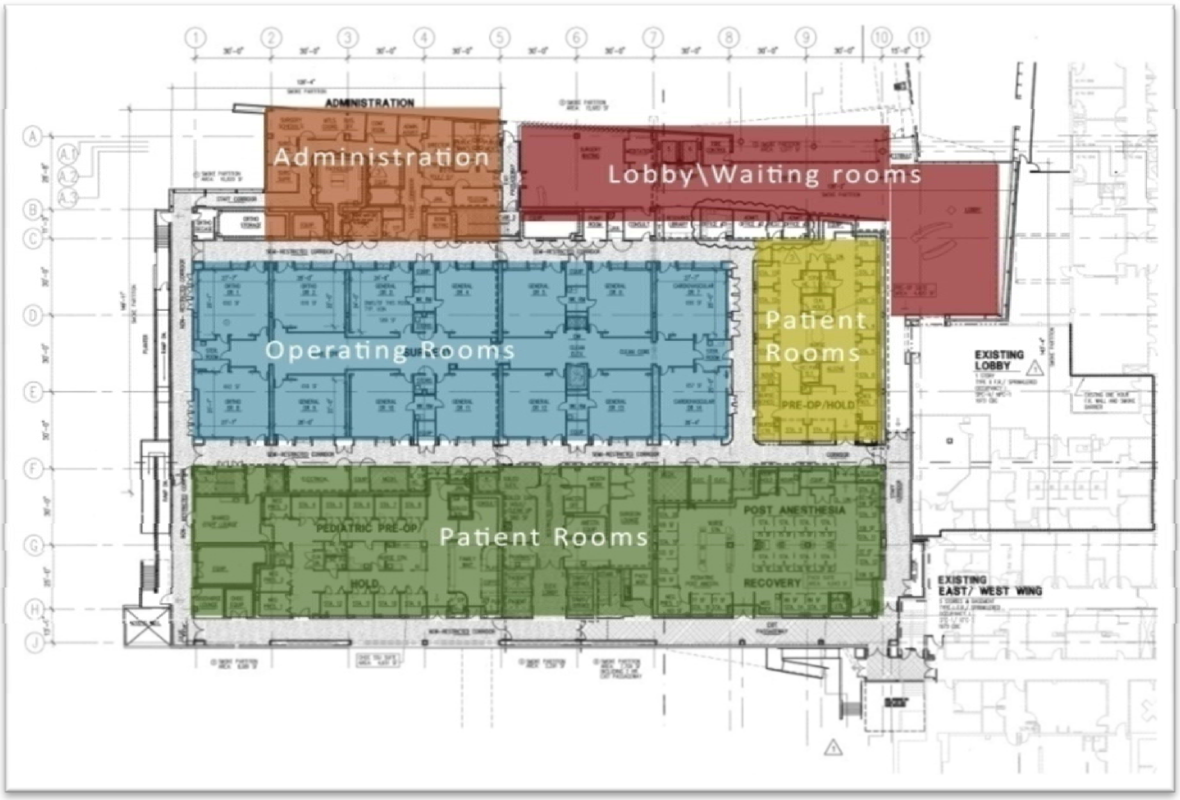


Figure 5. First floor plan showing occupant use.

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.

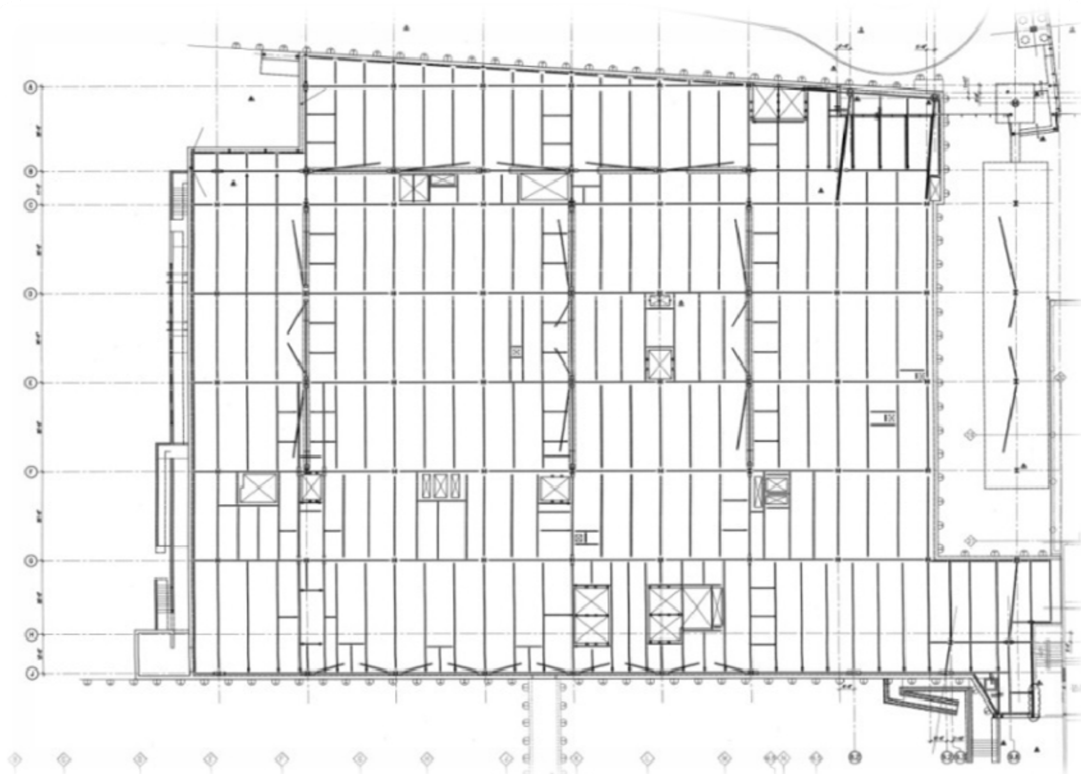


Figure 6. First floor framing plan.

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.

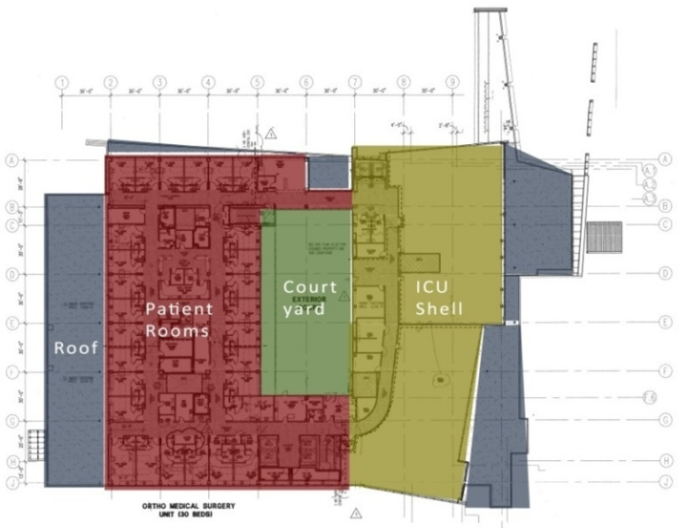


Figure 7. 2nd floor plan showing occupant use.

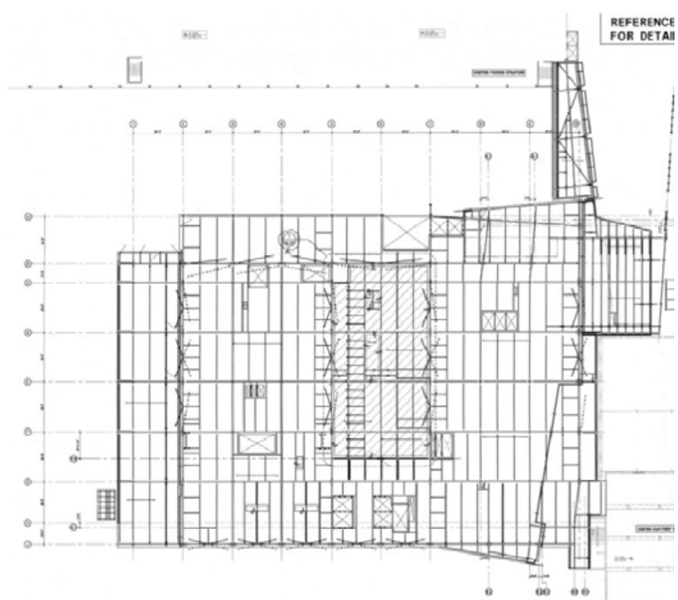


Figure 8. 2nd floor plan showing framing plan.

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.

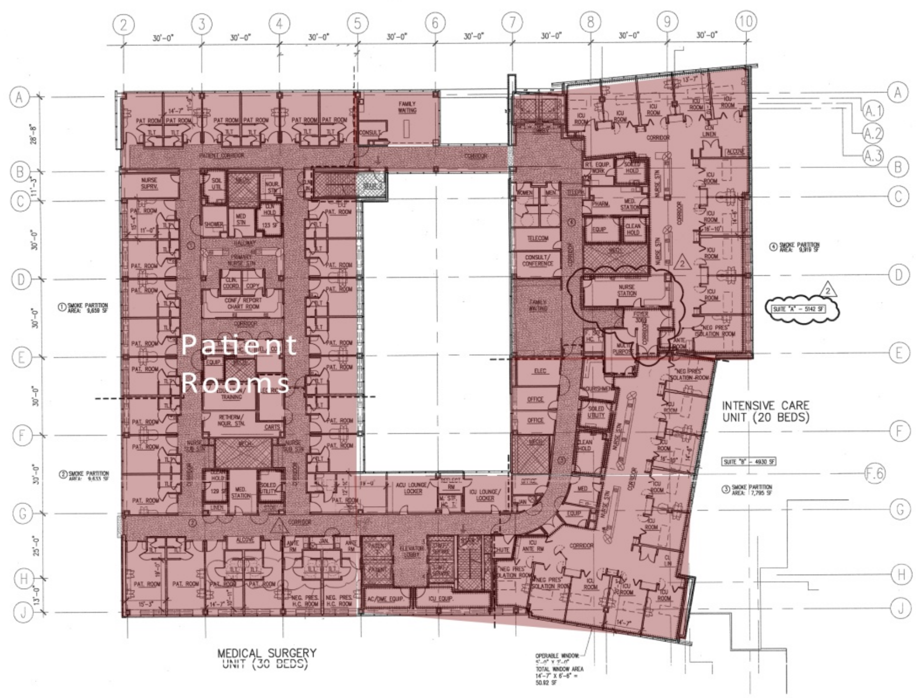


Figure 9. 3rd Floor Plan showing occupant use.

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.

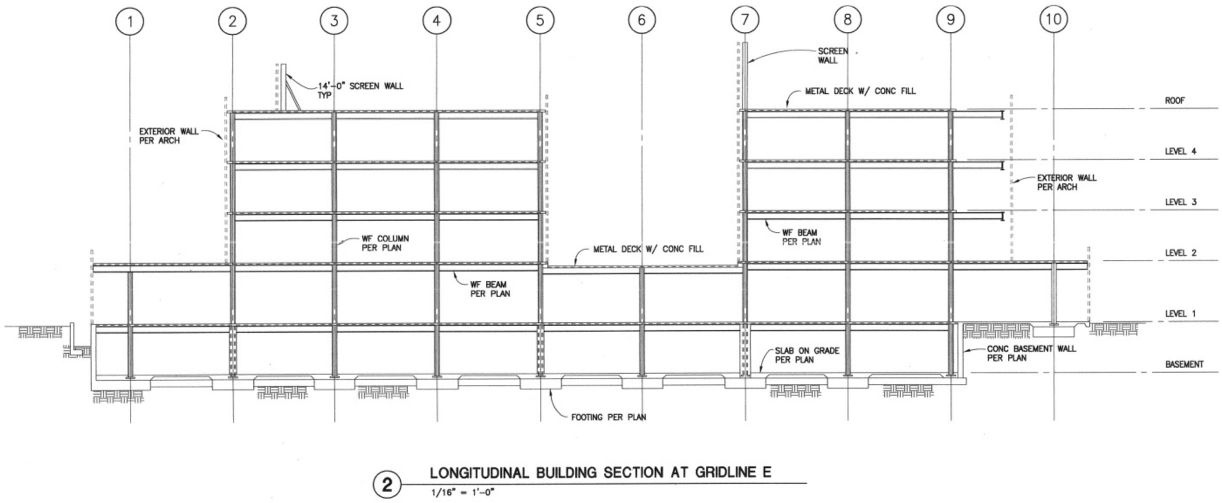


Figure 10. Longitudinal Section at gridline E.

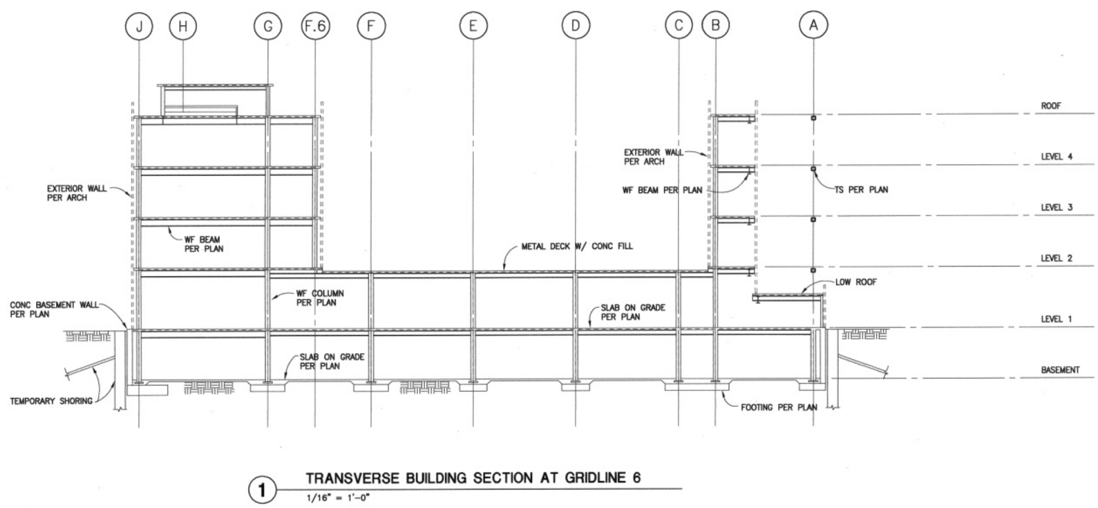


Figure 11. Transverse Building Section at gridline 6.

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.

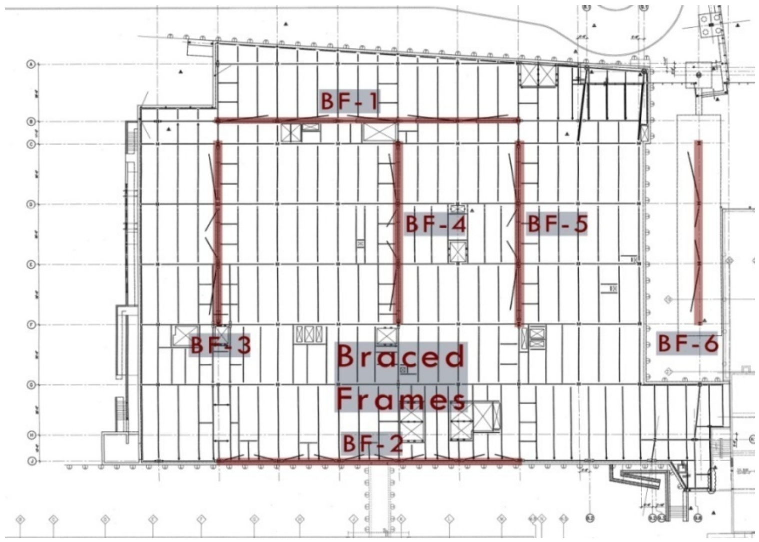


Figure 12. 1st Floor plan labeling all braced frames

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.

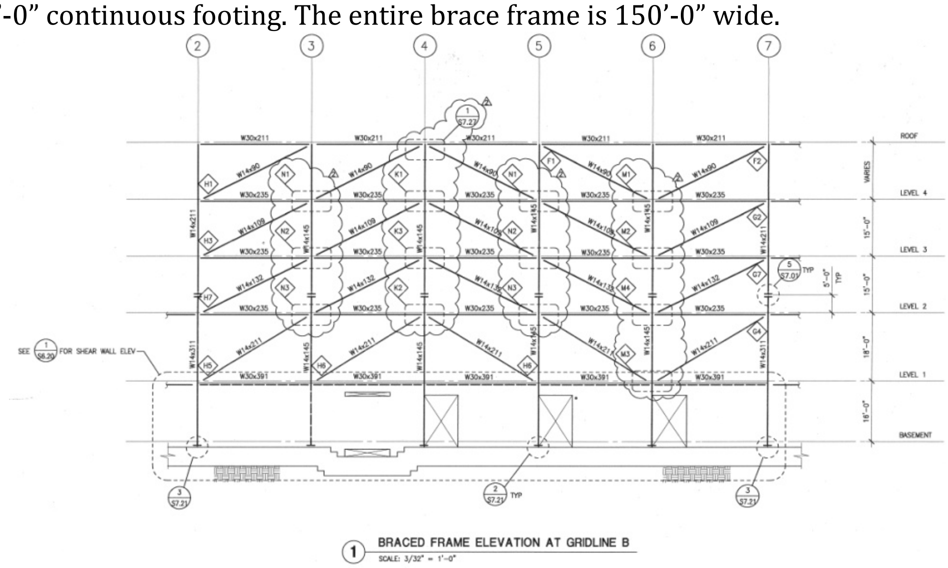


Figure 13. BF-1 Braced Frame Elevation

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.

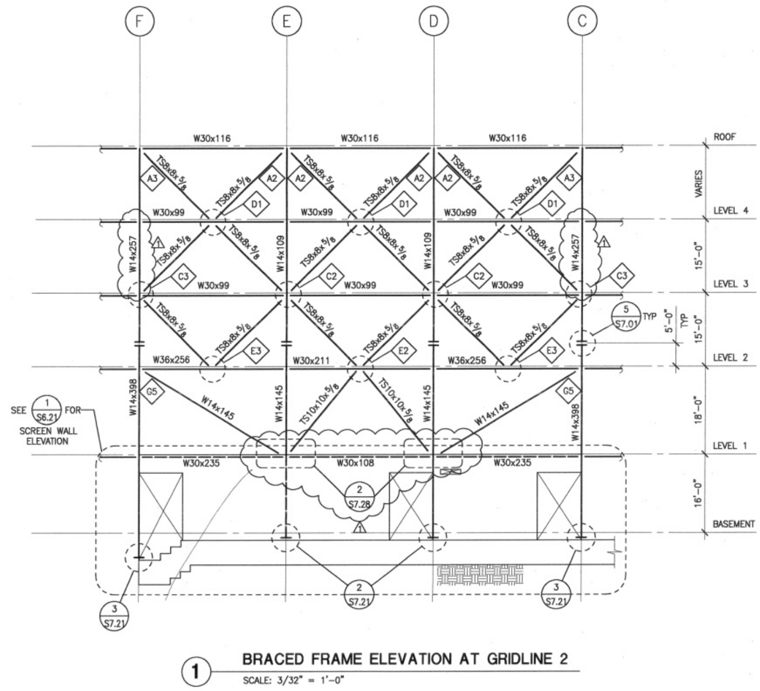


Figure 14. BF-3 Braced Frame Elevation

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.

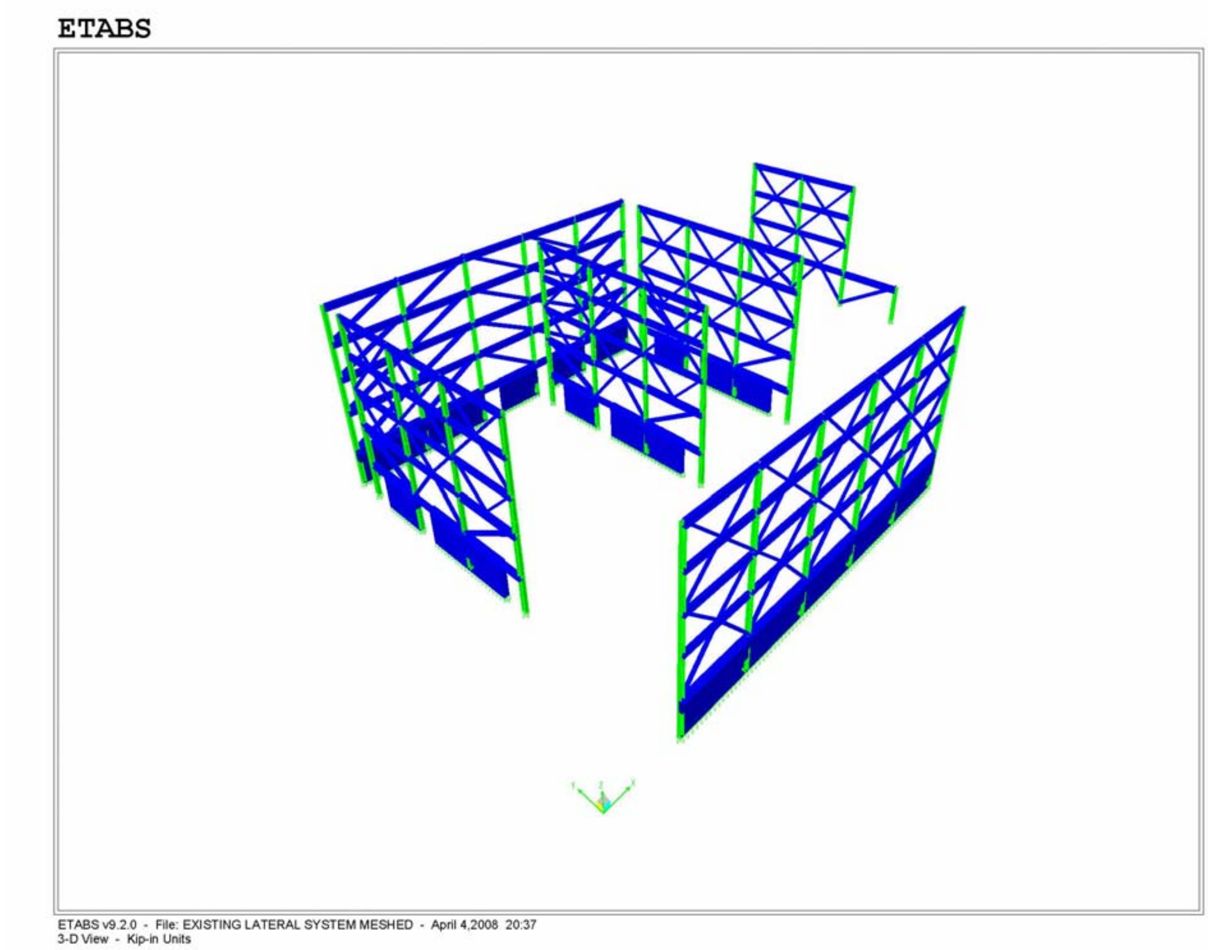


Figure 15. 3-D ETABS Model

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.

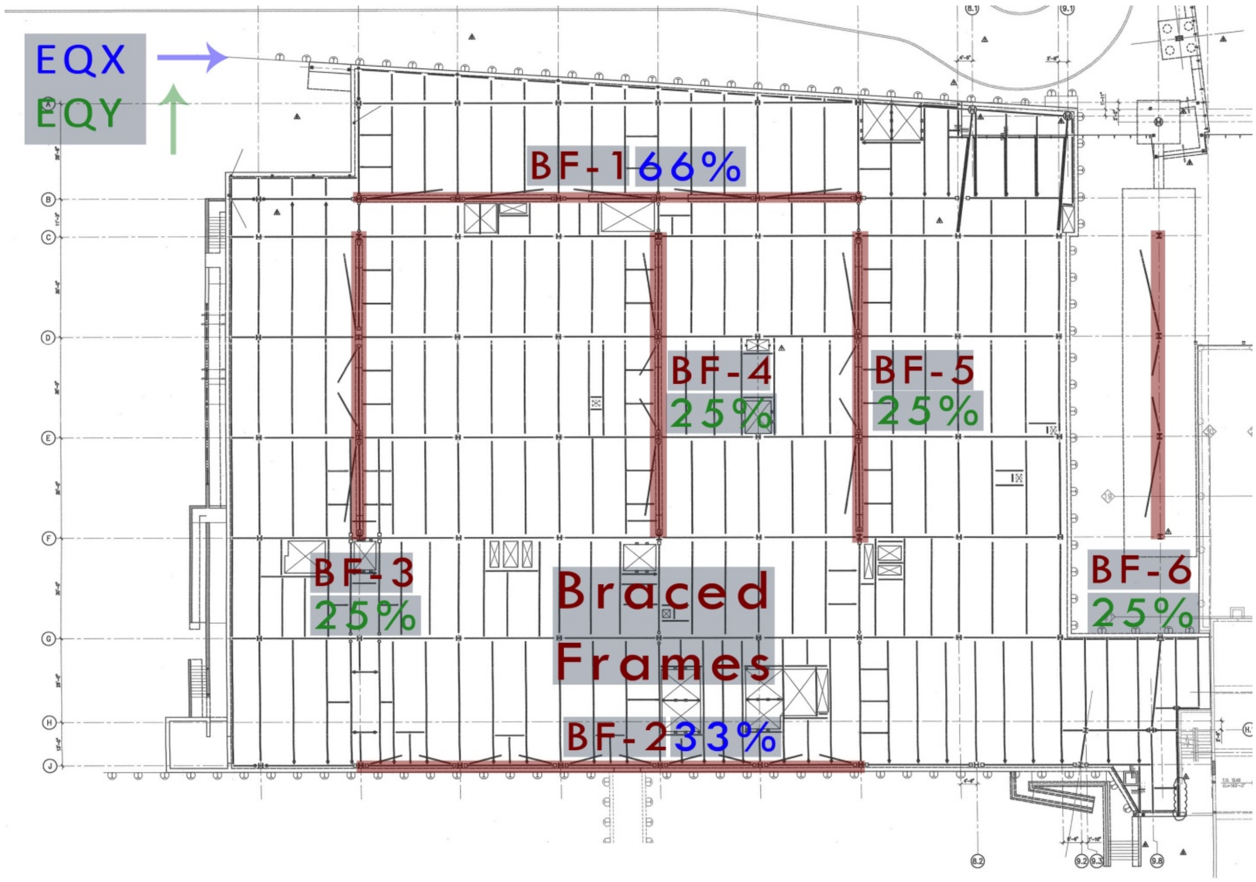


Figure 16. Distribution of Lateral Forces among frames plan.

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.

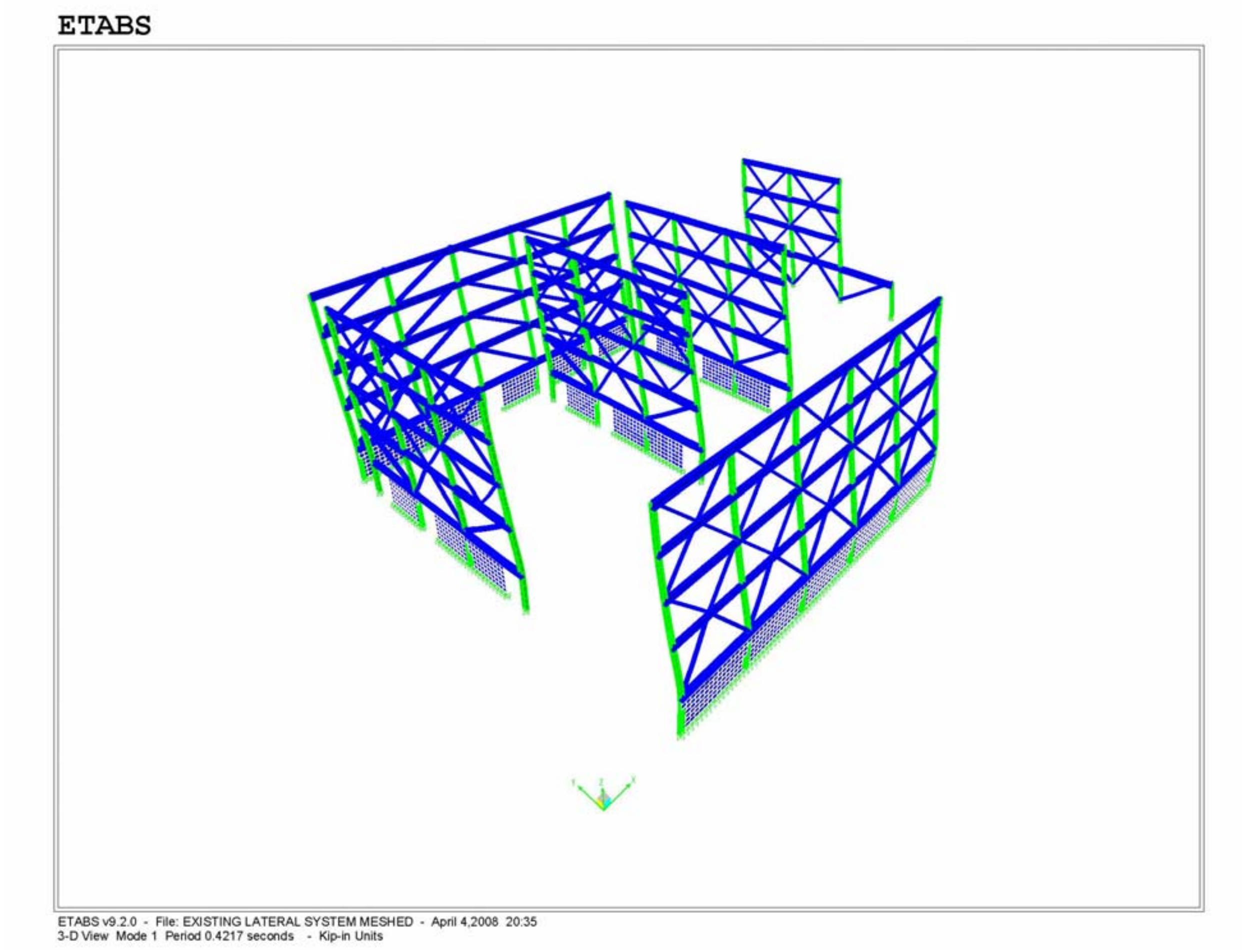


Figure 17. 3-D Modal Period

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.

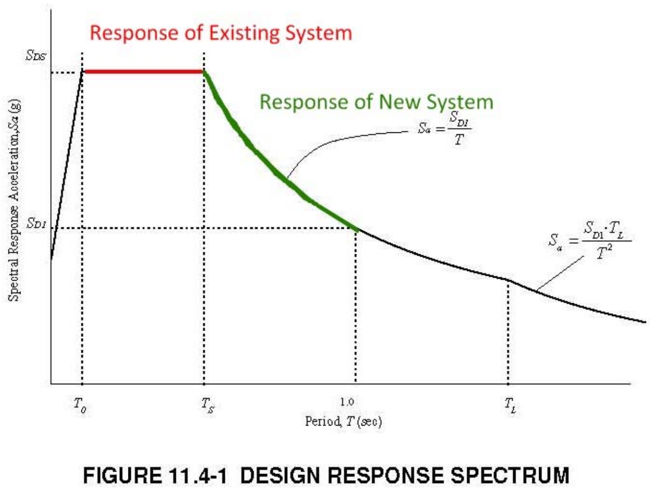


Figure 18. ASCE 7-05 Design Response Spectrum Chart

- 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.

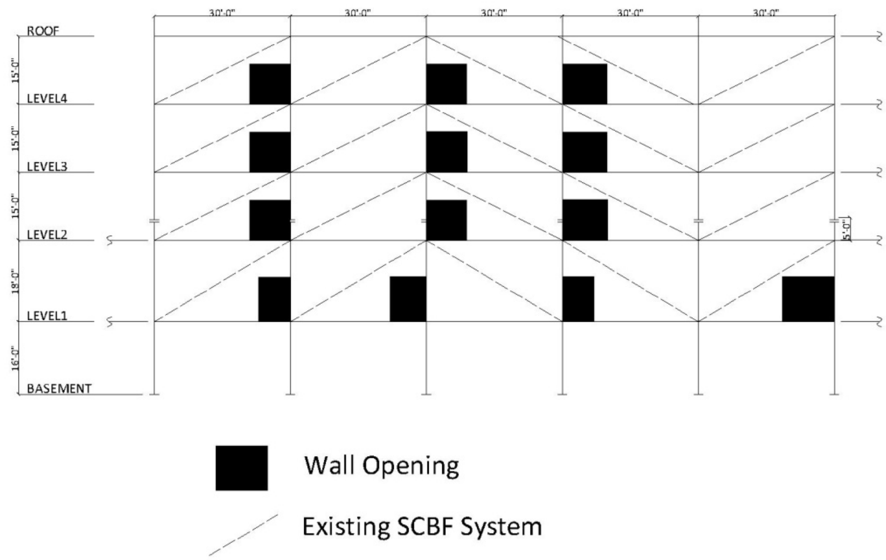


Figure 19. Existing Lateral Frame in the X Direction

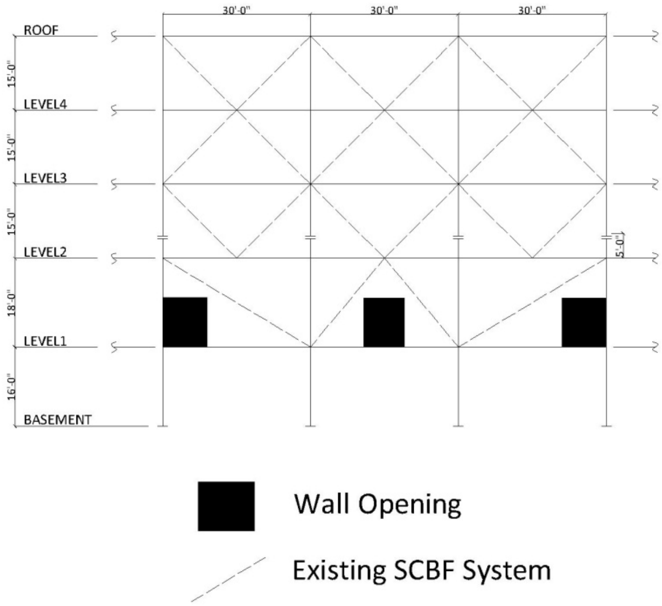


Figure 20. Existing Lateral Frame in the Y Direction

- 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.

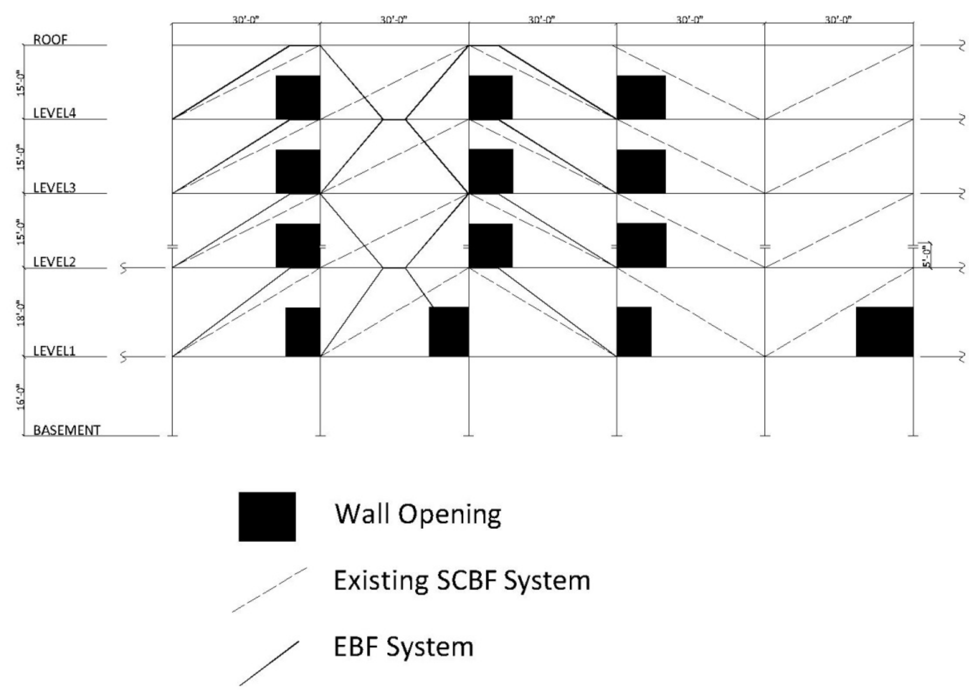


Figure 21. Proposed Lateral Frame in the X Direction

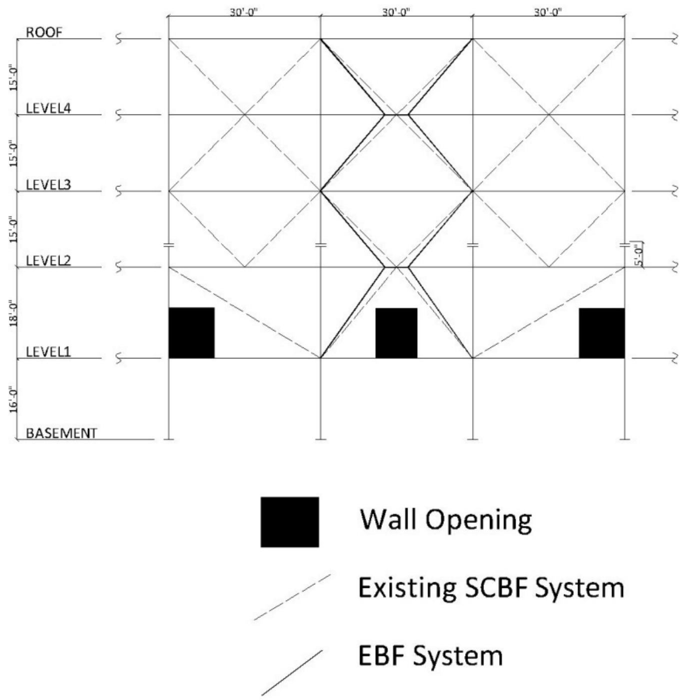


Figure 22. Proposed Lateral Frame in the Y Direction

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.

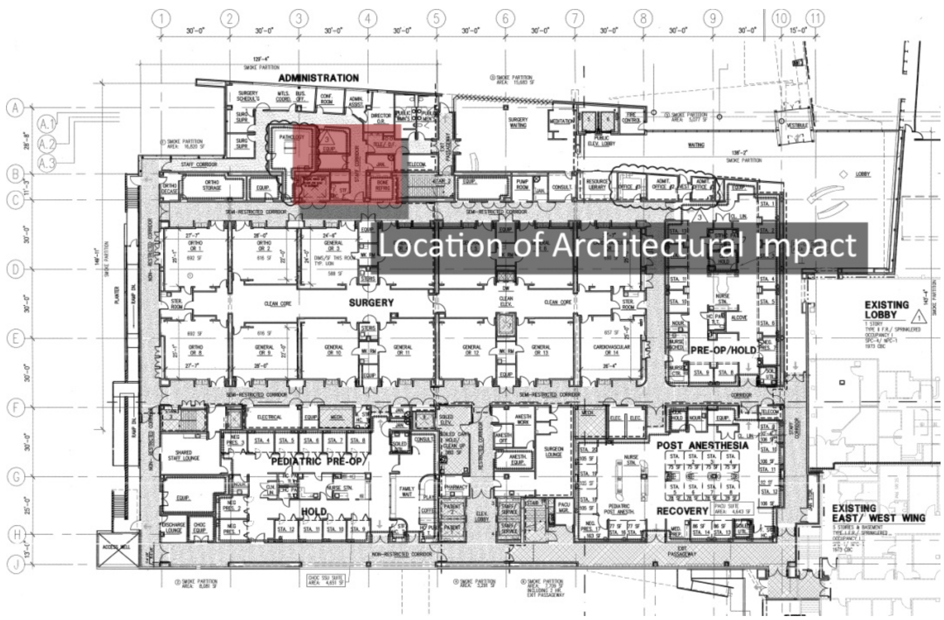


Figure 23. Location of Architectural Impact

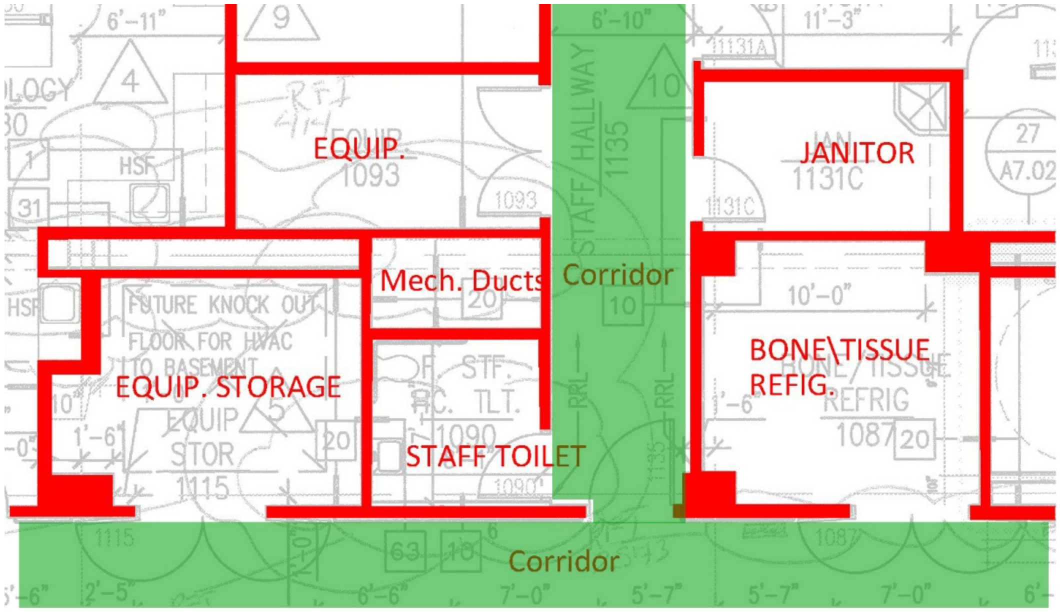


Figure 24. Original Architectural Layout

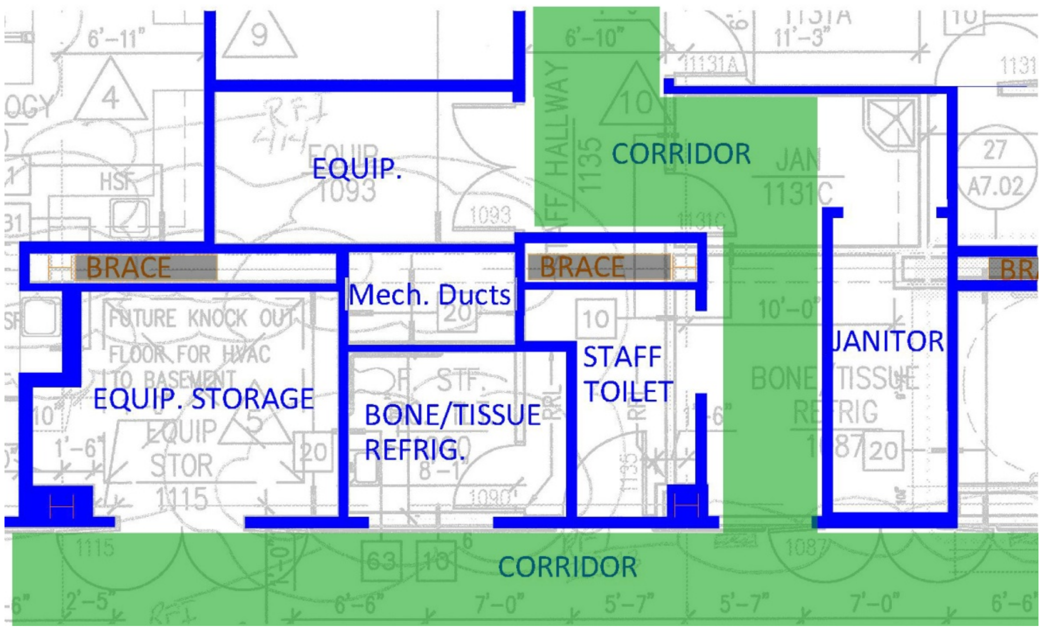


Figure 25. Architectural Layout Change

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.

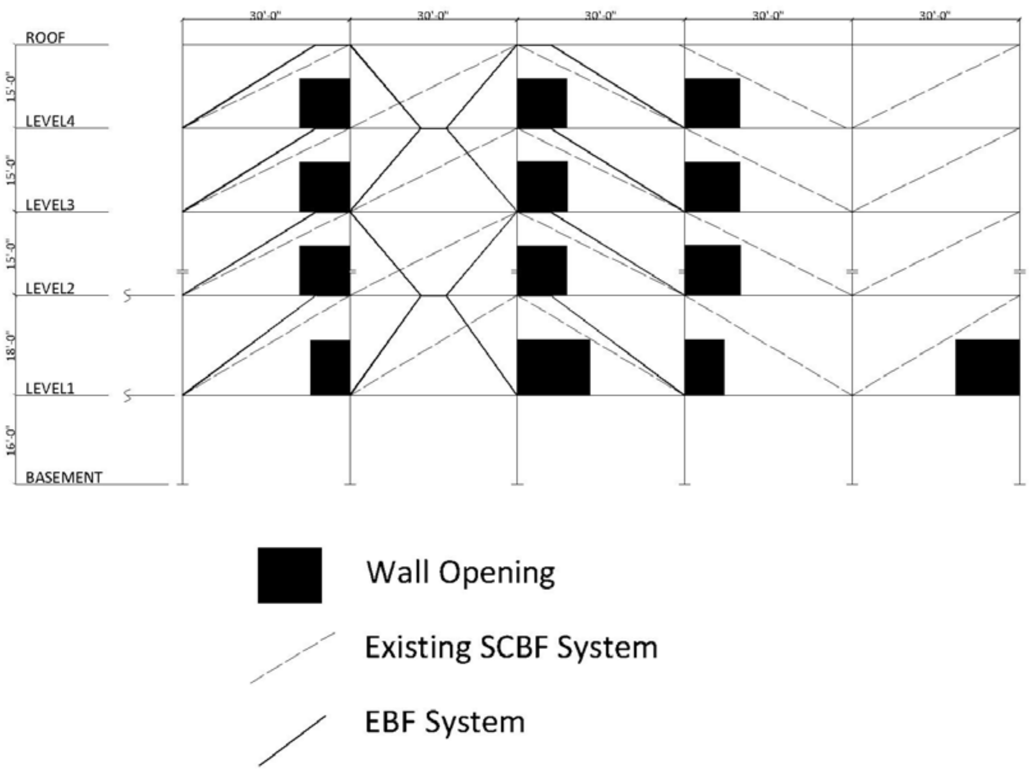


Figure 26. BF-1 Revised Wall Openings

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.

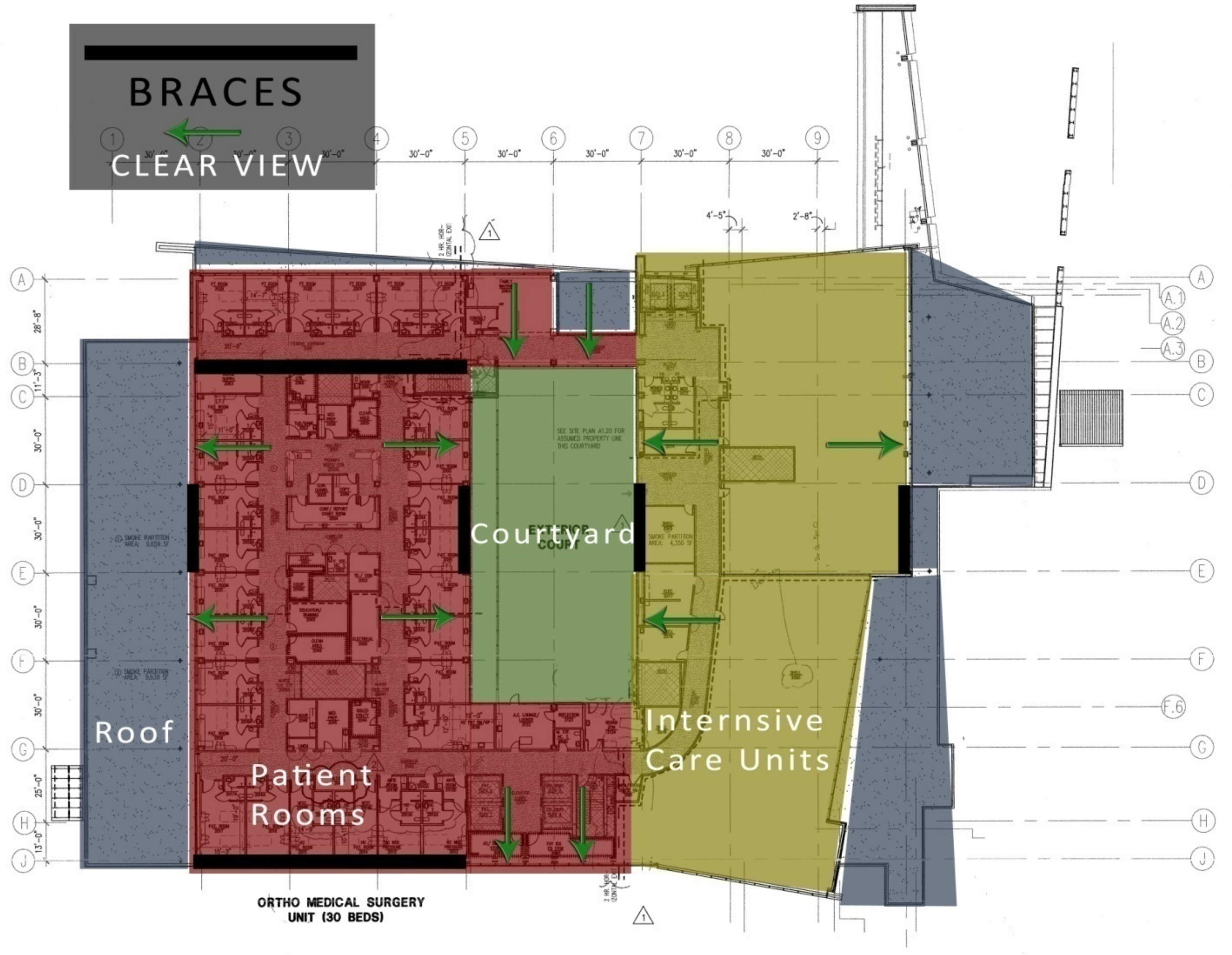


Figure 27. Architectural plan of level 2 showing where the new clear views to the outside exist.

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.

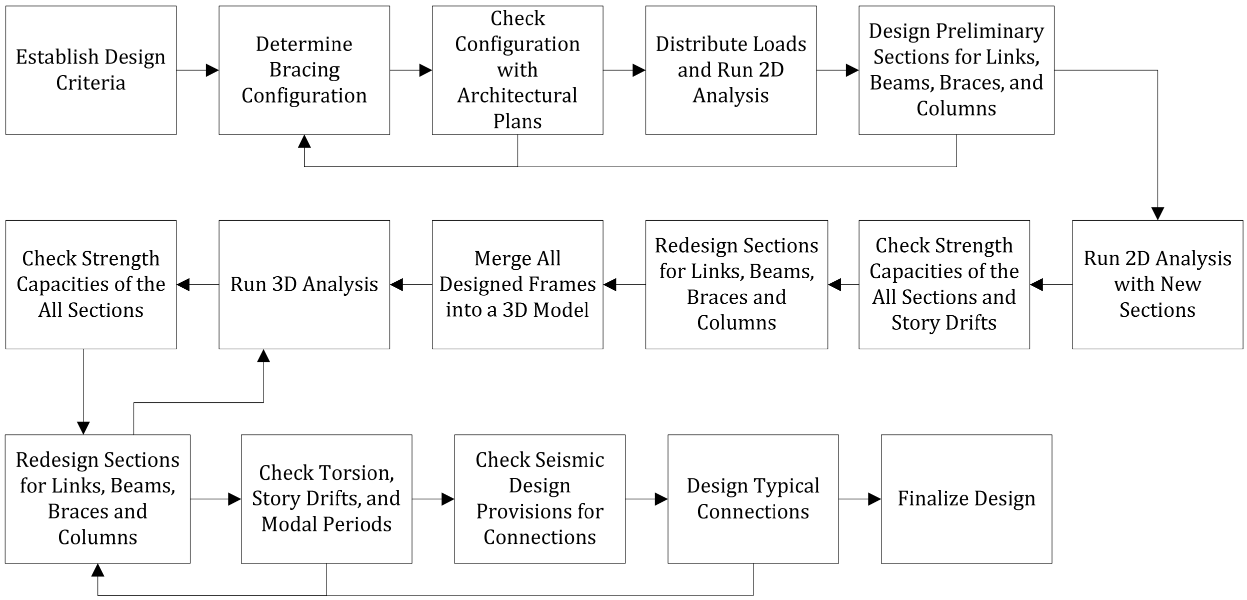


Figure 28. EBF Design Process Flow Chart

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.

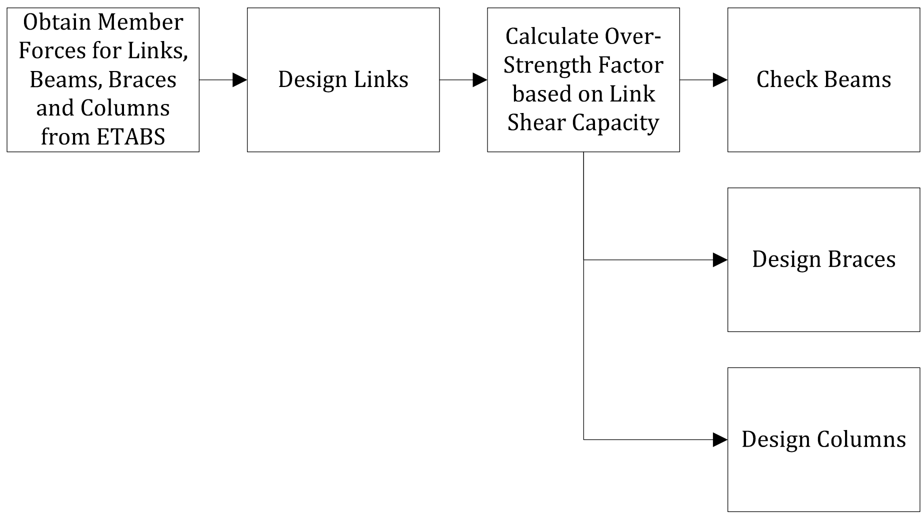


Figure 29. Member Design Flow Chart

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.

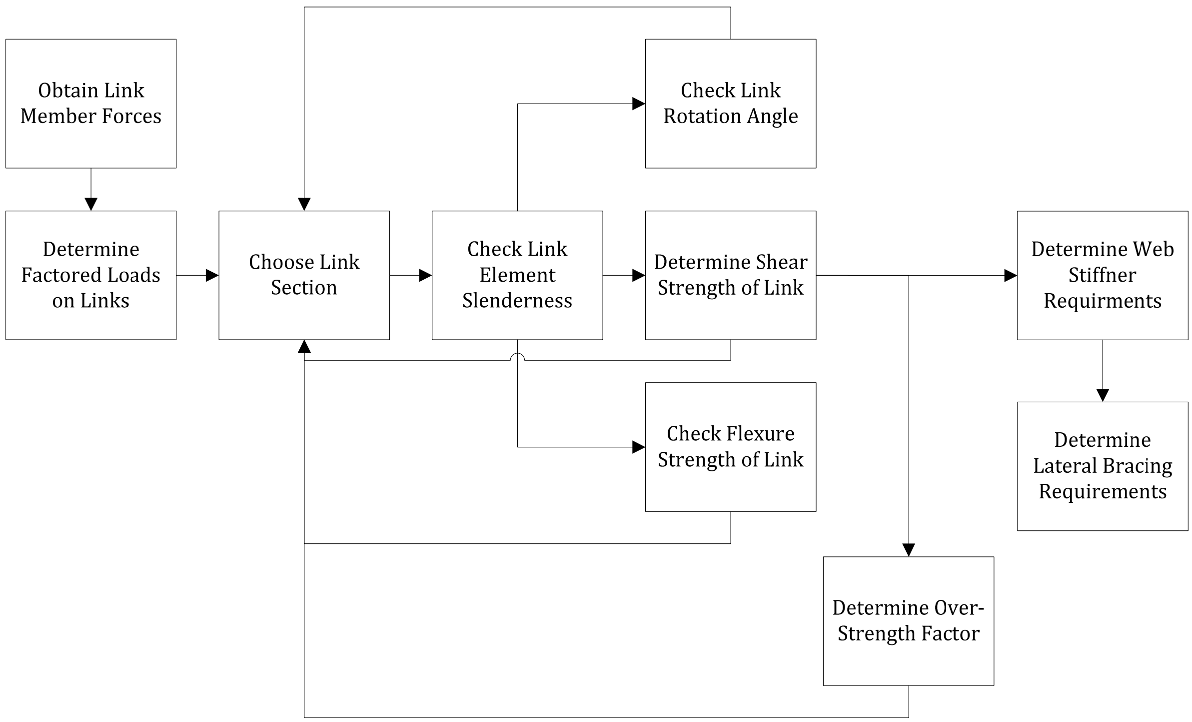
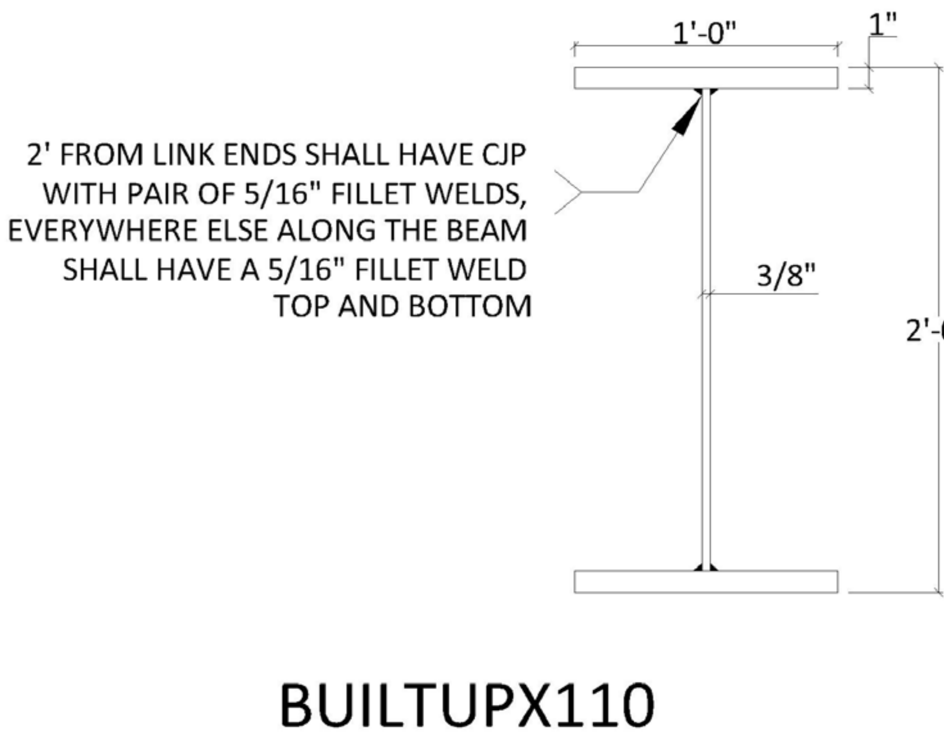


Figure 30. Link Design Flow Chart

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.

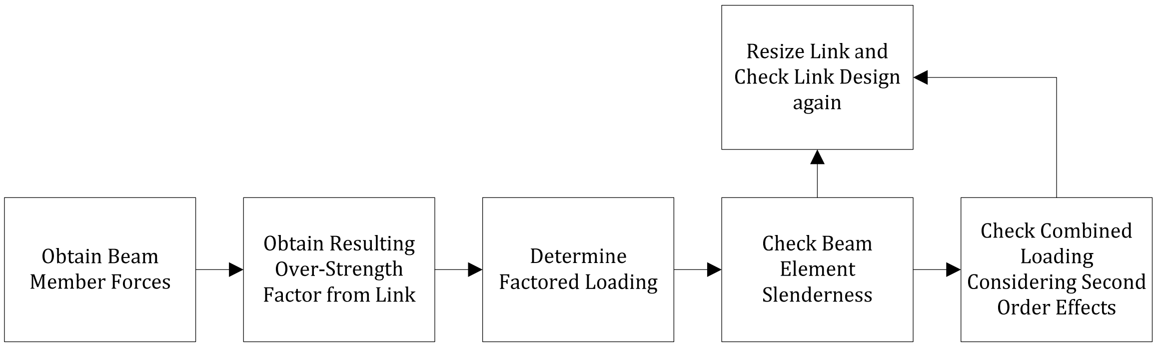


Figure 32. Beam Design Flow Chart

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.

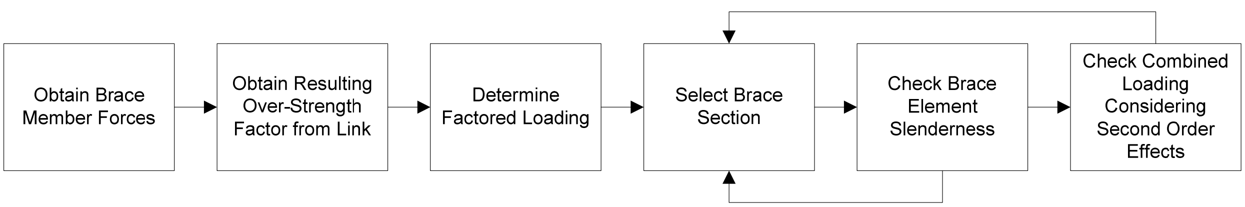


Figure 33. Brace Design Flow Chart

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.

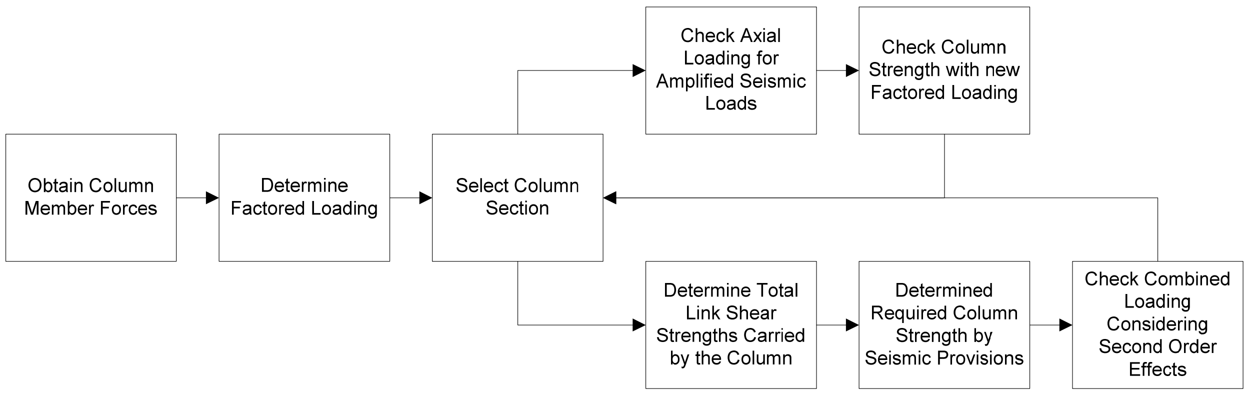


Figure 34. Column Design Flow Chart

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.

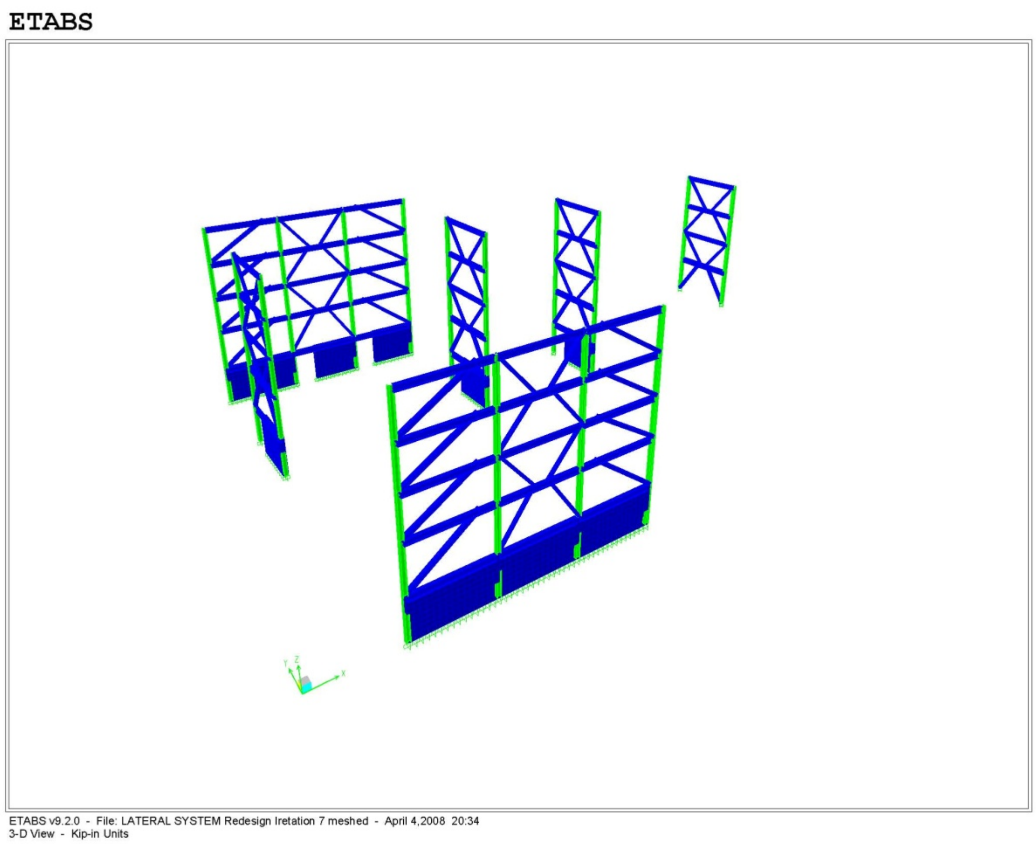


Figure 35. 3D ETABS Model of the EBF system

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.

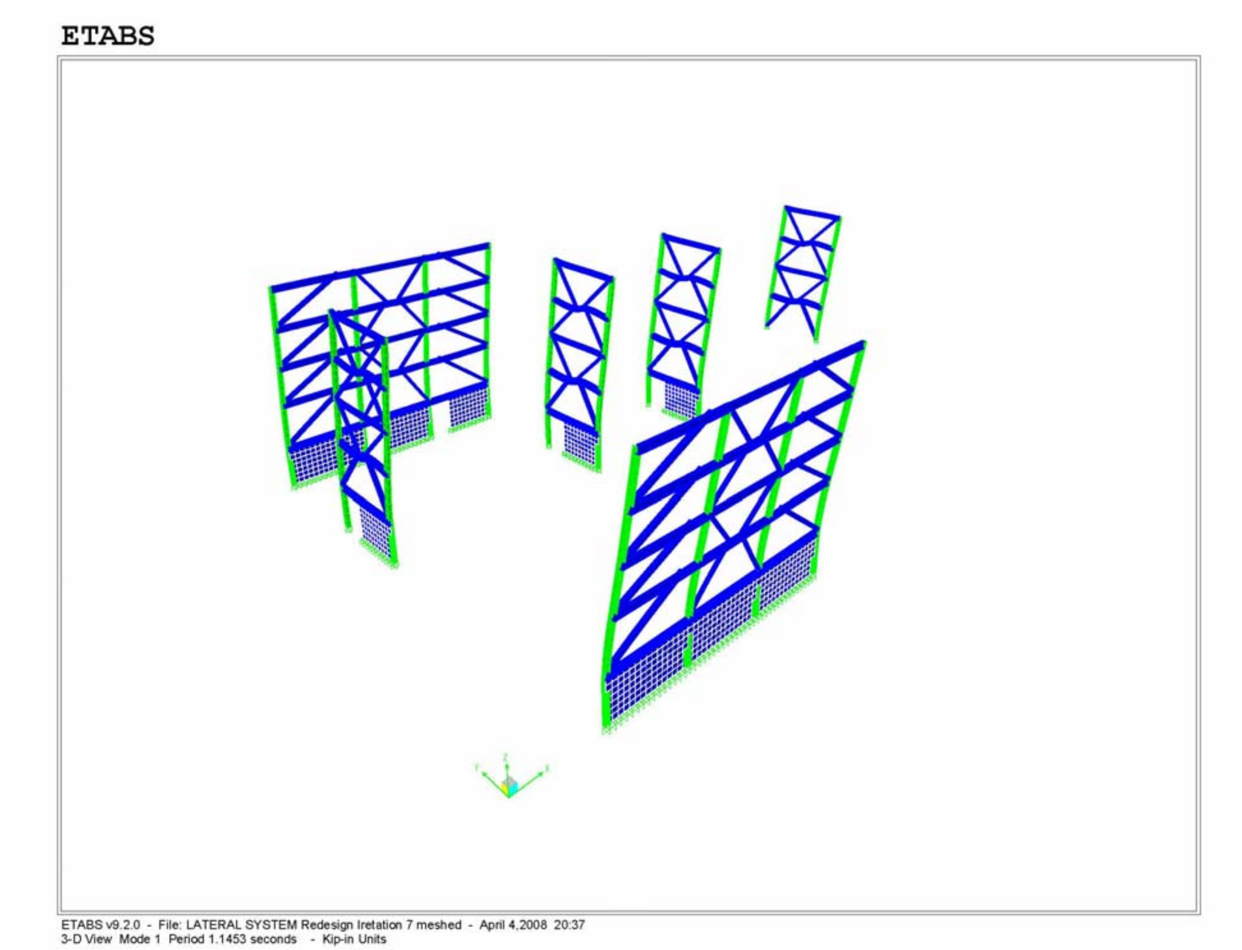


Figure 36. 3D Model in ETABS of the structure's first modal excitation.

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.

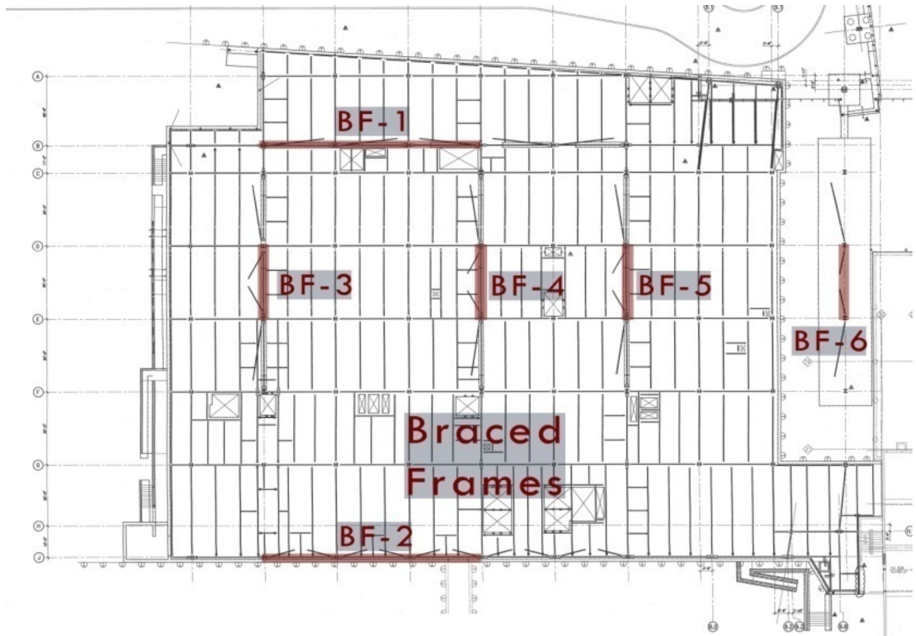


Figure 37. Level 1 plan with EBF system location

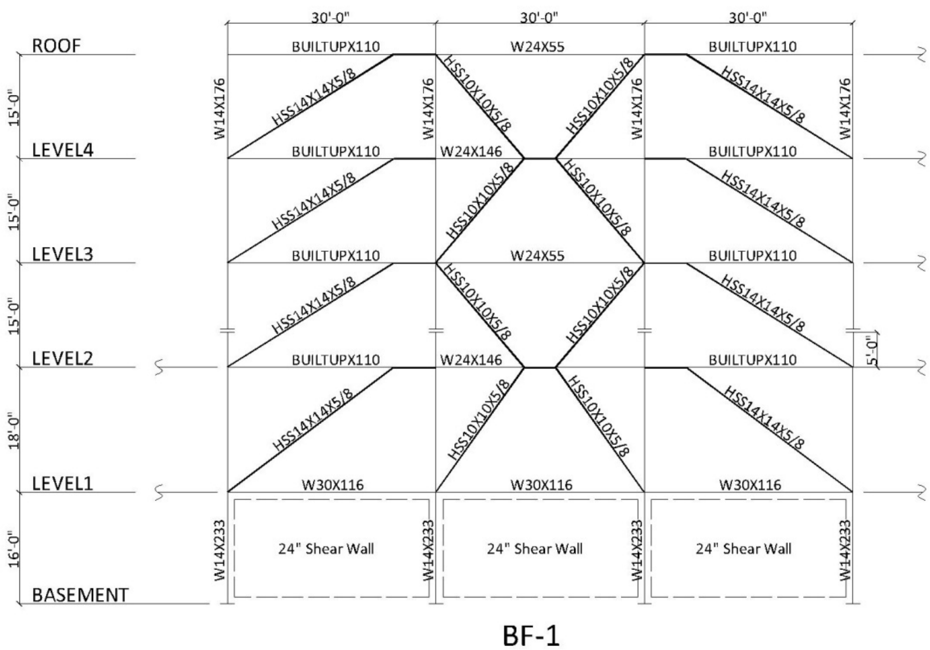
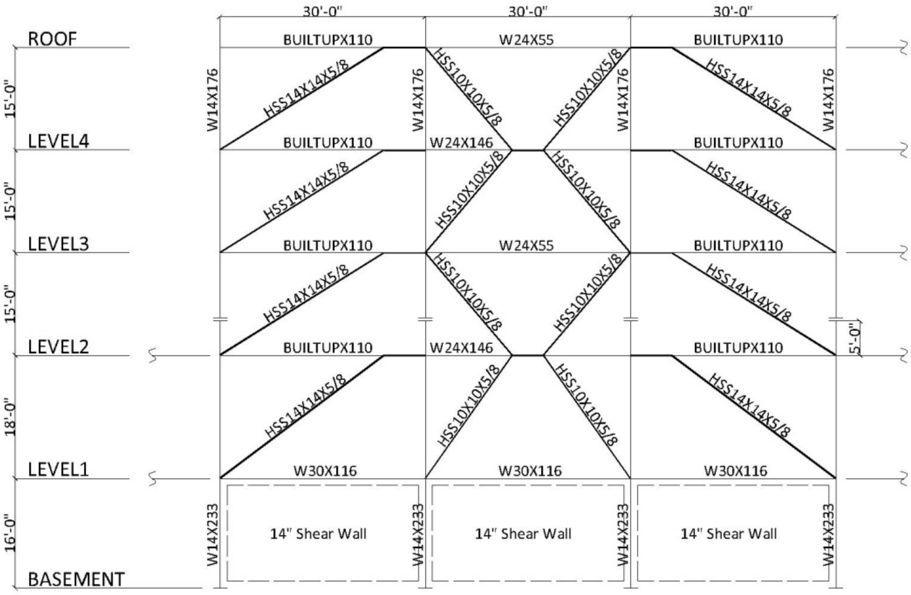
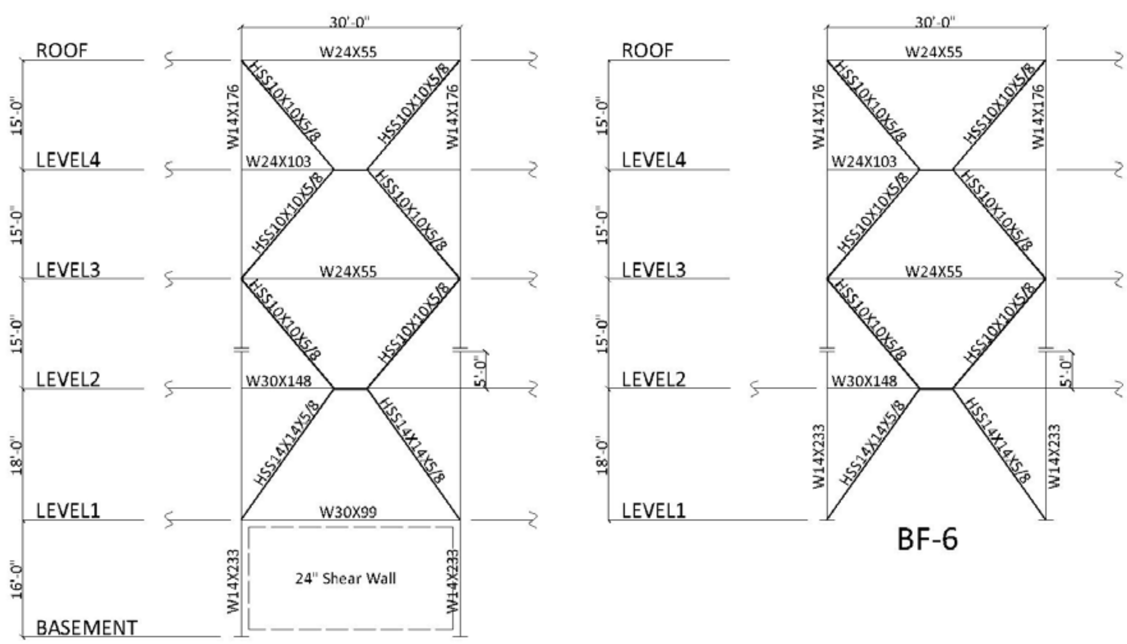


Figure 38. Elevation of BF-1



BF-2

Figure 39. Elevation of BF-2



BF-3,4,5

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.

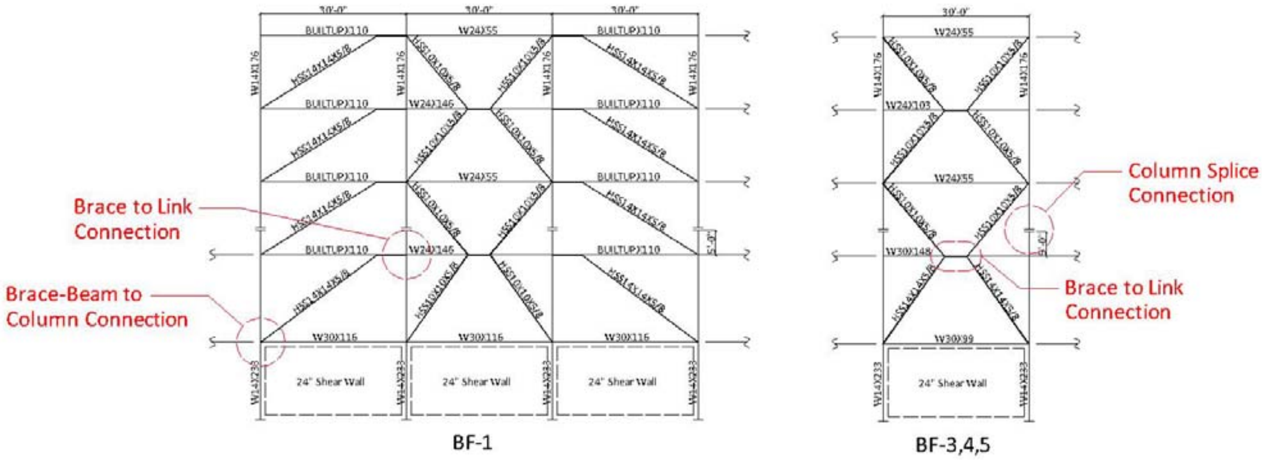


Figure 41. Location of Typical Connections in Elevation view.

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.

[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.

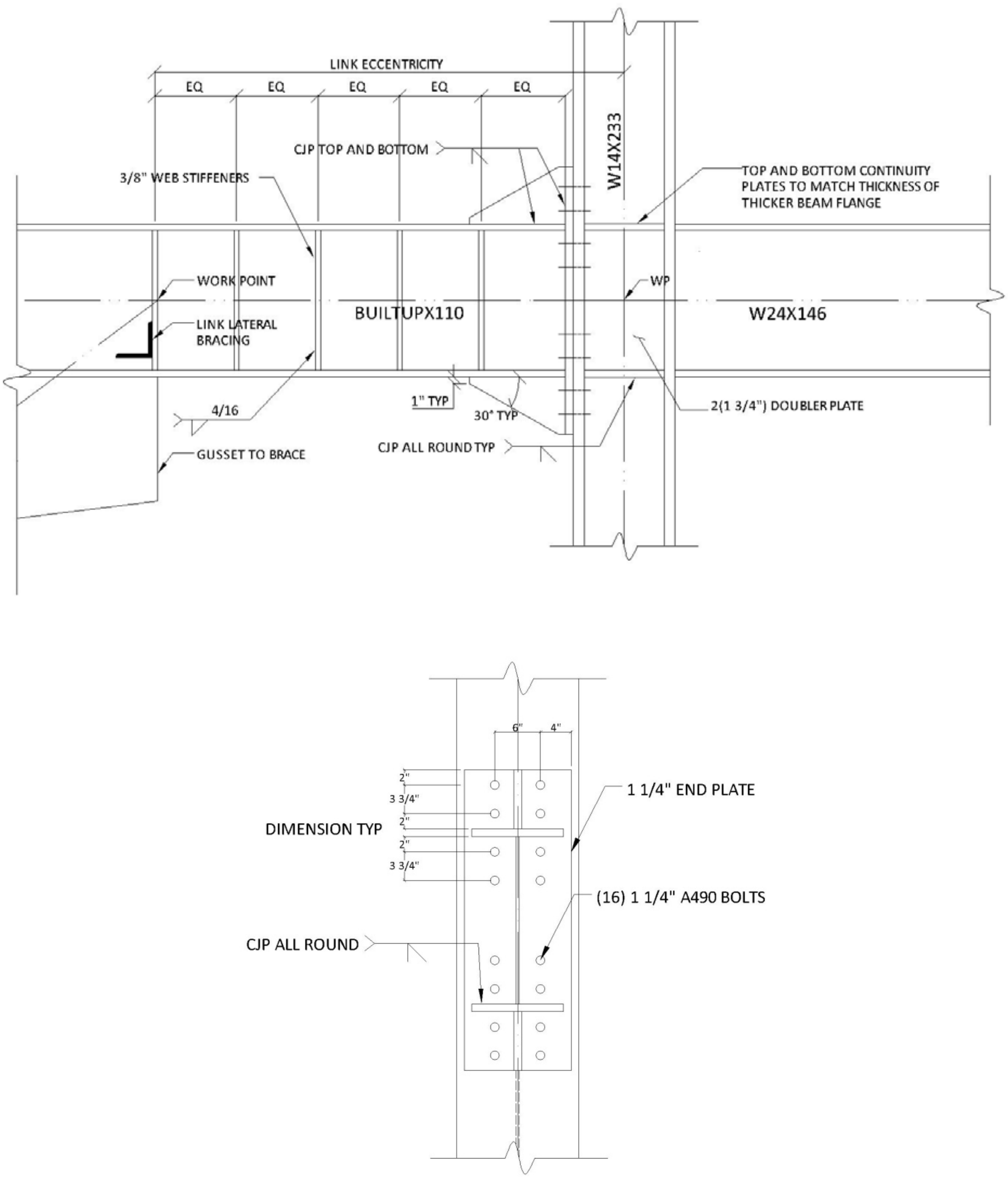


Figure 42. Link to Column Connection Option 1

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.

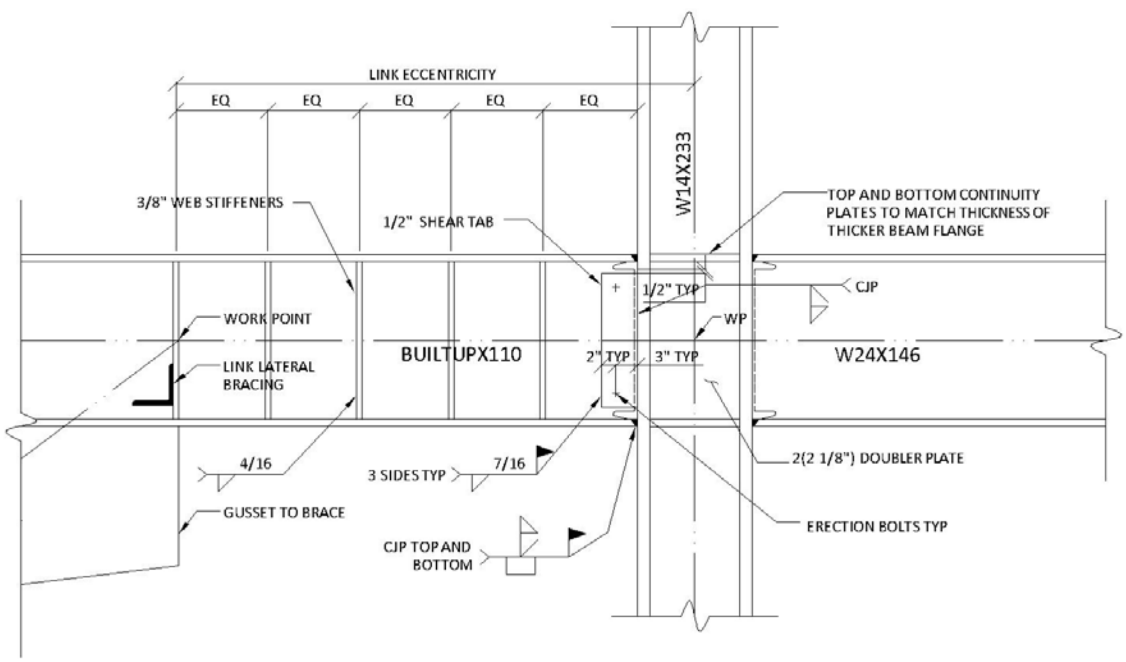


Figure 43. Link to Column Connection Option 2.

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.

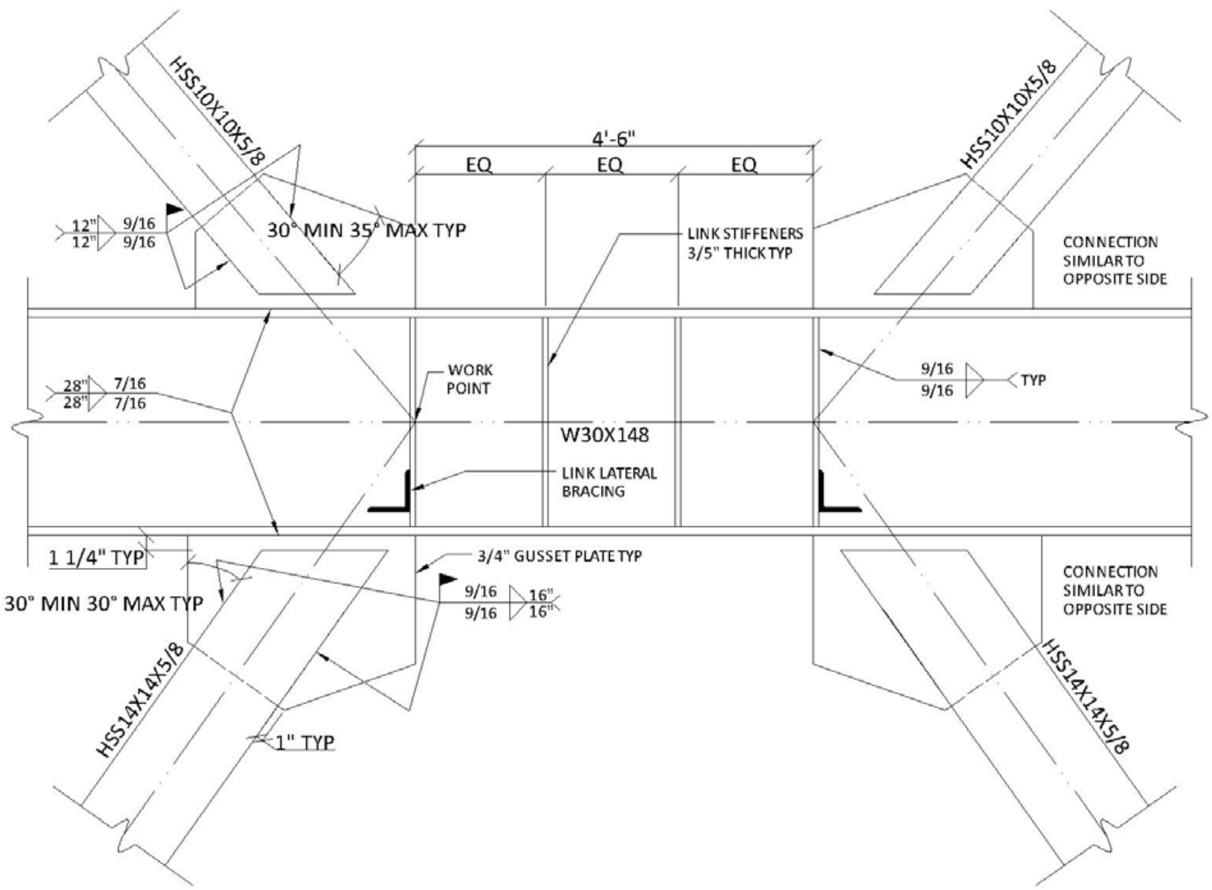


Figure 44. Brace to Link Connection

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.

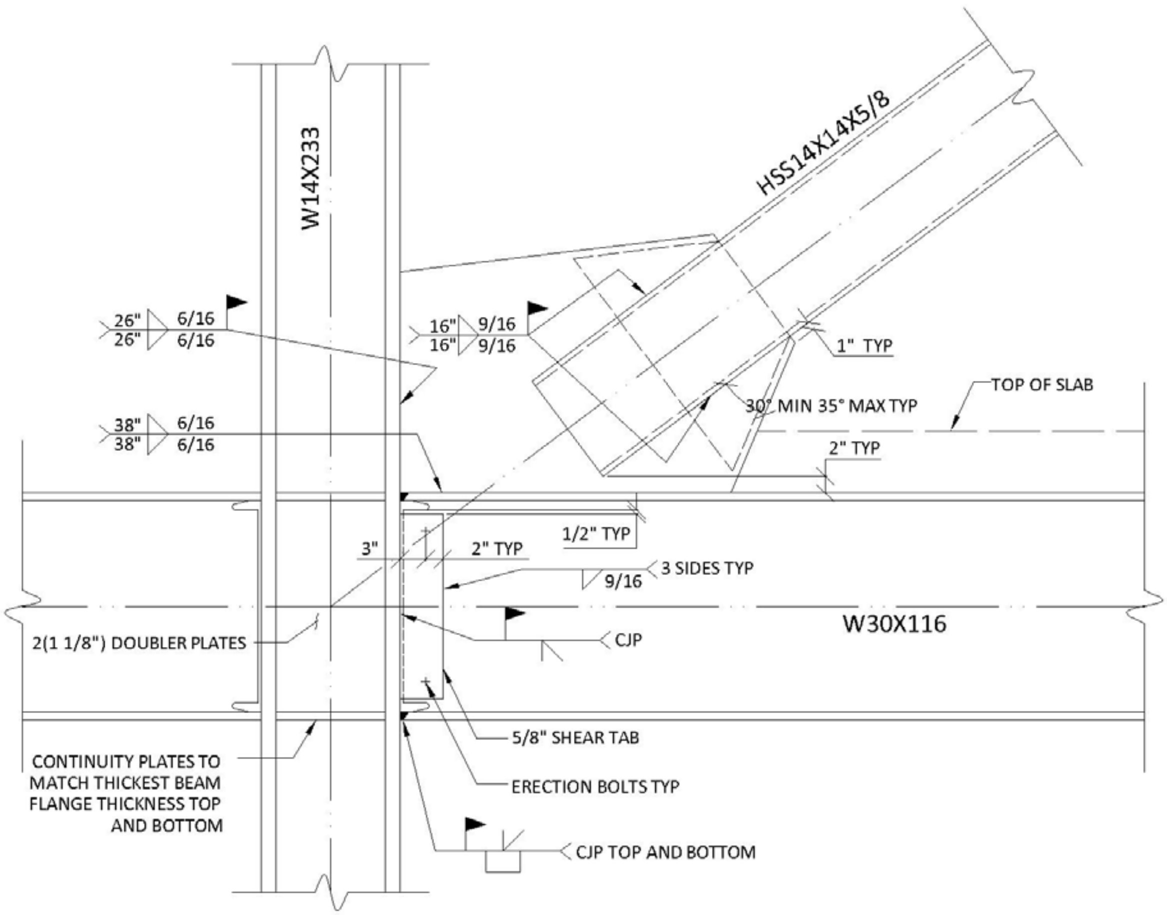


Figure 45. Brace-Beam to Column Connection

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.

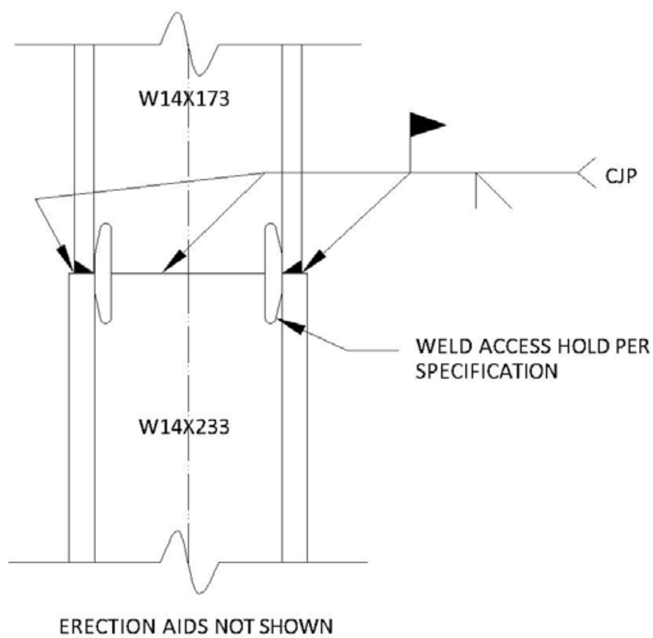


Figure 46. Column Splice Connection

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

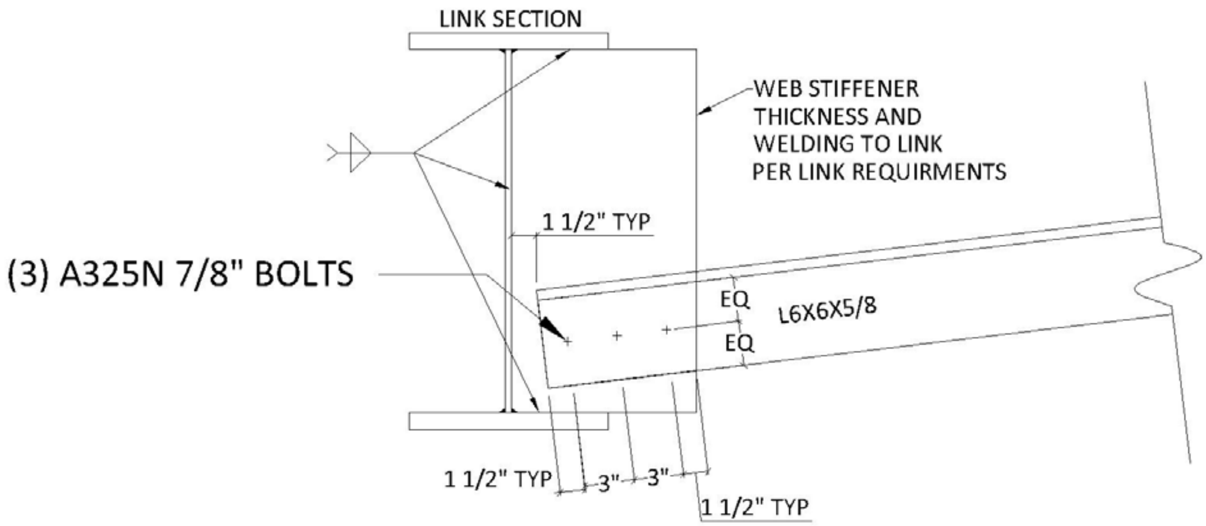


Figure 47. Link Lateral Brace Connection

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.

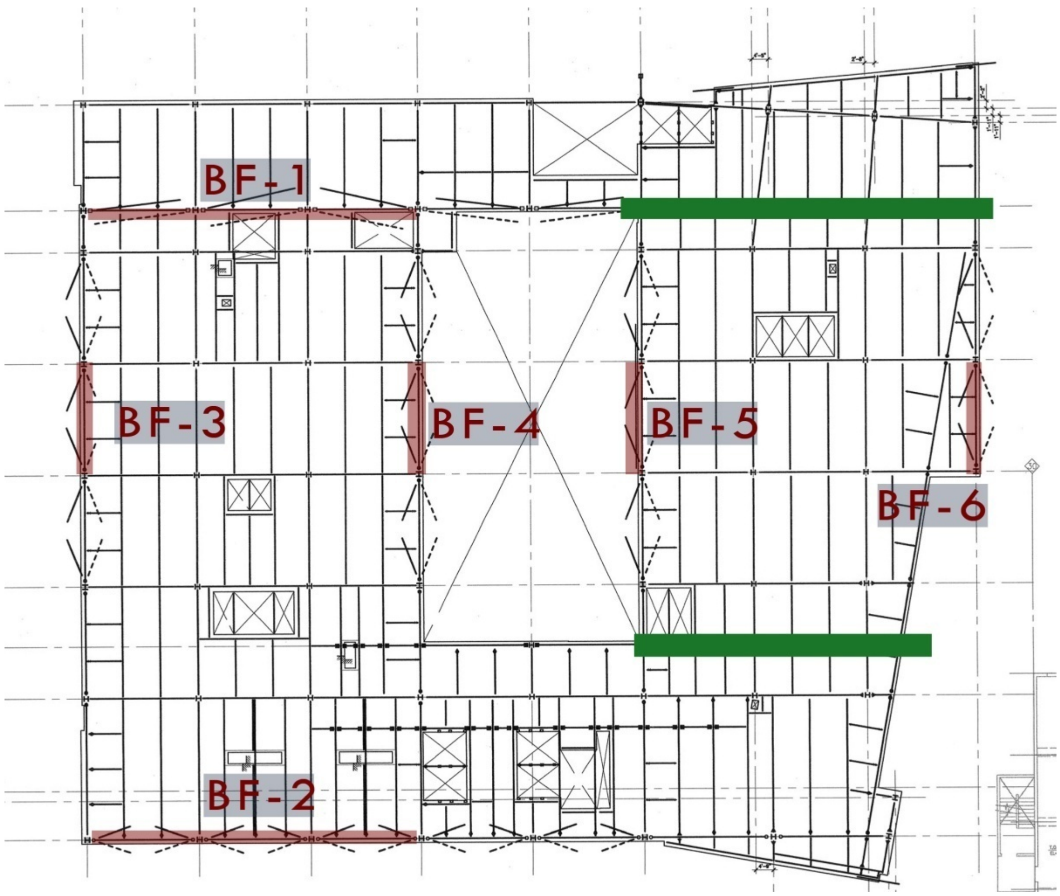


Figure 48. Plan of Level 3 showing EBF system and critical diaphragm locations

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

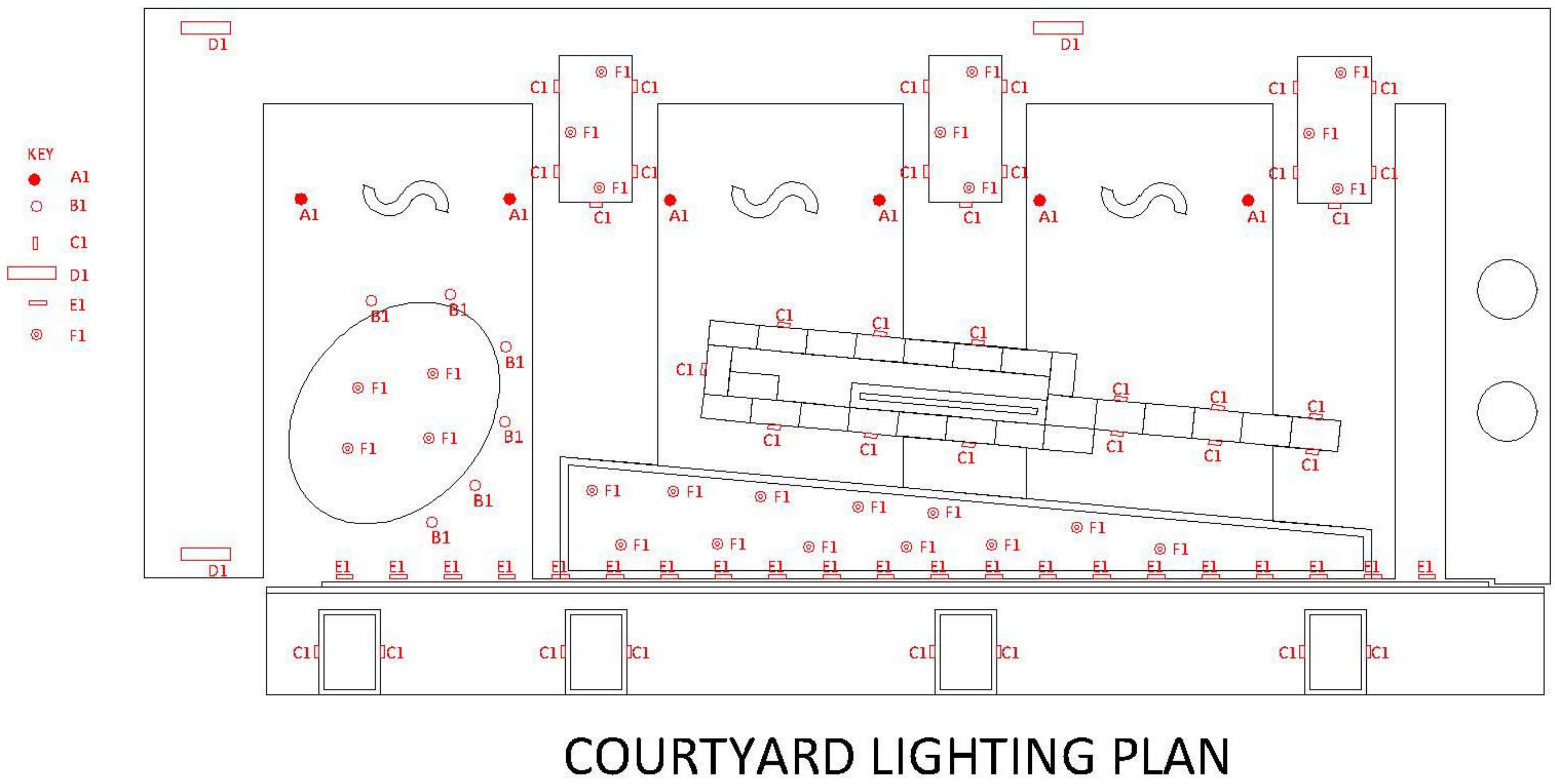
Light Fixture	Lighting Comment
Compact Fluorescent Bollard	Comfortable lighting, general illumination next to seating
Compact Fluorescent In grade Circular up light	Visual interest, highlight tree planter
Compact Fluorescent Step Light	Compliments architecture, directional flow
LED Wall Washers	Creates 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



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.

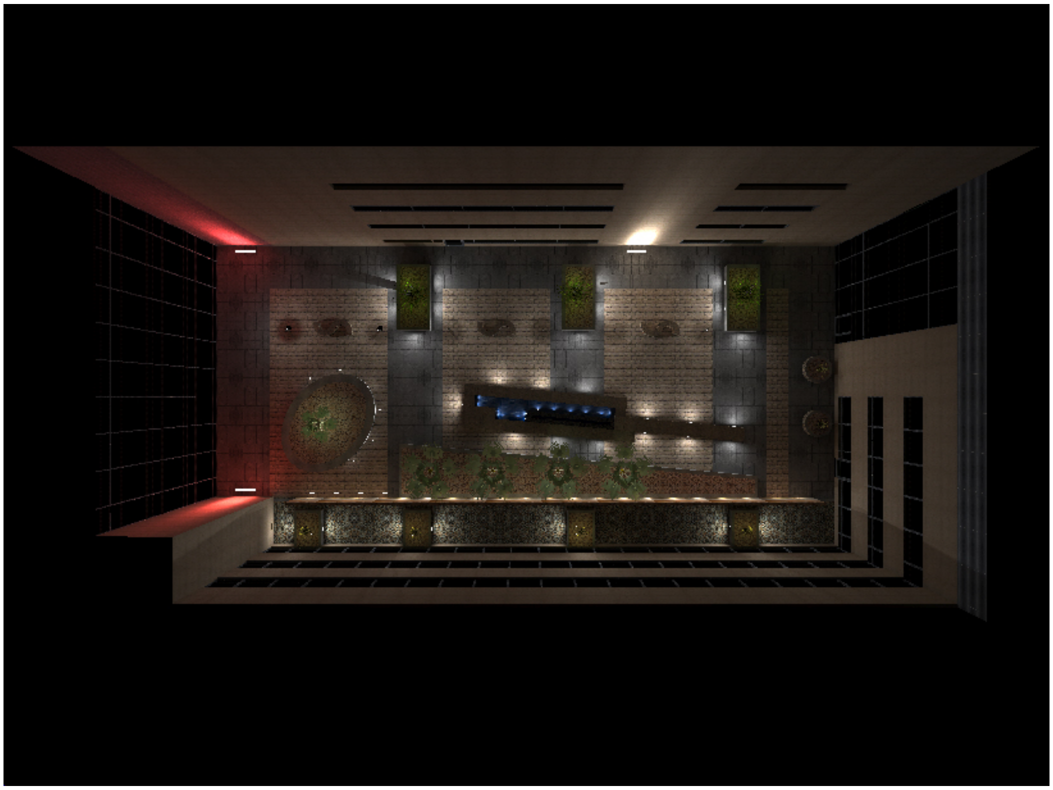


Figure 50. AGI Rendering of the Central Courtyard in plan.

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.

Appendix

Table of Contents

Appendix A. Structural Depth

SCBF System Design Check (Existing Structure)

Seismic Response Coefficient

Calculating Seismic Response Coefficient

		Refrences			Refrences			Refrences
Ss	1.378		height	63		TL	8	
S1	0.497		Ct	0.02	Table 12.8-2	Tb	0.413	Modal Period
I	1.5	Table 11.5-1	x	0.75	Table 12.8-2	Sds	0.918667	
Fa	1	Table 11.4-1	Ta	0.447234613		Sd1	0.497	
Fv	1.5	Table 11.4-2	Cu	1.4	Table 12.8-1	SDC	D	Table 11.6-1
			CuTa	0.626128458				

X Direction

Resisting System	Concentricalled Braced Frames			
R	6			
Cd	5			
Cs min(_,_,_)	0.229667	0.300847458	5.827554	
min Cs Check	N/A	0.01	OK	Check
Cs	0.229667			

Y Direction

Resisting System	Concentricalled Braced Frames		
R	6		
Cd	5		
Cs min(_,_,_)	0.229667	0.300847	5.827553659
min Cs Check	N/A	0.01	OK
Cs	0.229667		

Vertical Distribution of Seismic Forces

Vertical Distribution of Seismic Forces

X-Direction Loading

T	0.626	s
k	1.063	(ASCE 7-05 12.8.3)
Cs	0.23	
W	19376.1	kips
V	4456.503	

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	1769	1769	198	9.9	1.0	17515
4	15	48	3566	218444	0.25	1095	2864	198	9.9	1.0	10836
3	15	33	3566	146677	0.16	735	3599	198	9.9	1.0	7276
2	18	18	7927	171189	0.19	858	4457	198	9.9	1.0	8492
		Sum	19376	889388		4457					44119

Y-Direction Loading

T	0.626	s
k	1.063	(ASCE 7-05 12.8.3)
Cs	0.23	
W	19376.1	kips
V	4456.503	

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

Accidental Torsion

From EQX, the displacements are;

Δ_a = .64713", Δ_b = .650098"

Therefore Δ_{avg} = .649"

A_x =[.65/(1.2)(.648)]^2 = .84 **Therefore use 1**

From EQY, the displacements are;

Δ_a = .6296", Δ_b = .7411"

Therefore Δ_{avg} = .6854"

A_x =[.7411/(1.2)(1.854)]^2 = .90 **Therefore use 1**

5% (Accidental Torsion) (1) = **5%**

Therefore a 5% displacement eccentricity ratio to the center of mass was applied to each level where the lateral loads were applied. The analysis was then run again and the story drift from the controlling load combination was taken for comparison to the allowable story drift.

Story Drift Check

Story Drift Check

Importance Factor	1.5	
Cd	5	
Drift Limit	0.015	Table 12.12-1

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

Spot Checks

Using AISC Steel Manual Table 4-1, the following Available Strength Axial Compression Capacities are computed to check for braces which are connected with the moments released in its local x and y.

Member	Type	KL	$\Phi_c P_n$	P_u	$P_u / \Phi_c P_n$
HSS10x10x5/8	Brace	23.4	603	343	.57
HSS8x8x5/8	Brace	21.2	420	296	.70
W14x109	Brace	33.5	617	584	.95
W14x132	Brace	33.5	757	700	.92
W14x211	Brace	35	1280	902	.70

The following checks for steel columns are done using the steel frame design in ETABS; a hand calculation was done to verify this answer. Refer to appendix for comparison. Note that all the load combinations for the column design are with over strength factors.

Member	Grid Location	Member Type	KL	Demand/Capacity Ratio
W14x145	B-3	Column	18'	.645
W14x311	B-7	Column	18'	.77
W14x398	C-2	Column	18'	.565

EBF Design Calculations (New System)

Seismic Response Coefficient

Calculating Seismic Response Coefficient

		Refrences			Refrences			Refrences
Ss	1.378		height	63		TL	8	
S1	0.497		Ct	0.03	Table 12.8-2	Tb	1.27	Modal Period
I	1.5	Table 11.5-1	x	0.75	Table 12.8-2	Sds	0.918667	
Fa	1	Table 11.4-1	Ta	0.67085192		Sd1	0.497	
Fv	1.5	Table 11.4-2	Cu	1.4	Table 12.8-1	SDC	D	Table 11.6-1
			CuTa	0.939192687				

X Direction

Resisting System	EBF w/o Mom. Conn.			
R	8			
Cd	4			
Cs min(_,_,_)	0.17225	0.099220853	0.845159	
min Cs Check	N/A	0.01	OK	Check
Cs	0.099221			

Y Direction

Resisting System	EBF w/o Mom. Conn.		
R	8		
Cd	4		
Cs min(_,_,_)	0.17225	0.099221	0.845158654
min Cs Check	N/A	0.01	OK
Cs	0.099221		

Vertical Distribution of Forces

Vertical Distribution of Seismic Forces

X-Direction Loading

T	0.94	s
k	1.22	(ASCE 7-05 12.8.3)
Cs	0.099	
W	19376.1	kips
V	1918.234	

`	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	676655	0.42	811	811	198	9.9	1.0	8025
4	15	48	3566	401140	0.25	481	1291	198	9.9	1.0	4757
3	15	33	3566	253962	0.16	304	1595	198	9.9	1.0	3012
2	18	18	7927	269496	0.17	323	1918	198	9.9	1.0	3196
		Sum	19376	1601253		1918					18991

Y-Direction Loading

T	0.94	s
k	1.22	(ASCE 7-05 12.8.3)
Cs	0.099	
W	19376.1	kips
V	1918.234	

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	676655	0.42	811	811	240	12.0	1.0	9727
4	15	48	3566	401140	0.25	481	1291	240	12.0	1.0	5767
3	15	33	3566	253962	0.16	304	1595	240	12.0	1.0	3651
2	18	18	7927	269496	0.17	323	1918	285	14.3	1.0	4601
		Sum	19376	1601253		1918					23745

Torsion

Accidental Torsion

The analysis was first run with the seismic loads running in the x and y assigned to the center of mass + 5 percent accidental torsion. The following calculation was done to calculate the amplified accidental torsion according to ASCE 7-05 section 12.8.4.2.

From EQX, the displacements are;

$\Delta_a = 1.121''$, $\Delta_b = 1.064''$

Therefore $\Delta_{avg} = 1.09''$

$A_x = [\ 1.121/(1.2)(1.09)\ ]^2 = .86$ **Therefore use 1**

From EQY, the displacements are;

$\Delta_a = 1.8518''$, $\Delta_b = 1.8568''$

Therefore $\Delta_{avg} = 1.8543''$

$A_x = [\ 1.8568/(1.2)(1.854)\ ]^2 = .70$ **Therefore use 1**

5% (Accidental Torsion) (1) = **5%**

Therefore a 5% displacement eccentricity ratio to the center of mass was applied to each level where the lateral loads were applied. The analysis was then run again and the story drift from the controlling load combination was taken for comparison to the allowable story drift.

Story Drifts

Story Drift Check

Importance Factor	1.5	
Cd	4	
Drift Limit	0.015	Table 12.12-1

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

EBF Design Spread Sheet

Links Design Spread Sheet

Link Design

Link Location	Link Shape	Ag	d	tw	tf	Zx (in3)	bf	Ry	e Link Length (ft)	Fy (ksi)	E (ksi)	Vu (K)	Mu (in-K)	Pu (kips)	Story Drift (in)	height (ft)	Bay L. (ft)	λf	λps	Flange?
		Dimentions								Steel Properties		Loading			Deflection					
TEST	W16X77	22.9	16.5	0.455	0.76	152	10.3	1.1	4	50	29000	112	2916	20.2	0.7	12.5	30	6.776	7.225	OK
BF-3		#N/A	#N/A	#N/A	#N/A	#N/A	#N/A													
BF-3 4th Floor	W24X103	30.3	24.5	0.55	0.98	280	9	1.1	4.5	50	29000	261	6983	112	1.5594	15	30	4.592	7.225	OK
BF-3 2th Floor	W30X148	43.5	30.7	0.65	1.18	500	10.5	1.1	4.5	50	29000	468	12778	66	1.49112	18	30	4.449	7.225	OK
BF-1 BAY 2-3		#N/A	#N/A	#N/A	#N/A	#N/A	#N/A											#N/A	#DIV/0!	#N/A
BF-1 Roof B1	BUILTUP4	32.25	24	0.375	1	321	12	1.1	6	50	29000	119	7653	167	0.6042	15	30	6.000	7.225	OK
BF-1 4th B1	BUILTUP4	32.25	24	0.375	1	321	12	1.1	6	50	29000	134	7955	92	0.7626	15	30	6.000	7.225	OK
BF-1 3rd B1	BUILTUP4	32.25	24	0.375	1	321	12	1.1	6	50	29000	163	9149	92	1.0002	15	30	6.000	7.225	OK
BF-1 2nd B1	BUILTUP4	32.25	24	0.375	1	321	12	1.1	6	50	29000	219	12056	50	1.36872	18	30	6.000	7.225	OK
BF-1 BAY 3-4		#N/A	#N/A	#N/A	#N/A	#N/A	#N/A											#N/A	#DIV/0!	#N/A
BF-1 4th B2	W24X146	43	24.7	0.65	1.09	418	12.9	1.1	4.5	50	29000	219	6458	183	0.7626	15	30	5.917	7.225	OK
BF-1 2nd B2	W24X146	43	24.7	0.65	1.09	418	12.9	1.1	4.5	50	29000	316	9419	52	1.36872	18	30	5.917	7.225	OK
BF-6		#N/A	#N/A	#N/A	#N/A	#N/A	#N/A											#N/A	#DIV/0!	#N/A
BF-6 4th Floor	W24X103	30.3	24.5	0.55	0.98	280	9	1.1	4.5	50	29000	278	7431	112	1.5594	15	30	4.592	7.225	OK
BF-6 2th Floor	W30X148	43.5	30.7	0.65	1.18	500	10.5	1.1	4.5	50	29000	504	13762	66	1.49112	18	30	4.449	7.225	OK
		#N/A	#N/A	#N/A	#N/A	#N/A	#N/A											#N/A	#DIV/0!	#N/A
		#N/A	#N/A	#N/A	#N/A	#N/A	#N/A											#N/A	#DIV/0!	#N/A
		#N/A	#N/A	#N/A	#N/A	#N/A	#N/A											#N/A	#DIV/0!	#N/A
		#N/A	#N/A	#N/A	#N/A	#N/A	#N/A											#N/A	#DIV/0!	#N/A
		#N/A	#N/A	#N/A	#N/A	#N/A	#N/A											#N/A	#DIV/0!	#N/A

Link Design

Link	Link	Slenderness Check									Calculations							Check
Location	Shape	λw	Ca	λps	λps	Web?	Aw	Mp (ft-K)	2Mp/e (kips)	Vp (kips)	e/(Mp/Vp)	Py (kips)	ΦVn (kips)	Mu/ΦMn	Pu/ΦPn	Vu/ΦVn	Shear	Flexural
TEST	W16X77	29.9	0.019602	73.3384	62.3188	OK	6.816	633.33333	316.6666667	204.477	1.29	1,145.00	184.03	0.43	0.02	0.61	OK	OK
BF-3												#N/A	-	#DIV/0!	#N/A	#DIV/0!		
BF-3 4th Floor	W24X103	39.2	0.082142	66.0553	60.6319	OK	12.4	1166.6667	518.5185185	371.91	1.43	1,515.00	334.72	0.55	0.08	0.78	OK	OK
BF-3 2th Floor	W30X148	41.6	0.033716	71.6947	61.9381	OK	18.42	2083.3333	925.9259259	552.63	1.19	2,175.00	497.37	0.57	0.03	0.94	OK	OK
BF-1 BAY 2-3		#N/A	#N/A	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
BF-1 Roof B1	BUILTUP4	61.33	0.115073	62.2202	59.7436	OK	8.25	1337.5	445.8333333	247.5	1.11	1,612.50	222.75	0.53	0.12	0.53	OK	OK
BF-1 4th B1	BUILTUP4	61.33	0.063394	68.2386	61.1376	OK	8.25	1337.5	445.8333333	247.5	1.11	1,612.50	222.75	0.55	0.06	0.60	OK	OK
BF-1 3rd B1	BUILTUP4	61.33	0.063394	68.2386	61.1376	OK	8.25	1337.5	445.8333333	247.5	1.11	1,612.50	222.75	0.63	0.06	0.73	OK	OK
BF-1 2nd B1	BUILTUP4	61.33	0.034453	71.6089	61.9182	OK	8.25	1337.5	445.8333333	247.5	1.11	1,612.50	222.75	0.83	0.03	0.98	OK	OK
BF-1 BAY 3-4		#N/A	#N/A	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
BF-1 4th B2	W24X146	33.2	0.094574	64.6075	60.2965	OK	14.64	1741.6667	774.0740741	439.14	1.13	2,150.00	395.23	0.34	0.09	0.55	OK	OK
BF-1 2nd B2	W24X146	33.2	0.026873	72.4916	62.1226	OK	14.64	1741.6667	774.0740741	439.14	1.13	2,150.00	395.23	0.50	0.03	0.80	OK	OK
BF-6		#N/A	#N/A	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
BF-6 4th Floor	W24X103	39.2	0.082142	66.0553	60.6319	OK	12.4	1166.6667	518.5185185	371.91	1.43	1,515.00	334.72	0.59	0.08	0.83	OK	OK
BF-6 2th Floor	W30X148	41.6	0.033716	71.6947	61.9381	OK	18.42	2083.3333	925.9259259	552.63	1.19	2,175.00	497.37	0.61	0.03	1.01	Not OK	OK
		#N/A	#N/A	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
		#N/A	#N/A	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
		#N/A	#N/A	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
		#N/A	#N/A	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
		#N/A	#N/A	#DIV/0!	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Link Design

L

Link	Link	s		Rotation		Link End Stiffners (Double Sided)					Intermediate Web Stiffners
Location	Shape	Axial Effect?	Rotation (Radians)	Limit (Radians)	Check?	Width (in)	Thickness (in)	< 1.6Mp/Vp 0.08 rad	< 1.6 .02 rad	<1.6 Interpolation w/ Rot.	2.6 to 5
TEST	W16X77	No	0.035	0.08	OK	4.70	0.375	10.35	20.36	17.86	15.45
BF-3											
BF-3 4th Floor	W24X103	No	0.057755556	0.08	OK	3.95	0.4125	11.60	23.70	16.09	13.50
BF-3 2th Floor	W30X148	No	0.046022222	0.08	OK	4.60	0.4875	13.36	27.66	21.46	15.75
BF-1 BAY 2-3		#N/A	#DIV/0!	#N/A	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#DIV/0!	#N/A
BF-1 Roof B1	BUILTUP4	No	0.016783333	0.08	OK	5.63	0.375	6.45	14.70	14.70	18.00
BF-1 4th B1	BUILTUP4	No	0.021183333	0.08	OK	5.63	0.375	6.45	14.70	14.54	18.00
BF-1 3rd B1	BUILTUP4	No	0.027783333	0.08	OK	5.63	0.375	6.45	14.70	13.63	18.00
BF-1 2nd B1	BUILTUP4	No	0.031683333	0.08	OK	5.63	0.375	6.45	14.70	13.09	18.00
BF-1 BAY 3-4		#N/A	#DIV/0!	#N/A	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#DIV/0!	#N/A
BF-1 4th B2	W24X146	No	0.028244444	0.08	OK	5.80	0.4875	14.56	28.86	26.90	19.35
BF-1 2nd B2	W24X146	No	0.042244444	0.08	OK	5.80	0.4875	14.56	28.86	23.56	19.35
BF-6		#N/A	#DIV/0!	#N/A	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#DIV/0!	#N/A
BF-6 4th Floor	W24X103	No	0.057755556	0.08	OK	3.95	0.4125	11.60	23.70	16.09	13.50
BF-6 2th Floor	W30X148	No	0.046022222	0.08	OK	4.60	0.4875	13.36	27.66	21.46	15.75
		#N/A	#DIV/0!	#N/A	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#DIV/0!	#N/A
		#N/A	#DIV/0!	#N/A	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#DIV/0!	#N/A
		#N/A	#DIV/0!	#N/A	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#DIV/0!	#N/A
		#N/A	#DIV/0!	#N/A	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#DIV/0!	#N/A
		#N/A	#DIV/0!	#N/A	#DIV/0!	#N/A	#N/A	#N/A	#N/A	#DIV/0!	#N/A

Link Design

Link Location	Link Shape	ink Stiffners Spacing (in)	1x or 2x Stiffners	Thickness (in)	Width (in)	USE Width (in)	t (in)	Double Sided Fillet 70ksi (1/16ths) Ast	to Web	to Flange	min	Link Lateral Brace Design Mr (ft-K)	Pb (Kips)	Section
TEST	W16X77	17.86	1x Stiffner OK	0.455	4.70	4.75	1/2	2.38	4.17	2.67	3	8,360.00	31.87	
BF-3				#N/A	#N/A			-	#N/A	-	1			
BF-3 4th Floor	W24X103	16.09	1x Stiffner OK	0.550	3.95	4.00	5/8	2.50	2.52	3.45	4	15,400.00	39.29	L6X6X5/8
BF-3 2th Floor	W30X148	21.46	2x Req.	0.650	4.60	4.75	3/4	3.56	2.71	4.00	4	27,500.00	55.89	L6X6X5/8
BF-1 BAY 2-3		#N/A	#N/A	#N/A	#N/A			-	#N/A	-	1	#N/A	#N/A	
BF-1 Roof B1	BUILTUP4	14.70	1x Stiffner OK	0.375	5.63	5.75	3/8	2.16	2.24	1.94	3	17,655.00	46.06	L6X6X5/8
BF-1 4th B1	BUILTUP4	14.54	1x Stiffner OK	0.375	5.63	5.75	3/8	2.16	2.24	1.94	3	17,655.00	46.06	L6X6X5/8
BF-1 3rd B1	BUILTUP4	13.63	1x Stiffner OK	0.375	5.63	5.75	3/8	2.16	2.24	1.94	3	17,655.00	46.06	L6X6X5/8
BF-1 2nd B1	BUILTUP4	13.09	1x Stiffner OK	0.375	5.63	5.75	3/8	2.16	2.24	1.94	3	17,655.00	46.06	L6X6X5/8
BF-1 BAY 3-4		#N/A	#N/A	#N/A	#N/A			-	#N/A	-	1	#N/A	#N/A	
BF-1 4th B2	W24X146	26.90	1x Stiffner OK	0.650	5.80	6.00	3/4	4.50	4.55	3.85	4	22,990.00	58.42	L6X6X5/8
BF-1 2nd B2	W24X146	23.56	1x Stiffner OK	0.650	5.80	6.00	3/4	4.50	4.55	3.85	4	22,990.00	58.42	L6X6X5/8
BF-6		#N/A	#N/A	#N/A	#N/A			-	#N/A	-	1	#N/A	#N/A	
BF-6 4th Floor	W24X103	16.09	1x Stiffner OK	0.550	3.95	4.00	5/8	2.50	2.52	3.45	4	15,400.00	39.29	L6X6X5/8
BF-6 2th Floor	W30X148	21.46	2x Req.	0.650	4.60	4.75	3/4	3.56	2.71	4.00	4	27,500.00	55.89	L6X6X5/8
		#N/A	#N/A	#N/A	#N/A			-	#N/A	-	1	#N/A	#N/A	
		#N/A	#N/A	#N/A	#N/A			-	#N/A	-	1	#N/A	#N/A	
		#N/A	#N/A	#N/A	#N/A			-	#N/A	-	1	#N/A	#N/A	
		#N/A	#N/A	#N/A	#N/A			-	#N/A	-	1	#N/A	#N/A	
		#N/A	#N/A	#N/A	#N/A			-	#N/A	-	1	#N/A	#N/A	

Beams Design Spread Sheet

Beam Outside Link Design

		Link								Link Loading from Elastic Analysis		
Location	Shape	Ry	Vn	height (ft)	Bay Length (ft)	e Link Length (ft)	Pe (kips)	end Moment (ft-K)	1.1(Ry)(Vn)	Veq (kips)	Overstrength Factor	Vdead (kips)
TEST	W16X77	1.1	204.477	12.5	30	4	296.9	494.8	247.42	84	2.95	6.8
BF-3 4th Floor	W24X103	1.1	371.91	15	30	4.5	450.0	1012.5	450.01	256	1.76	9
BF-3 2th Floor	W30X148	1.1	552.63	15	30	4.5	668.7	1504.5	668.68	457	1.46	20
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
BF-1 Roof B1	BUILTUP4	1.1	247.5	15	30	6	299.5	898.4	299.48	59	5.08	33
BF-1 4th B1	BUILTUP4	1.1	247.5	15	30	6	299.5	898.4	299.48	106	2.83	25
BF-1 3rd B1	BUILTUP4	1.1	247.5	15	30	6	299.5	898.4	299.48	135	2.22	23
BF-1 2nd B1	BUILTUP4	1.1	247.5	18	30	6	249.6	898.4	299.48	197	1.52	24
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
BF-1 4th B2	W24X146	1.1	439.14	15	30	4.5	531.4	1195.6	531.36	204	2.60	16
BF-1 2nd B2	W24X146	1.1	439.14	18	30	4.5	442.8	1195.6	531.36	305	1.74	15
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	

Beam No Links

	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	
	#N/A	#N/A	#N/A				#N/A	#N/A	#N/A		#N/A	

Beam Outside

	Beam Loadings from Elastic Analysis								Load Combination Factors			Factored Beam Loadi	
Location	Mdead (in-ft)	Pdead (kips)	Veq (kips)	Meq (in-ft)	Peq (kips)	Vlive (kips)	Mlive (in-ft)	Plive (kips)	D	EQ	L	Vu	Mu
TEST	204	1	8.7	1356	105	4.8	135.6	0.7	1.4	1	1	**39.9**	**4415.2**
BF-3 4th Floor	419	6	53	7371	4	8	428	7	1.4	1	0.5	**109.8**	**13757.8**
BF-3 2th Floor	398	1	84	13333	4	4	141	1	1.4	1	0.5	**152.9**	**20136.5**
												#N/A	**#N/A**
BF-1 Roof B1	1309	11	7	1697	3	37	1429	11	1.4	1	1	**118.7**	**11875.3**
BF-1 4th B1	1173	37	8	1863	132	40	1780	42	1.4	1	0.5	**77.6**	**7795.6**
BF-1 3rd B1	998	22	14	3178	192	20	885	19	1.4	1	0.5	**73.3**	**8889.6**
BF-1 2nd B1	1047	27	13	3469	195	20	879	22	1.4	1	0.5	**63.4**	**7178.8**
												#N/A	**#N/A**
BF-1 4th B2	696	11	55	6079	84	25	1083	17	1.4	1	0.5	**178.2**	**17349.9**
BF-1 2nd B2	584	1	87	9091	47	13	487	1	1.4	1	0.5	**179.1**	**16899.1**
												#N/A	**#N/A**
												#N/A	**#N/A**
												#N/A	**#N/A**
												#N/A	**#N/A**
												#N/A	**#N/A**
												#N/A	**#N/A**

Beam No Links

Beam No Links

												52.0	2990.0
												34.0	2360.0
												37.0	2393.0
												4.0	1017.0
												3.0	533.0
												68.0	4127.0

Beam Outside

	ngs	Beam	Beam						Dimentions								
Location	Pu	Location	Shape	Ag	d	tw	tf	Zx (in3)	Zy	bf	Ix	Iy	ry	Jc	Sx	Sy	ho
TEST	311.4	TEST	W16X77	22.9	16.5	0.455	0.76	152	41.2	10.3	1120	138	2.46	3.86	136	26.9	15.74
BF-3 4th Floor	18.9	BF-3 4th Floor	W24X103	30.3	24.5	0.55	0.98	280	41.5	9	3000	119	1.99	7.07	245	26.5	23.52
BF-3 2th Floor	7.8	BF-3 2th Floor	W30X148	43.5	30.7	0.65	1.18	500	68	10.5	6680	227	2.28	14.5	436	43.3	29.52
	#N/A	0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
BF-1 Roof B1	41.6	BF-1 Roof B1	BUILTUP4	32.25	24	0.375	1	321	48	12	3509	288	2.99	7.96	292	72.8	23
BF-1 4th B1	445.7	BF-1 4th B1	BUILTUP4	32.25	24	0.375	1	321	48	12	3509	288	2.99	7.96	292	72.8	23
BF-1 3rd B1	466.2	BF-1 3rd B1	BUILTUP4	32.25	24	0.375	1	321	48	12	3509	288	2.99	7.96	292	72.8	23
BF-1 2nd B1	345.2	BF-1 2nd B1	BUILTUP4	32.25	24	0.375	1	321	48	12	3509	288	2.99	7.96	292	72.8	23
	#N/A	0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
BF-1 4th B2	242.7	BF-1 4th B2	W24X146	43	24.7	0.65	1.09	418	93.2	12.9	4580	391	3.01	13.4	371	60.5	23.61
BF-1 2nd B2	83.8	BF-1 2nd B2	W24X146	43	24.7	0.65	1.09	418	93.2	12.9	4580	391	3.01	13.4	371	60.5	23.61
	#N/A	0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	#N/A	0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	#N/A	0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	#N/A	0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	#N/A	0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	#N/A	0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Beam No Links

	103.0	BF-3 ROOF	W24X55	16.3	23.6	0.395	0.505	135	13.4	7.01	1360	29.1	1.34	1.24	115	8.3	23.095
	3.0	BF-3 LVL3	W24X55	16.3	23.6	0.395	0.505	135	13.4	7.01	1360	29.1	1.34	1.24	115	8.3	23.095
	704.0	BF-3 LVL 1	W30X116	34.2	30	0.565	0.85	378	49.2	10.5	4930	164	2.19	6.43	329	31.3	29.15
				#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	226.0	BF-1 ROOF	W24X55	16.3	23.6	0.395	0.505	135	13.4	7.01	1360	29.1	1.34	1.24	115	8.3	23.095
	72.0	BF-1 LVL3	W24X55	16.3	23.6	0.395	0.505	135	13.4	7.01	1360	29.1	1.34	1.24	115	8.3	23.095
	1056.0	BF-1 LVL 1	W30X116	34.2	30	0.565	0.85	378	49.2	10.5	4930	164	2.19	6.43	329	31.3	29.15
				#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Beam Outside

														Stabili
			Steel Properties				Check Beam Slenderness		Unbraced Length					2
Location	Cw	Ry	Fy (ksi)	E (ksi)	Col Depth dc (in)	λw	λp	Web Check	Possible Lb (ft)	Lb (ft)	K	Pe1	Cm	α
TEST	*8570*	1.1	50	29000	14	**29.90**	**90.55**	OK	12.41666667	12.4	1.0	**14478.0**	1.0	1.0
BF-3 4th Floor	*16500*	1.1	50	29000	14	**39.20**	**90.55**	OK	12.16666667	10.0	1.0	**59628.9**	1.0	1.0
BF-3 2th Floor	*49400*	1.1	50	29000	14	**41.60**	**90.55**	OK	12.16666667	10.0	1.0	**132773.6**	1.0	1.0
	#N/A	1.1	50	29000	14	**#N/A**	**90.55**	#N/A	-0.583333333	10.0	1.0	**#N/A**	1.0	1.0
BF-1 Roof B1	*38088*	1.1	50	29000	14	**61.33**	**90.55**	OK	11.41666667	10.0	1.0	**69745.9**	1.0	1.0
BF-1 4th B1	*38088*	1.1	50	29000	14	**61.33**	**90.55**	OK	11.41666667	10.0	1.0	**69745.9**	1.0	1.0
BF-1 3rd B1	*38088*	1.1	50	29000	14	**61.33**	**90.55**	OK	11.41666667	10.0	1.0	**69745.9**	1.0	1.0
BF-1 2nd B1	*38088*	1.1	50	29000	14	**61.33**	**90.55**	OK	11.41666667	10.0	1.0	**69745.9**	1.0	1.0
	#N/A	1.1	50	29000	14	**#N/A**	**90.55**	#N/A	-0.583333333	10.0	1.0	**#N/A**	1.0	1.0
BF-1 4th B2	*54700*	1.1	50	29000	14	**33.20**	**90.55**	OK	12.16666667	10.0	1.0	**91033.4**	1.0	1.0
BF-1 2nd B2	*54700*	1.1	50	29000	14	**33.20**	**90.55**	OK	12.16666667	10.0	1.0	**91033.4**	1.0	1.0
	#N/A	1.1	50	29000	14	**#N/A**	**90.55**	#N/A	-0.583333333	10.0	1.0	**#N/A**	1.0	1.0
	#N/A	1.1	50	29000	14	**#N/A**	**90.55**	#N/A	-0.583333333	10.0	1.0	**#N/A**	1.0	1.0
	#N/A	1.1	50	29000	14	**#N/A**	**90.55**	#N/A	-0.583333333	10.0	1.0	**#N/A**	1.0	1.0
	#N/A	1.1	50	29000	14	**#N/A**	**90.55**	#N/A	-0.583333333	10.0	1.0	**#N/A**	1.0	1.0
	#N/A	1.1	50	29000	14	**#N/A**	**90.55**	#N/A	-0.583333333	10.0	1.0	**#N/A**	1.0	1.0
	#N/A	1.1	50	29000	14	**#N/A**	**90.55**	#N/A	-0.583333333	10.0	1.0	**#N/A**	1.0	1.0

Beam No Links

	3870	1.1	50	29000	14	**54.10**	**90.55**	OK	-0.583333333	10.0	1.0	**27031.7**	1.0	1.0
	3870	1.1	50	29000	14	**54.10**	**90.55**	OK	-0.583333333	10.0	1.0	**27031.7**	1.0	1.0
	34900	1.1	50	29000	14	**47.80**	**90.55**	OK	-0.583333333	10.0	1.0	**97990.1**	1.0	1.0
	#N/A	1.1	50	29000	14	**#N/A**	**90.55**	#N/A	-0.583333333	10.0	1.0	**#N/A**	1.0	1.0
	3870	1.1	50	29000	14	**54.10**	**90.55**	OK	-0.583333333	10.0	1.0	**27031.7**	1.0	1.0
	3870	1.1	50	29000	14	**54.10**	**90.55**	OK	-0.583333333	10.0	1.0	**27031.7**	1.0	1.0
	34900	1.1	50	29000	14	**47.80**	**90.55**	OK	-0.583333333	10.0	1.0	**97990.1**	1.0	1.0
	#N/A	1.1	50	29000	14	**#N/A**	**90.55**	#N/A	-0.583333333	10.0	1.0	**#N/A**	1.0	1.0

Beam Outside

	ty and Analysis Design															Combined Loading Strtength Check
	nd Order Analysis							Compressive Strength								Members
Location	B1	B2	Pr (Kips)	Mr (in-K)	K	Lb	r	KL/r	Fe	Fcr	ΦPn	Lb	rts	Lp	Lr	Mp
TEST	1.0	1.0	311.4	4512.3	1	148.8	2.46	60.4878	78.22798	38.26377	788.6162	148.8	2.827779	104.2706	756.4784	7600
BF-3 4th Floor	1.0	1.0	18.9	13762.1	1	120	1.99	60.30151	78.71208	38.32677	1045.171	120	2.391524	84.34896	796.5225	14000
BF-3 2th Floor	1.0	1.0	7.8	20137.7	1	120	2.28	52.63158	103.3249	40.83265	1598.598	120	2.771373	96.64102	948.9793	25000
	#N/A	1.0	#N/A	#N/A	1	120	#N/A	#N/A	#N/A	#N/A	#N/A	120	#N/A	#N/A	#N/A	#N/A
BF-1 Roof B1	1.0	1.0	41.6	11882.4	1	120	2.99	40.13378	177.696	44.44496	1290.015	120	3.367858	126.7354	734.2599	16050
BF-1 4th B1	1.0	1.0	445.7	7845.8	1	120	2.99	40.13378	177.696	44.44496	1290.015	120	3.367858	126.7354	734.2599	16050
BF-1 3rd B1	1.0	1.0	466.2	8949.4	1	120	2.99	40.13378	177.696	44.44496	1290.015	120	3.367858	126.7354	734.2599	16050
BF-1 2nd B1	1.0	1.0	345.2	7214.5	1	120	2.99	40.13378	177.696	44.44496	1290.015	120	3.367858	126.7354	734.2599	16050
	#N/A	1.0	#N/A	#N/A	1	120	#N/A	#N/A	#N/A	#N/A	#N/A	120	#N/A	#N/A	#N/A	#N/A
BF-1 4th B2	1.0	1.0	242.7	17396.3	1	120	3.01	39.86711	180.0811	44.51434	1722.705	120	3.530645	127.5831	1000.952	20900
BF-1 2nd B2	1.0	1.0	83.8	16914.7	1	120	3.01	39.86711	180.0811	44.51434	1722.705	120	3.530645	127.5831	1000.952	20900
	#N/A	1.0	#N/A	#N/A	1	120	#N/A	#N/A	#N/A	#N/A	#N/A	120	#N/A	#N/A	#N/A	#N/A
	#N/A	1.0	#N/A	#N/A	1	120	#N/A	#N/A	#N/A	#N/A	#N/A	120	#N/A	#N/A	#N/A	#N/A
	#N/A	1.0	#N/A	#N/A	1	120	#N/A	#N/A	#N/A	#N/A	#N/A	120	#N/A	#N/A	#N/A	#N/A
	#N/A	1.0	#N/A	#N/A	1	120	#N/A	#N/A	#N/A	#N/A	#N/A	120	#N/A	#N/A	#N/A	#N/A
	#N/A	1.0	#N/A	#N/A	1	120	#N/A	#N/A	#N/A	#N/A	#N/A	120	#N/A	#N/A	#N/A	#N/A
	#N/A	1.0	#N/A	#N/A	1	120	#N/A	#N/A	#N/A	#N/A	#N/A	120	#N/A	#N/A	#N/A	#N/A

Beam No Links

	1.0	1.0	103.0	3001.4	1	120	1.34	89.55224	35.68986	27.81712	408.0771	120	1.708253	56.79779	619.7816	6750
	1.0	1.0	3.0	2360.3	1	120	1.34	89.55224	35.68986	27.81712	408.0771	120	1.708253	56.79779	619.7816	6750
	1.0	1.0	704.0	2410.3	1	120	2.19	54.79452	95.32866	40.1448	1235.657	120	2.696618	92.82624	879.7964	18900
	#N/A	1.0	0.0	#N/A	1	120	#N/A	#N/A	#N/A	#N/A	#N/A	120	#N/A	#N/A	#N/A	#N/A
	1.0	1.0	226.0	1025.6	1	120	1.34	89.55224	35.68986	27.81712	408.0771	120	1.708253	56.79779	619.7816	6750
	1.0	1.0	72.0	534.4	1	120	1.34	89.55224	35.68986	27.81712	408.0771	120	1.708253	56.79779	619.7816	6750
	1.0	1.0	1056.0	4172.0	1	120	2.19	54.79452	95.32866	40.1448	1235.657	120	2.696618	92.82624	879.7964	18900
	#N/A	1.0	0.0	#N/A	1	120	#N/A	#N/A	#N/A	#N/A	#N/A	120	#N/A	#N/A	#N/A	#N/A

Beam Outside

:

	in Flexure						Check		
Location	Cb	Mn w/ LTB	Fcr	Mn w/ LTB	ΦMn	Pr/Pc	Mr/Mc	H1-1	
TEST	1	7406.099	121.8441	16570.8	6665.489	0.358939	0.615417	0.91	OK
BF-3 4th Floor	1	13728.43	126.6372	31026.12	12355.58	0.016467	1.01258	1.02	Not OK
BF-3 2th Floor	1	24733.07	164.7563	71833.74	22259.76	0.004409	0.822426	0.82	OK
	1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
BF-1 Roof B1	1	16114.63	238.3093	69586.31	14445	0.029335	0.747815	0.76	OK
BF-1 4th B1	1	16114.63	238.3093	69586.31	14445	0.314113	0.49377	0.75	OK
BF-1 3rd B1	1	16114.63	238.3093	69586.31	14445	0.328552	0.563226	0.83	OK
BF-1 2nd B1	1	16114.63	238.3093	69586.31	14445	0.243292	0.454042	0.65	OK
	1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
BF-1 4th B2	1	20968.72	264.2923	98052.44	18810	0.128073	0.840765	0.90	OK
BF-1 2nd B2	1	20968.72	264.2923	98052.44	18810	0.044213	0.817489	0.84	OK
	1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
	1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Beam No Links

1	6444.084	62.99789	7244.757	5799.675	0.229458	0.470471	0.647654	OK
1	6444.084	62.99789	7244.757	5799.675	0.006683	0.369968	0.373309	OK
1	18645	151.8351	49953.74	16780.5	0.517943	0.13058	0.634014	OK
1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
1	6444.084	62.99789	7244.757	5799.675	0.50347	0.160757	0.646365	OK
1	6444.084	62.99789	7244.757	5799.675	0.160398	0.08377	0.163969	OK
1	18645	151.8351	49953.74	16780.5	0.776915	0.226018	0.977819	OK
1	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Braces Design Spread Sheet

Brace Design

	Connected Link						Link Loading from Elastic Analysis						Brace Loadings from Elastic
Location	Shape	Ry	Vn	height (ft)	Bay Length (ft)	1.25RyVn	Veq (kips)	Overstrength Factor	Vdead (kips)	Mdead (in-ft)	Pdead (kips)	Veq (kips)	Meq (in-ft)
Test	W16X77	1.1	204.477	12.5	30	281.155875	84	3.34709375	0.2	38.4	11.8	3.02	654
BF-3													
BF-3 4th Floor	W24X103	1.1	371.91	15	30	511.37625	256	1.997563477	0	0	3	0	0
BF-3 4th Floor	W24X103	1.1	371.91	15	30	511.37625	256	1.997563477	0	0	13	0	0
BF-3 2th Floor	W30X148	1.1	552.63	15	30	759.86625	457	1.662727024	0	0	3	0	0
BF-3 2th Floor	W30X148	1.1	552.63	18	30	759.86625	457	1.662727024	0	0	37	0	0
BF-1 Bay1													
BF-1 Roof B1	BUILTUP4	1.1	247.5	15	30	340.3125	59	5.768008475	0	0	49	0	0
BF-1 4th B1	BUILTUP4	1.1	247.5	15	30	340.3125	106	3.210495283	0	0	45	0	0
BF-1 3rd B1	BUILTUP4	1.1	247.5	15	30	340.3125	135	2.520833333	0	0	40	0	0
BF-1 2nd B1	BUILTUP4	1.1	247.5	18	30	340.3125	197	1.727474619	0	0	46	0	0
BF-1 Bay2													
BF-1 4th B2	W24X146	1.1	439.14	15	30	603.8175	204	2.959889706	0	0	9	0	0
BF-1 4th B2	W24X146	1.1	439.14	15	30	603.8175	204	2.959889706	0	0	27	0	0
BF-1 2nd B2	W24X146	1.1	439.14	15	30	603.8175	305	1.979729508	0	0	23	0	0
BF-1 2nd B2	W24X146	1.1	439.14	18	30	603.8175	305	1.979729508	0	0	39	0	0
	#N/A	#N/A	#N/A			#N/A		#N/A					
	#N/A	#N/A	#N/A			#N/A		#N/A					
	#N/A	#N/A	#N/A			#N/A		#N/A					
	#N/A	#N/A	#N/A			#N/A		#N/A					
	#N/A	#N/A	#N/A			#N/A		#N/A					
	#N/A	#N/A	#N/A			#N/A		#N/A					

Brace Design

	: Analysis				Load Combination Factors			Factored Brace Loadings			Brace	Brace		
Location	Peq (kips)	Vlive (kips)	Mlive (in-ft)	Plive (kips)	D	EQ	L	Vu (kips)	Mu (in-K)	Pu (kips)	Location	Shape	Shape	Ag
Test	136	0.12	26.4	8.3	1.4	1	0.5	10.44822313	2255.959313	475.87475	TEST	W10X112	W	32.9
BF-3														
BF-3 4th Floor	164	0	0	4	1.4	1	1	0	0	335.8004102	BF-3 4th Floor	HSS10X10X5/8	HSS	21
BF-3 4th Floor	263	0	0	10	1.4	1	0.5	0	0	548.5591943	BF-3 3rd Floor	HSS10X10X5/8	HSS	21
BF-3 2th Floor	314	0	0	10	1.4	1	0.5	0	0	531.2962856	BF-3 2th Floor	HSS10X10X5/8	HSS	21
BF-3 2th Floor	416	0	0	14	1.4	1	0.5	0	0	750.494442	BF-3 1th Floor	HSS14X14X5/8	HSS	30.3
BF-1 Bay1														
BF-1 Roof B1	125	0	0	50	1.4	1	1	0	0	839.6010593	BF-1 Roof B1	HSS14X14X5/8	HSS	30.3
BF-1 4th B1	215	0	0	67	1.4	1	0.5	0	0	786.7564858	BF-1 4th B1	HSS14X14X5/8	HSS	30.3
BF-1 3rd B1	281	0	0	33	1.4	1	0.5	0	0	780.8541667	BF-1 3rd B1	HSS14X14X5/8	HSS	30.3
BF-1 2nd B1	350	0	0	39	1.4	1	0.5	0	0	688.5161168	BF-1 2nd B1	HSS14X14X5/8	HSS	30.3
BF-1 Bay2														
BF-1 4th B2	138	0	0	8	1.4	1	1	0	0	429.0647794	BF-1 Roof B2	HSS10X10X5/8	HSS	21
BF-1 4th B2	200	0	0	34	1.4	1	0.5	0	0	646.7779412	BF-1 4th B2	HSS10X10X5/8	HSS	21
BF-1 2nd B2	233	0	0	27	1.4	1	0.5	0	0	506.9769754	BF-1 3rd B2	HSS10X10X5/8	HSS	21
BF-1 2nd B2	263	0	0	39	1.4	1	0.5	0	0	594.7688607	BF-1 2nd B2	HSS10X10X5/8	HSS	21
								#N/A	#N/A	#N/A				#N/A
								#N/A	#N/A	#N/A				#N/A
								#N/A	#N/A	#N/A				#N/A
								#N/A	#N/A	#N/A				#N/A
								#N/A	#N/A	#N/A				#N/A
								#N/A	#N/A	#N/A				#N/A

Brace Design

	Dimentions															Steel Properties			
Location	d	tw	tdesign	h	b	tf	Zx (in3)	bf	Ix	Iy	ry	Jc	Sx	ho	Cw	Ry	Fy (ksi)	E (ksi)	λf	λp
Test	11.4	0.755	0	0	0	1.25	147	10.4	716	236	2.68	15.1	126	10.15	6030	1.1	50	29000	4.16	9.151612
BF-3																				
BF-3 4th Floor	0	0	0.581	10	10	0	73.2	0	304	304	3.8	498	60.8	0	0	1.4	46	29000	17.211704	28.12148
BF-3 4th Floor	0	0	0.581	10	10	0	73.2	0	304	304	3.8	498	60.8	0	0	1.4	46	29000	17.211704	28.12148
BF-3 2th Floor	0	0	0.581	10	10	0	73.2	0	304	304	3.8	498	60.8	0	0	1.4	46	29000	17.211704	28.12148
BF-3 2th Floor	0	0	0.581	14	14	0	151	0	897	897	5.44	1430	128	0	0	1.4	46	29000	24.096386	28.12148
BF-1 Bay1																				
BF-1 Roof B1	0	0	0.581	14	14	0	151	0	897	897	5.44	1430	128	0	0	1.4	46	29000	24.096386	28.12148
BF-1 4th B1	0	0	0.581	14	14	0	151	0	897	897	5.44	1430	128	0	0	1.4	46	29000	24.096386	28.12148
BF-1 3rd B1	0	0	0.581	14	14	0	151	0	897	897	5.44	1430	128	0	0	1.4	46	29000	24.096386	28.12148
BF-1 2nd B1	0	0	0.581	14	14	0	151	0	897	897	5.44	1430	128	0	0	1.4	46	29000	24.096386	28.12148
BF-1 Bay2																				
BF-1 4th B2	0	0	0.581	10	10	0	73.2	0	304	304	3.8	498	60.8	0	0	1.4	46	29000	17.211704	28.12148
BF-1 4th B2	0	0	0.581	10	10	0	73.2	0	304	304	3.8	498	60.8	0	0	1.4	46	29000	17.211704	28.12148
BF-1 2nd B2	0	0	0.581	10	10	0	73.2	0	304	304	3.8	498	60.8	0	0	1.4	46	29000	17.211704	28.12148
BF-1 2nd B2	0	0	0.581	10	10	0	73.2	0	304	304	3.8	498	60.8	0	0	1.4	46	29000	17.211704	28.12148
	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A				Incorrect Shap	correct Sha
	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A				Incorrect Shap	correct Sha
	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A				Incorrect Shap	correct Sha
	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A				Incorrect Shap	correct Sha
	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A				Incorrect Shap	correct Sha
	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A				Incorrect Shap	correct Sha

Brace Design

Location	Check Brace Slenderness				Unbraced Length	2nd Order Effects									
	Flange?	λw	λp	Web?	Lb (ft)	K	Pe1	Cm	α	B1	B2	Pr (Kips)	Mr (in-K)	K	L
Test	OK	10.4	90.55279	OK	18	1.0	4392.4	0.6	1.0	1.0	1.0	475.9	2256.0	1	216
BF-3															
BF-3 4th Floor	OK	17.2117	60.76247	OK	19.69	1.0	1558.5	0.6	1.0	1.0	1.0	335.8	0.0	1	236.28
BF-3 4th Floor	OK	17.2117	60.76247	OK	19.69	1.0	1558.5	0.6	1.0	1.0	1.0	548.6	0.0	1	236.28
BF-3 2th Floor	OK	17.2117	60.76247	OK	19.69	1.0	1558.5	0.6	1.0	1.0	1.0	531.3	0.0	1	236.28
BF-3 2th Floor	OK	24.09639	60.76247	OK	22.06	1.0	3663.7	0.6	1.0	1.0	1.0	750.5	0.0	1	264.72
BF-1 Bay1															
BF-1 Roof B1	OK	24.09639	60.76247	OK	28.3	1.0	2226.2	0.6	1.0	1.0	1.0	839.6	0.0	1	339.6
BF-1 4th B1	OK	24.09639	60.76247	OK	28.3	1.0	2226.2	0.6	1.0	1.0	1.0	786.8	0.0	1	339.6
BF-1 3rd B1	OK	24.09639	60.76247	OK	28.3	1.0	2226.2	0.6	1.0	1.0	1.0	780.9	0.0	1	339.6
BF-1 2nd B1	OK	24.09639	60.76247	OK	30	1.0	1981.0	0.6	1.0	1.0	1.0	688.5	0.0	1	360
BF-1 Bay2															
BF-1 4th B2	OK	17.2117	60.76247	OK	19.69	1.0	1558.5	0.6	1.0	1.0	1.0	429.1	0.0	1	236.28
BF-1 4th B2	OK	17.2117	60.76247	OK	19.69	1.0	1558.5	0.6	1.0	1.0	1.0	646.8	0.0	1	236.28
BF-1 2nd B2	OK	17.2117	60.76247	OK	19.69	1.0	1558.5	0.6	1.0	1.0	1.0	507.0	0.0	1	236.28
BF-1 2nd B2	OK	17.2117	60.76247	OK	22.06	1.0	1241.6	0.6	1.0	1.2	1.0	594.8	0.0	1	264.72
	Not OK	correct Sha	correct Sha	Not OK		1.0	#N/A	0.6	1.0	#N/A	1.0	#N/A	#N/A	1	0
	Not OK	correct Sha	correct Sha	Not OK		1.0	#N/A	0.6	1.0	#N/A	1.0	#N/A	#N/A	1	0
	Not OK	correct Sha	correct Sha	Not OK		1.0	#N/A	0.6	1.0	#N/A	1.0	#N/A	#N/A	1	0
	Not OK	correct Sha	correct Sha	Not OK		1.0	#N/A	0.6	1.0	#N/A	1.0	#N/A	#N/A	1	0
	Not OK	correct Sha	correct Sha	Not OK		1.0	#N/A	0.6	1.0	#N/A	1.0	#N/A	#N/A	1	0
	Not OK	correct Sha	correct Sha	Not OK		1.0	#N/A	0.6	1.0	#N/A	1.0	#N/A	#N/A	1	0

Brace Design

	Compressive Strength				
Location	r	KL/r	Fe	Fcr	ΦPn
Test	2.68	80.59701493	44.06155592	31.09546784	920.7368026
BF-3					
BF-3 4th Floor	3.8	62.17894737	74.03055617	35.46579958	670.3036122
BF-3 4th Floor	3.8	62.17894737	74.03055617	35.46579958	670.3036122
BF-3 2th Floor	3.8	62.17894737	74.03055617	35.46579958	670.3036122
BF-3 2th Floor	5.44	48.66176471	120.8709775	39.22652724	1069.707398
BF-1 Bay1					
BF-1 Roof B1	5.44	62.42647059	73.44465237	35.39229406	965.147859
BF-1 4th B1	5.44	62.42647059	73.44465237	35.39229406	965.147859
BF-1 3rd B1	5.44	62.42647059	73.44465237	35.39229406	965.147859
BF-1 2nd B1	5.44	66.17647059	65.35676404	34.26256756	934.3402173
BF-1 Bay2					
BF-1 4th B2	3.8	62.17894737	74.03055617	35.46579958	670.3036122
BF-1 4th B2	3.8	62.17894737	74.03055617	35.46579958	670.3036122
BF-1 2nd B2	3.8	62.17894737	74.03055617	35.46579958	670.3036122
BF-1 2nd B2	3.8	69.66315789	58.97818837	33.18816454	627.2563098
	#N/A	#N/A	#N/A	#N/A	#N/A
	#N/A	#N/A	#N/A	#N/A	#N/A
	#N/A	#N/A	#N/A	#N/A	#N/A
	#N/A	#N/A	#N/A	#N/A	#N/A
	#N/A	#N/A	#N/A	#N/A	#N/A
	#N/A	#N/A	#N/A	#N/A	#N/A

Combined Loading Strtength Check

	Members in Flexure							
Location	Lb	rts	Lp	Lr	Mp	Cb	Mn w/ LTB	Fcr
Test	216	3.076961457	113.5955866	770.563567	7350	1	6891.729606	136.6862103
BF-3								
BF-3 4th Floor	236.28	0	167.925383	#DIV/0!	3367.2	1	#DIV/0!	#DIV/0!
BF-3 4th Floor	236.28	0	167.925383	#DIV/0!	3367.2	1	#DIV/0!	#DIV/0!
BF-3 2th Floor	236.28	0	167.925383	#DIV/0!	3367.2	1	#DIV/0!	#DIV/0!
BF-3 2th Floor	264.72	0	240.3984431	#DIV/0!	6946	1	#DIV/0!	#DIV/0!
BF-1 Bay1								
BF-1 Roof B1	339.6	0	240.3984431	#DIV/0!	6946	1	#DIV/0!	#DIV/0!
BF-1 4th B1	339.6	0	240.3984431	#DIV/0!	6946	1	#DIV/0!	#DIV/0!
BF-1 3rd B1	339.6	0	240.3984431	#DIV/0!	6946	1	#DIV/0!	#DIV/0!
BF-1 2nd B1	360	0	240.3984431	#DIV/0!	6946	1	#DIV/0!	#DIV/0!
BF-1 Bay2								
BF-1 4th B2	236.28	0	167.925383	#DIV/0!	3367.2	1	#DIV/0!	#DIV/0!
BF-1 4th B2	236.28	0	167.925383	#DIV/0!	3367.2	1	#DIV/0!	#DIV/0!
BF-1 2nd B2	236.28	0	167.925383	#DIV/0!	3367.2	1	#DIV/0!	#DIV/0!
BF-1 2nd B2	264.72	0	167.925383	#DIV/0!	3367.2	1	#DIV/0!	#DIV/0!
	0	#N/A	#N/A	#N/A	#N/A	1	#N/A	#N/A
	0	#N/A	#N/A	#N/A	#N/A	1	#N/A	#N/A
	0	#N/A	#N/A	#N/A	#N/A	1	#N/A	#N/A
	0	#N/A	#N/A	#N/A	#N/A	1	#N/A	#N/A
	0	#N/A	#N/A	#N/A	#N/A	1	#N/A	#N/A
	0	#N/A	#N/A	#N/A	#N/A	1	#N/A	#N/A

Brace Design

Location	Mn w/ LTB	ΦMn	Pr/Pc	Mr/Mc	H1-1	Check	2.24(E/Fy)^.5	h/tw	Shear Strength Check	Cv	ΦVn (kips)	Shear Check
Test	17222.46249	6202.556645	0.516841	0.3637144	0.840143	OK	53.94634371	10.4	OK	1	258.21	OK
BF-3												
BF-3 4th Floor	#DIV/0!	3030.48	0.500968	0	0.500968	OK	56.24295125	17.2117	OK	1	320.712	OK
BF-3 4th Floor	#DIV/0!	3030.48	0.818374	0	0.818374	OK	56.24295125	17.2117	OK	1	320.712	OK
BF-3 2th Floor	#DIV/0!	3030.48	0.79262	0	0.79262	OK	56.24295125	17.2117	OK	1	320.712	OK
BF-3 2th Floor	#DIV/0!	6251.4	0.701589	0	0.701589	OK	56.24295125	24.09639	OK	1	448.9968	OK
BF-1 Bay1												
BF-1 Roof B1	#DIV/0!	6251.4	0.86992	0	0.86992	OK	56.24295125	24.09639	OK	1	448.9968	OK
BF-1 4th B1	#DIV/0!	6251.4	0.815167	0	0.815167	OK	56.24295125	24.09639	OK	1	448.9968	OK
BF-1 3rd B1	#DIV/0!	6251.4	0.809051	0	0.809051	OK	56.24295125	24.09639	OK	1	448.9968	OK
BF-1 2nd B1	#DIV/0!	6251.4	0.736901	0	0.736901	OK	56.24295125	24.09639	OK	1	448.9968	OK
BF-1 Bay2												
BF-1 4th B2	#DIV/0!	3030.48	0.640105	0	0.640105	OK	56.24295125	17.2117	OK	1	320.712	OK
BF-1 4th B2	#DIV/0!	3030.48	0.964903	0	0.964903	OK	56.24295125	17.2117	OK	1	320.712	OK
BF-1 2nd B2	#DIV/0!	3030.48	0.756339	0	0.756339	OK	56.24295125	17.2117	OK	1	320.712	OK
BF-1 2nd B2	#DIV/0!	3030.48	0.948207	0	0.948207	OK	56.24295125	17.2117	OK	1	320.712	OK
	#N/A	Incorrect Shape	#N/A	#N/A	#N/A	#N/A	#DIV/0!	correct Sha	#DIV/0!	correct Shap	correct Shap	#N/A
	#N/A	Incorrect Shape	#N/A	#N/A	#N/A	#N/A	#DIV/0!	correct Sha	#DIV/0!	correct Shap	correct Shap	#N/A
	#N/A	Incorrect Shape	#N/A	#N/A	#N/A	#N/A	#DIV/0!	correct Sha	#DIV/0!	correct Shap	correct Shap	#N/A
	#N/A	Incorrect Shape	#N/A	#N/A	#N/A	#N/A	#DIV/0!	correct Sha	#DIV/0!	correct Shap	correct Shap	#N/A
	#N/A	Incorrect Shape	#N/A	#N/A	#N/A	#N/A	#DIV/0!	correct Sha	#DIV/0!	correct Shap	correct Shap	#N/A
	#N/A	Incorrect Shape	#N/A	#N/A	#N/A	#N/A	#DIV/0!	correct Sha	#DIV/0!	correct Shap	correct Shap	#N/A

Columns Design Spread Sheet

Column Design

Column Location	Column Shape	Ag	d	tw	tf	Zx (in3)	Zy	bf	Ix	Iy	ry	Jc	Sx	Sy	ho	Cw	Ry	Fy (ksi)	E (ksi)	Pdead (kips)
								Dimentions									Steel Properties			
TEST	W14X99	29.1	14.2	0.485	0.78	173	83.6	14.6	1110	402	3.71	5.37	157	55.2	13.42	18000	1.1	50	29000	151
BF-3																				
BF-3 LOWER	W14X233	68.5	16	1.07	1.72	436	221	15.9	3010	1150	4.1	59.5	375	145	14.28	59000	1.1	50	29000	81
BF-3 UPPER	W14X176	51.8	15.2	0.83	1.31	320	163	15.7	2140	838	4.02	26.5	281	107	13.89	40500	1.1	50	29000	56
BF-1 GRID 2																				
BF-1 LOWER	W14X233	68.5	16	1.07	1.72	436	221	15.9	3010	1150	4.1	59.5	375	145	14.28	59000	1.1	50	29000	192
BF-1 UPPER	W14X176	51.8	15.2	0.83	1.31	320	163	15.7	2140	838	4.02	26.5	281	107	13.89	40500	1.1	50	29000	141
BF-1 GRID 3																				
BF-1 LOWER	W14X233	68.5	16	1.07	1.72	436	221	15.9	3010	1150	4.1	59.5	375	145	14.28	59000	1.1	50	29000	161
BF-1 UPPER	W14X176	51.8	15.2	0.83	1.31	320	163	15.7	2140	838	4.02	26.5	281	107	13.89	40500	1.1	50	29000	135

Column Design

Column Location	Mdead X (in-K)	Mdead Y (in-K)	Peq (kips)	Meq X (in-K)	Meq Y (in-K)	Plive (kips)	Mlive X (in-K)	Mlive Y (in-K)	D	EQ	L	Pu (kips)	Mu X (in-K)
	Column Loadings from Elastic Analysis with expected yeild strength forces								Load Combination Factors			Factored Beam Lo	
TEST	180	120	172	0	0	46	108	72	1.4	1	0.5	**406**	**306**
BF-3													
BF-3 LOWER	119	0	517	2732	0	44	155	0	1.4	2	1	**1295**	**6332**
BF-3 UPPER	340	0	517	450	0	40	305	0	1.4	2	1	**1256**	**1771**
BF-1 GRID 2			0	0									
BF-1 LOWER	466	0	287	2770	0	197	404	0	1.4	1	1	**782**	**4103**
BF-1 UPPER	505	0	152	817	0	159	533	0	1.4	1	1	**524**	**2139**
BF-1 GRID 3			0	0									
BF-1 LOWER	179	0	208	3888	0	163	65	0	1.4	1	1	**617**	**4592**
BF-1 UPPER	69	0	118	2727	0	146	160	0	1.4	1	1	**465**	**3256**

Column Design

Column Location	adings Mu Y (in-K)	K	Lb (ft)	r	KL/r	4.71(E/Fy)^.5	Fe	Fcr	ΦPn	Pu/ΦPn	Check	ΣVn	Σ1.1RyVn
						Check Axial Strength						Link Expected Yield Strength Forces	
TEST	204	1	14	3.71	45.28	113.4318209	139.6	43.04	1127.2	0.360547	OK	523	632.83
BF-3													
BF-3 LOWER	0	1	18	4.1	52.68	113.4318209	103.1	40.82	2516.3	0.514557	Consider Amplified Seismic Loads	925	1119.25
BF-3 UPPER	0	1	15	4.02	44.78	113.4318209	142.8	43.18	2013.2	0.623796	Consider Amplified Seismic Loads	648.5	784.685
BF-1 GRID 2													
BF-1 LOWER	0	1	18	4.1	52.68	113.4318209	103.1	40.82	2516.3	0.31057	OK	992	1200.32
BF-1 UPPER	0	1	15	4.02	44.78	113.4318209	142.8	43.18	2013.2	0.260089	OK	744	900.24
BF-1 GRID 3													
BF-1 LOWER	0	1	18	4.1	52.68	113.4318209	103.1	40.82	2516.3	0.245277	OK	992	1200.32
BF-1 UPPER	0	1	15	4.02	44.78	113.4318209	142.8	43.18	2013.2	0.230881	OK	744	900.24

*Insert Expected Yeild Strength Force if less than earthquake loads

Column Design

Column Location	Column Loadings from Elastic Analysis with expected yeild strength forces									Load Combination Factors		
	PVdead (kips	Mdead X (in-K)	Mdead Y (in-K)	Peq (kips)	Meq X (in-K)	Meq Y (in-K)	Plive (kips)	Mlive X (in-K)	Mlive Y (in-K)	D	EQ	L
TEST	151	180	120	633	0	0	46	108	72	1.4	1	0.5
BF-3												
BF-3 LOWER	81	119	0	1119	2732	0	44	155	0	1.4	1	1
BF-3 UPPER	56	340	0	785	450	0	40	305	0	1.4	1	1
BF-1 GRID 2				0	0							
BF-1 LOWER	192	466	0	1200	2770	0	197	404	0	1.4	1	1
BF-1 UPPER	141	505	0	900	817	0	159	533	0	1.4	1	1
BF-1 GRID 3				0	0							
BF-1 LOWER	161	179	0	1200	3888	0	163	65	0	1.4	1	1
BF-1 UPPER	135	69	0	900	2727	0	146	160	0	1.4	1	1

Column Design

Column Location	Factored Beam Loadings Pu (kips)	Mu X (in-K)	Mu Y (in-K)	Max. Beam Loadings Pu (kips)	Mu X (in-K)	Mu Y (in-K)	2nd Order Effects K	Pe1x	Pe1y	Cm	α	B1x	B1y	B2	Pr (Kips)	Mrx (in-K)	Mry (in-K)
TEST	867.4	306	204	867	306	204	1	11256.47	4076.667	1	1	1.083492	1.27028	1	867.4	331.5484311	259.1370773
BF-3																	
BF-3 LOWER	1276.4	3053.6	0	1295	6332	0	1	18465.32	7054.855	1	1	1.075408	1.22479	1	1294.8	6809.485578	0
BF-3 UPPER	903.4	1231	0	1256	1771	0	1	18904.56	7402.813	1	1	1.071155	1.204294	1	1255.8	1897.015775	0
BF-1 GRID 2																	
BF-1 LOWER	1665.8	3826.4	0	1666	4103	0	1	18465.32	7054.855	1	1	1.099158	1.309108	1	1665.8	4510.28338	0
BF-1 UPPER	1256.4	2057	0	1256	2139	0	1	18904.56	7402.813	1	1	1.071192	1.204412	1	1256.4	2290.95741	0
BF-1 GRID 3																	
BF-1 LOWER	1588.4	4203.6	0	1588	4592	0	1	18465.32	7054.855	1	1	1.094117	1.290572	1	1588.4	5024.621639	0
BF-1 UPPER	1235	2983.6	0	1235	3256	0	1	18904.56	7402.813	1	1	1.069894	1.200233	1	1235	3483.896565	0

*Conservative Traverse Loading

Column Design

Combined Loading Strtength Check

Column Location	K	L	r	KL/r	Fe	Fcr	ΦPn
	Compressive Strength						
TEST	1	168	3.71	45.28302	139.5812	43.03838	1127.175
BF-3							
BF-3 LOWER	1	216	4.1	52.68293	103.1236	40.81651	2516.338
BF-3 UPPER	1	180	4.02	44.77612	142.7594	43.18228	2013.158
BF-1 GRID 2							
BF-1 LOWER	1	216	4.1	52.68293	103.1236	40.81651	2516.338
BF-1 UPPER	1	180	4.02	44.77612	142.7594	43.18228	2013.158
BF-1 GRID 3							
BF-1 LOWER	1	216	4.1	52.68293	103.1236	40.81651	2516.338
BF-1 UPPER	1	180	4.02	44.77612	142.7594	43.18228	2013.158

Column Location	Lb	rts	Lp	Lr	Mp	Cb	Mn w/ LTB	Fcr	Mn w/ LTB	ΦMn
	Members in Flexure X Direction									
TEST	168	4.13928	157.2536	542.9695	8650	1	8562.098737	200.1902435	31429.8682	7705.889
BF-3										
BF-3 LOWER	216	4.68675	173.7843	1139.819	21800	1	21420.90257	227.1209794	85170.3673	19278.81
BF-3 UPPER	180	4.553251	170.3934	878.7872	16000	1	15916.39564	247.5936523	69573.8163	14324.76
BF-1 GRID 2										
BF-1 LOWER	216	4.68675	173.7843	1139.819	21800	1	21420.90257	227.1209794	85170.3673	19278.81
BF-1 UPPER	180	4.553251	170.3934	878.7872	16000	1	15916.39564	247.5936523	69573.8163	14324.76
BF-1 GRID 3										
BF-1 LOWER	216	4.68675	173.7843	1139.819	21800	1	21420.90257	227.1209794	85170.3673	19278.81
BF-1 UPPER	180	4.553251	170.3934	878.7872	16000	1	15916.39564	247.5936523	69573.8163	14324.76

Column Design

Column Location	Members in Flexure Y Direction							Combined Check				
	Mp	λ	λp	λr	Mn NonCompact	Mn Slender	ΦMn	Pr/Pc	Mr/Mc	Mr/Mc	H1-1	
TEST	4180	9.358974	9.151612	24.08319	4148.78087	12610.42291	4609.75652	0.769534353	0.043025	0.056215	0.857748	OK
BF-3												
BF-3 LOWER	11050	4.622093	9.151612	24.08319	12862.52621	135811.8695	12277.7778	0.514557277	0.353211	0	0.828522	OK
BF-3 UPPER	8150	5.992366	9.151612	24.08319	9082.016493	59625.78972	9055.55556	0.62379614	0.132429	0	0.741511	OK
BF-1 GRID 2												
BF-1 LOWER	11050	4.622093	9.151612	24.08319	12862.52621	135811.8695	12277.7778	0.661993754	0.23395	0	0.86995	OK
BF-1 UPPER	8150	5.992366	9.151612	24.08319	9082.016493	59625.78972	9055.55556	0.62409418	0.15993	0	0.766254	OK
BF-1 GRID 3												
BF-1 LOWER	11050	4.622093	9.151612	24.08319	12862.52621	135811.8695	12277.7778	0.631234769	0.260629	0	0.862905	OK
BF-1 UPPER	8150	5.992366	9.151612	24.08319	9082.016493	59625.78972	9055.55556	0.613464113	0.243208	0	0.829649	OK

Typical Connections

Link to Column Connection

connection #2

Builtup x 110.

$d = 24$

$b_f = 12$

$t_f = 1$

$t_w = 3/8$

$A_g = 32.25$

$R_y = 1.1$

$S_x = 292$

$Z_x = 321$

W14 x 233.

$d = 16$

$b_f = 15.9$

$t_f = 1.72$

$t_w = 1.07$

$A_g = 68.5$

$R_y = 1.1$

$S_x = 375$

$Z_x = 436$

$w_{Dead} = 1.74$

$w_{live} = 1.6$

W24 x 146.

$d = 24.7$

$b_f = 12.9$

$t_f = 1.09$

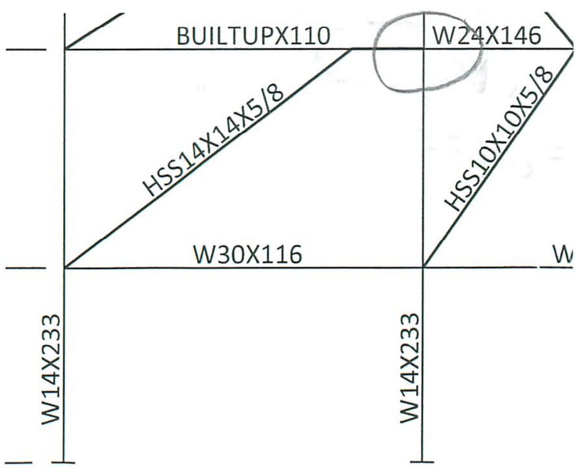
$t_w = .65$

$A_g = 43$

$R_y = 1.1$

$S_x = 371$

$Z_x = 418$



$V_u = 219^{k}$

$M_u = 12056.$ k-in

$P_u = 50.^{k}$

$V_p = 248^{kips}.$

Design option 1

AISC 358-05 Chapter 6. prequalified.

bolted unstiffened & stiffened Extended End plate moment connections.

must be an Eight bolt stiffened connection according to prequalification limits.

Design Procedure

(1) Determine moment @ face of column.

Etabs:

$$M_{u@column} = 12056 \text{ k-in.}$$

$$V_u @ \text{ column} = 219.$$

$$M_c = M_{u\,col.} \times \frac{C_{pr} R_y V_p}{V_{u\,Etabs.}}$$

$C_{pr} = 1.15$
$R_y = 1.1$

$$M_{pe} = 12056 \times \frac{1.15 \times 1.1 \times 248}{219} = 17,270 \text{ k-in.}$$

$$V_u = 219.$$

$$S_h = d/2 = 24/2 = 12 \text{ ⇐ controls.}$$

$$3 \times 12 = 36$$

$$M_f = 17,270 + 219 \times 12 = 19,898 \text{ k-in}$$

table 6.1.

(2)

	max	min	
t_p	2 1/2	3/4	t_p Plate
b_p	15	9	width of End plate.
g	6	5	horizontal distance between bolts.
p_{ti}, p_{to}	2	1 3/4	
p_b	3 3/4	3 1/2	
d	36	18.5	
t_{wt}	1	14/32	
b_{bf}	12 1/4	7 3/4	

Refer to following page for preliminary plate dimentions

(3) Determine the req dia of bolts.
&
(4) $d_{b\,req} = \sqrt{\dfrac{2(19,898)}{\pi \times .75 \times 113 \times (29.25 + 25.5 + 20.5 + 16.75)}}$

$= 1.27$ bolts $\therefore$ use A490 $1.\frac{1}{4}$" bolts.

$\approx$ 1.25" or else connection wont match spacing requirements.

(5) Req End plate thickness.
(6) $t_{p\,req} = \sqrt{\dfrac{1.11 \times 19,898}{.9 \times 50 \times 384.4}} = 1.12 \approx$ use $1.\frac{1}{8}$" plate

table 6.4

$s = \frac{1}{2}\sqrt{14.6} = 4.58.$

$d_e = 2"$ $\quad d_e < s \therefore$ case 1 applies.

$$Y_p = \frac{14}{2}\left[29\tfrac{1}{4}\left(\frac{1}{2.2}\right) + 25.5\left(\frac{1}{2}\right) + 20.5\left(\frac{1}{2}\right) + 16.75\left(\frac{1}{4.58}\right)\right]$$

$$+ \frac{2}{6}\left[29\left(2 + \frac{3.75}{4}\right) + 25.5\left(2 + \frac{3(3.75)}{4}\right)\right.$$

$$+ 20.5\left(3 + \frac{3.75}{4}\right) + 16.75\left(4.58 + \left(\frac{3(3.75)}{4}\right)\right)$$

$$\left. + 3.75^2\right] + 6.$$

$Y_p = 7(33.97) + .33(412 + 14.1) + 6$

$= 384$

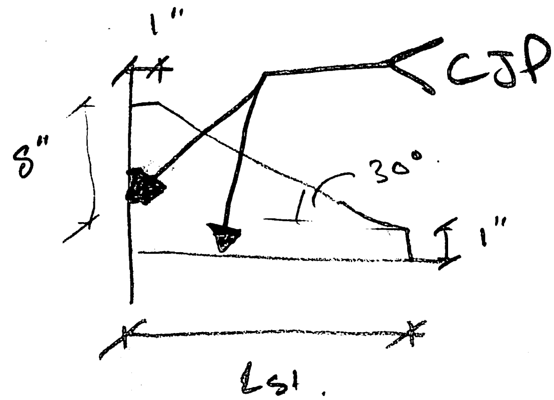
(7) calculate facture beam flange force

$F_{fu} = \dfrac{M_f}{d - t_{bf}} = \dfrac{19,898}{23} = 865$

(10) Stiffened : Design.

$t_{s\,min} = 3/8(1) = 3/8$ use $3/8"$

$l_{st} = \dfrac{h_{st}}{\tan 30°} = \dfrac{7.75"}{\tan 30} = 13.4.$ use $15"$



CJP weld.

check Stiffener Slenderness

$\dfrac{9"}{3/8} \leq .56\sqrt{\dfrac{29E3}{50}} = 13.5$ ∴ not good, use $t_p = 1\tfrac{1}{8}$ for Repitition.

$\dfrac{9}{1\tfrac{1}{8}"} = 8" \leq 13.5$

(11) Bolt Sher Ropture.

table 7-1 SM.

$V_u \leq \phi_n R_n = .75 \times 8 \times 55.2 = 331.$ kips.

$V_u = 219 \leq 331$ ∴ ✓

(12) Bolt Bearing / tear out failure of End plate & connection flange.

$V_u = 319 \leq \phi R_n =$

$n_i = 4$

$n_o = 4.$

$r_{ni} = 1.2(2.5)(1\,1/8)(65) < 2.4\,(1\,1/4)(1\,1/8)(65)$

$= 219.4 \qquad < 219.$

$t_{fc} = 1.72''$

$t_{EP} = 1\,1/8''$

(13) flange to End plate & web to End plate welds.

358-05, 6.9.7

1) no weld Access holes.

2) $V_p = 248$ kips.

$\phi R_n = 1.392 \times 2 \times 22 \times 5 = 306$ kips.

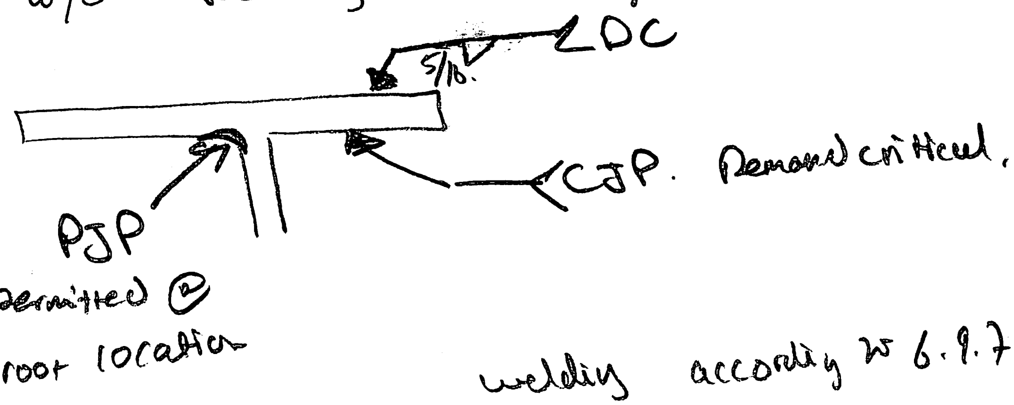
max weld $= \dfrac{16''}{16''}$ ∴ use (2) (5/16ths) fillet welds

or CJP.

min weld. $= 5/16$

3) Demand critical welds.

CJP w/o backing on flanges.



4) welding according to 6.9.7.

(14) Column flange flexural yielding.

$$t_{cf} = \sqrt{\frac{1.11 \times 19890}{.9 \times 50 \times 275}} = 1.33 < t_{cf} = 1.7'' \therefore \checkmark$$

See next page for Y_c computation.

unstiffened.

$$Y_c = \frac{16}{2}\left[29.25\left(\frac{1}{4.24}\right) + 16.75\left(\frac{1}{2}\right)\right.$$

$$+ \frac{2}{6}\left[29.25\left(3.75 + \frac{5}{2} + 2\right) + (20.5)\left(\frac{3.75}{2} + \frac{5}{2}\right)\right.$$

$$\left. + 20.5\left(\frac{3.75}{2} + \frac{5}{2}\right) + 16.75(2)\right] + 6/2$$

$$= 8(15.3) + .33(454) + 3 = 275.$$

$$S = \frac{1}{2}\sqrt{12.6} = 4.24.$$

$$t_{cf} = \sqrt{\frac{1.11 \times 19898}{.9 \times 50 \times 275}} = 1.33 < t_{cf} \therefore \text{not}$$

stiffeners required.

$Y_c = 8[32] + .33[489] + 6$

$= 423$

(15) no stiffeners required.

(16) Check colom web yielding strength at the unstiffened col. web @ the beam flange.

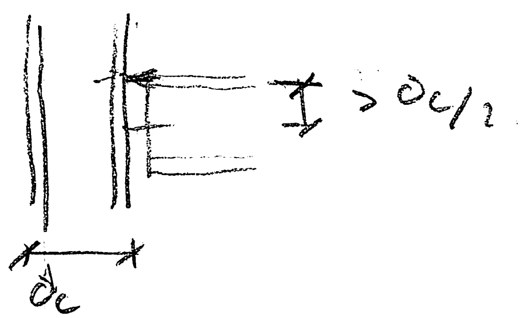
$$\phi d, R_n = 0.9 \times 0.5(6 \times 2.32 + 1 + (2\tfrac{1}{2})2) 50 \times 1.07)$$

$= 468.$ ∴ Column continuity plates are required.

(17) check unstiffened column web buckling strength

a)

$$R_n = \frac{24 \times 1.07^3 \sqrt{29E3 \cdot 50}}{22}$$



$= 1504 \geq F_{fu}$ ∴ ✓

(18) Colom web crippling.

$$\phi R_n = 0.75 \times 0.8 \times 1.07^2 \left[1 + 3 \left(\frac{1}{16} \right) \left(\frac{1.07}{1.72} \right)^{1.5} \right] \sqrt{\frac{29E3 (50)(1.72)}{1.07}}$$

$$\phi R_n = (0.687)(1.092)(1527)$$

$$= 1145 \text{ kips} \quad > F_{fu}.$$

∴ no stiffeners.

(19) $f_{su} = 865 - 468 = 397$

weld continuity plates. according to 6.7.3.

$t_{plate} = 1''$ weld.

Use CJP welds for continuity plates.

According to AISC 358-05 2.4.4.b.

(20) Check Panel Zone strength

$$\Sigma f_u = \frac{19{,}898}{24} \times 2$$

Assume thicker now. Equal moment @ both sides

assume

$$= 1658$$

$\square$

Doubler plates Design.

$V_{udp} = \Sigma F_u - 482$

from previous calc.

$= 1658 - 482 = 1176.$

$h = 16 - 1.72 \times 2 = 12.56$

$1176 = 0.6 \times 50 \times 12.56 \times t_{dp} \times 1$ ∴ $C_v = 1$

$t_{dp} = 3.1136.$ Assume $K_v = 6.34.$

use $\underline{\underline{2(1.75)}}$ Doubler plates

Design option 2.

After looking @ the Prequalified Connections:
the following connection was designed.

- CJP @ Link flanges.
    - check for continuity plates in colum.
    - Panel zone strength in column.

- Design shear tab connection. @ Link web w col flange.

Limit States.

Column. -
- Panel zone
- continuity plates req.

Beam -
- Web Shear yield.

Shear plate
- shear yield.
- Instantaneous center of rotation.
- weld Rupture due to eccentric loading.

* Determine Max moment of Link

$M_{pr} = 1.15 \times 1.1 \times 321 \times 50 = 20{,}303$ k-in.

$C_{pr} = \frac{50+65}{2(50)} = 1.15$ $\qquad$ $M_{pr} = 27{,}577.$
(W24+146)

* Determine shear force @ hinge location.

$$V_p = \frac{(20{,}303 + 27{,}577)}{12(30 - \frac{20}{12})} + 43 = 184 \text{ kips.}$$

$$V_{grave} = 3.2 \times \left(30 - \frac{20 \times 2}{12}\right) \times 1/2 = 43$$

$$w_u = 1.4(1.74)\ 0.5(1.6) = 3.2$$

Hinge location $S_H = \frac{24}{2} + 16/2 = 20''$

* Determine moment @ colum face.

$$M_f = 20{,}303 + 184 \times 20 = 23{,}983 \text{ k-in.}$$

- Check continuity Plates requirement AISC 358-05 Sec. 2.4.4.

$t_{cf} \geq 0.4\sqrt{1.8(12)(1)(1)(1)} = 1.86''$

$t_{cf} \geq \frac{12}{6} = 2''$

$t_{cf} = 1.72$ ∴ continuity plates are required.

$t_p \geq \max[t_f \text{ of Beam}]$.

- Check Panel Zone Strength. AISC 341-05 Sec. 9.3.

W14x233

$P_r = 1588 \Leftarrow P_u$ Peter no Spread Sheet.

$P_y = 50(68.5) = 3425$

$P_c = .6 \times 3425 = 2035$

Panel Zone Def. Considered in Analysis.

∴ $P_r > 0.7(2035) = 1541$ ∴ SPEC J10.6.b.ii applies.

$$R_n = 0.6 \times 50 \times 16 \times 1.07 \times \left(1 + \frac{3(15.9)(1.72)^2}{16 \times (24)(1.07)}\right)\left(1.9 - \frac{1.2(1588)}{1588}\right)$$

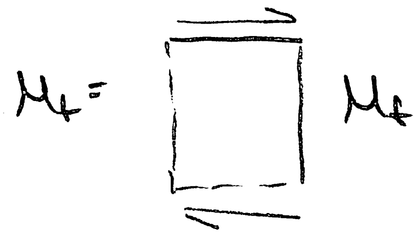
$= (513.6)(1.34)(0.7) = 481.8^{\text{kips}}$

$\phi R_n = 482^{k}$

$M_f = 21{,}136$ Assume W 24 x 146 has

$M_f = 25{,}289 + 143 \times (20 - 16/2)$

$= 27{,}005$



$\sum f_u = \dfrac{21{,}136 + 27{,}005}{24} - \cancel{V_u} \rightarrow 0$ braces carry story shear.

$= 2006 > \phi R_n \therefore$ Doubler plates required.

Doubler. plate Design

AISC 358-05 9.3b.

$t \geq [(24 - 2(1)) + (16 - 1.72 \times 2)] / 90 = .384$

Sizing Doubler plates.

$V_{u\,dp} = \sum f_u - \phi R_{v_c} = 2006 - 482 = 1524$ kips

$h = 16 - 1.72 \times 2 = 12.56$

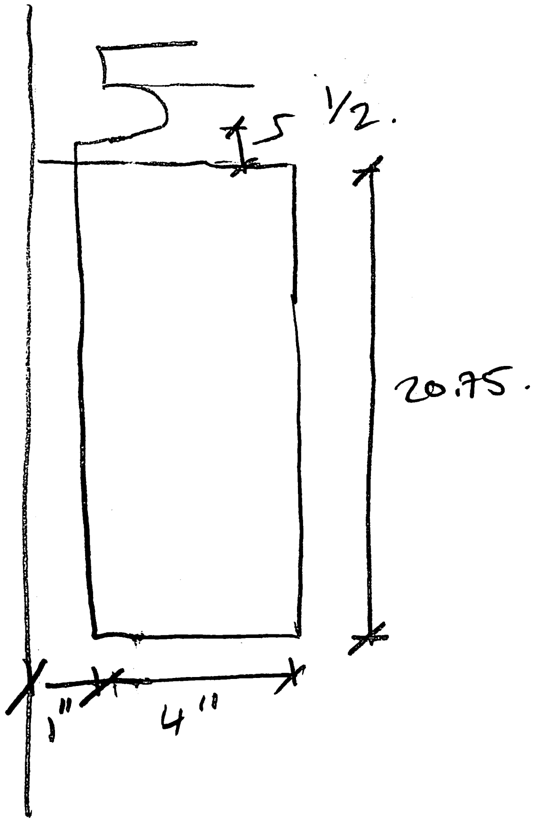
$1524 = 0.6 \times 50 \times 12.56 \times t_{dp} \times 1 \rightarrow C_v = 1$

$t_{dp} = 4.04''$

$\dfrac{12.56}{4.04} = 3.1 \leq 1.1 \sqrt{\dfrac{6.34 \times 29E3}{50}} = 66.7 \therefore \checkmark$

use $C_v = 1$.

Shear tab weld using table 8-9 Steel Manual.



$l = 20.75$

$k = 4/20.75 = 0.19$

$$x = \frac{0.029 - 0.008}{0.2 - 0.1}(0.19 - 0.1) + .008$$

$= 0.0269.$

$xl = 20.75 \times 0.0269 = 0.56"$

$al = a \times 20.75 = 5 - 0.56 \Rightarrow a = 0.214$

$$\therefore \quad C = \frac{2.51 - 2.63}{.250 - .2}(.214 - .2) + 2.63$$

$= 2.596.$

$P = V_n$

$$C_{min} = \frac{248}{.75(1)(D)(20.75)} = 2.596$$

$D = 6.138$ use $\underline{7/16^{th}}$ $\Leftarrow$

$t_p = 1/2$

$t_w \max = 1/2 - 1/16 = 7/16^{th}$ $\therefore$ ✓

$t_w\min = 1/4"$ $\therefore$ ✓

Stiffened web:

$$k_v = 5 + \frac{5}{(1.93)^2} = 6.34.$$

$h = 10.7 \times 1.07 = 11.4$

$a = 24 - 2 = 22''$

$$\frac{a}{h} = \frac{22}{11.4} = 1.93$$

Use $t_{dp} = 4.04''$ $\Rightarrow$ $2(2\,1/8'')$ plates.

Design Shear tab connection.

$V_n = 247.5^{kips}$ $\Leftarrow$ connection strength.

Beam web strength,

$\phi R_n = 1 \times .6 \times 50 \times 3/8 \times d_{min} \geq 248$

$d_{min} \geq 22''$ full depth

Check Access hole dimension

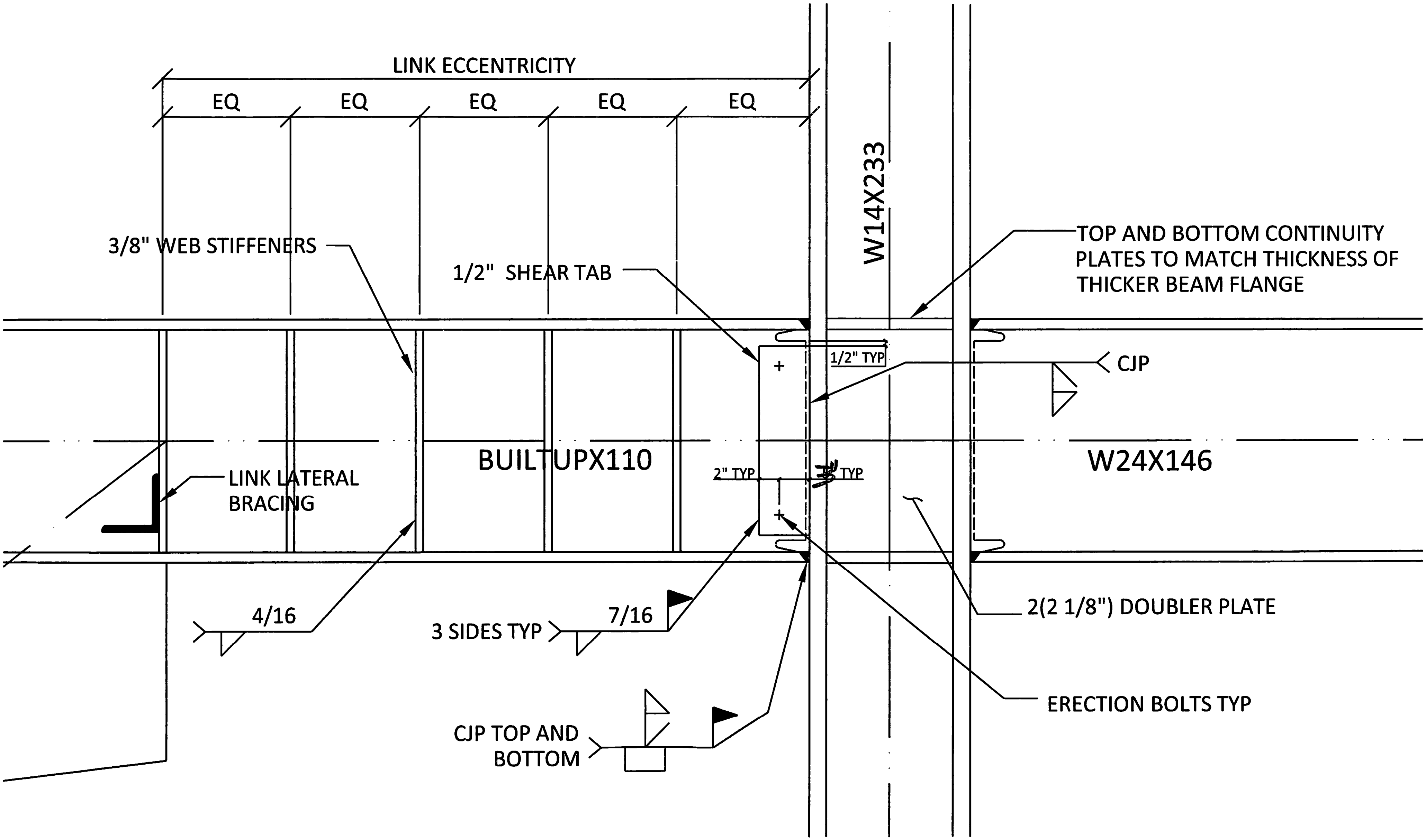
$d = 24 - (3/8 + 3/4 \times 1) \times 2 - 1 = 20.75$

Size plate thickness

$\phi R_n = 1 \times .6 \times 50 \times t_p \times 20.75 = 248$

$t_p = 0.39$ $\Rightarrow$ Use $1/2''$ plate.

Design Summary



Brace to Link Connection

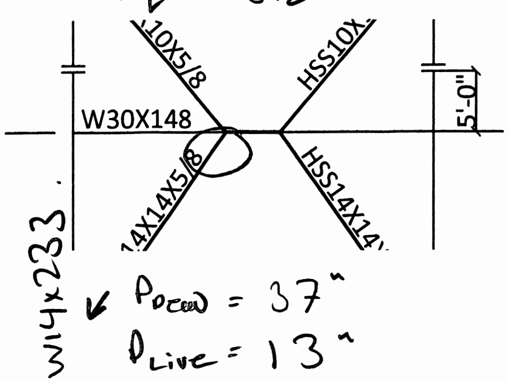
Connection #1

Beam/Link W30x148	Col. W14x233	Brace HSS14x14x5/8	HSS10x10x5/8
$F_y = 50$	$F_y = 50$	$F_y = 46$	$F_y = 46$
$F_u = 65$	$F_u = 65$	$F_u = 58$	$F_u = 58$
$d = 30.7$	$d = 16.0$	$t_w = .581$	$t_w = .581$
$t_w = .650$	$t_w = 1.07$	$A_g = 30.3$	$A_g = 21.0$
$b_f = 10.5$	$b_f = 15.9$	$d = 14"$	$d = 10"$
$t_f = 1.18$	$t_f = 1.72$	workable flat $= 11\ 3/16$.	work flat $= 7\ 3/16"$
$k = 1.83$	$k = 2.32$		
$T = 26.5$			
$N =$			

$P_{Dead} = 4^k$

$P_{Live} = 9^k$

$P_{Eq} = 312^k$



$P_{Dead} = 37^k$

$P_{Live} = 13^k$

$P_{Eq} = 414^k$

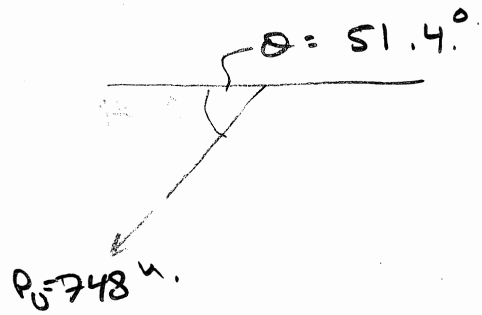
$$\Omega_{overstreng} = \frac{1.25 R_y V_n}{V_{Eq}} = 1.66$$

Limit States: connection #1.

- ✓ - plate Rupture @ weld
- ✓ - Brace Rupture @ weld.
- ✓ - Shear yeild Strength @ brace gosset weld.
- ✓ - Uniform force method & Design gosset-beam. connection
- ✓ - Shear lag Rupture of Brace.
- ✓ - compression buckling. Strength of gosset. top & Bottom.
- ✓ - Yeilding of gosset plates.
- ✓ - Check beam web local yielding.
- ✓ - Check beam web crippling.

Connection #1 Design.

$$P_u = \underset{\text{Dead}}{1.38(37)} + \underset{\text{live}}{0.5(13)} + \underset{\text{Eq}}{414 \times 1.66} = 748^k.$$



$R_u = 748$.

Determine weld size Brace to gusset.

- Fillet E70.

min $t_w = 5/16"$

max $t_w = 3/4 \quad 1/16 = 11/16"$

$L_{weld} \geq 14"$

Try. 14" weld.

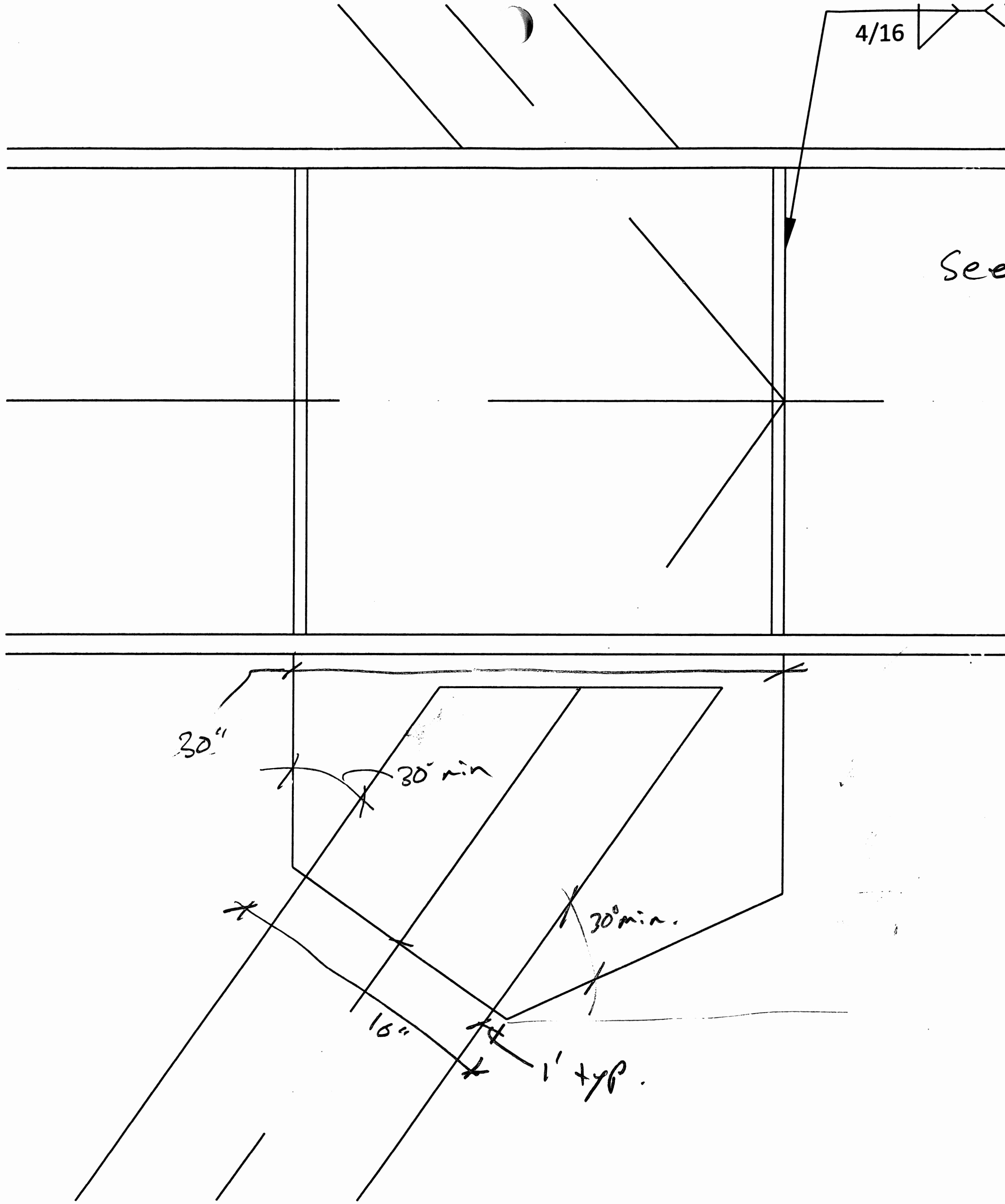
$\phi R_n = 4 \times 1.392 \times 5 \times 14 = 389.7$

$= 4 \times 1.392 \times 6 \times 14 = 468^k$

$4 \times 1.392 \quad 9 \times 16 = 802$ kips

Use $\frac{9}{16}$" fillet weld 16" Long

Page 2.



See Page ___ for more dimentions.

whichever is thinner assume Brace

- Base metal. Rupture @ weld.

Rupture $\phi R_n = 0.75 \times .6 \times 58 \times .581 \times 16 \quad \times 4$

$= 971^k > R_u \therefore$ ✓

Whitemore Section.

Refer to diagram on. page 2

- Determine comp. buckling of gosset plate.

$L_{avg} = 8.75''$

$Kl = 8.75 \qquad l_w = 36''$

$\phi r_n = \dfrac{748}{30.19} = 24.8$ Using table 1-7 Seismic manual.

Use $\underline{\underline{3/4''}}$ gosset. $\phi r_n = 29.7$ k/in width

$L_{fe} > 13.5''$ Edge length

- check shear lag Rupture of Brace

$\bar{x} = \dfrac{14^2 + 2(14)^2}{4(14+14)} = 5.25''$

$U = 1 - \dfrac{5.25}{14} = 0.625$ assume 1/16" gap.

$A_n = 30.3 - 2(3/4 + 1/16)\ 5/8 = 29.3$

$A_e = 58(29.3) = 1698$

$\phi P_n = .75 \times 1698 = 1274 > P_u \therefore \checkmark$

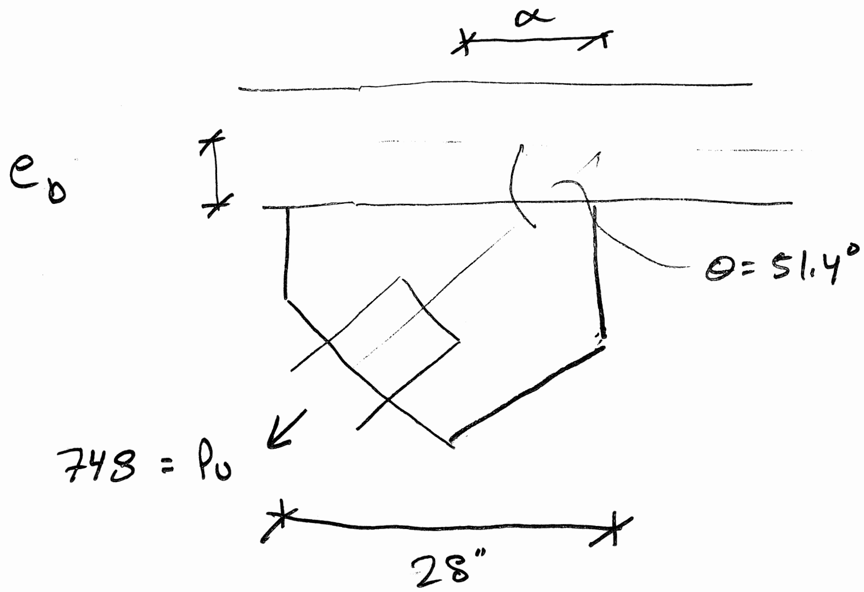
- check Shear Strength at brace gusset weld.

$l_w = 16''$

$A_e = 4 \times 16 \times .581 = 37.2$

$\phi V_n = .9 \times .6 \times 46 \times 37.2 = 924 > P_u \therefore \checkmark$

Determine gusset to beam-Link connection.



$\alpha = 30/2 = 15$

$\beta = 0.$

$e_c = 0$

$e_b = 15.35''$

Using the uniform force method.

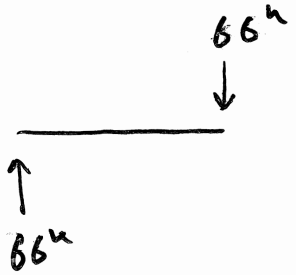
$\alpha = 15.35 \tan 51.4 = 19.22$

$\therefore \quad r = \sqrt{19.22^2 + 15.35^2} = 24.6$

$H_{UB} = \dfrac{19.22}{24.6} \times 748 = 584^k.$

$V_{UB} = \dfrac{15.35}{24.6} \times 748 = 467^k$

$M_{UB} = 467\,(19.22 - 15) = 1970 \quad k\text{-in.}$



$\dfrac{1970}{30''} = 66^k$

Design weld. for H_{UB} = 584^{k}.

min weld t_w = 5/16" max weld t_w > 15/16".

ϕR_n = 2 welds x 1.392 x t_w x 30 = 584

t_w = 7 => use 7/16" weld.

check Shear yeilding at gusset

ϕR_n = .9 x .6 x 50 x 3/4 x 30 = 608 > 584 ∴ ✓

gusset thinner than beam flange ∴ gusset limit states controls.

Design Brace - gosset - Link connection of HSS10x10x5/8.

$P_u = 1.38(4) + 0.5(9) + 312 \times 1.66 = 543^{k}$.

Brace w gosset connection

weld.

min $t_w = 5/16"$ max $t_w = 3/4 - 1/16 = 11/16"$

$L_{weld} = 10"$

$\phi R_n = 4 \times 1.392 \times 9 \times 12 = 601$ kips

Use $9/16"$ weld, 12" Long for consistency with HSS14x14x5/8 Brace.

Base metal Rupture @ Weld.

$\phi R_n = .75 \times .6 \times 58 \times .581 \times 12 \times 4 = 728 > P_u$ ∴ ✓

Determine Comp. buckling of gosset

$L_{avg} = 7.958$

$Kl = 8$ $l_w = 23$.

$\phi r_n = \dfrac{543}{23} = 23.6^{k}/in.$ table 1-7 SM.

Use gosset $3/4"$ thick ⇒ use $5/8"$ instead for consistency.

$L_{fe} > 13.5$ Edge length

$\phi r_n = 30.5^{k}/in.$

- Shear lag Rupture of Brace

$$x = \frac{10^2 + 2(10)^2}{4(20)} = 3.75.$$

$$U = 1 - \frac{3.75}{10} = .625.$$

$A_n = 21. - 2(3/4 + 1/16)5/8 = 19.98$

$A_e = .58(19.98) = 11.59$

$\phi P_n = .75(1159) = 869 > P_u \therefore \checkmark$.

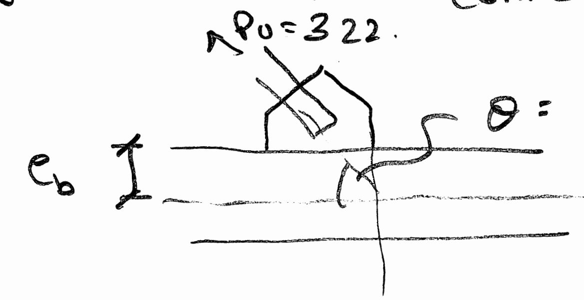
- Shear. yeild strength @ brace gusset weld.

$L_w = 10$

$A_e = 4 \times 12 \times .581 = 27.9$

$\phi V_n = .9 \times .6 \times 46 \times 27.9 = 693 > P_u \therefore \checkmark$

- gusset - Beam Connection



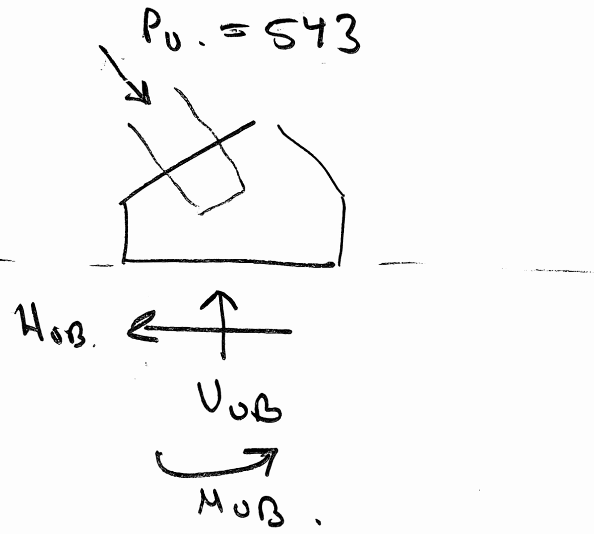
$\alpha = 29/2 = 14.5$

$e_b = 15.35$

$\alpha = 15.35 \tan 49.6. = 18$

$r = \sqrt{18^2 + 15.35^2} = 23.68.$

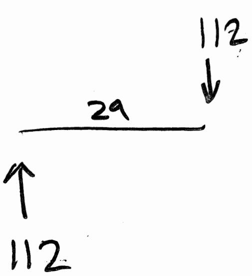
UFM



$$H_{uB} = \frac{18}{23.68}(543) = 413^{k}$$

$$V_{uB} = \frac{15.38}{23.68}(543) = 353^{k}$$

$$M_{uB} = 353\,(23.68 - 14.5) = 3241 \text{ k·in.}$$



- weld @ Beam - gosset. for H_{uB}.

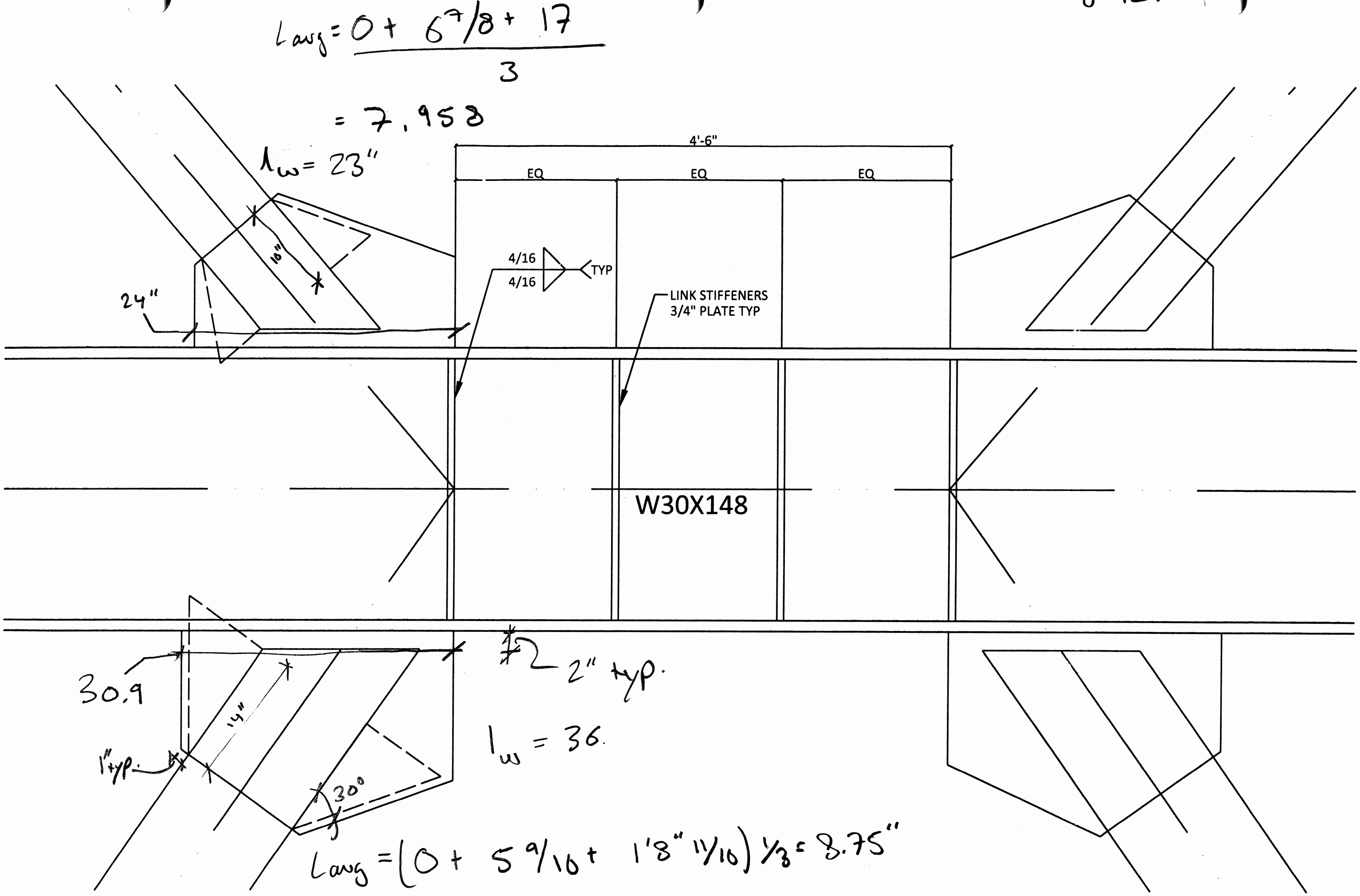
min 5/16" weld.

$$\phi R_n = 2 \times 1.392 \times 6 \times 30 = 501 > H_{uB} \therefore ✓$$

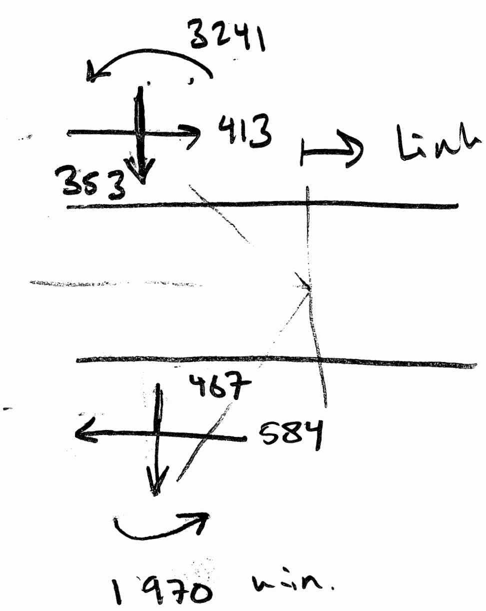
Use 5/16" weld.

- Shear yeild of gosset.

$$\phi R_n = .9 \times .6 \times 50 \times 3/4 \times 29 = 587 > H_{uB} \therefore ✓$$


L avg = (0 + 6 7/8 + 17) / 3
= 7.958
l w = 23"
4'-6"
EQ
EQ
EQ
10"
24"
4/16
4/16
TYP
LINK STIFFENERS
3/4" PLATE TYP
W30X148
30.9
14"
2" typ.
l w = 36.
1" typ.
30°
L avg = (0 + 5 9/16 + 1'8" 11/16) 1/3 = 8.75"

combine forces from brace above & Below.



$\Sigma V = 353 + 467$

$= 820$

Due to moment Axial comp. in web.

$$\frac{(3241 + 1970)}{19} = 274$$

local yeilding strength of the beam web.

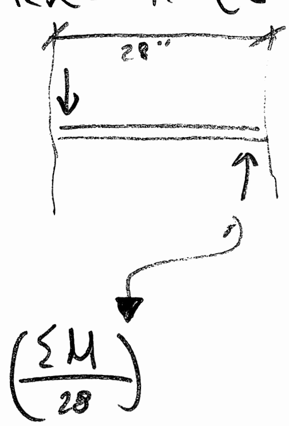
- $\phi R_n = 1.0(5(1.83) + 29) \times 50 \times .65 = 1207$ kips.

$\phi R_n = 1207 > (820)$

no stiffeners. required.

- $\phi R_n = 1.0(5(1.83) + 5/8) \times 50 \times .65 = 318. > 274$ ∴

no stiffers.



- Chen Beam web crippling.

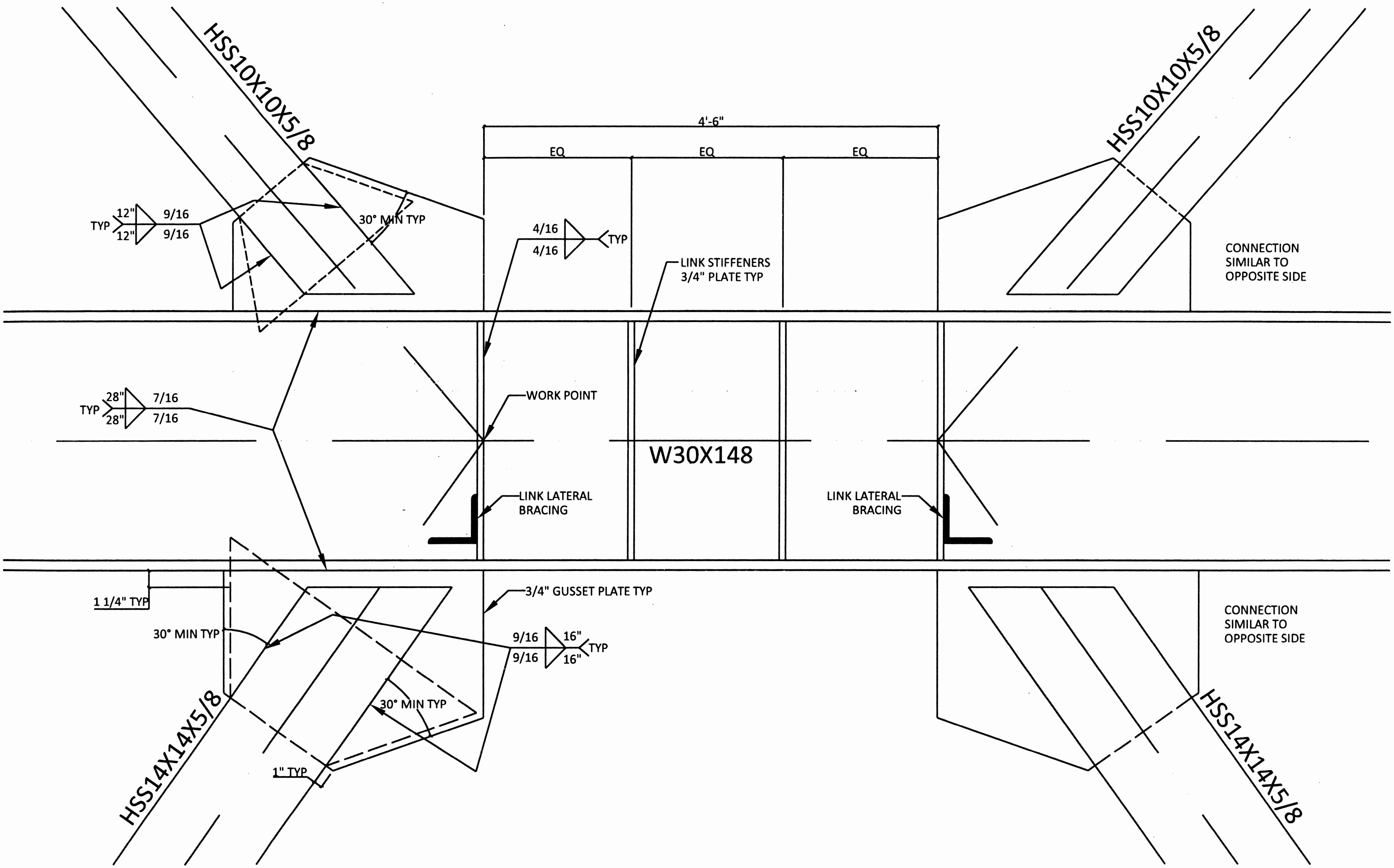
$N/d = 28/30.7 = 0.912.$

$$R_n = 0.8 \times .65^2 \left[1 + (3 \times .912)\left(\frac{.65}{1.18}\right)^{1.5} \right] \sqrt{\frac{29E3\,(50)(1.18)}{.65}}$$

$$= (0.338)(2.12)(1622) = 1{,}162 > 820$$

$\therefore$ ✓

Connection Design Summary



Brace-Beam to Column Connection

Page 1

Connection #3.

W30x116.

$f_y = 50$

$f_u = 65$

$d = 30"$

$t_w = .565$

$b_f = 10.5$

$t_f = .85$

$k = 1.5$

$T = 26\frac{1}{2}$

$Z_x = 329$

W14x233

$f_y = 50$

$f_u = 65$

$d = 16$

$b_f = 15.9$

$t_f = 1.72$

$t_w = 1.07$

$A_g = 68.5$

$R_y = 1.1$

$S_x = 375$

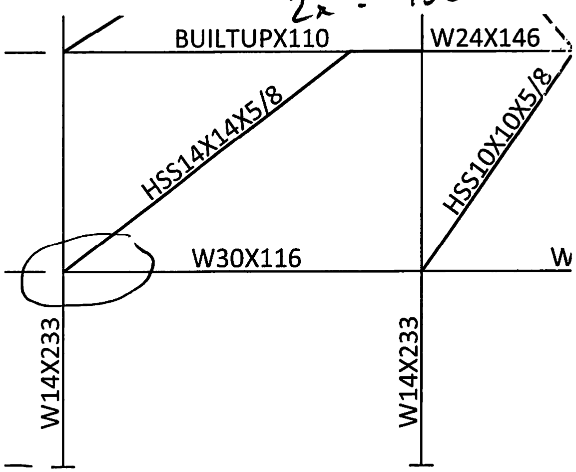
$Z_x = 436$

HSS 14x14x5/8.

$d = 14$

$t_{design} = .581$

$A = 30.3$



Brace:

$P_{Dead} = 48$

$P_{Eq} = 352$

$P_{Live} = 41$

Beam:

$V = 37$ $M = 2223$

$V = 24$ $M = 1449$

$V = 0$ $M = 14$

$P_{Eq} = 352$ kips.

$$\Omega_o = \frac{1.25 \times 1.1 \times 247.5}{197} = 1.73$$

Limit States of Connection:
- Rupture of weld @ Brace
- Base metal Rupture @ weld (gosset & Brace)
- Comp buckling of gosset
- Shear lag Tension Rupture of Brace.
- Shear yeild @ brace & gosset weld.
- Rupture of weld @ gosset w beam
& w column.
- Shear yeild @ gosset. beam & column face.
- beam Web local yeilding.
- beam web crippling.
- Base metal Rupture @ Column flange.
- Column Continuity plates requirement per Seismic provisions.
- Panel Zone Shear yeilding of Column.
- Beam web yeilding

Determine the required strength of the brace

$P_u = 1.38(48) + 352(1.73)+41 \times .5 = 696.^{kips.}$

neglect V_u

$R_u = 696^{kips.}$

Determine the required strength of beam outside link.

$V = 37 \times 1.38 + 1.73(24) = 93^{k}$

$M = 2223(1.38) + 1.73(1449) + 14(0.5) = 5,582.^{k\text{-}in.}$

$P_{Eq} = 2(352) = 704^{kips.}$

Required Strength to gusset - from Brace

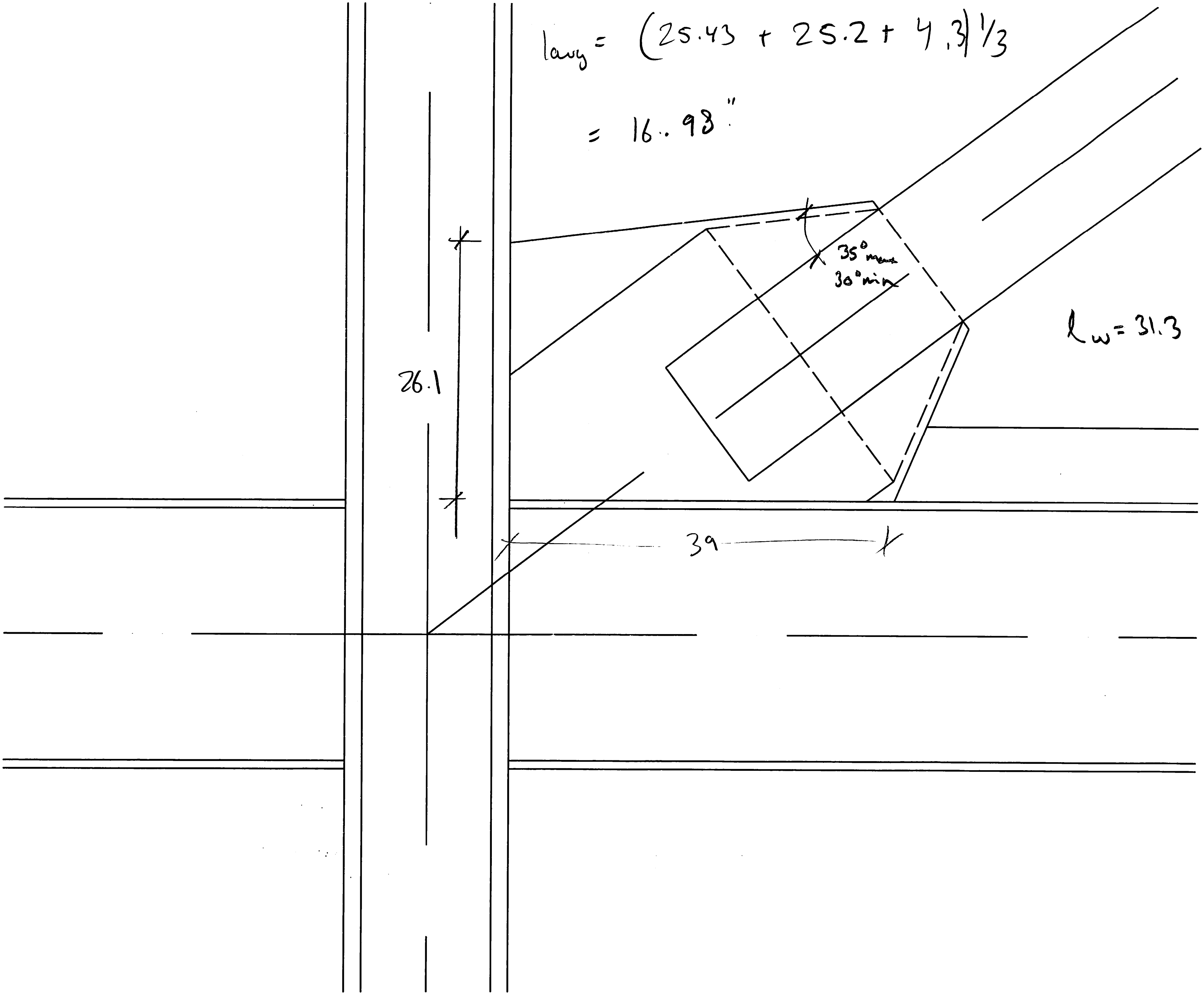
$R_u = 696^{kips.}$

min $t_w = 5/16''$ max $= 5/8 - 1/16 = 9/16''$

$\phi R_n = 4 \times 1.392 \times 9 \times 16 = 802^{k} > R_u \therefore ✓$

Base metal Rupture @ weld.

$\phi R_n = .75 \times .6 \times 58 \times .581 \times 16 \times 4 = 971 > R_u \therefore ✓$


$l_{avg} = (25.43 + 25.2 + 4.3) 1/3$
$= 16.98''$
26.1
39
35° max
30° min
$l_w = 31.3$

Determine comp Buckling of gosset plate.

$L_{avg} = 16.98''$

$L_w = 31.3 \qquad kl = 17''$

$$\phi r_n = \frac{696}{31.3} = 22.3 \text{ k/in.}$$

table 1-8.

$\phi R_n = 27.2$ k/in. use $\underline{\underline{7/8''}}$ gosset plate.

Shear lag Tension Rupture. of the Brace.

$$\bar{x} = \frac{14^2 + 2(14)^2}{4(14 \times 2)} = 5.25''$$

$$U = 1 - \frac{5.25}{14} = 0.625$$

$$A_n = 30.3 - 2\left(7/8 + 1/16\right) 5/8 = 29.1$$

$$A_e = 0.625 \times 29.1 = 18.2$$

$\phi P_n = .75 \times 58 \times 18.2 = 792^k \quad > P_u. \;\therefore \checkmark$

Shear strength @ brace to gosset weld.

$L_w = 16$

$A_e = 4 \times 16 \times .581 = 37.2 \text{ in}^2$

$\phi V_n = 0.9 \times 0.6 \times 46 \times 37.2 = 924 > P_u \;\therefore \checkmark$

Determine gusset to beam column connection interface forces.

$e_b = \frac{30}{2} = 15''$

$e_c = 16/2 = 8''$

$\theta = 36.9^\circ$

$\alpha = 1/2(39-1) + 1 = 20''$

$\bar{\beta} = 1/2(30-1)+1 = 15.5''$

$\beta = \bar{\beta}$

$\alpha - \beta(\tan\theta) = e_b \tan\theta - e_c$

$= 15 \tan 36.9 - 8$

$\alpha - 15.5 \tan 36.9 = 3.26.$

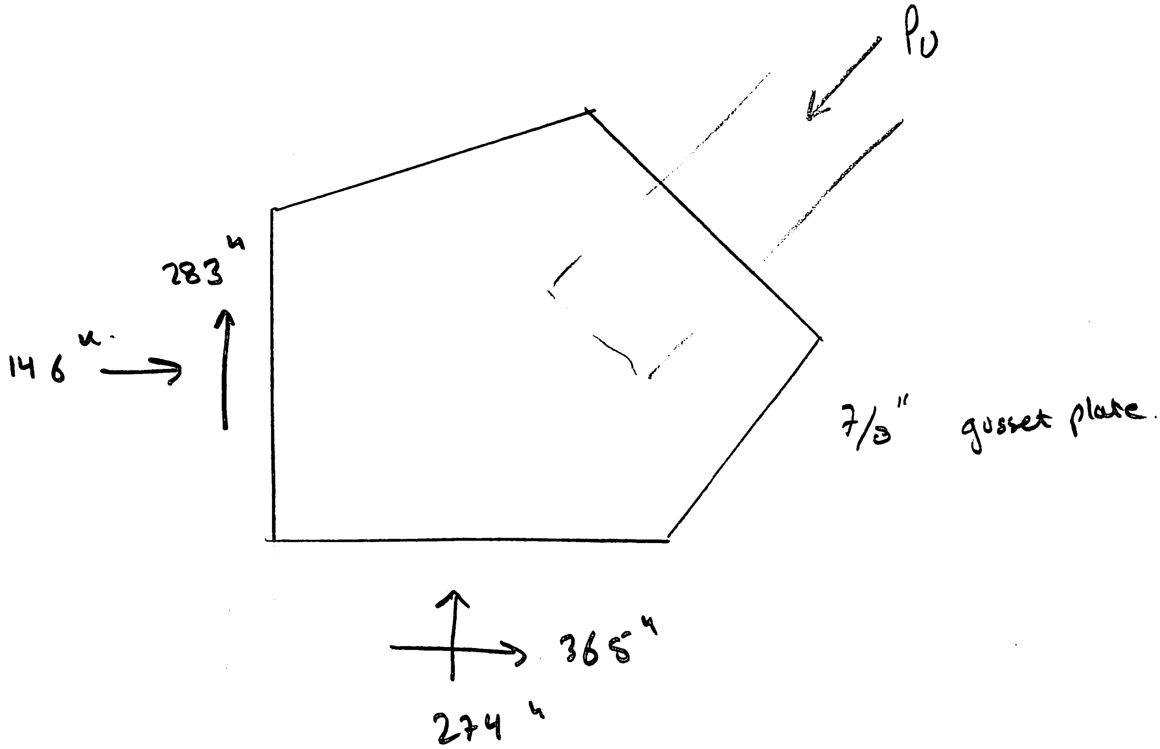
$\alpha = 14.898$

$r = \sqrt{(14.898+8)^2 + (15.5+15)^2} = 38.14$

$V_c = \frac{15.5}{38.14}(696) = 283^k$ $H_c = \frac{8}{38.14}(696) = 146.^k$

$H_b = \frac{20}{38.14}(696) = 365^k$ $V_b = \frac{15}{38.14}(696) = 274^k.$

gusset free body diagram.



Design Weld @ gusset / Beam Interface.

$$l_w = 39 - 1 = 38''$$

$$f_v = \frac{365}{38} = 9.6 \ k/in$$

$$f_a = \frac{274}{38} = 7.16 \ k/in.$$

$$f_{peak} = f_{avg} = \sqrt{9.6^2 + 7.16^2} = 12 \ k/in.$$

$$\theta = \tan^{-1}\left(\frac{274}{365}\right) = 36.9^\circ.$$

fillet weld strength

$$\phi r_n = 1.392 \ kips/in \ (1.0 + .5 \sin^{1.5} 36.9) = 1.72 \ k/in$$

$$D_{min} = \frac{1.25(12)}{2(1.72)} = 5.23.$$ use $2\left(6/16^{\text{ths}}\right)$ fillet welds.

check yeild of the gosset.

$$\phi R_n = .9 \times 0.6 \times 50 \times 7/8 \times 38 = 898^{\text{kips}}. > H_B.$$

check beam web local yeilding.

$$\phi R_n = 1.0\left[2.5(1.5) + 38\right] 50 \times .565$$

$$= 1{,}179^{\text{kips}}. \quad > V_{UB}. \therefore \checkmark$$

Check beam web crippling.

$$N/d = 39/16 = 2.44.$$

$$R_n = 0.8(.565)^2\left[1 + (3 \times 2.44)\left(\frac{.565}{.85}\right)^{1.5}\right]\sqrt{\frac{29E3 \times 50 \times .85}{.565}}$$

$$= 0.255 \times 4.47 \times 1477 = 1875^{\text{kips}}. \quad > V_{UB}. \therefore \checkmark$$

Design weld gusset to colom flange.

$$f_v = \frac{283}{26} = 10.9$$

$$f_a = \frac{146}{26} = 5.62$$

$$f_r = \sqrt{10.9^2 + 5.62^2} = 12.26 \text{ kips/in.}$$

$$f_r = 1.25\,(12.26) = 15.3$$

$$\theta = \tan^{-1}\left(\frac{146}{283}\right) = 27.3^\circ$$

$$\phi r_n = 1.392\,(1 + .5 \sin^{1.5} 27.3)$$

$$= 1.61 \text{ k/in.}$$

$$D_{min} = \frac{15.3}{2(1.61)} = 4.75 \Rightarrow \text{ use } 2(5/16^{th}) \text{ fillet welds.}$$

Check yeilding of gusset.

$$\phi R_n = .9 \times .6 \times 50 \times 7/8 \times 26 = 614 > V_{uc}.$$

Check flange Rupture @ gusset.

$$\phi R_n = 0.75(0.6) \times 65 \times 7/8^{*} \times 26 = 665 > V_{uc}$$

* controls over col. flange

Design Beam to Col. for WUF-W Connection.

Design Beam to Col. Connection according to AISC 341-05 Fully Restrained moment Connection. section 15.7.

① $M_{pr} = 1.15 \times 1.1 \times 329 \times 50 = 20{,}809$ k-in.

$$C_{pr} = \frac{50+65}{2(50)} = 1.15$$

② $V_p = \dfrac{(2 \times 20809)}{12\left(30 - \frac{23}{12}\right)} + 58$ kips.

$S_H = \dfrac{d_c}{2} + \dfrac{d_b}{2} = \dfrac{16}{2} + \dfrac{30}{2} = 23''$

$= 181^k$.

$$V_{grav} = \frac{4.1 \times \left(30 - \frac{23}{12}\right)}{2} = 58 \text{ kips.}$$

$w_u = 1.38(2.36) + 0.5(1.6) = 4.1$ ⇐ gravity loads.

③ $M_f = 20{,}809 + 181 \times 23 = 24{,}972$ k-in.

Check to see if Continuity Plates are required. per AISC 358-05 Section 2.4.4.

$$t_{cf} \geq 0.4\sqrt{1.8 \times 10.5 \times .85 \times 1}$$

$$\geq 1.6.$$

$$t_{cf} \geq \frac{10.5}{6.}$$

$$\geq 1.75$$

$t_{cf} = 1.72$ ∴ Continuity plates are required.

t_{plates} shall equal the thickness of the beam flange.

Check Panel Zone strength

According to AISC 341-05 Section 9.3a.

W14x233

$P_r = 1666 \Leftarrow$ Etabs Refer to Column Design Spread Sheet

$P_y = 50(68.5) = 3{,}425$

$P_c = .6 \times 3425 = 2055$

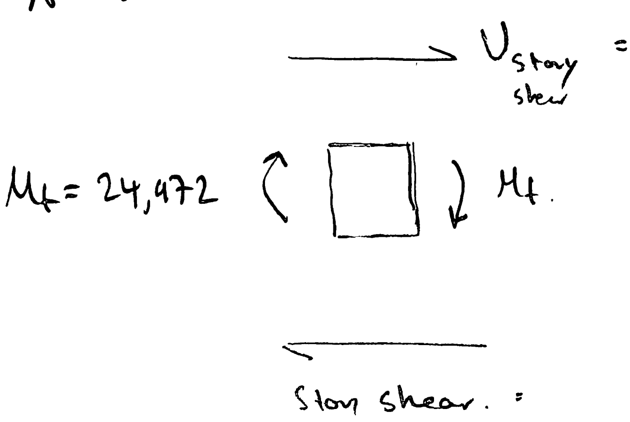
Panel Zone deformation considered in Analysis.

$1666 > .75(2055) = 1541 \quad \therefore$ spec J10.6.b.ii applies.

$$R_n = 0.6 \times 50 \times 16 \times 1.07\left(1 + \frac{3(15.9)(1.72)^2}{16(30)(1.07)}\right)\left(1.9 - \frac{1.2(1666)}{2055}\right)$$

$$= 513.6(1.075)(0.927) = 511.9$$

$\phi_v R_n = 512^k$.



$d = 30''$

0 braces carry shear.

$$\Sigma F_u = \frac{24\,972}{30} \times 2 + \cancel{V_u} = 1665 > \phi R_n \therefore$$

Doubler plates required.

Doublerplate Design
per. AISC 358-05 9.3

9.3b

$$t \geq [(30 - .85\times2) + (16 - 1.72\times2)] / 90$$

$$\geq 0.45''$$

Sizing doubler plates.

$$V_{udp} = \Sigma F_u - \phi R_{vc} = 1665 - 512 = 1153 \text{ kips.}$$

$$h = 16 - 1.72\times2 = 12.56.$$

use Cv=1 for now

$$1153 = 0.6\times50\times12.56\times t_{dp} \times 1$$

$$t_{dp} = 3.07''$$

$$\frac{12.56}{3.07} = 4.09 \leq 1.1\sqrt{\frac{5.81\times29E3}{50}} = 63.85 \therefore \checkmark$$

use Cv=1.

Stiffened web ∴

$$K_v = 5 + \frac{5}{\left(\frac{30-1}{11.4}\right)^2} = 5.81$$

$$h = 10.7\times1.07 = 11.4$$

$$\frac{a}{h} = \frac{30-21.85)}{11.4} = 2.5$$

$t_{dp} = 3.07''$ use $2(1\frac{1}{8})$ doubler plates.

Design Shear tab connection.

$V_u = 93^{kips}$ from beam outside link.

$V_{uB} = 274$ kips. from gusset - Brace.

Using CJP to connect Beam web plate w column flange.

Check Beam web strength.

$\phi R_n = 1 \times 0.6 \times 50 \times .565 \times d_{min} \geq (274 + 93)$

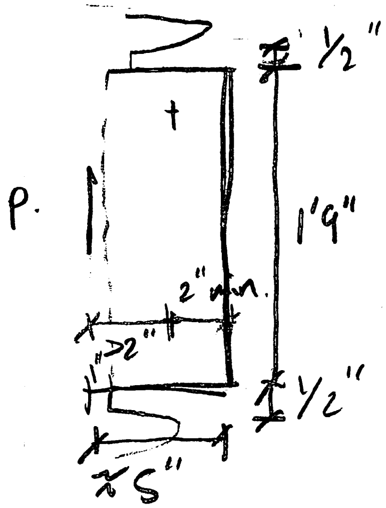
$d_{min} = 21.65''$

Check Access hole dimentions.

$d = 30 - (3/8 + 3/4 + .85) \times 2 - 1$

$= 25.05 > d_{min}$ ∴ ✓

Shear tab weld.



using table 8-9. SM.

$l = 21''$

$k = 4/21 = 0.19$.

$x \approx 0.029$

$xl = 21 \times 0.029 = 0.609$

$al = a \times 21 = 5 - 0.609 \Rightarrow a = 0.209$.

∴ $C = 2.63$

$P = V_n = 0.9 \times 0.6 \times 50 \times 5/8 \times 21 = 354^{\text{kips}}$.

Shear tab $t_p = 5/8''$ plate $> t_w$ beam $\therefore$

$C_{min} = \dfrac{354}{.75(1)(D)(21)} = 2.63 \Rightarrow D = 8.54$ 16th

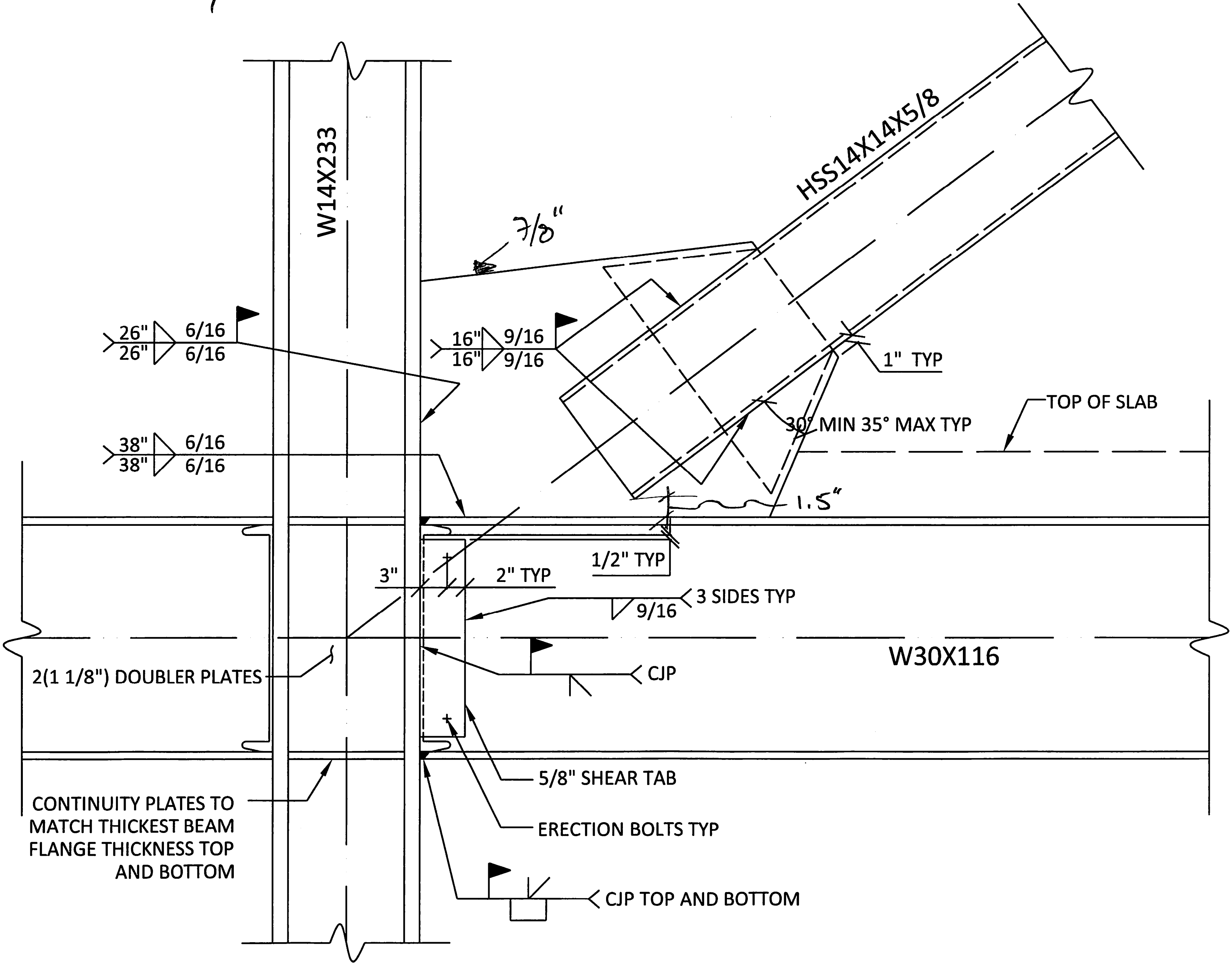
use $9/16^{\text{ths}}$

$\therefore$ t_w max $= 5/8 - 1/16 = 9/16$.

t_w min $= 1/4''$

use $\underline{\underline{9/16^{\text{ths}}}}$.

Design Summary



Link Lateral Bracing Connection

Link Lateral Bracing

Link Lateral Bracing Connection:

$A_g = 7.13\ in^2$

Brace Shape: L6x6x5/8 A36.

Web Stiffener: $t_p = 3/8''$

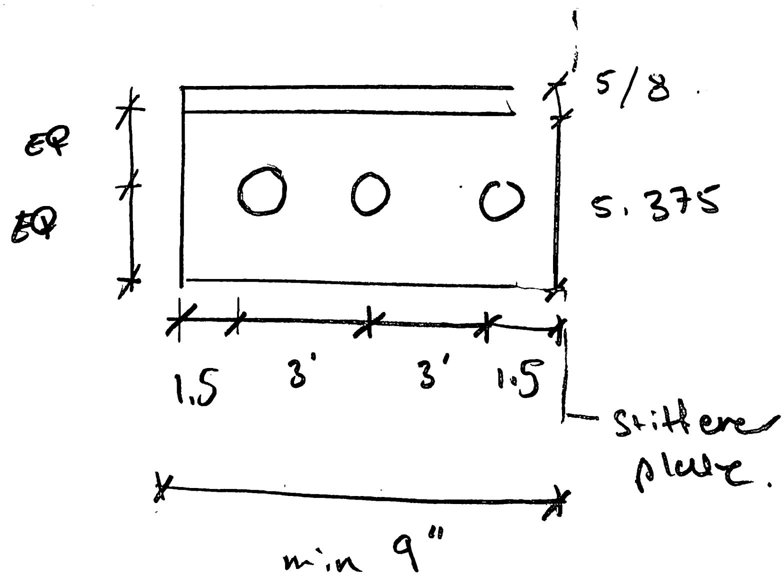
Assume A36 for now. $d \approx 20''$ ← worst case

$P_u = \underline{\underline{58}}^{\text{kips}}$

Design for bolted connection.

Limit States

- Bolt shear.
- Bearing on Angle & stiffener.
- Tearout - Edge Bolts
  other bolts.
- Angle Tension yield
- Angle Tension Rupture
- Block shear. Angle
- web stiffener tension Rupture
- " " " yield.



try A325N
7/8" (3)

Bolt shear. table 7-1 SM

$\phi R_n = 21.6 \times 3 = 64.8$ kips. $> P_u \therefore \checkmark$

Bearing Strength on L shape. STD holes. table 7-5 SM

$\phi r_n = 91.4$ k/in Bolt Spacing

$\phi r_n = 40.8$ Bolt Edge (use 1.25" conservative)

$\phi R_n = (2(91.4) + 40.8)\, 5/8 = 139.75 > P_u \therefore \checkmark$

Bearing on plate. plate.

$\phi r_n = (2(91.4) + 40.8) \times 3/8 = 84 > P_u \therefore \checkmark$

Tear out. Edge bolts.

$L_c = 1.5 - (7/8 + 1/16)/2 = 1.03$

$\phi r_{nv} = .75\,(1.2) \times 1.03 \times 58 \times 3/8 = 20.2^k$

Interior bolts.

$L_c = 3 - (7/8 + 1/16) = 2.06$

$\phi r_{nv} = 0.75 \times 1.2 \times 2.06 \times 58 \times 3/8 = 40.32$

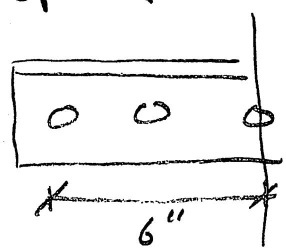
$\phi R_{nv} = 40.32 + 20.2 = 60.52^k > P_u$

Angle Tension yeilding.

$\phi R_n = \phi F_y A_g = 0.9 \times 36 \times 7.13 = 231 > P_u \therefore ✓$

Angle tension Rupture

$L = 6''$



$\bar{x} = 1.72$

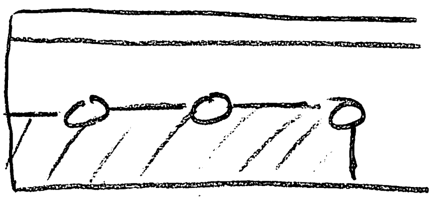
Shear lag.

$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.72}{6} = 0.713$

$A_n = 7.13 - 5/8 \times (7/8 + 1/8) = 6.505\ in^2$

$\phi R_n = .75 \times 58 \times 0.713 \times 6.505 = 202 > P_u \therefore ✓$

Angle Block shear



BS Tension Rupture table 9-3a

$\phi r_n = 43.5$ k/in

BS sher yeild table 9-3b.

$\phi r_n = 121$ k/in $\Leftarrow$ controls.

BS Sher Rupture

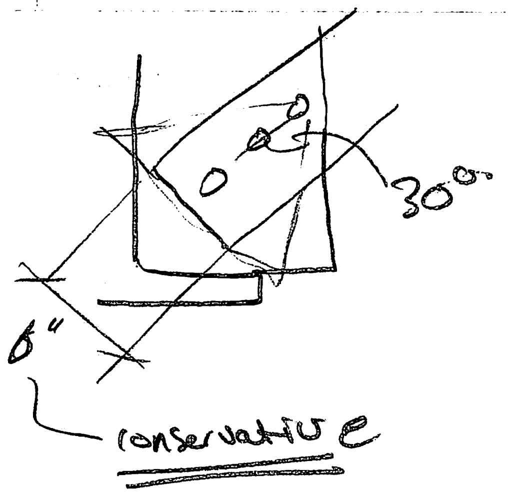
$\phi r_n = 131$ k/in.

$\phi R_n = (43.5 + 121)\, 5/8 = 103 > P_u \therefore \checkmark$

Stiffener Tension Rupture

$\phi R_n = .75 \times 58 \times 3/8 \times 6$

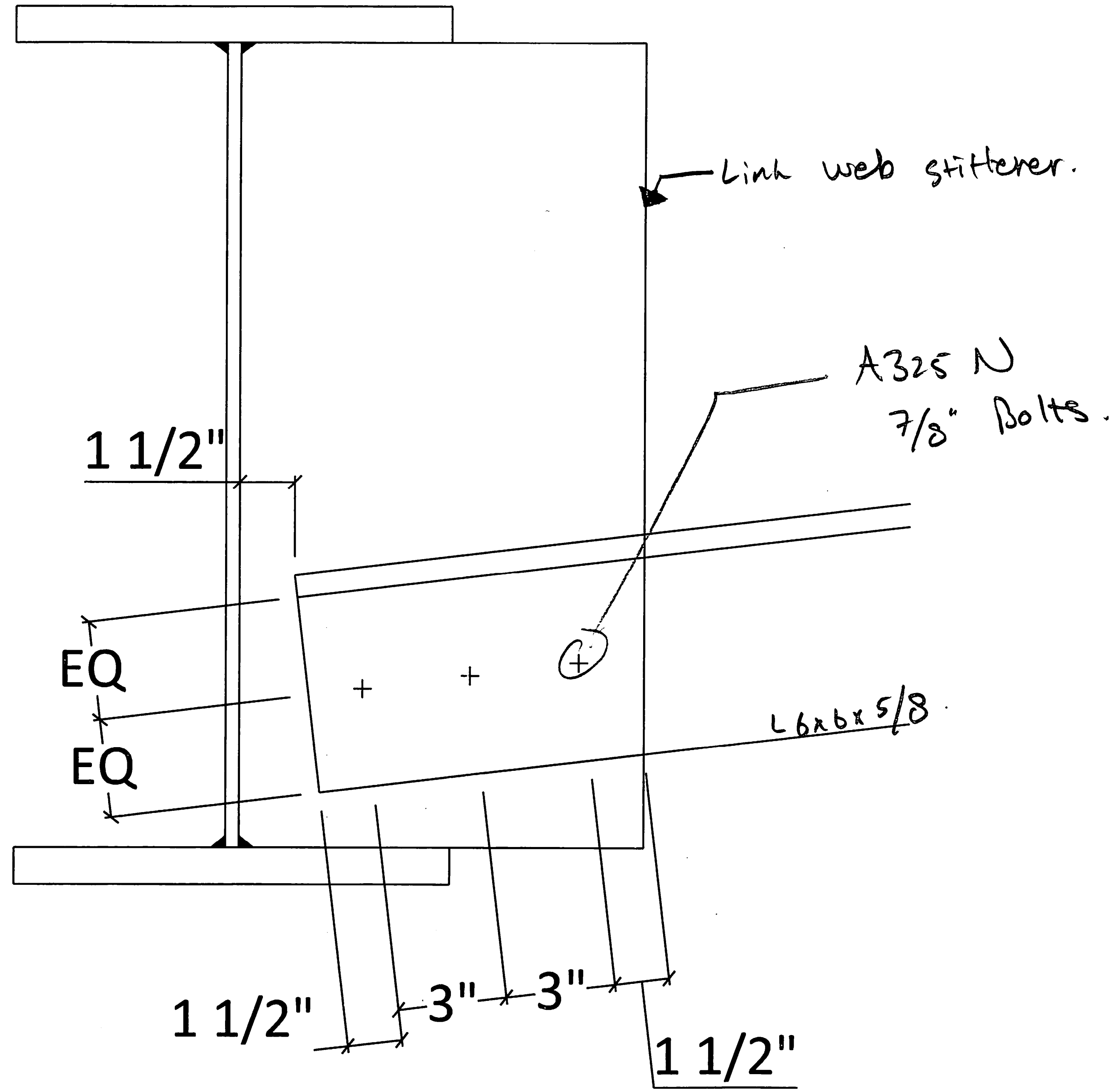
$= 97.9 > P_u \therefore$ ✓



stiffener tension yeild

$\phi R_n = 0.9 \times 36 \times 3/8 \times 6$

$= 73^k > P_u \therefore$ ✓



Link web stiffener.
A325 N
7/8" Bolts.
1 1/2"
EQ
EQ
L6x6x5/8
1 1/2"
3"
3"
1 1/2"

Column Splice Connection

Column Splice Connection per AISC 341-05 Sec. 8.4

W14x233

$d = 16$

$b_f = 15.9$

$t_f = 1.72$

$t_w = 1.07$

W14x173

$d = 15.2$

$b_f = 15.7$

$t_f = 1.31$

$t_w = .83$

upper colum loading.

unbraced length = 15'

$P_D = 135^{k}$.

$P_L = 146.^{k}$

$P_{QE} = \Sigma\, 1.1 R_y V_n = 900^{k}$

determine factored loadings.

$P_u = 1.38 \times 135 + 900 + 146 \times 1 = 1232$

$T_u = (.9 - .18) \times 135 - 900 = -803$ kips.

Determine required flexural strength of splice

$$M_n = \frac{4055.6}{.9} = 10{,}062^{k\text{-}in}.$$

$$R_u = \frac{10{,}062}{15.2 - 1.31} = 724 \text{ kips.}$$

Design strength @ CJP weld.

$\phi R_n = .9 \times 50 \times 15.7 \times 1.31 = 926$ kips.

$\phi R_n > R_u \therefore$ ✓

Determine the required strength of splice

W14x233 $\quad M_n = \frac{12277}{.9} = 13{,}641$ k-in.

$$\Sigma M_{uc} = \frac{13{,}641 + 10{,}064}{15' \times 12} = 132^k.$$

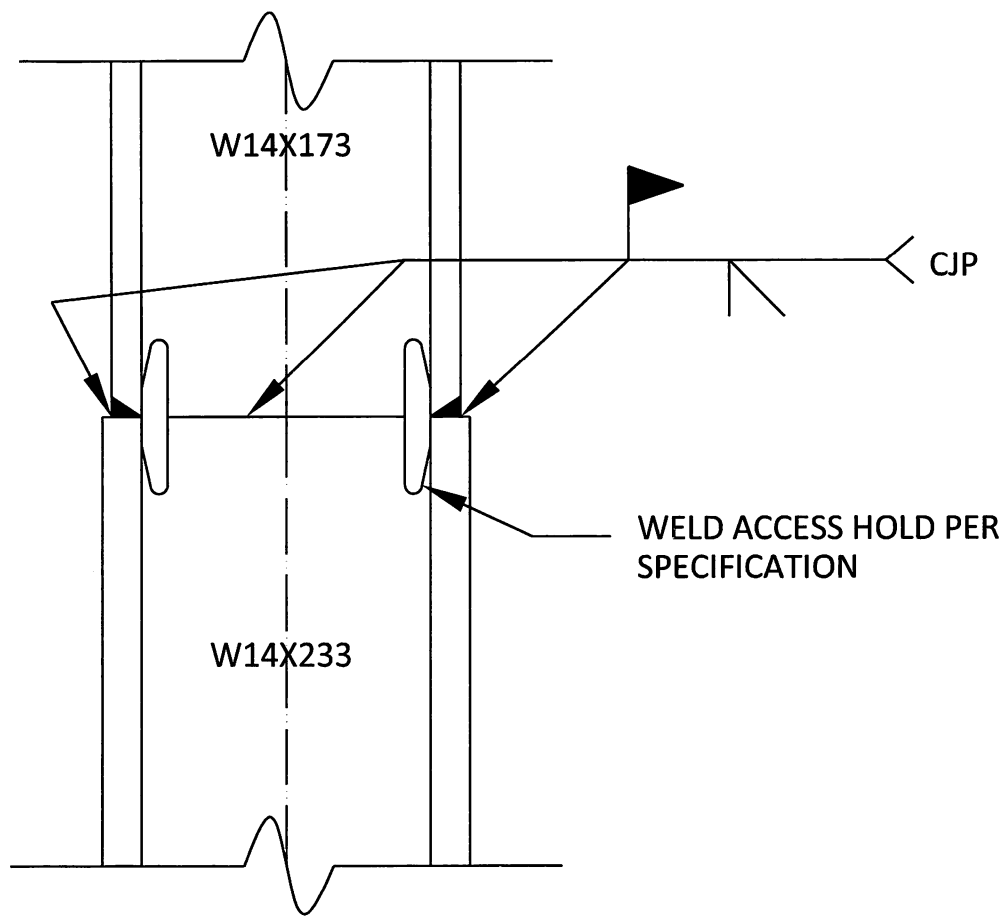
Access hole $\quad 1.5(t_w) = 1.5(.83) = 1.245''$

weld @ web.

$$\phi R_n = 0.9 \times 0.6 \times 50 \times .83 \left[15.2 - 2(1.31) - 2(1.245)\right]$$

$$= 226 > 132^k \therefore \checkmark$$

Use CJP @ column webs.



ERECTION AIDS NOT SHOWN

Miscellaneous Calculations

Building Weight

Weight of Building

Floor	Component	Wt. (psf)	Area (SF)	Wt. (Kips)		
		LVL 1				
	Beams			231		
	Girders			168		
	Columns			132		
	Curtain Wall			218		
	Composite Deck	75	57000	4275		
	Paritions	20	57000	1140		
	Super Imposed	12	57000	684	Wt.	Mass
			Total Weight	6848	128	3.98
		LVL 2			*psf*	
All of level 2						
	Beams			231		
	Girders			168		
	Columns			132		
	Curtain Wall			218		
	Composite Deck	47	37000	1739		
	Paritions	20	37000	740		
	Super Imposed	12	37000	444		
Courtyard						
	Composite Deck	94	7100	667.4		
	SI & Patio w/ Pavers	102	5900	601.8		
	SI Tree	552	320	176.64		
	SI Planters	342	861	294.462		
Lvl 2 Roof & Future Green roof						
	Composite Deck	47	7200	338.4		
	Super Imposed	222	7200	1598.4		
	Topping	18	7200	129.6		
LVL 2 Roof						
	Composite Deck	47	6500	305.5		
	Super Imposed	22	6500	143		Mass
			Total Weight	7927	148	4.61
		LVL 3			*psf*	
	Beams			151		

Girders			94		
Columns			180		
Curtain Wall			218		
Composite Deck	47	37000	1739		
Paritions	20	37000	740		
Super Imposed	12	37000	444		Mass
		Total Weight	3566	96	2.99
				psf	

LVL 4

Beams			151		
Girders			94		
Columns			180		
Curtain Wall			218		
Composite Deck	47	37000	1739		
Paritions	20	37000	740		
Super Imposed	12	37000	444		Mass
		Total Weight	3566	96	2.99
				psf	

ROOF

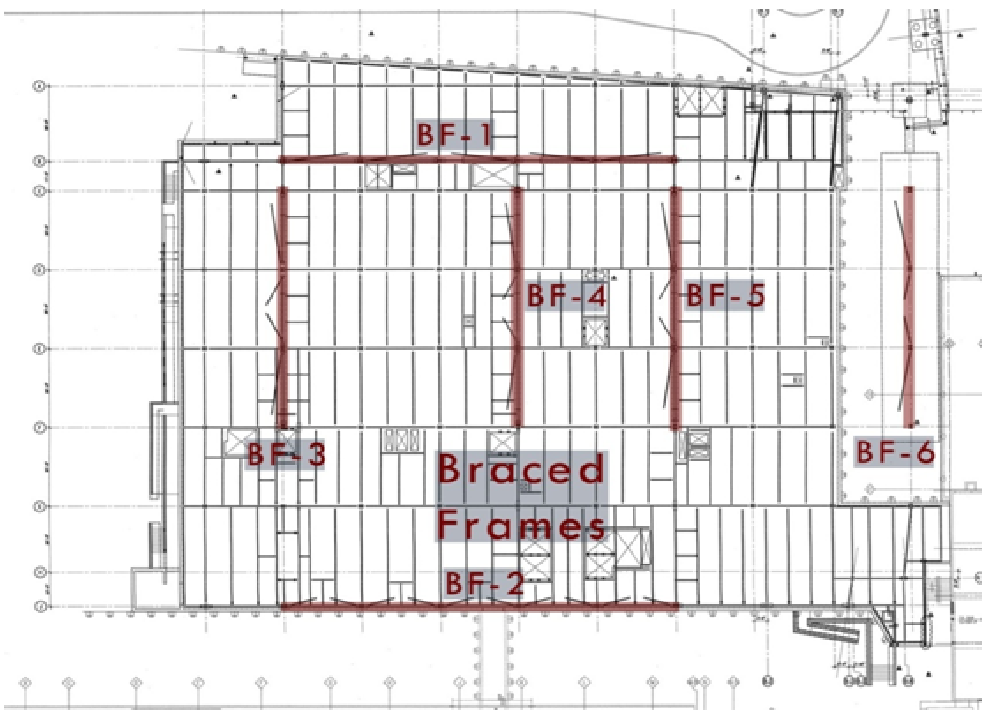
	Beams			234		
	Girders			116		
	Composite Deck	97	37000	3589		
	Super Imposed	25	3700	92.5		
	AHU	25	7200	180		
				0		
Pent House	*Pent House*	170	620	105.4		Mass
			Total Weight	4317	117	3.62
					psf	

Building Weight	19376	kips

Dead & Live Loads on Lateral System

Dead and Live Loadings

	Dead (klf)					Live (klf)				
Brace	lvl1	lvl2	lvl3	lvl4	Roof	lvl1	lvl2	lvl3	lvl4	Roof
BF-1	2.36	1.74	1.74	1.74	2.58	1.60	1.60	1.60	1.74	3.00
BF-2	0.77	0.57	0.57	0.57	0.84	0.52	0.52	0.52	0.57	0.98
BF-3	1.18	2.93	0.87	0.87	1.29	0.80	0.50	0.80	0.87	1.50
BF-4	1.18	2.06	0.87	0.87	1.29	0.80	1.00	0.80	0.87	1.50
BF-5	1.18	2.06	0.87	0.87	1.29	0.80	1.00	0.80	0.87	1.50
BF-6	0.00	0.44	0.44	0.44	0.65	0.00	0.40	0.40	0.44	0.75



ETABS Load Combinations

The following is a list of load combinations that were imported into the ETABS model. Note that EQTX and EQTY are the moments due to torsion based on the code provisions in ASCE 7-05 Section 12.8.4.3.

Load Combination Name	Load Combination
EQ1	1.38D + EQX + EQTX + L
EQ2	1.38D - EQX + EQTX + L
EQ3	1.38D + EQX - EQTX + L
EQ4	1.38D - EQX - EQTX + L
EQ5	1.38D + EQY + EQTY + L
EQ6	1.38D - EQY + EQTY + L
EQ7	1.38D + EQY - EQTY + L
EQ8	1.38D - EQY - EQTY + L
EQ9	.72D + EQX + EQTX
EQ10	.72D - EQX + EQTX
EQ11	.72D + EQX - EQTX
EQ12	.72D - EQX - EQTX
EQ13	.72D + EQY + EQTY
EQ14	.72D - EQY + EQTY
EQ15	.72D + EQY - EQTY
EQ16	.72D - EQY - EQTY

Other Calculations

Refer to next page.

Weight of Structural steel Calculations.

1st $(231 + 168 + 132) 1000 / 252,712 = 2.1$ psf.

2nd $(231 + 168 + 132) 1000 / 252,712 = 2.1$ psf.

3rd $(151 + 94 + 180) E3 / 252,712 = 1.68$

4th $= 1.68$

Roof. $(234 + 116) E3 / 252,712 = 1.38$

Braces. $(121 \times 2) E3 / 252712. = 0.96.$

10 psf.

Lateral system.

SCBF. $637 \times 2 \times E3 / 252,712 = 5$ psf.

EBF. $329 \times 2 \times E3 / 252,712 = 2.6.$ psf.

gravity. $5 - 5 = 5$ psf.

Builtup. 4

$d = 24$ $b_f = 12$ $t_f = 1$

$t_w = 3/8''$ $A = 32.25$

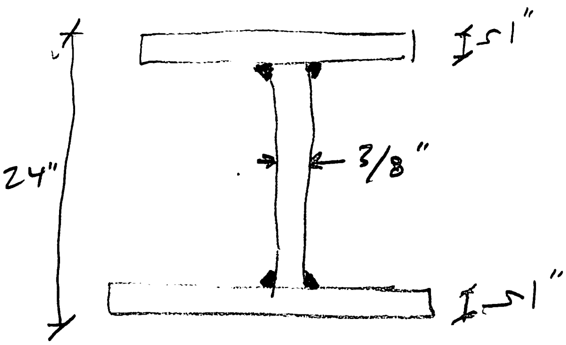
$J = 7.96$

$I_x = 3509$ $I_y = 288$

$S_x = 292$ $S_y = 48$

$Z_x = 321$ $Z_y = 72.8$

$r_x = 10.4$ $r_y = 2.99$



$C_w = 38{,}088$. $h/t_w = 61.33$ $w_{self} = 110$ lb/ft

$$C_w = \frac{(24-)^2 \times 12^3 \times 1}{24} = 38{,}088$$

Slenderness check

$$\frac{h}{t_w} = \frac{23}{.375} = 61.33 \quad \therefore < 62.8 \therefore \text{Slenderness ratio } \underline{\underline{ok}}$$

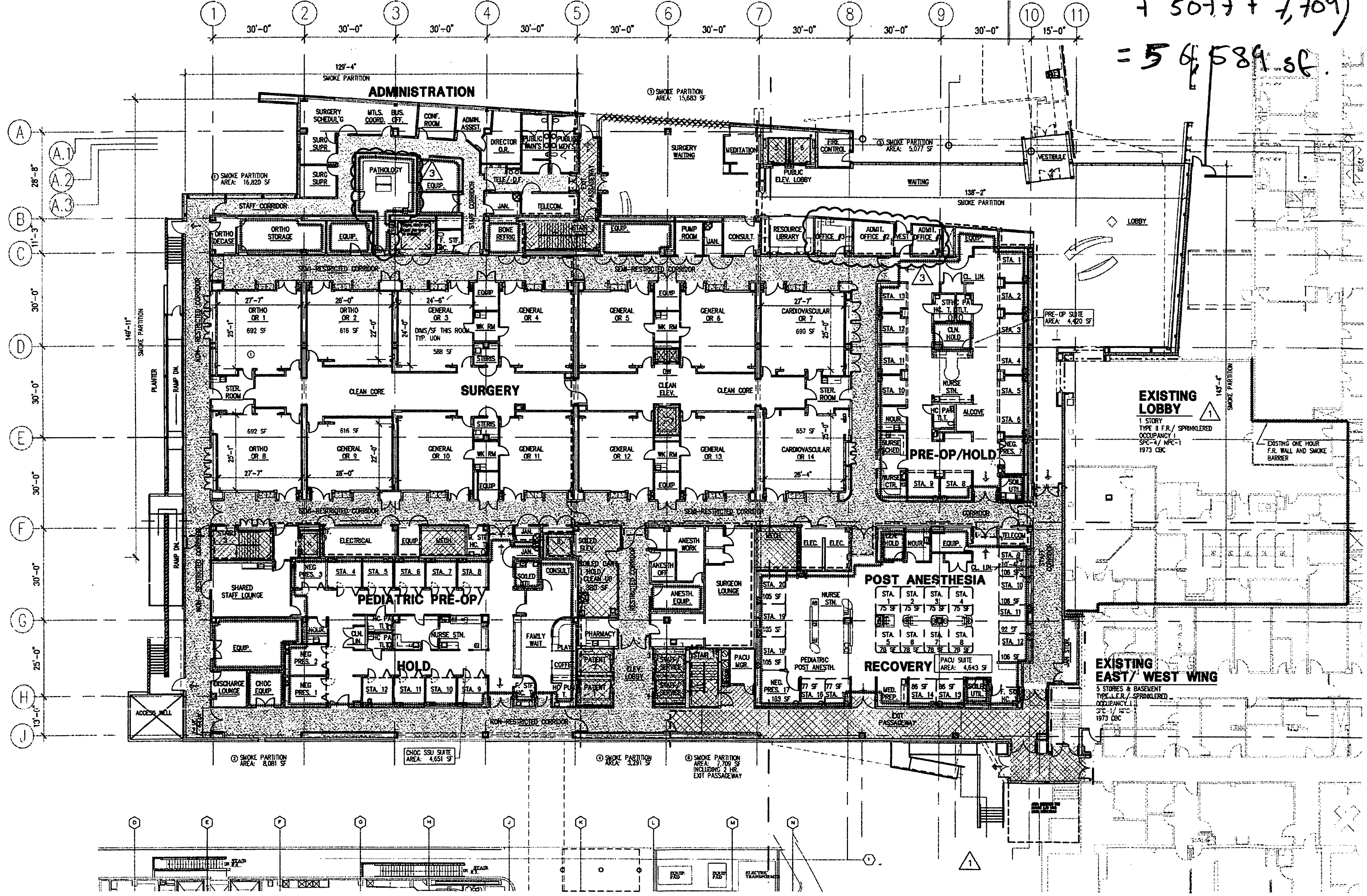
$$\frac{b_f}{2t_w} = \frac{12}{2(1)} = 6 \quad < 7.22 \therefore \checkmark$$

$w_{self} = 32.25 \times 2.83E{-}4 \times E3 \times 12 = \underline{\underline{110}}$ plf

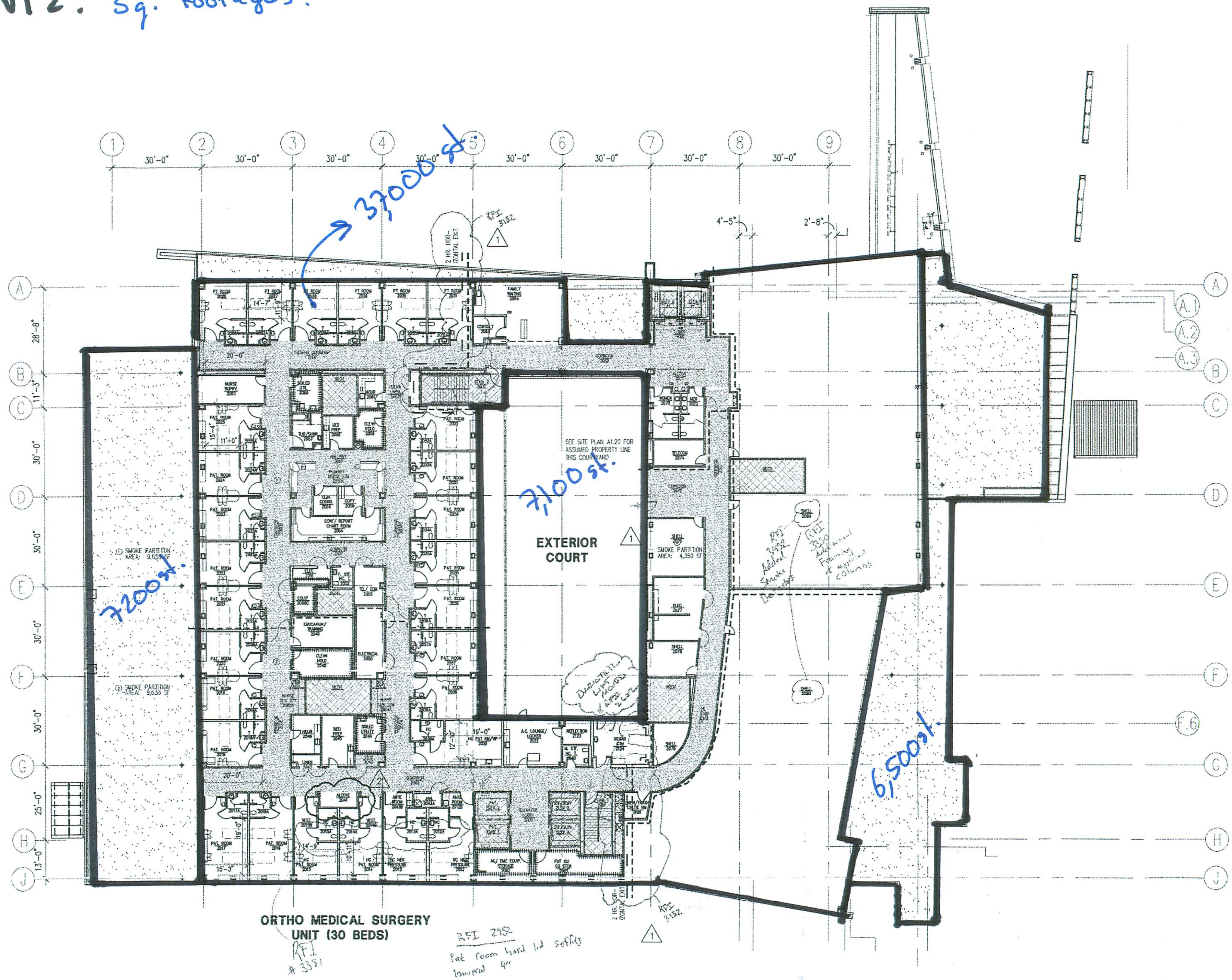
1011 Sq. footages.

$\Sigma(16,820 + 8,081 + 15,683 + 3291 + 5077 + 7,709)$

$= 56,589$ sf.

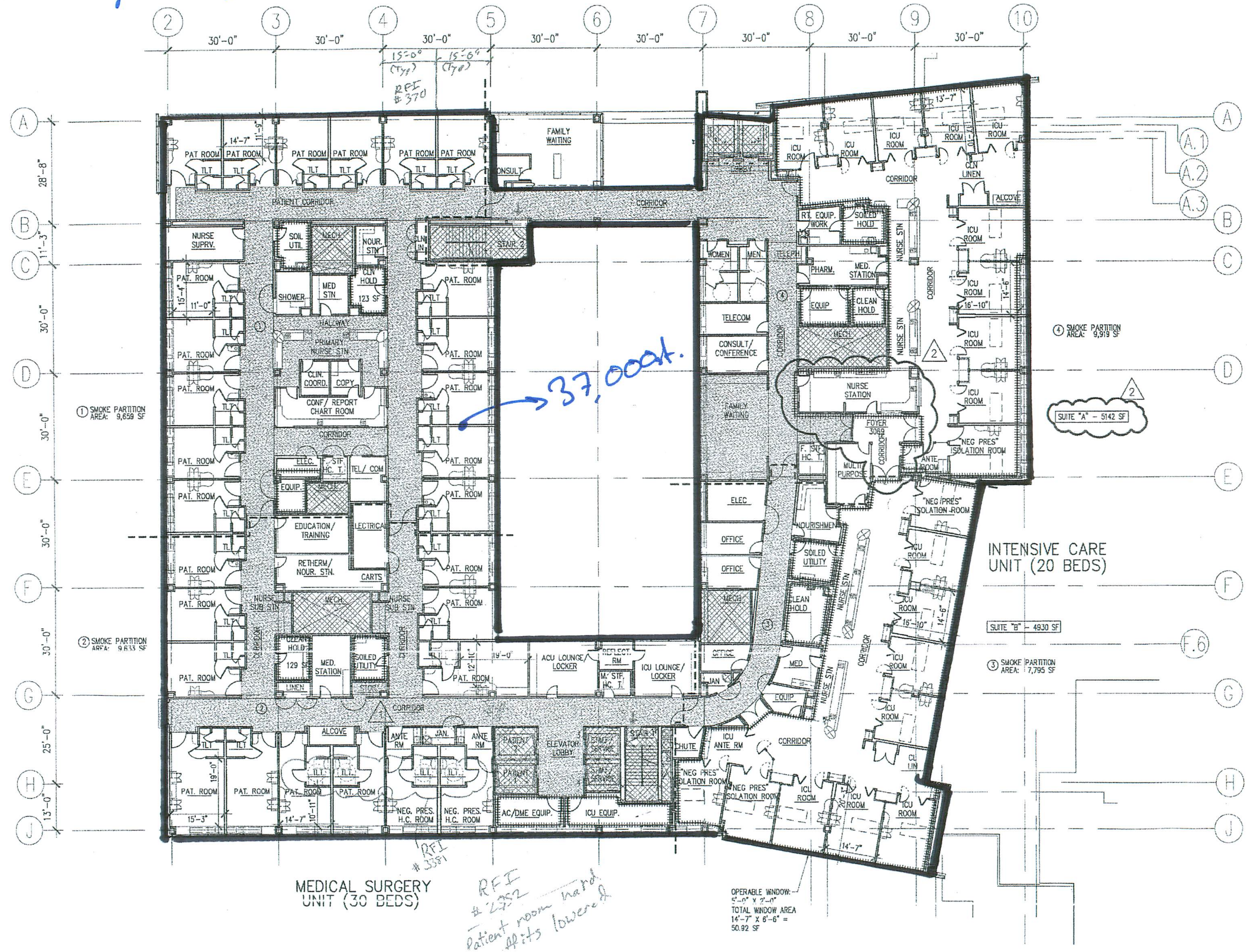


lvl 2. sq. footages.

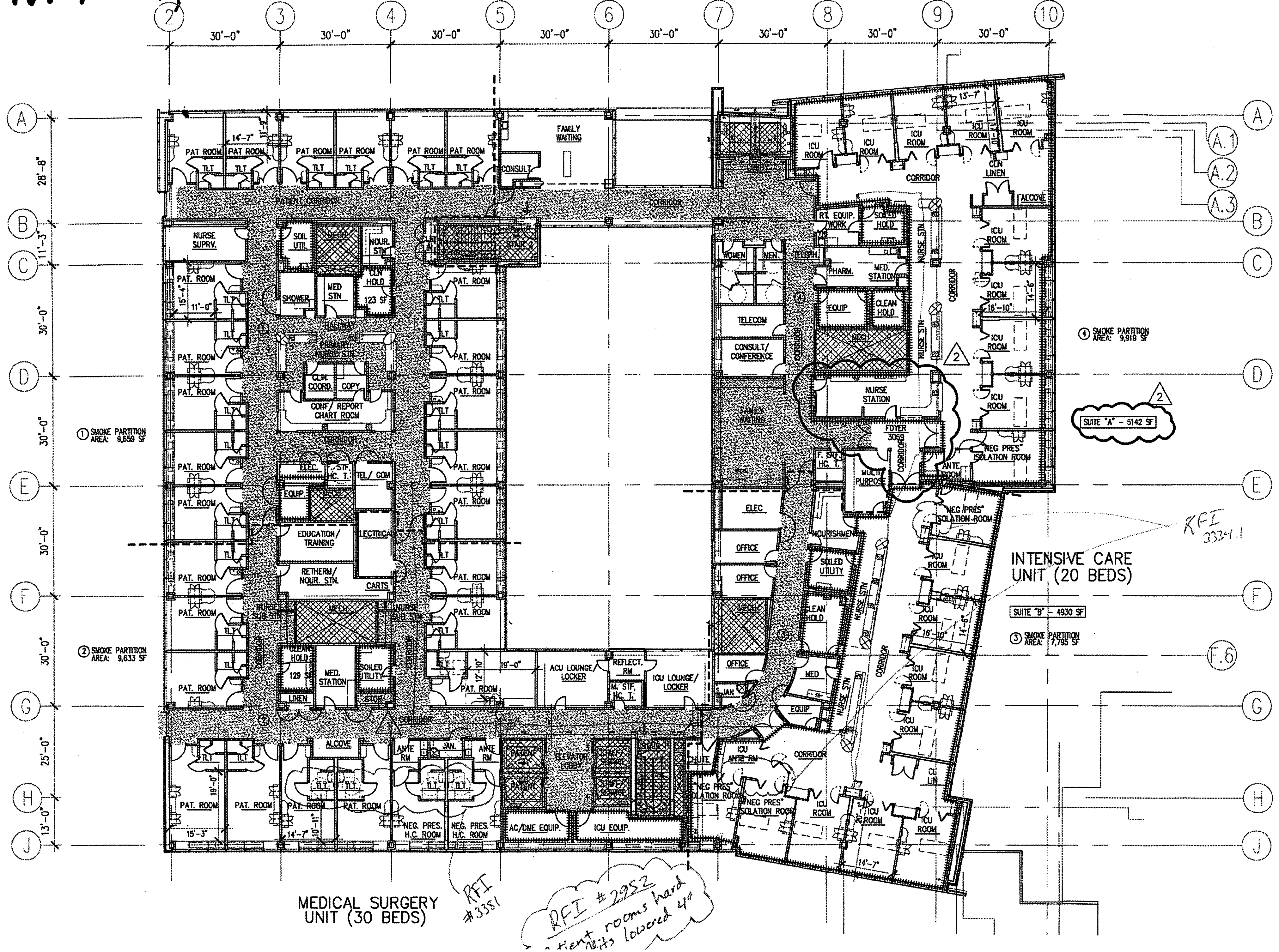


ORTHO MEDICAL SURGERY UNIT (30 BEDS)

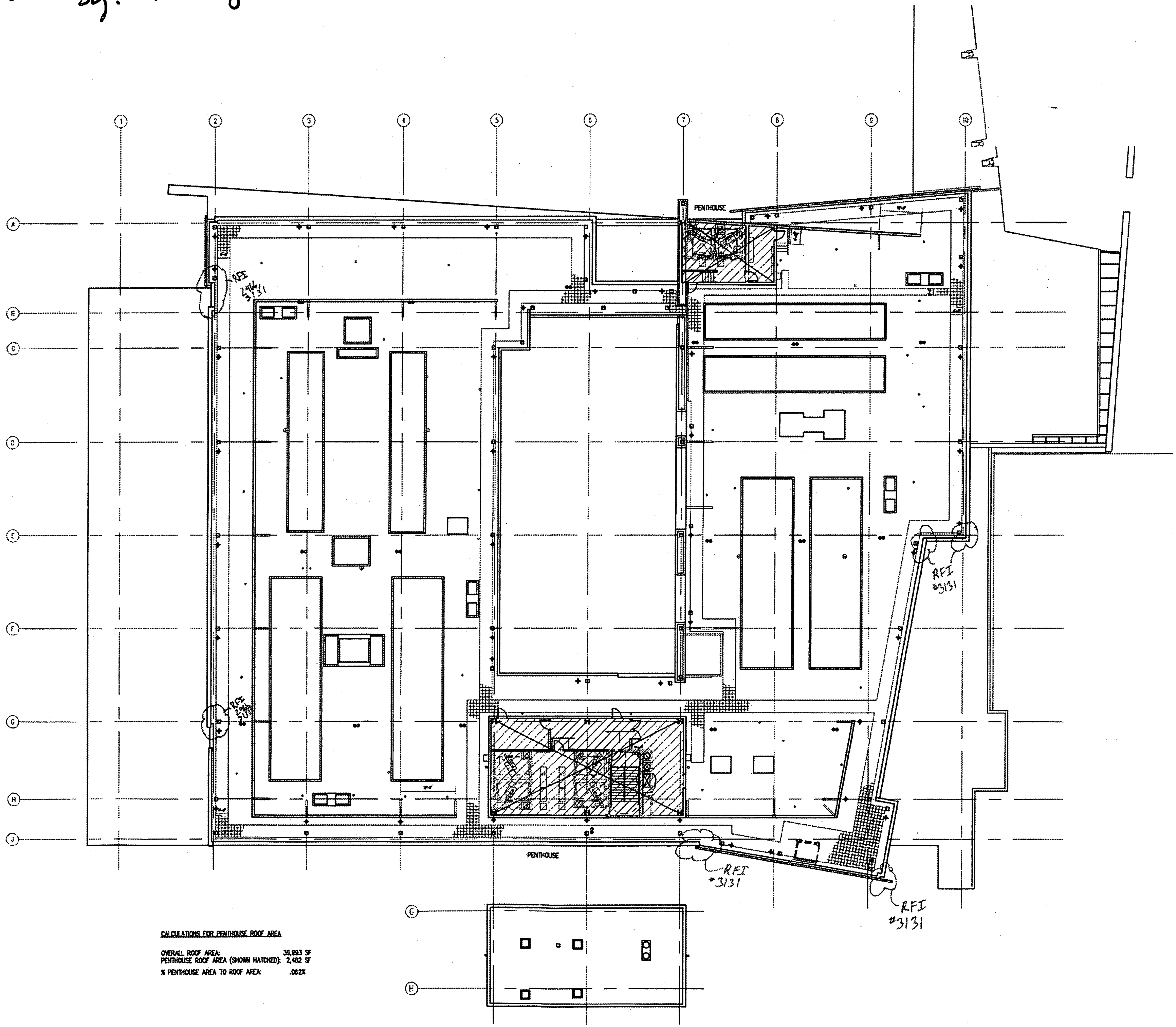
lvl 3 sq. footages



lvl 4 sq. footages see lvl 3.



Roof. sq. footages. see lvl 3.



<u>CALCULATIONS FOR PENTHOUSE ROOF AREA</u>

OVERALL ROOF AREA: 39,893 SF
PENTHOUSE ROOF AREA (SHOWN HATCHED): 2,482 SF
% PENTHOUSE AREA TO ROOF AREA: .062%

Appendix B.

Construction Management Breadth

Tonnage of Steel Calculations

Eccentrically Braced System					
Steel Grade	**MEMBER**	**LENGTH**	**SELF WT.**	**#**	**TONAGE**
A992	W14X176	40	176	16	56.3
	W14X233	23	233	2	4.9
	W14X233	39	233	14	57.7
	W24X103	30	103	4	5.6
	W24X146	30	146	4	7.9
	W30X99	30	99	3	4.0
	W30X116	30	116	6	9.5
	W30X148	30	148	4	8.1
	BUILTUPX110	30	110	16	26.4
				SUM	**180.4**
A500B	HSS10X10X5/8	20	76	36	27.4
	HSS10X10X5/8	22	76	4	3.3
	HSS14X14X5/8	22	110	8	9.7
	HSS14X14X5/8	28	110	12	18.5
	HSS14X14X5/8	39	110	4	8.6
				SUM	**67.4**
A36	L6X6X5/8	10	24.2	28	3.4
				SUM	**3.4**
Gravity System					
Steel Grade	**MEMBER**	**LENGTH**	**SELF WT.**	**#**	**TONAGE**
A992	W14X99	40	99	12	23.8
	W14X145	39	145	12	33.9
	W24X68	30	68	20	20.4
				SUM	**78.1**

Concentrically Braced System					
Steel Grade	**MEMBER**	**LENGTH**	**SELF WT.**	**#**	**TONAGE**
A992	W14X90	33.5	90	5	7.5
	W14X109	33.5	109	5	9.1
	W14X132	33.5	132	5	11.1
	W14X145	23	145	1	1.7
	W14X145	35	145	8	20.3
	W14X145	39	145	14	39.6
	W14X145	40	145	14	40.6
	W14X159	40	159	2	6.4
	W14X211	18	211	1	1.9
	W14X211	35	211	5	18.5
	W14X211	39	211	2	8.2
	W14X211	40	211	2	8.4
	W14X257	40	257	8	41.1
	W14X283	23	283	1	3.3
	W14X311	39	311	2	12.1
	W14X398	39	398	7	54.3
	W30X99	30	99	38	56.4
	W30X108	30	108	3	4.9
	W30X116	30	116	11	19.1
	W30X148	30	148	3	6.7
	W30X173	30	173	6	15.6
	W30X211	30	211	9	28.5
	W30X235	30	235	22	77.6
	W30X391	30	391	6	35.2
	W36X256	30	256	8	30.7
				SUM	**559**
A500B	HSS8X8X5/8	21	59	96	59.5
	HSS10X10X5/8	23	76	18	15.7
				SUM	**75**
A36	L6X6X5/8	10	24.2	28	3.4
				SUM	**3.4**

Cost and Schedule Estimate Spread Sheet

RS Means Detailed Cost Estimate

Name	ID	Type	Amount	Crew	LH	Daily Output	Labor Hours	Units	Mat.	Labor	Equip.	Total	Total Incl O&P	Mat.
Gravity System	800	Hospitals Steel Bearing 3 to 6 Stories	632.0	E-5	80	14.4	8.889	Ton	2250	375	130	2812	3350	1422000
EBF Structural System	800	Hospitals Steel Bearing 3 to 6 Stories	329.3	E-5	80	14.4	8.889	Ton	2250	375	130	2812	3350	740955.7
SCBF Structural System	800	Hospitals Steel Bearing 3 to 6 Stories	637	E-5	80	14.4	8.889	Ton	2250	375	130	2812	3350	1433906
Strip Footings 60' Long														
Formwork, 4 uses	5150	Spread Footings, job built lumber, 4 use	300	C-1	32	414	0.077	SFCA	0.63	2.79	0	3.42	5.05	189
Reinforcing, fy=60,000	550	Footings, #8 to #18	8.8	4 Rodm		3.6	8.889	Ton	840	380	0	1220	1550	7392
Concrete, f'c = 4000	300	Normal Weight 4000 psi Conc	156					CY	106	0	0	106	117	16536
Placing Concrete, Direct Chute	2650	Footings Over 5 CY Pimped	156	C-7	72	100	0.72	CY	0	23.5	11.9	35.4	49	0
Screed Finish		Screed Finish	1140					SFCA	0	0.92	0	0.92	1.288	0
Shear Walls 24" Thick 60' Long														
Forms in Place, 4 uses		Walls, 8' to16' high 4 use	1920	C-2	48	395	0.122	SFCA	2.66	7.6	0	10.26	14.75	5107.2
Reinforcing in Place, Walls	750	Walls, #8 to #18	2.88	4 Rodm		4	8	TON	890	345	0	1235	1550	2563.2
Concrete Ready mix, regular weight, 4000psi	300	Normal Weight 4000 psi Conc	72			0	0	CY	106	0	0	106	117	7632
Place and Vibrate Concete, 14" Thick, pumped	5250	Wall 15" Pumped	72	C-20	64	120	0.533	CY	0	17.35	6.5	23.85	33.5	0
Finish Wall, Break Ties, Patch Voids			60					LF	0.12	2.48	0	2.6	3.64	7.2
Shear Walls 14" Thick 60' Long														
Forms in Place, 4 uses		Walls, 8' to16' high 4 use	1920	C-2	48	395	0.122	SFCA	2.66	7.6	0	10.26	14.75	5107.2
Reinforcing in Place, Walls	750	Walls, #8 to #18	2.88	4 Rodm		4	8	TON	890	345	0	1235	1550	2563.2
Concrete Ready mix, regular weight, 4000psi	300	Normal Weight 4000 psi Conc	42			0	0	CY	106	0	0	106	117	4452
Place and Vibrate Concete, 24" Thick, pumped	5250	Wall 15" Pumped	42	C-20	64	120	0.533	CY	0	17.35	6.5	23.85	33.5	0
Finish Wall, Break Ties, Patch Voids		Finish Wall, Break Ties, Patch Voids	60					LF	0.12	2.48	0	2.6	3.64	7.2
Gravity Foundation 14'x14' Footing														
Forms in Place, 4 uses	6150	12'x12' footing interpolate to 14'x14' footing	1	C-1	32	17	1.882	Ea.	34.2	81.6	0	115.8	165.6	34.2
Reinforcing in Place, Footings		Footings, #4 to #7	0.25	4 Rodm		2.1	15.238	Ton	890	655	0	1545	2050	222.5
Concrete Ready mix, regular weight, 4000psi	300	Normal Weight 4000 psi Conc	22			0	0	CY	106	0	0	106	117	2332
Placing Concrete, Direct Chute	2650	Footings Over 5 CY Pimped	22	C-7	72	100	0.72	CY	0	23.5	11.9	35.4	49	0
Screed Finish		Screed Finish	364					SFCA	0	0.92	0	0.92	1.288	0

RS Means Detailed Cost Estimate

Name	Labor	Equip.	Total	Total Incl O&P	w/ Factors	Construction Days	Man Hours	ΣDays	ΣHours	ΣCost
Gravity System	$237,000.00	$82,160.00	$1,777,184.00	**$2,117,200.00**	**$2,292,927.60**	43.89	3511.11	**44**	3512	**$2,292,927.60**
EBF Structural System	$123,492.62	$42,810.78	$926,030.00	$1,103,200.75	$1,194,766.41	22.87	1829.52	23	1830	**$1,194,766.41**
SCBF Structural System	$238,984.41	$82,847.93	$1,792,064.40	**$2,134,927.36**	**$2,312,126.33**	44.26	3540.51	**45**	3541	**$2,312,126.33**
Strip Footings 60' Long										
Formwork, 4 uses	$837.00	$0.00	$1,026.00	$1,515.00		0.72	23.19			
Reinforcing, fy=60,000	$3,344.00	$0.00	$10,736.00	$13,640.00		2.44				
Concrete, f'c = 4000	$0.00	$0.00	$16,536.00	$18,252.00						
Placing Concrete, Direct Chute	$3,666.00	$1,856.40	$5,522.40	$7,644.00		1.56	112.32			
Screed Finish	$1,048.80	$0.00	$1,048.80	$1,468.32						
			SUM	**$42,519.32**	**$46,048.42**	**4.73**		**29**		**$276,290.54**
Shear Walls 24" Thick 60' Long										
Forms in Place, 4 uses	$14,592.00	$0.00	$19,699.20	$28,320.00		4.86	233.32			
Reinforcing in Place, Walls	$993.60	$0.00	$3,556.80	$4,464.00		0.72				
Concrete Ready mix, regular weight, 4000psi	$0.00	$0.00	$7,632.00	$8,424.00						
Place and Vibrate Concete, 14" Thick, pumped	$1,249.20	$468.00	$1,717.20	$2,412.00		0.60	38.40			
Finish Wall, Break Ties, Patch Voids	$148.80	$0.00	$156.00	$218.40						
			SUM	**$43,838.40**	**$47,476.99**	6.18		31		**237384.936**
Shear Walls 14" Thick 60' Long										
Forms in Place, 4 uses	$14,592.00	$0.00	$19,699.20	$28,320.00		4.86	233.32			
Reinforcing in Place, Walls	$993.60	$0.00	$3,556.80	$4,464.00		0.72				
Concrete Ready mix, regular weight, 4000psi	$0.00	$0.00	$4,452.00	$4,914.00						
Place and Vibrate Concete, 24" Thick, pumped	$728.70	$273.00	$1,001.70	$1,407.00		0.35	22.40			
Finish Wall, Break Ties, Patch Voids	$148.80	$0.00	$156.00	$218.40						
			SUM	**$39,323.40**	**$42,587.24**	**5.93**		**6**		**$42,587.24**
Gravity Foundation 14'x14' Footing										
Forms in Place, 4 uses	$81.60	$0.00	$115.80	$165.60		0.06	1.88			
Reinforcing in Place, Footings	$163.75	$0.00	$386.25	$512.50		0.12				
Concrete Ready mix, regular weight, 4000psi	$0.00	$0.00	$2,332.00	$2,574.00						
Placing Concrete, Direct Chute	$517.00	$261.80	$778.80	$1,078.00		0.22	15.84			
Screed Finish	$334.88	$0.00	$334.88	$468.83						
			SUM	**$4,798.93**	**$5,197.24**	0.40		5		**$62,366.92**

Appendix C.

Lighting Breadth

Light Fixture Cut Sheets

Horizontal louver bollards with 360° light distribution

Type:
BEGA Product #:
Project:
Voltage:
Color:
Options:
Modified:

Post construction: One piece extruded aluminum, 3/16" wall thickness with one piece die cast aluminum top housing and base, internally welded into an assembly.

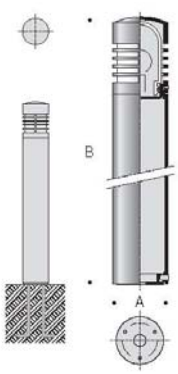
Lamp enclosure: Heavy wall, die cast aluminum cap with louver/guard secured by one (1) stainless steel set screw with a tamper resistant hex driver. Hand blown three-ply opal glass diffuser with screw neck. Fully gasketed for weather tight operation with a molded silicone rubber o-ring.

Electrical: Lampholders; Type G24q-1 rated 75W, 250V (13W) rated 75W, 600V. Ballasts are electronic universal voltage 120V thru 277V.

Anchor base: Heavy cast aluminum, slotted for precise alignment. Mounts to BEGA #895A anchorage kit. Bollards are secured to the base with one (1) socket head stainless steel screw.

Finish: These luminaires are available in five standard BEGA colors: Black (BLK); White (WHT); Bronze (BRZ); Silver (SLV); Eurocoat™ (URO). To specify, add appropriate suffix to catalog number. For complete description of BEGA finishing process, refer to technical information section at end of catalog. Custom colors supplied on special order.

U.L. listed, suitable for wet locations. Protection class: IP 65.



Bollards with diffused 360° light distribution and die cast aluminum horizontal louver guard. Three-ply opal diffuser. U.L. listed, suitable for wet locations. IP 65.
Color: Standard BEGA finishes.

Lamp				Lumen	A	B
8429P	Bollard	1	13W CF quad-4p	900	5½	39⅜
895A	Anchorage included					

BEGA/US 1000 BEGA Way, Carpinteria, CA 93013 [P] 805·684·0533 [F] 805·684·6682

Drive over buried luminaires with fluorescent light sources

Type:
BEGA Product #:
Project:
Voltage:
Color:
Options:
Modified:

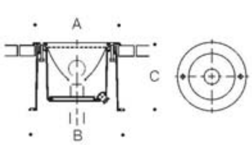
Enclosure: Outer housing: Constructed of high tensile strength, copper free die cast aluminum alloy.
Inner housing: Constructed of copper free die cast aluminum alloy.
Trim/Clamping ring is heavy gauge, machined stainless steel secured to inner housing by four (4) stainless steel threaded weld studs.
Relamping requires removal of inner housing/trim/clamping ring assembly from outer housing by means of two (2) flush socket head stainless steel access screws on trim.

Electrical: Flourescent lamholders are G23-2 rated 750W, 250V Ballasts are magnetic 1120V or 277V - specify. Inner housing pre-wired with three (3) feet of 18/3 waterproof cable, cable clamp, and waterproof cable gland entry into housing. A separate waterproof wiring box for power supply must be provided (by contractor).

Finish: Machined stainless steel. Custom colors are not available.

U.L. listed, suitable for wet locations and vehicle drive over. Protection class: IP 67.

Luminaires are designed to withstand loads of up to 11,000 lbs. at speeds up to 30 mph when installed on a proper foundation. Proper drainage must be provided.



Flush mounted uplights utilizing efficient fluorescent light sources. Machined, tempered glass diffuser, ¾" thick, with internal white ceramic coating.
U.L. listed, suitable for wet locations. IP 67.
Finish: Machined stainless steel.
Caution: The column "T" in this chart indicates the temperature in degrees C which is reached on the center of the glass.

		Lamp	Lumen	A	B	C	T
8703P	Symmetrical	1 9W CF quad-2p	525	6 5/16	7 7/8	7 1/2	35°

Suffix 'R' for optional skid resistant glass

BEGA/US 1000 BEGA Way, Carpinteria, CA 93013 [P] 805·684·0533 [F] 805·684·6682

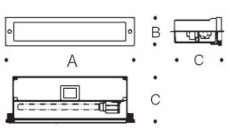
Recessed wall luminaires

Type:
BEGA Product #:
Project:
Voltage:
Color:
Options:
Modified:

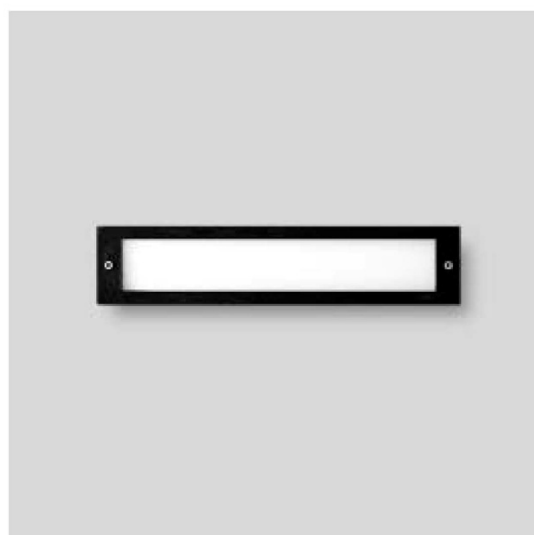
Housing: Constructed of die cast and extruded aluminum with integral wiring compartment. Mounting tabs provided.
Enclosure: One piece die cast aluminum faceplate, ⅛" thick. Clear tempered glass with translucent white ceramic coating. Faceplate is secured by two (2) socket head, stainless steel, captive screws threaded into stainless steel inserts in the housing casting. Continuous high temperature O-ring gasket for weather tight operation.

Electrical (Fluorescent): Lampholder; type G23 (9W) rated 75W, 250V. Ballasts are magnetic available 120V HPF or 277V NPF - specify. Through Wiring: All units are suitable for a maximum of four (4) No. 12 AWG conductors (plus ground) suitable for 75°C. Two ⅞" knockouts provided for ½" conduit.

Finish: These luminaires are available in five standard BEGA colors: Black (BLK); White (WHT); Bronze (BRZ); Silver (SLV); Eurocoat™ (URO). To specify, add appropriate suffix to catalog number. For complete description of BEGA finishing process, refer to technical information section at end of catalog. Custom colors supplied on special order.
U.L. listed, suitable for wet locations and for installation within 3 feet of ground. Suitable for all types of construction including poured concrete. Type non-IC. Protection class: IP 64



Recessed luminaires with white tempered glass diffusers.
U.L. listed, suitable for wet locations. IP 64.
Color: Standard BEGA finishes.
Opening: 9 13/32" x 2 7/16" x 4"



			Lamp	Lumen	A	B	C
2289P	Recessed	ADA	1 9W CF twin-2p	600	9½	2 9/16	4¼

BEGA/US 1000 BEGA Way, Carpinteria, CA 93013 [P] 805·684·0533 [F] 805·684·6682

COLOR KINETICS

PHILIPS

COLORBLAZE 48

The ColorBlaze® 48 fixture washes large areas with far-reaching, rich, saturated colors and color-changing lighting effects. The streamlined, four-foot black metal housing provides a simple yet powerful solution for large-area scenery and wash lighting for theaters, TV and video studios, concerts, events, casinos, and exhibits. On-board power supplies and addressing capabilities eliminate the need for dedicated support equipment and simplifies specification and installation. The auto-switching power supplies work around the world.

Designed in a rugged extruded aluminum housing, each fixture features attached mounting brackets with two, 1/2-inch (13 mm) mounting holes for use with Cheeseborough clamps or pipe clamps. Locking knobs located on the mounting brackets allow for 180° rotational adjustment and locking without the use of special tools. Optional mounting brackets are available for T-handle mount applications. The housing is equipped to support spread lenses, louvers, and other attachments. A single 3-wire, 18AWG 6-foot (1.8 m) UL/cUL rated cord with IEC and flying leads is supplied. (Consult distribution for cord sets listed for PSE or CE).

Each ColorBlaze 48 fixture has eight individual circuit board assemblies, each with 18 high-intensity LEDs. This makes it sequentially controllable in 6-inch increments by a Color Kinetics DMX controller or a third-party DMX512 controller. Each circuit board is pre-addressed for Light# 1-8/DMX# 1-24. Data can be daisy-chained from fixture to fixture with an RJ-45 data cable or an XLR-5 data cable.

For protection from overheating, ColorBlaze 48 has been designed with a temperature monitoring feature. If operating temperatures rise to an unsafe level, a compensation circuit is triggered and ColorBlaze 48 operation is interrupted causing the lights to turn dull red. After 30 minutes the lights will auto-cycle and return to full intensity.

COLORBLAZE 48 SPECIFICATIONS

COLOR RANGE	16.7 million (24 bit) additive RGB colors; continuously variable intensity output range
SOURCE	High intensity power light emitting diodes (LEDs)
BEAM ANGLE	10°
HOUSING	Extruded aluminum with black finish
POWER CONNECTOR	IEC 15A (max) with C13 plug, UL/cUL rated 2-pole, 3-wire, grounded, 15A, flying leads
DATA CONNECTORS	RJ-45 or XLR-5
LISTINGS	UL/cUL, CE, PSE

COMMUNICATION SPECIFICATIONS

DATA INTERFACE	DMX512
CONTROL	Color Kinetics' line of DMX controllers or other DMX512 (RS-485) controllers

ELECTRICAL SPECIFICATIONS

POWER REQUIREMENT	100-240VAC
POWER CONSUMPTION	280W, 2.5A nominal at full intensity (full RGB)

ENVIRONMENTAL SPECIFICATIONS

TEMPERATURE RANGE	-40°F to 122°F (-40°C to 50°C) operating temperature 14°F to 122°F (-10°C to 50°C) starting temperature

LED SOURCE LIFE

In traditional lamp sources, lifetime is defined as the point at which 50% of the lamps fail. This is also termed Mean Time Between Failure (MTBF). LEDs are semiconductor devices and have a much longer MTBF than conventional sources. However, MTBF is not the only consideration in determining useful life. Color Kinetics uses the concept of useful light output for rating source lifetimes. Like traditional sources, LED output degrades over time (lumen depreciation) and this is the metric for SSL lifetime.

LED lumen depreciation is affected by numerous environmental conditions such as ambient temperature, humidity and ventilation. Lumen depreciation is also affected by means of control, thermal management, current levels, and a host of other electrical design considerations. Color Kinetics systems are expertly engineered to optimize LED life when used under normal operating conditions. Lumen depreciation information is based on LED manufacturers' source life data as well as other third party testing. Low temperatures and controlled effects have a beneficial effect on lumen depreciation. Overall system lifetime could vary substantially based on usage and the environment in which the system is installed.

Temperature and effects will affect lifetime. Color Kinetics rates product lifetime using lumen depreciation to 50% of original light output. When the fixture is running at room temperature using a color wash effect, the range of lifetime is in the range of 80,000-100,000 hours. This is LED manufacturers' test data. High output is defined as any LED device that is 1/2 watt or above. For more detailed information on source life, please see www.colorkinetics.com/lifetime.

OPTIBIN®

There are inherent variations in the fabrication processes of all semiconductor materials. For LEDs, this variance results in differences in the color and intensity of light output as well as electrical characteristics. Due to these differences, LED manufacturers sort production into "bins," but insuring the availability of a single bin is very difficult. To minimize this issue and achieve optimal color consistency in its products, Color Kinetics has developed and uses a proprietary technology called Optibin. Optibin is an advanced production binning optimization process that minimizes the effects of LED variance for the best possible output uniformity in the final product. Color Kinetics Optibin technology gives the most consistent control of color and intensity from product to product.

CHROMACORE® BY COLOR KINETICS

OPTIBIN® BY COLOR KINETICS

LISTED

ITEM#116-000016-00

This product is protected by one or more of the following U.S. patents and their foreign counterparts: 6,016,038, 6,150,774, 6,292,901, 6,340,868, 6,777,891, 6,788,011, 6,806,659, 6,969,954, 6,975,079, 7,186,003, and 7,221,104. Other patents pending.

BR0116 Rev 07

Specifications subject to change without notice. Refer to www.colorkinetics.com for the most recent data sheet versions.

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COLORBLAZE 48

PHOTOMETRIC PERFORMANCE

Photometric data is based on test results from an independent testing lab.

SOURCE SPECIFICATIONS

Optics:	Clear polycarbonate
Source:	144 LEDs (48 Red, 48 Green, 48 Blue)
Beam Angle:	10° (at 50% of peak illuminance)
Distribution:	Symmetric direct illumination
CCT:	Adjustable 1,000 – 10,000K
CRI:	Not measurable (CIE 13.3-1995)

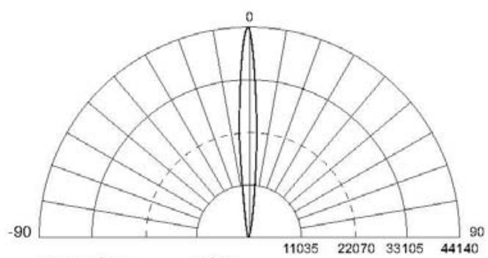
ILLUMINANCE DISTRIBUTION

7.9 / 85.0	10.7 / 115.2	11.9 / 128.1	11.4 / 122.7	9.6 / 103.3	6.9 / 74.3	6.0'/2.0m
15.3 / 164.7	25.3 / 272.3	29.3 / 315.4	27.6 / 297.1	19.1 / 205.6	10.0 / 107.6	
52.8 / 568.3	99.1 / 1066.7	107.0 / 1151.7	109.0 / 1173.3	68.0 / 732.0	18.0 / 193.8	3.0'/1.0m
59.0 / 635.1	144.0 / 1550.0	183.0 / 1969.8	183.0 / 1969.8	140.0 / 1507.0	54.6 / 587.7	
23.4 / 251.9	82.5 / 888.0	127.0 / 1367.0	125.0 / 1345.5	112.0 / 1205.6	57.3 / 616.8	
10.1 / 108.7	25.5 / 274.5	38.9 / 418.7	40.5 / 435.9	35.4 / 381.0	19.6 / 211.0	0.0'/0.0m

3.0'/1.0m 0'/0m 3.0'/1.0m

Units:	Footcandles (top)/Lux (bottom) 10.8 lux = 1 fc
Measured on:	All, reflectance model 80/50/20%
Distance from surface:	Bottom of grid, 3' (1.0 m) from surface, light at a 45° angle off horizontal

CANDLE POWER DISTRIBUTION



Measured on:	White
Beam center:	44140 cd
Thin dashed line:	Indicates 50% of peak
Multipliers:	0.33 Red, 0.50 Green, 0.18 Blue

ILLUMINANCE

COLOR	3' 1m	6' 2m	9' 3m	15' 5m
WHITE	2162.0 / 23271.8	675.0 / 7265.7	253.0 / 2723.3	127.0 / 1367.0
RED	721.2 / 7763.5	225.2 / 2423.8	84.4 / 908.5	42.4 / 456.0
GREEN	1070.2 / 11519.5	334.1 / 3596.5	125.2 / 1348.0	62.9 / 676.7
BLUE	393.5 / 4235.5	122.9 / 1322.4	46.0 / 495.6	23.1 / 248.8

Measured in Footcandles (top)/Lux (bottom) on axis.
Measured on: All, reflectance 0.

LIGHT OUTPUT

COLOR	TOTAL OUTPUT (lumens)	POWER (Watts)	EFFICACY (Lm/W)
WHITE	2282	240.0	9.5
RED	761.3	84.0	9.1
GREEN	1129.6	84.0	13.4
BLUE	415.3	84.0	4.9

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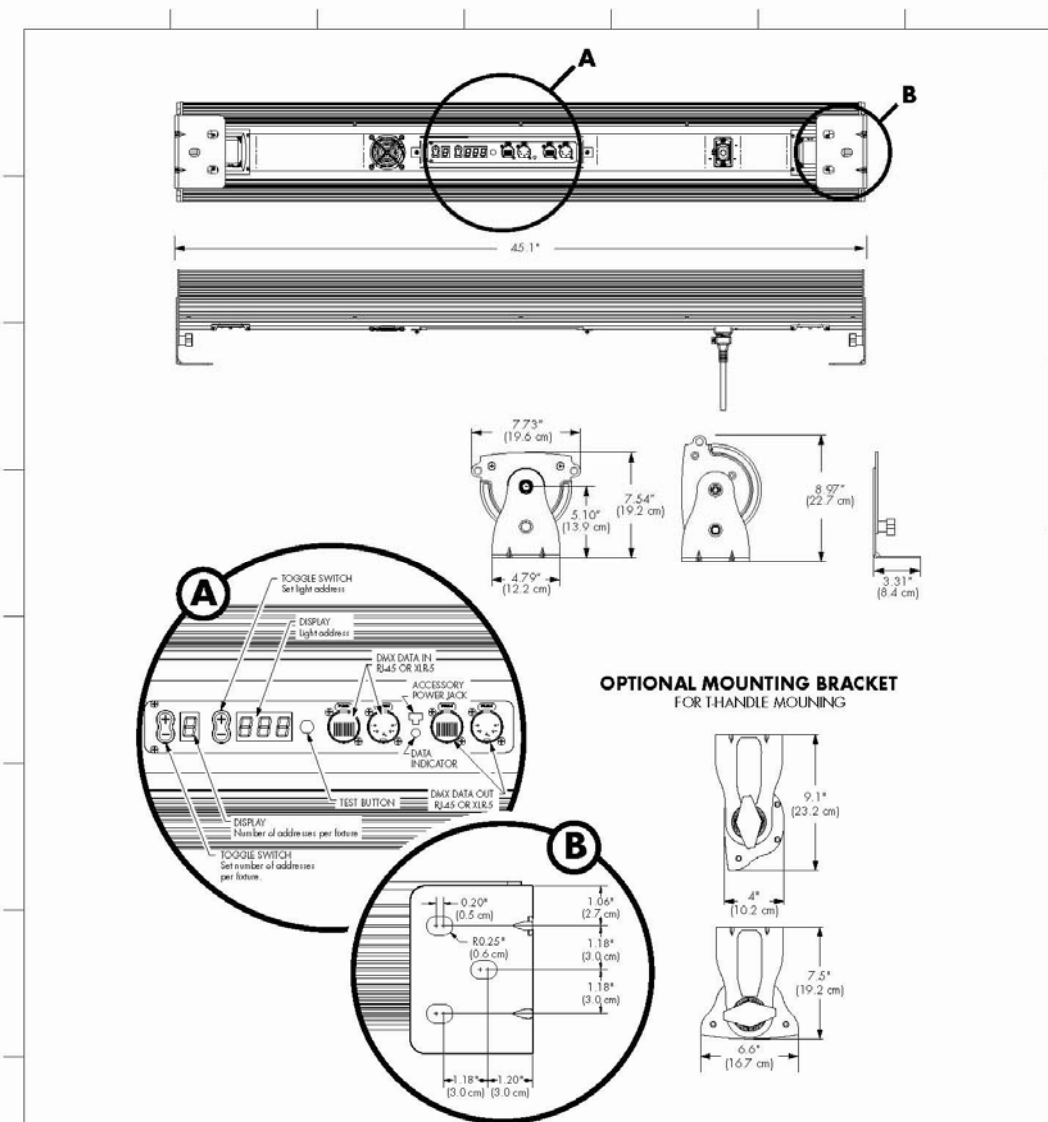
COLORBLAZE 48

PHYSICAL DIMENSIONS



COLORBLAZE 48 ITEM# 116-000016-00	
WEIGHT (APPROX)	40.0 lb (18.1 kg)
POWER CABLE	IEC 15A (max) with C13 plug
POWER REQUIREMENT	100-240VAC (280W)

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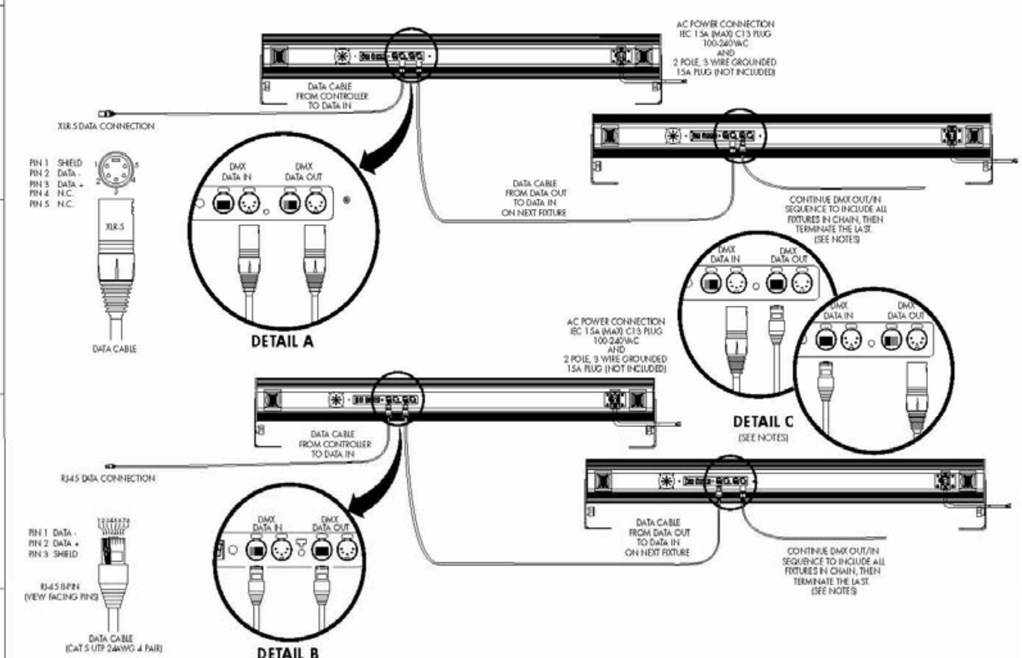
COLORBLAZE 48

FUNCTIONAL FLOW DIAGRAM



NOTES:

1. PLACE TERMINATOR IN DMX DATA OUT PORT OF THE LAST FIXTURE IN A CHAIN.
2. MAXIMUM DMX DATA RUN FROM DMX SOURCE TO LAST FIXTURE IN CHAIN IS 1000 FEET (OR 300 m).
3. DMX DATA CHAINS NEED NOT BE CONNECTOR SPECIFIC, FOR EXAMPLE: XLR-5 INPUT WITH RJ-45 OUTPUT AND VICE VERSA. SEE DETAIL C.
4. THE ACCESSORY POWERJACK IS FOR USE WITH COLOR KINETICS PRODUCTS ONLY.

For complete installation instructions and safety precautions, refer to the ColorBlaze 48/72 User Guide and wiring diagrams located at www.colorkinetics.com/support.

Additional Items	
Power Plug	2 Pole, 3 Wire, Grounded, 15A Power Plug
Controller	Color Kinetics DMX controller or DMX512 compatible controller
Data Cables	RJ-45 or XLR-5 data cables

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COLOR KINETICS

PHILIPS

iW COVE POWERCORE

An IntelliWhite™ Product

The iW™ Cove Powercore fixture combines the intelligent white LED illumination systems of the IntelliWhite™ family with the versatile housing of the iColor Cove® MX fixture. This high performance 12" (30.2 cm) linear fixture is designed for common interior alcove applications, such as lobbies, atriums, schools, museums, malls, or other public spaces. Integrated Powercore® technology provides greater operational efficiency and simplified installation.

Powercore technology, a digital power processing technology that drives LED systems, integrates power and data management directly into the fixture and eliminates the need for an external power supply. Powercore surpasses traditional power supply technology by streamlining multiple conversion and regulation stages into a single, flexible, microprocessor-controlled power stage that controls power output to LED systems directly from line voltage and significantly increases overall system efficiency.

iW Cove Powercore is available in medium and asymmetric beam angles. The asymmetric version provides a true forward-throw asymmetric projection of light. This option is designed to deliver superior surface uniformity with no waste, resulting in a better ratio of light projected onto the receiving surface with an even distribution from top to bottom when used in alcove applications.

iW Cove Powercore's integral, two-point mounting bracket simplifies installation. It permits 180° of rotation, with detents every 10°. This provides adjustable aiming to tailor the illumination performance to the application. The fixture can be rotated to the desired position and locked in place with the included set screws. The end-to-end locking connectors, capable of making 180° turns, make iW Cove Powercore extremely versatile and easily adaptable to even the most challenging mounting requirements. An optional mounting track is available for linear runs.

iW Cove Powercore receives data via Color Kinetics® iW Data Enabler – a data formatting device specifically designed for use with Color Kinetics IntelliWhite Powercore based fixtures and line of iW controllers. Each iW Data Enabler will support up to 48 (110VAC), 74 (220VAC), or 78 (240VAC) iW Cove Powercore fixtures, using a 40-foot (12 m) field-cuttable leader cable. One-foot (0.3 m) and five-foot (1.5 m) jumper cables are available for installations that require spacing between fixtures.

iW COVE POWERCORE SPECIFICATIONS

COLOR TEMP RANGE	3000K to 6500K adjustable
SOURCE	High intensity LEDs
BEAM ANGLE	Medium (50° x 50°), Forward-Throw Asymmetric (20° x 30° x 160°)
HOUSING	Die cast aluminum, powder coated 12.00" x 2.00" x 1.54" (30.48 cm x 5.09 cm x 3.90 cm)
CONNECTORS	Integral male/female connectors
LISTINGS	UL/cUL, CE

COMMUNICATION SPECIFICATIONS

DATA INTERFACE	Color Kinetics iW Data Enabler
CONTROL	Color Kinetics full line of controllers including iW Scene Controller or Light System Manager**

ELECTRICAL SPECIFICATIONS

POWER REQUIREMENT	100-240VAC, 50-60 Hz
POWER CONSUMPTION	15W
POWER FACTOR	0.95 or greater at 120VAC

ENVIRONMENTAL SPECIFICATIONS

TEMPERATURE RANGE	-4°F to 122°F (-20°C to 50°C) based on testing of specific product
PROTECTION RATING	IP50

** For large or complex installations, consider controlling iW Cove Powercore with Light System Manager (LSM). Refer to the LSM data sheets or contact support@colorkinetics.com for more information.

CHROMACORE® BY COLOR KINETICS

POWERCORE® BY COLOR KINETICS

OPTIBIN® BY COLOR KINETICS

DRY

DAMP

CE c UL us LISTED 13PE

ITEM# 523-000002-00 (Medium)
ITEM # 523-000002-01 (Asymmetric)

This product is protected by one or more of the following patents: U.S. Patent Nos. 6,016,038, 6,150,774 and other patents listed at http://colorkinetics.com/patents/. Other patents pending.

BRO186 Rev 04

Specifications subject to change without notice. Refer to www.colorkinetics.com for the most recent data sheet versions.

LED SOURCE LIFE

In traditional lamp sources, lifetime is defined as the point at which 50% of the lamps fail. This is also termed Mean Time Between Failure (MTBF). LEDs are semiconductor devices and have a much longer MTBF than conventional sources. However, MTBF is not the only consideration in determining useful life. Color Kinetics uses the concept of useful light output for rating source lifetimes. Like traditional sources, LED output degrades over time (lumen depreciation) and this is the metric for SSL lifetime.

LED lumen depreciation is affected by numerous environmental conditions such as ambient temperature, humidity, and ventilation. Lumen depreciation is also affected by means of control, thermal management, current levels, and a host of other electrical design considerations. Color Kinetics systems are expertly engineered to optimize LED life when used under normal operating conditions. Lumen depreciation information is based on LED manufacturers' source life data as well as other third party testing. Low temperatures and controlled effects have a beneficial effect on lumen depreciation. Overall system lifetime could vary substantially based on usage and the environment in which the system is installed.

Temperature and effects will affect lifetime. Color Kinetics rates product lifetime using lumen depreciation to 70% of original light output. When the fixture is running on warm or cool, at room temperature, the LED lifetime is in the range 50,000 – 70,000 hours. This is based on LED manufacturers' test data. High output is defined as any LED device that is 1/2 watt or above. For more detailed information on source life, please see www.colorkinetics.com/lifetime.

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iW COVE POWERCORE - MEDIUM

PHOTOMETRIC PERFORMANCE

SOURCE SPECIFICATIONS

Lens:	UV-resistant soft-focus polycarbonate lens
Source:	10 LEDs (5 warm white, 5 cool white)
Beam Angle:	50° X 50° (at 50% of peak illuminance)
Distribution:	Symmetric direct illumination
CCT:	Adjustable 3000–6500K
CRI:	81 All, 73 Warm, 86 Cool

ILLUMINANCE DISTRIBUTION

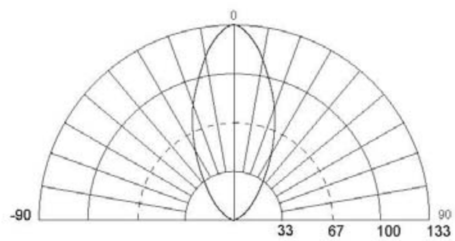
0.2 / 2.2	0.3 / 3.2	0.4 / 4.3	0.3 / 3.2	0.2 / 2.2	0.2 / 2.2	1.0'/0.3m
0.3 / 3.2	3.9 / 42.0	7.4 / 79.7	4.0 / 43.1	0.4 / 4.3	0.2 / 2.2	2.0'/0.6m
0.3 / 3.2	7.4 / 79.7	17.8 / 191.6	14.4 / 155.0	4.0 / 43.1	0.3 / 3.2	3.0'/1.0m
0.3 / 3.2	4.0 / 43.1	14.4 / 155.0	17.8 / 191.6	7.4 / 79.7	0.3 / 3.2	4.0'/1.2m
0.2 / 2.2	0.4 / 4.3	4.0 / 43.1	7.4 / 79.7	3.9 / 42.0	0.3 / 3.2	5.0'/1.5m
0.2 / 2.2	0.2 / 2.2	0.3 / 3.2	0.4 / 4.3	0.3 / 3.2	0.3 / 3.2	6.0'/2.0m
3.0'/1.0m		0'/0m		3.0'/1.0m		

Units: Footcandles (top)/Lux (bottom)
10.8 lux = 1 fc

Location: Centered 1'/0.3m from, and perpendicular to, surface

Measured on: All, Reflectance 50%

CANDLE POWER DISTRIBUTION



Measured on: All

Beam peak: 133 cd

Thin dashed line: Indicates 50% of peak

ILLUMINANCE

	3' / 1m	6' / 2m	9' / 3m	15' / 5m
ALL	19.3 / 207.7	4.2 / 45.2	1.8 / 19.4	0.6 / 6.5

Measured in Footcandles (top)/Lux (bottom) on axis.
Measured on all, reflectance 0.

LIGHT OUTPUT

	TOTAL OUTPUT (lumens)	POWER (Watts)	EFFICACY (lm/W)
ALL	120	15.0	8.0
WARM	109	15.0	7.3
COOL	116	15.0	7.7

Note: Efficacy figures are for a complete tested fixture not simply a lamp source.

CRI

It is common practice in the lighting industry to use color rendering index (CRI) to compare the properties of various light sources. There are known deficiencies and limitations associated with CRI and as a result, it is not always an accurate indicator of good object color appearance. This is especially true for LED-based sources. Until a better method for measuring color rendering in LEDs is accepted, Color Kinetics measures CRI in accordance with the current CIE 13.3-1995 standard using the Ra calculation. The reference illuminants employed are the Planckian locus below 5000K and CIE Daylight reference above 5000K. All measurements for Color Kinetics products are performed by third party laboratories using NIST-traceable instruments.

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iW COVE POWERCORE - ASYMMETRIC

PHOTOMETRIC PERFORMANCE

SOURCE SPECIFICATIONS

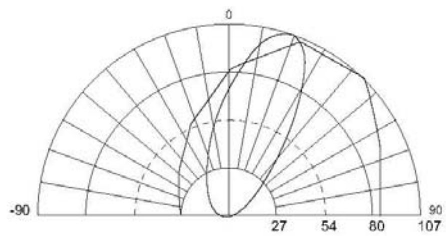
Lens:	UV-resistant soft-focus polycarbonate lens
Source:	10 LEDs (5 warm white, 5 cool white)
Beam Angle:	20° x 30° x 160° (at 50% of peak illuminance)
Distribution:	Asymmetric direct illumination
CCT:	Adjustable 3000–6500K
CRI:	84 All, 78 Warm, 87 Cool

ILLUMINANCE DISTRIBUTION

0.7 / 7.5	0.9 / 9.7	0.9 / 9.7	0.9 / 9.7	0.8 / 8.6	0.6 / 6.5	6'/2m
1.1 / 11.8	1.7 / 18.3	1.8 / 19.4	1.7 / 18.3	1.4 / 15.1	0.9 / 9.7	
1.6 / 17.2	3.1 / 33.4	3.9 / 42.0	3.5 / 37.7	2.4 / 25.8	1.3 / 14.0	3'/1m
1.7 / 18.3	4.1 / 44.1	6.0 / 64.6	5.5 / 59.2	3.5 / 37.7	1.7 / 18.3	
1.4 / 15.1	3.6 / 38.8	6.0 / 64.6	6.0 / 64.6	3.8 / 40.9	1.6 / 17.2	
1.1 / 11.8	2.7 / 29.1	4.8 / 51.7	5.0 / 53.8	3.0 / 32.3	1.2 / 12.9	0'/0m
3'/1.0m		0'/0m		3'/1.0m		

Units:	Footcandles (top)/Lux (bottom) 10.8 lux = 1 fc
Location:	2'/0.6m from surface, 45° tilt up from surface
Measured on:	Reflectance model 50/50/50

CANDLE POWER DISTRIBUTION



Measured on:	All
Beam peak:	107 cd
Thin dashed line:	Indicates 50% of peak

ILLUMINANCE

	0.5' / 0.15m	1' / 0.3m	2' / 0.6m	3' / 1m
ALL	4544.0 / 48911.6	182.0 / 1959.0	26.9 / 289.6	10.3 / 110.9

Measured in Footcandles (top)/Lux (bottom).
Measured on: Reflectance 0.

LIGHT OUTPUT

	TOTAL OUTPUT (lumems)	POWER (Watts)	EFFICACY (lm/W)
ALL	116	15	7.7
WARM	97	15	6.5
COOL	119	15	7.9

Note: Efficacy figures are for a complete tested fixture not simply a lamp source.

CRI

It is common practice in the lighting industry to use color rendering index (CRI) to compare the properties of various light sources. There are known deficiencies and limitations associated with CRI and as a result, it is not always an accurate indicator of good object color appearance. This is especially true for LED-based sources. Until a better method for measuring color rendering in LEDs is accepted, Color Kinetics measures CRI in accordance with the current CIE 13.3-1995 standard using the Ra calculation. The reference illuminants employed are the Planckian locus below 5000K and CIE Daylight reference above 5000K. All measurements for Color Kinetics products are performed by third party laboratories using NIST-traceable instruments.

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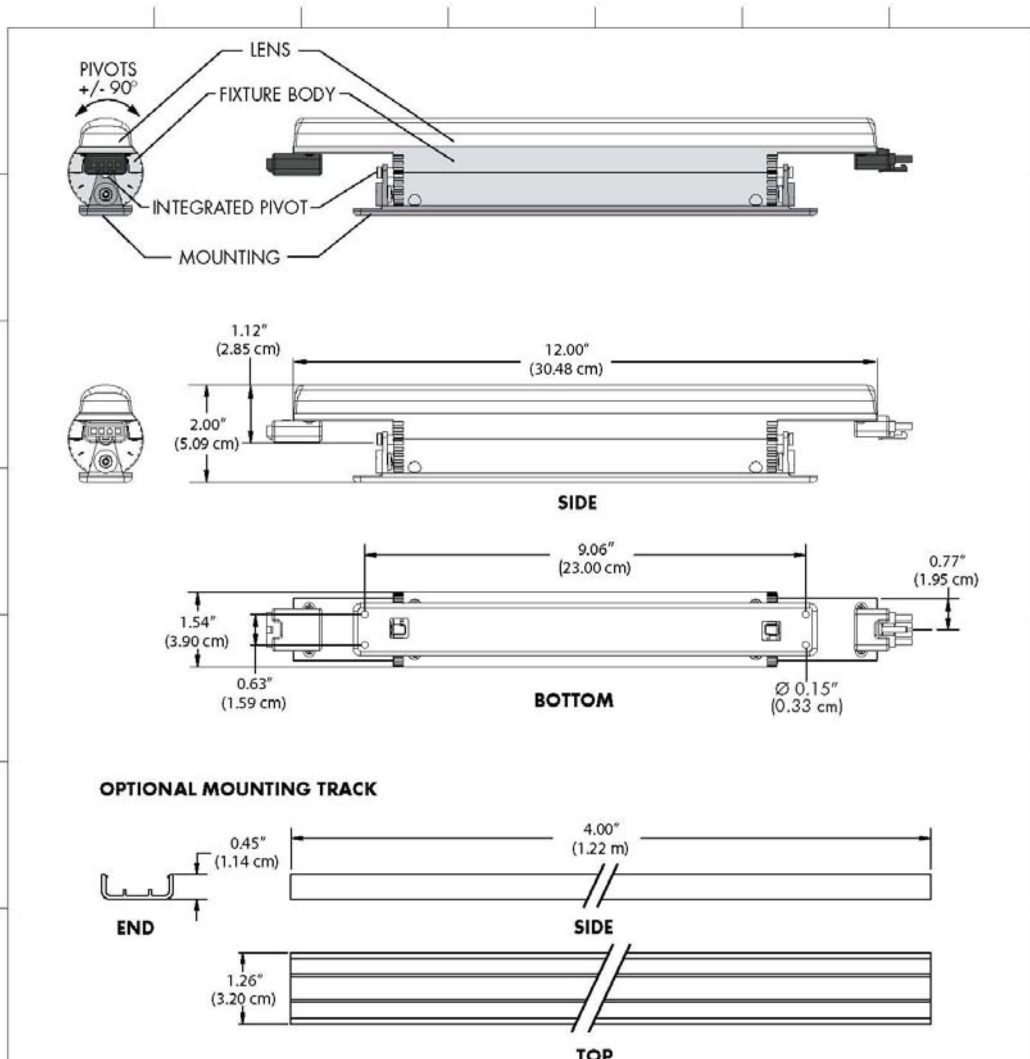
iW COVE POWERCORE

PHYSICAL DIMENSIONS



iW Cove Powercore Specification	
DATA/POWER CONNECTOR	Over-molded cable assembly
POWER REQUIREMENT	100-240VAC
WEIGHT	1.0 lbs (0.45 kg)

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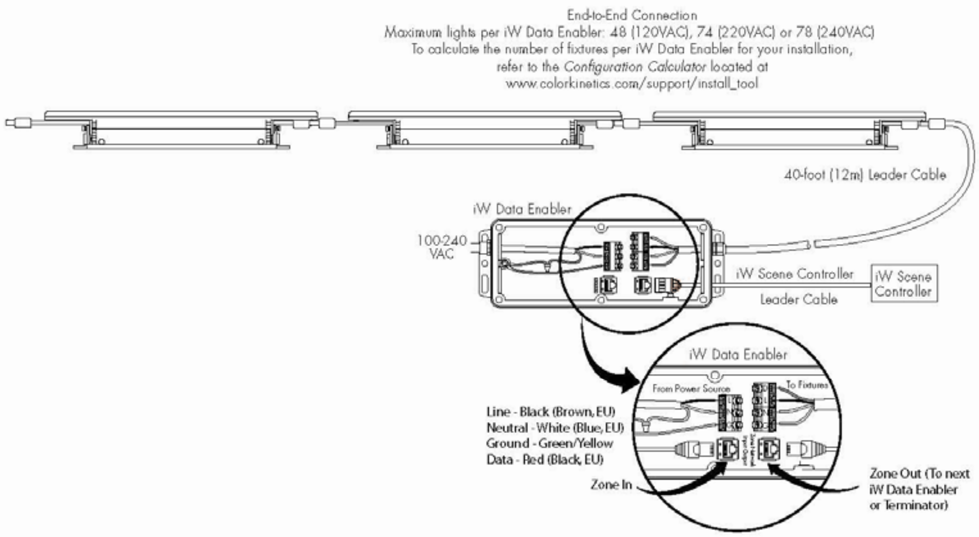
iW COVE POWERCORE

FUNCTIONAL FLOW DIAGRAM



For complete installation instructions and safety precautions, refer to the iW Cove Powercore User Guide and wiring diagrams located at www.colorkinetics.com/support.

Additional Items	
Data Device	iW Data Enabler (ITEM# 506-000001)
Controller	Color Kinetics controllers including iW Scene Controller or Light System Manager
Cable	Leader Cable (ITEM# 108-000021; 108-000022)

OPTIBIN®
There are inherent variations in the fabrication processes of all semiconductor materials. For LEDs, this variance results in differences in the color and intensity of light output as well as electrical characteristics. Due to these differences, LED manufacturers sort production into "bins," but insuring the availability of a single bin is very difficult. To minimize this issue and achieve optimal color consistency in its products, Color Kinetics has developed and uses a proprietary technology called Optibin. Optibin is an advanced production binning optimization process that minimizes the effects of LED variance for the best possible output uniformity in the final product. Color Kinetics Optibin technology gives the most consistent control of color and intensity from product to product.

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COLOR KINETICS

PHILIPS

iW MR

IntelliWhite™ iW MR is a compact lamp designed to retrofit into standard MR16 fixtures and sockets. It is well suited for use with tracks, rails, cables and pendants in interior architectural, retail, exhibit, display, and residential applications. The lamp features standard MR16 bi-pin connectors and a sleek housing with painted silver finish. iW MR is available in three distinct kelvin temperatures: 3000K, 3500K, and 6500K.

iW MR leverages Color Kinetics' new DIMand™ technology developed specifically for solid-state lighting devices. It uniquely allows the adjustment of light intensity using standard low-voltage dimmers, making the product suitable for retrofitting into existing low voltage MR16 systems. Unlike conventional light sources, which decrease in color temperature as they are dimmed, the iW MR will maintain a constant color temperature at any intensity.

iW MR fits most standard, low voltage MR16 lighting fixtures, including: track, cable, and rail styles. iW MR is equipped with a factory installed clear lens and includes a tempered sand blasted lens. An accessory/adapter ring (Item# 101-000050-00) is available for attaching lighting accessories and to ensure a proper fixture fit in some fixtures.

iW MR SPECIFICATIONS

TEMPERATURE RANGE	3000 kelvin (+250/-50K) 3500 kelvin (+/- 150K) 6500 Kelvin (+/- 500K)
SOURCE	4 white high power LEDs
BEAM ANGLE	18°
MIN. BEAM DISTANCE	6 inches, (15 cm)
HOUSING	Painted silver, die cast zinc, 2" (5 cm) diameter
BASE	GX5.3

ENVIRONMENTAL SPECIFICATIONS

TEMPERATURE RANGE	Ambient: - 4°F to 104°F (- 20°C to 40°C); Surface: 167°F (75°C)
HUMIDITY RANGE	0 to 95% non-condensing humidity

ELECTRICAL SPECIFICATIONS

POWER REQUIREMENT	12VAC
POWER CONSUMPTION	Maximum: 4 Watt
TRANSFORMER	Low-voltage magnetic or electronic to provide 12VAC

LED SOURCE LIFE

In traditional lamp sources, lifetime is defined as the point at which 50% of the lamps fail. This is also termed Mean Time Between Failure (MTBF). LEDs are semiconductor devices and have a much longer MTBF than conventional sources. However, MTBF is not the only consideration in determining useful life. Color Kinetics uses the concept of useful light output for rating source lifetimes. Like traditional sources, LED output degrades over time (lumen depreciation) and this is the metric for SSL lifetime.

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OPTIBIN®

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DRY

ITEM# 500-000002-00 (3000 K)
ITEM# 500-000002-01 (3500 K)
ITEM# 500-000002-02 (6500 K)

This product is protected by one or more of the following patents: U.S. Patent Nos. 6,016,038, 6,150,774 and other patents listed at http://colorkinetics.com/patents/. Other patents pending.

BRO142 Rev 03

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iW MR 3000K, 3500K, and 6500K

PHOTOMETRIC PERFORMANCE

Photometric data is based on test results from an independent testing lab.

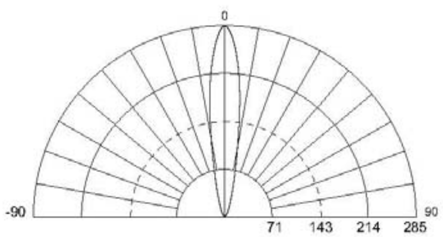
SOURCE SPECIFICATIONS

Optics:	Metalized polycarbonate reflectors
Lens:	Tempered clear glass lens
Source:	4 LEDs
Beam Angle:	18°
Distribution:	Symmetric direct illumination
CCT:	3000K, 3500K, 6500K
CRI:	72 (3000K), 77 (3500K), 83 (6500K)

ILLUMINANCE DISTRIBUTION PARAMETERS

Units: Footcandles (top)/Lux (bottom)
Location: 1.5'/0.6m from, and perpendicular to surface
Measured reflectance: 50

CANDLE POWER DISTRIBUTION



Measured on: All
Beam center: 285 cd (3000K), 272 cd (3500K), 309cd (6500K)
Dashed lined: Indicates 50% of peak

ILLUMINANCE

CCT	0.5' / 0.15m	1' / 0.3m	2' / 0.6m	3' / 1m
3000K	1161.0 / 12497.0	287.0 / 3089.3	71.6 / 770.7	31.8 / 342.3
3500K	1108.0 / 11926.5	274.0 / 2949.3	68.3 / 735.2	30.3 / 326.1
6500K	1259.0 / 13551.9	312.0 / 3358.4	77.6 / 835.3	34.4 / 370.3

Measured in Footcandles (top)/Lux (bottom) on axis
Measured on: Reflectance 0

LIGHT OUTPUT

	TOTAL OUTPUT (lumens)	POWER (Watts)	EFFICACY (lm/W)
3000K	41	3.5	11.7
3500K	38	3.4	11.2
6500K	49	3.7	13.2

Note: Efficacy figures are for a complete tested fixture not simply a lamp source.

ILLUMINANCE DISTRIBUTION 3000K

0.7 / 7.5	2.0 / 21.5	2.2 / 23.7	3.5 / 37.7	2.1 / 22.6	0.7 / 7.5	1.5'/0.5m
2.0 / 21.5	4.7 / 50.6	7.3 / 78.6	7.3 / 78.6	4.7 / 50.6	2.1 / 22.6	
3.4 / 36.6	7.3 / 78.6	12.4 / 133.5	11.2 / 120.6	7.3 / 78.6	3.5 / 37.7	0'/0m
2.2 / 23.7	7.3 / 78.6	11.2 / 120.6	12.4 / 133.5	7.3 / 78.6	2.3 / 24.8	
2.1 / 22.6	4.7 / 50.6	7.3 / 78.6	7.3 / 78.6	4.7 / 50.6	2.1 / 22.6	
0.7 / 7.5	2.1 / 22.6	2.3 / 24.8	3.5 / 37.7	2.1 / 22.6	0.7 / 7.5	1.5'/0.5m
1.5'/0.5m		0'/0m			1.5'/0.5m	

ILLUMINANCE DISTRIBUTION 3500K

0.5 / 5.4	1.9 / 20.5	2.0 / 21.5	3.2 / 34.4	1.9 / 20.5	0.5 / 5.4	1.5'/0.5m
1.8 / 19.4	4.4 / 47.4	6.9 / 74.3	6.9 / 74.3	4.4 / 47.2	1.8 / 19.4	
3.2 / 34.4	6.9 / 74.3	11.8 / 127.0	10.6 / 114.1	6.9 / 74.3	3.2 / 34.4	0'/0m
2.0 / 21.5	6.9 / 74.3	10.6 / 114.1	11.8 / 127.0	6.9 / 74.3	2.0 / 21.5	
1.9 / 20.5	4.4 / 47.4	6.9 / 74.3	6.9 / 74.3	4.4 / 47.4	1.9 / 20.5	
0.6 / 6.5	1.9 / 20.5	2.1 / 22.6	3.2 / 34.4	1.9 / 20.5	0.5 / 5.4	1.5'/0.5m
1.5'/0.5m		0'/0m			1.5'/0.5m	

ILLUMINANCE DISTRIBUTION 6500K

0.8 / 8.6	2.4 / 25.8	2.6 / 28.0	4.2 / 45.2	2.4 / 25.8	0.8 / 8.6	1.5'/0.5m
2.4 / 25.8	5.4 / 58.1	8.5 / 91.5	8.5 / 91.5	5.5 / 59.2	2.4 / 25.8	
4.0 / 43.1	8.5 / 91.5	14.4 / 155.0	13.0 / 139.9	8.5 / 91.5	4.0 / 43.1	0'/0m
2.6 / 28.0	8.5 / 91.5	13.0 / 139.9	14.4 / 155.0	8.5 / 91.5	2.6 / 28.0	
2.4 / 25.8	5.5 / 59.2	8.6 / 92.6	8.6 / 92.6	5.5 / 59.2	2.4 / 25.8	
0.8 / 8.6	2.5 / 26.9	2.7 / 29.1	4.1 / 44.1	2.4 / 25.8	0.8 / 8.6	1.5'/0.5m
1.5'/0.5m		0'/0m			1.5'/0.5m	

Units: Footcandles (top)/Lux (bottom); 10.8 lux = 1 fc
Location: 1.5' (0.46 m) perpendicular from surface
Measured on: Reflectance model 50%

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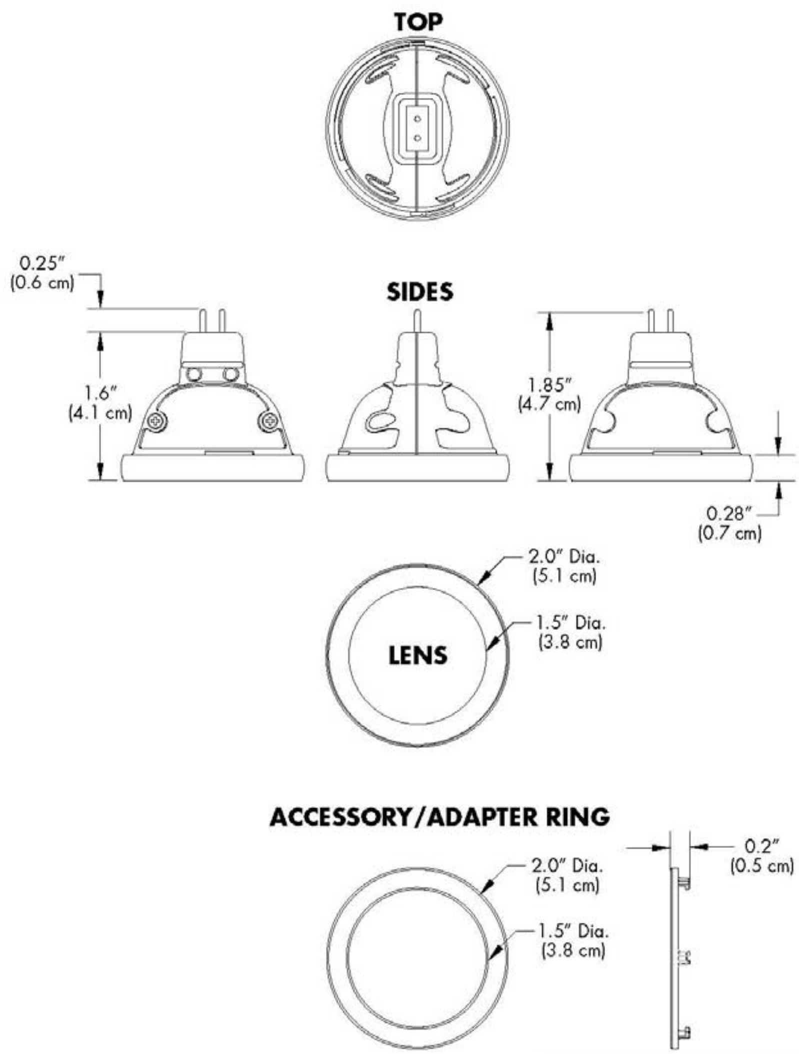
iW MR

PHYSICAL DIMENSIONS



For complete installation instructions and safety precautions, refer to the iW MR User Guide located at www.colorkinetics.com/support.

iW MR ITEM# 500-000002-00/01/02	
WEIGHT	3.4oz (96g)
POWER CONNECTOR	Standard MR16 fixture
POWER CONSUMPTION	4 Watts

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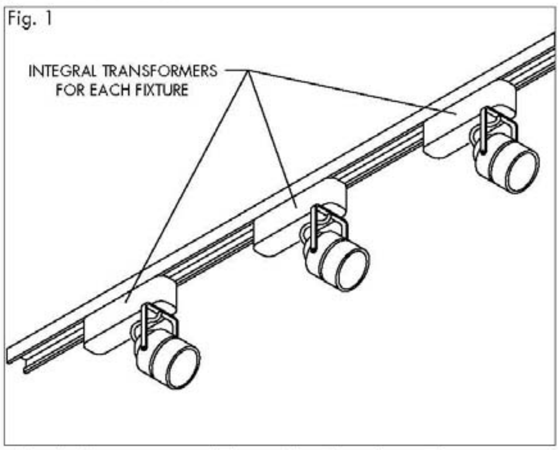
iW MR

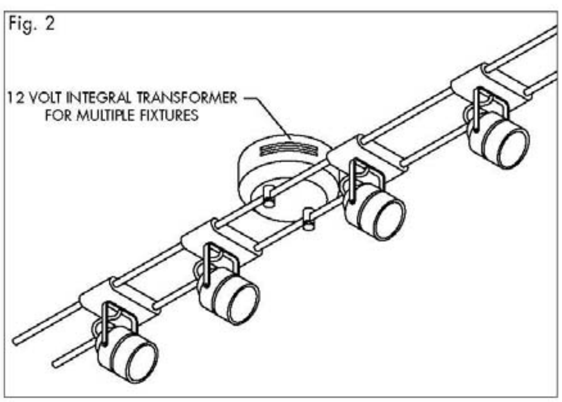
FIXTURE/TRANSFORMER COMPATIBILITY

Two types of low-voltage transformers are used with traditional MR16 lamps: magnetic and electronic. Consider the following when selecting the appropriate transformer for use with iW MR.

- **MAGNETIC:** iW MR is compatible with all types of magnetic transformers used with 12 volt lighting. Additionally, magnetic transformers are considered more reliable than electronic transformers; however, magnetic transformers are heavier, less efficient, and sometimes make noise (buzzing).
- **ELECTRONIC:** iW MR is not compatible with all electronic transformers due to its low power consumption of only 4 watts. Most electronic transformers require a minimum load greater than 4 watts in order to work properly. This is especially true of lighting fixtures that have an integral transformer for each. See Fig. 1. Symptoms of incompatibility include: no light output, flickering, strobing, or random shutdown.



- Transformers can be integral to the fixture or remotely located, and can be used to power a group of fixtures, tracks, cables, or rails See Fig. 2.



CRI

It is common practice in the lighting industry to use color rendering index (CRI) to compare the properties of various light sources. There are known deficiencies and limitations associated with CRI and as a result, it is not always an accurate indicator of good object color appearance. This is especially true for LED-based sources. Until a better method for measuring color rendering in LEDs is accepted, Color Kinetics measures CRI in accordance with the current CIE 13.3-1995 standard using the Ra calculation. The reference illuminants employed are the Planckian locus below 5000K and CIE Daylight reference above 5000K. All measurements for Color Kinetics products are performed by third party laboratories using NIST-traceable instruments.

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Ballast Cut Sheets

Ballast	Compact SE™ 5%	High-Performance Dimming

CompactSE-1 03.08.04

Compact SE Overview

For designs requiring the energy savings and aesthetic appeal of dimmed T4 compact fluorescent or T5 twin-tube lamps, Compact SE dimming ballasts are your solution. The Compact SE product family includes ballasts for nearly every type of dimmable compact fluorescent lamp.

Compact SE, case type A
3.00"w (76mm) x 1.00"h (25mm) x 4.90"l (124mm)

Features

- Continuous, flicker-free dimming from 100% to 5%
- Standard 3-wire line-voltage phase-control technology for consistent fixture-to-fixture dimming performance
- Models for 4-pin T4 compact lamps and T5 twin-tube lamps
- Programmed rapid start design will preheat lamp cathodes before applying full arc voltage
- Lamps turn on to any dimmed level without flashing to full brightness
- Low harmonic distortion throughout the entire dimming range maintains power quality
- Frequency of operation ensures that ballast does not interfere with infrared devices operating between 38 and 42 kHz
- Inrush current limiting circuitry eliminate circuit breaker tripping, switch arcing, and relay failure
- End-of-lamp-life protection circuitry ensures safe operation throughout entire lamp life cycle
- Ultra quiet operation
- Protected from miswires of any input power to control lead, or lamp leads to each other or ground
- 100% compatible with all Lutron 3-wire fluorescent controls
- 100% performance tested at factory
- Designed and assembled in the USA
- 5-year limited warranty with Lutron field service commissioning (3-year standard warranty) from date of purchase
- Ballasts that dim T4 compact fluorescent lamps are intended for factory installation by OEM fixture manufacturer.

Compact SE, case type B
3.00"w (76mm) x 1.00"h (25mm) x 6.75"l (171mm)

Compact SE, case type F
2.38"w (60mm) x 1.50"h (38mm) x 9.50"l (241mm)

LUTRON SPECIFICATION SUBMITTAL Page

Job Name:	Model Numbers:	
Job Number:		

Ballast	Compact SE™ 5%	High-Performance Dimming

CompactSE-2 03.08.04

Specifications

Performance

- Dimming Range: 100% to 5% measured relative light output (RLO)
- Lamp Starting: programmed rapid start
- Minimum Lamp Starting Temperature: 10°C (50°F)
- Ambient Temperature Operating Range: 10°C (50°F) to 60°C (140°F)
- Relative Humidity: maximum 90% non-condensing
- Operating Voltage: 120V or 277V at 60Hz
- Lamp Current Crest Factor: less than 1.7
- Lamp Flicker: none visible
- Light Output: constant ±2% light output for line voltage variations of ±10%
- Lamp Life: average lamp life meets or exceeds rating of lamp manufacturer
- Ballast Factor: greater than .95 for T4 quad or triple tube lamps, and greater than .85 for T5 twin-tube lamps
- Power Factor: greater than .95
- Total Harmonic Distortion (THD): less than 10%
- Maximum Inrush Current: 7 amps per ballast at 120V, 3 amps per ballast at 277V
- Sound Rating: Inaudible in a 27dBa ambient
- Maximum Ballast Case Temperature: 75°C (167°F)

Standards

- UL Listed (evaluated to the requirements of UL935)
- CSA certified (evaluated to the requirements of C22.2 No. 74)
- Class P thermally protected
- Meets ANSI C82.11 High Frequency Ballast Standard
- Meets FCC Part 18 Non-Consumer for EMI/RFI emissions requirements
- T4 compact fluorescent ballasts are MIL Std. 461E compliant (meets the requirements of CE101, RE101 and RE102)
- Meets ANSI C62.41 Category A surge protection standards to 6kV
- Manufacturing facilities employ ESD reduction practices that comply with the requirements of ANSI/ESD S20.20
- Lutron Quality Systems registered to ISO 9001

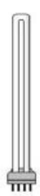
LUTRON® SPECIFICATION SUBMITTAL Page

Job Name:	Model Numbers:	
Job Number:		

Ballast   Compact SE™ 5%   High-Performance Dimming

CompactSE-3 03.08.04

Compact SE Ballast Models

Lamp Type	Lamp Watts	Lamps per ballast	Case Type	120 VOLTS Ballast Current (amps)	120 VOLTS Compact SE Model Number[1]	277 VOLTS Ballast Current (amps)	277 VOLTS Compact SE Model Number[1]
T4 4-Pin Quad-Tube	18W	1 2	A B	.20 .42	FDB-T418-120-1-S FDB-T418-120-2-S	.08 .17	FDB-T418-277-1-S FDB-T418-277-2-S
 1/2" diameter	26W	1 2	A B	.26 .50	FDB-T426-120-1-S FDB-T426-120-2-S	.12 .21	FDB-T426-277-1-S FDB-T426-277-2-S
T4 4-Pin Triple-Tube	18W	1 2	A B	.20 .42	FDB-T418-120-1-S FDB-T418-120-2-S	.08 .17	FDB-T418-277-1-S FDB-T418-277-2-S
 1/2" diameter	26W	1 2	A B	.26 .50	FDB-T426-120-1-S FDB-T426-120-2-S	.12 .21	FDB-T426-277-1-S FDB-T426-277-2-S
	32W	1 2	A B	.31 .59	FDB-T432-120-1-S FDB-T432-120-2-S	.13 .24	FDB-T432-277-1-S FDB-T432-277-2-S
	42W	1 2	B B	.36 .67	FDB-T442-120-1-S FDB-T442-120-2-S	.16 .29	FDB-T442-277-1-S FDB-T442-277-2-S
T5 Twin-Tube	36/39W (16")	1 2 3	F F F	.33 .58 .85	FDB-1643-120-1 FDB-1643-120-2 FDB-1643-120-3	.14 .25 .35	FDB-1643-277-1 FDB-1643-277-2 FDB-1643-277-3
 5/8" diameter	40W (22")	1 2 3	F F F	.33 .61 .88	FDB-2227-120-1 FDB-2227-120-2 FDB-2227-120-3	.14 .25 .38	FDB-2227-277-1 FDB-2227-277-2 FDB-2227-277-3
	50W (22")	1 2	F F	.38 .69	FDB-2243-120-1 FDB-2243-120-2	.17 .32	FDB-2243-277-1 FDB-2243-277-2

[1] Mounting studs standard for T4 ballasts. Delete suffix -S in the model number if mounting studs not needed.

LUTRON® SPECIFICATION SUBMITTAL   Page

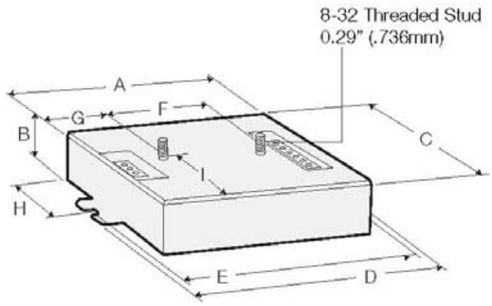
Job Name:	Model Numbers:
Job Number:	

Ballast | Compact SE™ 5% | High-Performance Dimming

CompactSE-4 03.08.04

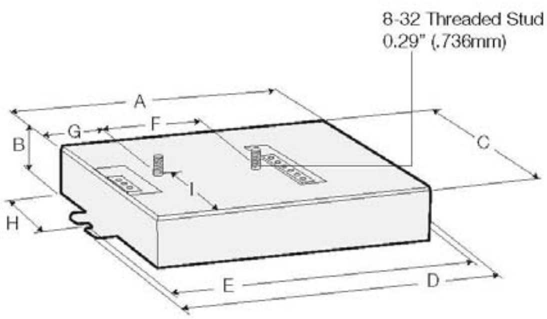
Case Dimensions

A[1]



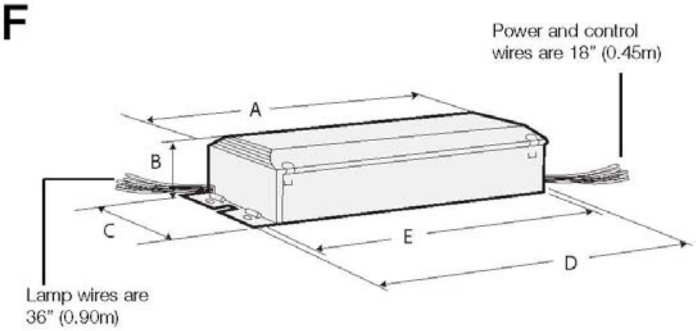
A	4.20"	(107 mm)
B	1.00"	(25 mm)
C	3.00"	(76 mm)
D	4.90"	(124 mm)
E	4.60" (mounting centers)	(117 mm)
F	2.00"	(51 mm)
G	1.08"	(27 mm)
H	1.60"	(41 mm)
I	1.39"	(35 mm)

B[1]



A	6.00"	(152 mm)
B	1.00"	(25 mm)
C	3.00"	(76 mm)
D	6.75"	(171 mm)
E	6.50" (mounting centers)	(165 mm)
F	2.00"	(51 mm)
G	1.00"	(29 mm)
H	1.60"	(41 mm)
I	1.39"	(35 mm)

F



A	8.30"	(211 mm)
B	1.50"	(38 mm)
C	2.38"	(60 mm)
D	9.50"	(241 mm)
E	8.91" (slot mounting centers)	(226 mm)

If using four hole mount, mounting centers are 9.21" (234 mm) x 1.70" (43 mm).

[1] Mounting studs standard. When ordering, delete suffix -S in the ballast model number if mounting studs not needed.

LUTRON® SPECIFICATION SUBMITTAL Page

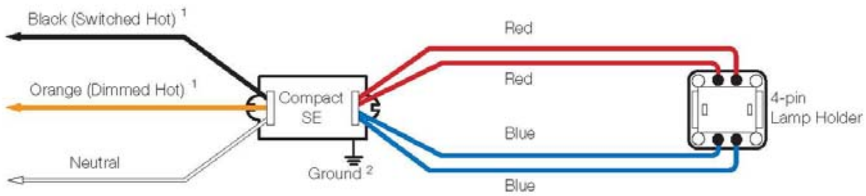
Job Name:	Model Numbers:	
Job Number:		

Ballast	Compact SE™ 5%	High-Performance Dimming

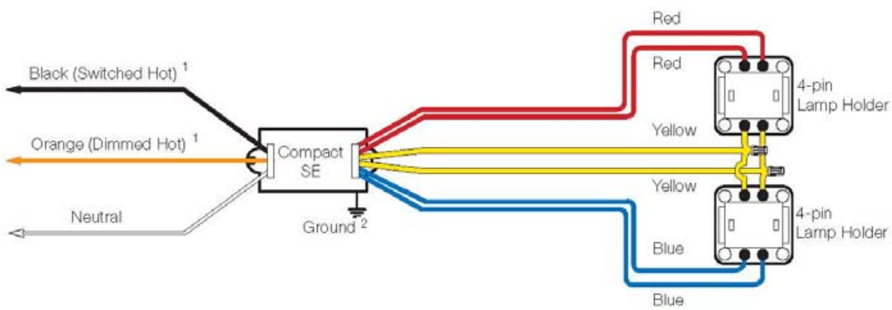
CompactSE-5 03.08.04

Wiring Diagrams

One compact fluorescent lamp



Two compact fluorescent lamps



1 Dimming control wire colors do not necessarily match ballast wire colors (e.g. control "dimmed hot" may be yellow and ballast "dimmed hot" may be orange). Wire colors shown are for Lutron controls and ballasts only.

2 Ballast and lighting fixture must be effectively grounded.

Note: For T4 compact lamps, maximum lamp-to-ballast wire length is 3 feet (1 m).

LUTRON® SPECIFICATION SUBMITTAL | Page

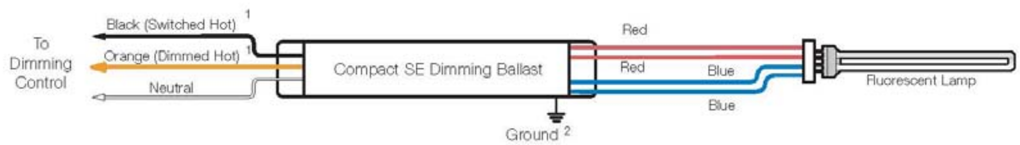
Job Name:	Model Numbers:	
Job Number:		

Ballast	Compact SE™ 5%	High-Performance Dimming

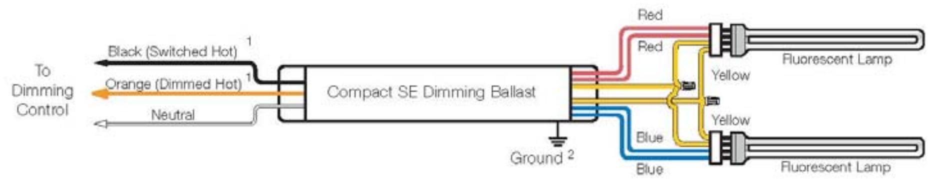
CompactSE-6 03.08.04

Compact SE Wiring Diagrams

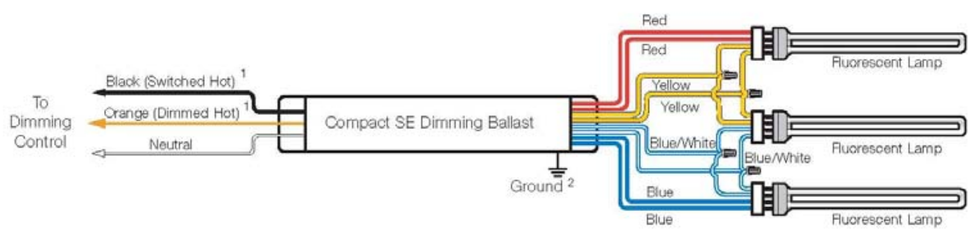
One T5 twin-tube lamp



Two T5 twin-tube lamps



Three T5 twin-tube lamps



[1] Dimming control wire colors do not necessarily match ballast wire colors (e.g. control 'dimmed hot' may be yellow, and ballast 'dimmed hot' may be orange). Wire colors shown are for Lutron ballasts and controls only.
[2] Ballast and lighting fixture must be effectively grounded.

Note: For T5 twin-tube lamps, maximum lamp-to-ballast wire length is 3 feet (1m).

LUTRON® SPECIFICATION SUBMITTAL Page

Job Name:	Model Numbers:	
Job Number:		

Ballast | Compact SE™ 5% | High-Performance Dimming

CompactSE-7 03.08.04

ATTENTION ELECTRICIANS AND CONTRACTORS

Ballast/Socket Leads

Lead lengths from ballast to socket must not exceed 3' (1m) for T4 and T5 twin-tube lamps.

Lamp Sockets

Quality lamp sockets are required to ensure positive lamp-pin to socket contact. T4 compact sockets must be the 4 pin type, and must be used with 4 pin compact lamps. T5 twin-tube lamps require sockets that lock into the lamp base and provide proper lamp support to hold lamp pins in full contact with socket.

Lamp Mounting for T5 Twin-Tube

Many fluorescent lamp sockets are available with mounting slots to vary the height of the lamp away from the grounded metal surface. Use these slots to keep the outside edge of the lamp at least 1/4" away from the grounded metal surface.

Having a fluorescent lamp too close to the grounded metal will make the minimum intensity too low and will reduce lamp life.

Wiring and Grounding

All wiring from the dimming control to Compact SE ballasts is line voltage wiring and may be run together in the same conduit.

Ballast and lighting fixture must be effectively grounded. Ballasts must be installed per national and local electrical codes.

Number of Ballasts Per Control

To calculate the maximum number of ballasts allowed per control, divide control's current capacity by individual ballast current. Certain controls allow a specific maximum number of ballasts.

Ballast Operating Temperature

Ballast case temperature must not exceed 75°C at any point on ballast.

ATTENTION FACILITIES MANAGERS

PERFORMANCE

Lamps Must Be Seasoned

New fluorescent lamps must be operated for 100 hours at full output ("seasoned") to render lamp impurities inert, so as to achieve proper dimming performance and average rated lamp life.

SERVICE

Replacement Parts

Use replacement parts with exact Lutron model numbers. Consult Lutron if you have any questions.

Further Information

Ballasts for other lamp types and voltages are available. Consult Lutron's Fluorescent Dimming Systems Selection Guide (P/N 366-002).

For further information, please visit us at http://www.lutron.com/ballasts or contact our 24-hour Technical Support Center at 1-800-523-9466.

LUTRON. SPECIFICATION SUBMITTAL | Page

Job Name:	Model Numbers:	
Job Number:		

Light Fixture IES Files and AGI Model

IES files and AGI model are on the attached compact disc.