

HUNTER'S POINT SOUTH SCHOOL

LONG ISLAND CITY, NEW YORK



MICHAEL PAYNE | STRUCTURAL OPTION

BUILDING STATISTICS

- **Location:** 1-50 51st Avenue, Long Island City, New York
- **Size:** 154,500 Square Feet
- **Height:** 5 Stories/ 75 Feet Tall
- **Dates of Construction:** January 10, 2011 - October 2013
- **Construction Cost:** \$61,098,000
- **Project Delivery Method:** Lump Sum Bid

PROJECT TEAM

- **Owner:** NYC School Construction Authority (SCA) per NYCDOE
- **General Contractor/CM:** SKANSKA
- **Architect:** FXFOWLE Architects, LLC
- **Structural Engineer:** Ysreale A. Seinuk, PC
- **MEP/Fire Protection:** Kallen & Lemelson, LLP
- **Site-Civil Engineering:** Langan Engineering & Environmental Service

ARCHITECTURE

- Mixed Intermediate School and High School
- Vertically Stacked Design with Spaces to Tie Both Schools Together
- Cubic Design with Vertical Shafts, Horizontal Windows, & Slanted Edges.
- Façade: Grey Brick, Slate, Orange Alum. Panels, & Glass Curtain Wall
- 4000 Square Foot Open Roof Terrace Outside Cafeteria on 5th Floor
- LEED Silver Certification for Sustainable Design

STRUCTURAL SYSTEM

- **Foundation:** 12" Slab on Grade Supported by Caissons and H-Piles
- **Floor System:** 3" Composite Deck with 3.25" LW Concrete Topping
- **Framing System:** Steel Frame Comprised of Wide Flange Members, Long Span Plate Girders, and Steel Columns.
- **Lateral System:** HSS and Wide Flange Lateral Truss Bracing, along with Steel Moment Connections at Specific Columns.

MEP SYSTEMS

- 3 VAV Systems Service Classroom, Office, and Corridor Spaces
- 3 CAV Systems Service the Gymnasium, Auditorium, and Cafeteria
- 4 Boilers Produce 1860MBH at 212°F/160psig for AHU and Heaters
- Main 208/120V 3 Phase System With Secondary Emergency Power
- Gas and Water Intake Lines are 4" Conduit; Sewage Lines Are 6"
- Wet Pipe Sprinkler System With Concealed and Upright Heads

THESIS REDESIGN



HUNTER'S POINT SOUTH INTERMEDIATE & HIGH SCHOOL

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4 April 2012

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EXECUTIVE SUMMARY

Hunter's Point South School is a 5 story combined intermediate and high school located in Long Island City, New York. At 154,000 square feet, this large school will hold over 1100 students from grades 6-12 and includes a gymnasium, auditorium, roof terrace, and many classrooms and laboratories. The structure includes a lightweight concrete composite floor supported by a steel framing system. Lateral loads are resisted by steel concentric braced frames and several moment frames along the gymnasium and auditorium spaces. The steel columns connect to a foundation of deep caissons, H-piles, and grade beams.

The goal of this thesis is to explore the effects of a more ductile lateral system, and to investigate whether a lateral system redesign for a higher seismic region is an effective and efficient possibility for Hunter's Point South. To start the investigation, the structure is moved to a SDC D seismic zone in Redding, CA, and an Eccentrically Braced Frame (EBF) system is chosen to replace the original lateral system. Using ASCE7-10, two different design methods are used to create two separate redesigns. This is done to help show the transition of design from the original location/design to final redesign/location. Equivalent Lateral Force Procedure (ELFP) and Modal Response Spectrum Analysis (MRSA) are used for design load calculations.

Using AISC 341-10 Seismic Provisions and AISC 327-05 Seismic Design Manual as design references, ETABS structural modeling program is used to design both EBF systems. Once both layouts are created and member sizes are designed, an analysis is performed to compare the strength and serviceability characteristics of each system against the other, as well as comparing each to the original design. Also, as part of an MAE requirement, seismic connection details are designed for each redesign system.

After analysis is performed on the performance of each new lateral system, several breadth studies must be completed to analyze the secondary effects the new systems have on the rest of the building project for Hunter's Point South. First, an architectural impact study is completed to investigate whether the new lateral systems are compatible with the original architectural layout. It is found that in the ELFP design, new EBF frames create façade issues and room lighting issues, so design changes are implemented to the façade and layout of several rooms. Also, a construction impact study is completed to determine the effects of each redesign on the overall construction cost and schedule. Using RS Means, original construction documents, and other research, cost increases are analyzed and the critical construction path is changed to accommodate the new lateral system designs and the seismic detailing that goes with it.

INTRODUCTION

Hunter's Point South School is a new 5 story educational building being constructed as part of the first phase of New York City's new mixed-use development plan on a 30 acre site of waterfront properties in Long Island City, NY. The new development focuses on creating an affordable middle-income area that includes several new mixed use housing towers, along with supporting retail spaces, a school, and new waterfront park. Hunter's Point South School is being developed by the NYC School Construction Authority (SCA) along with Skanska contracting and FXFowle Architects. The

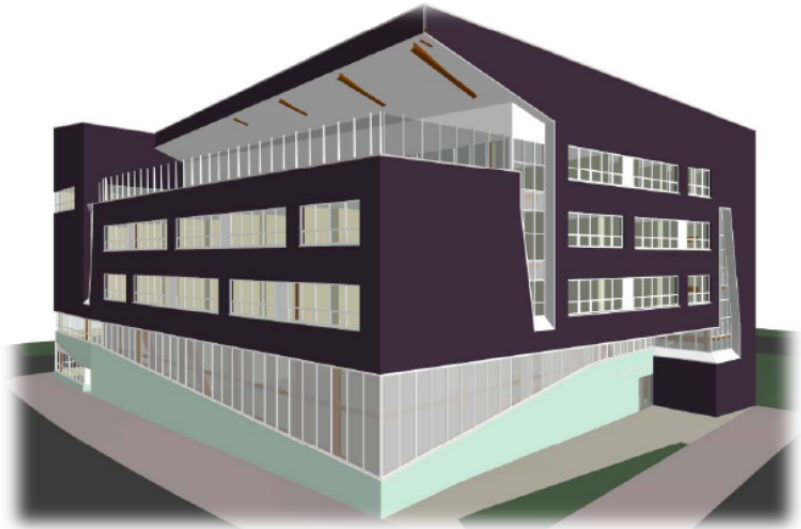


Figure 1: Building design rendering
Rendering by FXFowle Architects

The structural engineer on the project is Ysreale A. Seinuk, PC. Construction of the school will last from January 2011 to October 2013, and cost approximately \$61Million to complete. Project delivery is lump sum bid. It will open its doors to students in the fall of 2013.

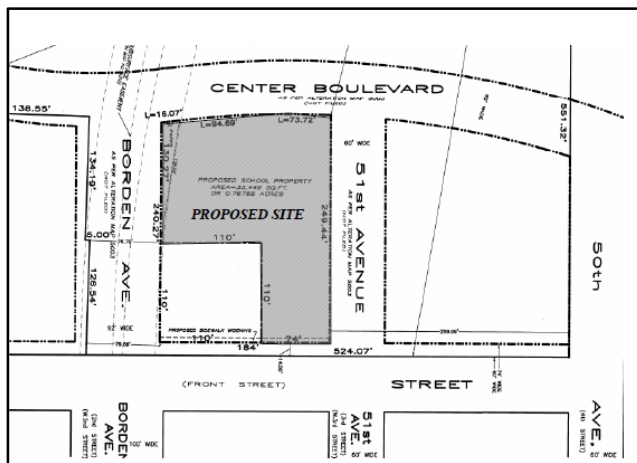


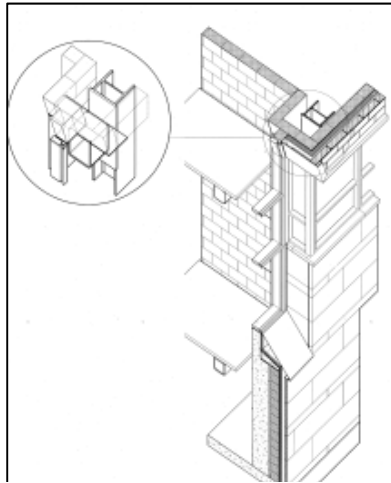
Figure 2: Building site plan
Drawing by FXFowle Architects

It should also be noted that the site sits right across the street from the bay.

The mixed use intermediate and high school will be nearly 154,500 square feet and house roughly 1100 students from grades 6-12 and District 75 (special needs) from the Queens School District. Being constructed on 51st Avenue, Hunter's Point will take up almost a full city block between 2nd Street and Center Boulevard with space in the corner of the lot reserved for the construction of a new 30 story housing tower to be built right next to the school. The site layout can be

THESIS REDESIGN

Following along with other city development ideals, the school building has a modern architectural feel as it incorporates interesting shapes, cantilevers, and sense of solids and voids together. The cubic shape of the building is broken up with vertical shafts, horizontal windows, and slanted edges. In addition, the SCA is aiming to achieve LEED Silver certification for this building through several different sustainable features and construction procedures.



**Figure 3: Typical Wall Section
Axonometric Detail**
Drawing by FXFowle Architects

The exterior façade is comprised of a unique blend of grey brick, slate veneer, concrete block, orange aluminum composite panels, and different types of glazing including translucent panels. Much of the shell is part of a curtain wall system that is supported by the floor above. There is, however, some load bearing masonry used in the design. **Figure 4** shows a current mock-up of the planned façade style.

The 5 story school rises roughly 75 feet off finished grade, with an irregular parapet rising as high as 98 feet on some elevations. It is mainly a structural steel building, with concrete on metal deck floors and an assorted exterior. The exterior façade is comprised of a unique blend of grey brick, slate veneer, concrete block, orange



Figure 4: Typical Wall mock-up
Photo by SKANSKA Inc.



Figure 5: Building Section
Rendering by FXFowle Architects

Inside, the building is vertically stacked to separate the schools, but includes ties to each other using shared spaces. The first floor contains athletic space, including a 2 story tall gymnasium and locker rooms for all grades. There are also support rooms/offices for the intermediate school and general storage areas. The second floor contains an auxiliary gym, library, and special education rooms for the

THESIS REDESIGN

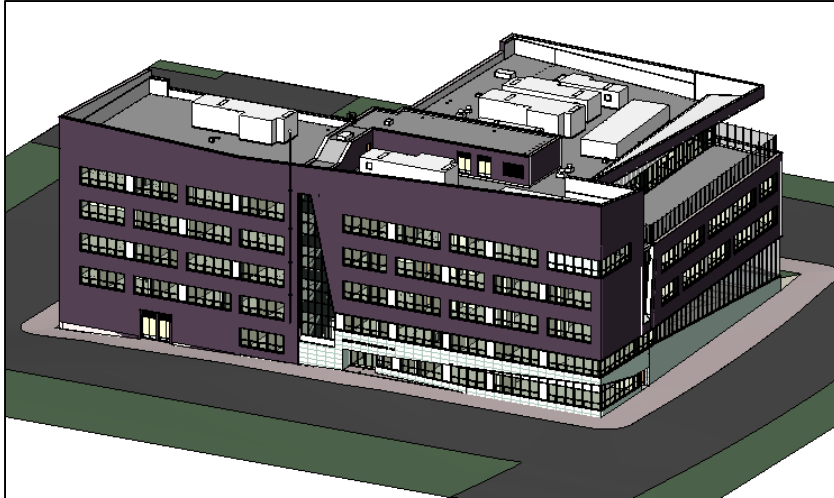


Figure 6: Building Perspective
Rendering by SKANSKA Inc.

District 75 students. The third floor contains a full sized 2 story auditorium that links the high school (HS) and intermediate school (IS) together, along with IS classrooms and IS support rooms/offices. The fourth floor contains high school classrooms with support rooms/offices and

access to the auditorium.

The fifth floor contains HS and IS cafeterias with a

central kitchen space, a connecting 4000sf roof terrace, science labs, and support rooms/offices for the high school. There is a small mechanical penthouse on the top roof.

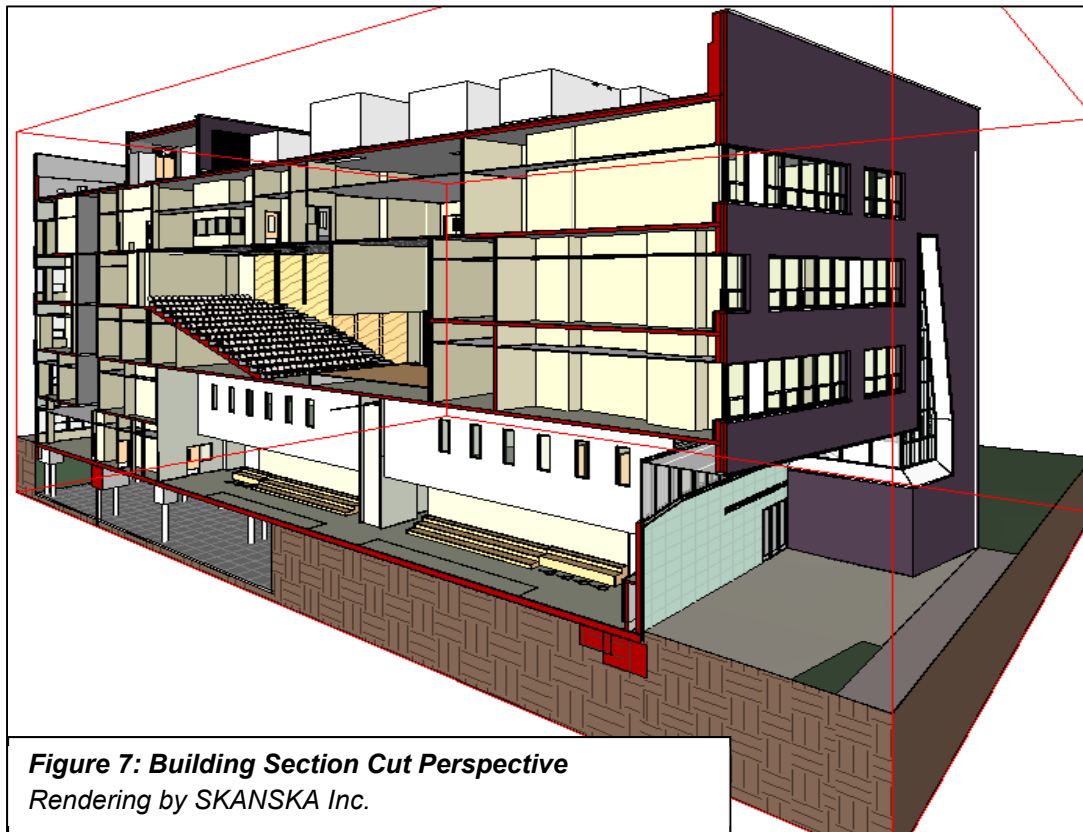


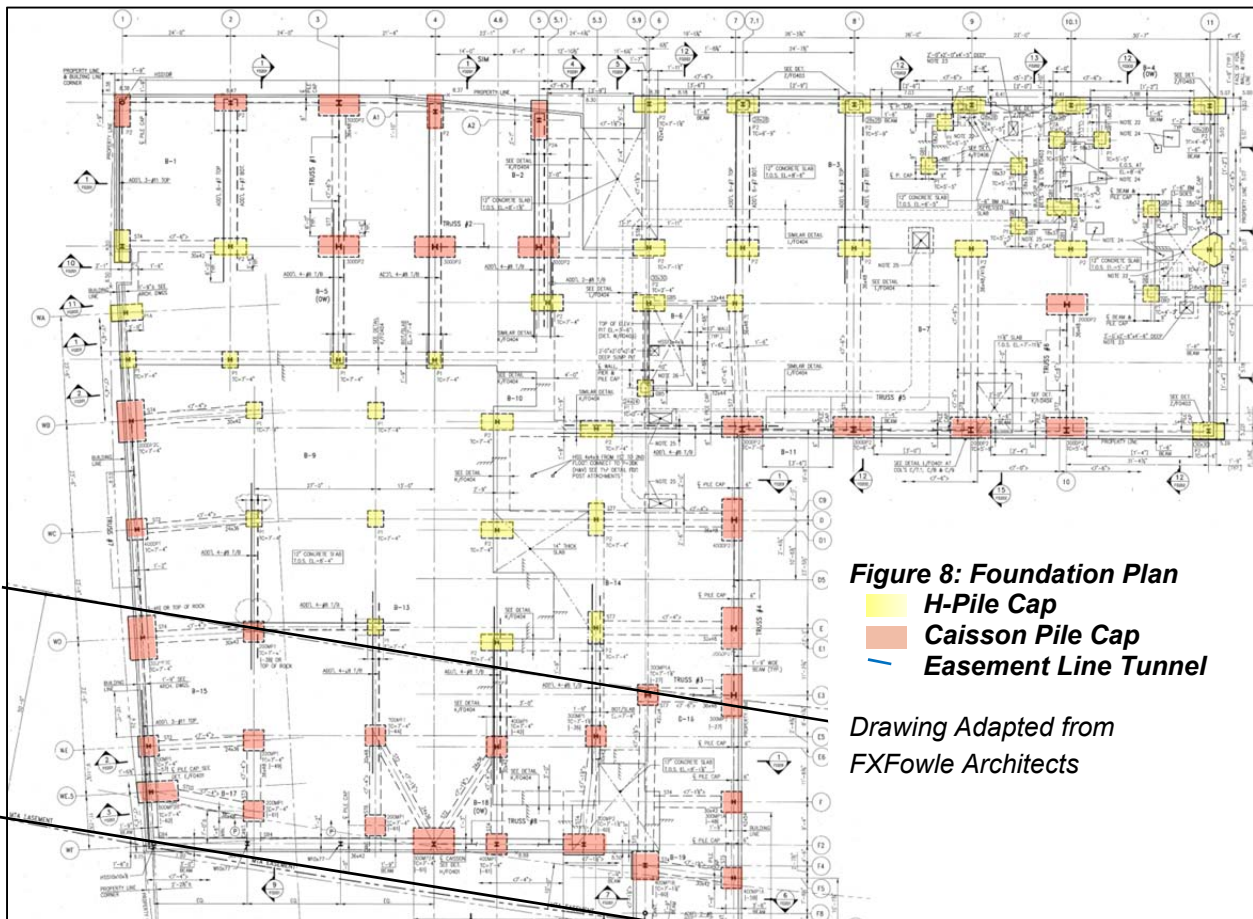
Figure 7: Building Section Cut Perspective
Rendering by SKANSKA Inc.

STRUCTURAL SYSTEMS

This section provides a brief overview of the different structural systems implemented in the Hunter's Point design. The structure consists of a steel framing system with concrete on metal deck floors. There are no subgrade levels, and structural height of the building is 72.3 feet to the roof level with a 13.5 foot parapet wall extending above. All exterior walls are non-loadbearing brick, slate, aluminum panel, or glazing. CMU masonry infill walls are used as a backup wall and are grout filled and reinforced against lateral forces. The steel frame makes up both the gravity and lateral load systems of this building.

Foundation

The foundation consists of a 12 inch 4000 psi reinforced slab on grade supported by a system of grade and strap beams, 14 inch caissons, and steel H-piles. All of these different foundation systems are required due to the poor soil properties on site. A geotechnical survey performed by Langan Engineering showed soil type ranges from grey silty sand fill to clay, with bedrock consisting of gneiss starting at about 40 feet below grade. Deep foundations are installed to at



THESIS REDESIGN

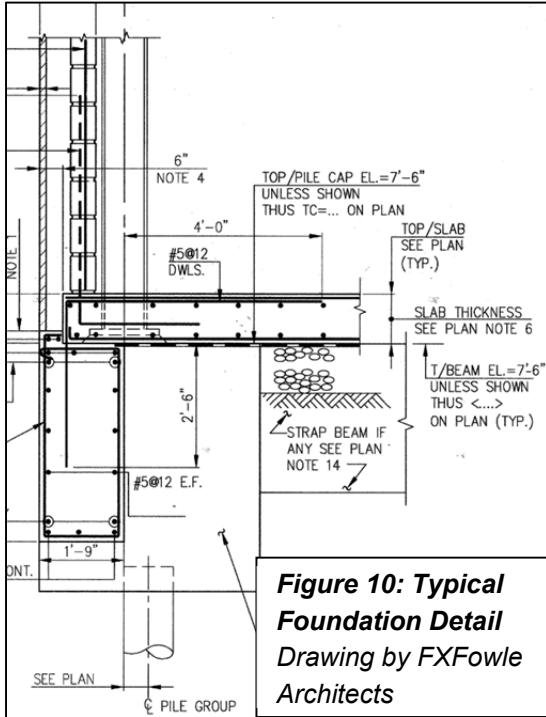


Figure 10: Typical Foundation Detail
 Drawing by FXFowle Architects

least this level. H-piles are used mainly within the interior and in the upper north east corner of the site where soil conditions are better.



Figure 9: Isolation Casing
 Photo by SKANSKA Inc.

Caissons are installed around the perimeter to help stabilize the building and take the majority of the dead load as it passes down and outward through the structural system. Special isolation caissons, as seen in **Figure 11**, were used for locations within 50 feet of two subsurface tunnels

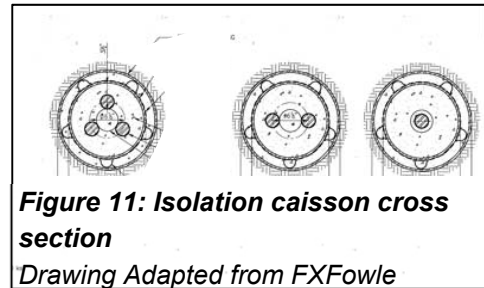


Figure 11: Isolation caisson cross section
 Drawing Adapted from FXFowle

used for the Queens-Midtown Tunnel easement lines that run E-W through the site. Each caisson has three 20 inch 75 ksi steel threadbars within 8000 psi grout, and can support up to 800kips of compressive force. Ground and strap beams are used to connect pile caps to help prevent lateral column base movement.

Floor and Roof Systems

As seen in **Figure 12**, the floor system consists typically of 3-¼ inch thick 3500 psi lightweight concrete on 3 inch deep composite 18 gage galvanized metal deck (6-¼ inch total depth) supported by a steel framing system. Concrete is reinforced with 6x6 W2.0xW2.0 WWF. The floor system above the gymnasium uses acoustical metal deck in place of typical deck. The auditorium stadium seating floor will have 16 gage deck in place of typical deck. The typical unsupported span length for the floor deck is 12 feet. All cast-in-place concrete slabs are reinforced by #4 reinforcing

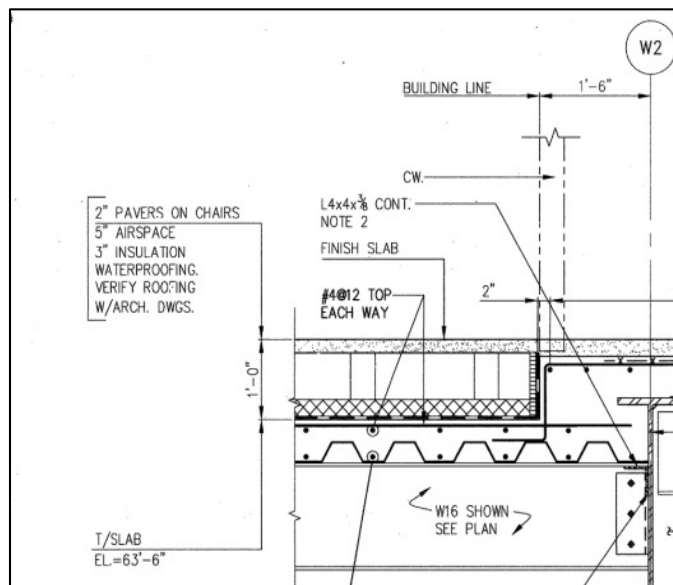


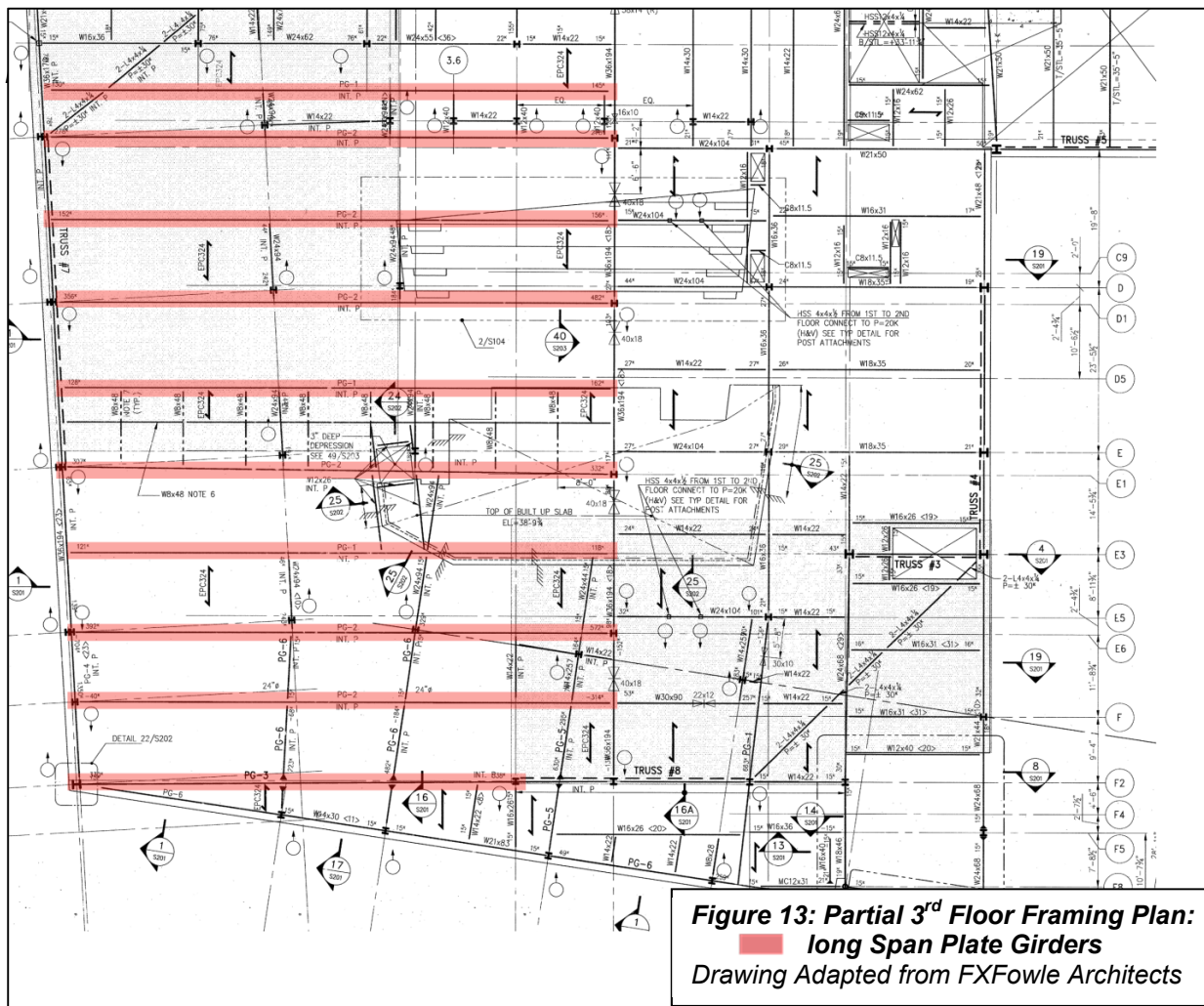
Figure 12: Typical floor system
 Drawing by FXFowle Architects

THESIS REDESIGN

bars spaced 12 inches in both directions. The top roof and terrace roof will have 2 inch thick lightweight concrete pavers over hot applied asphalt roofing membrane on top of the concrete slab.

Framing System

The superstructure of Hunter's Point is typically comprised of W10-W14 steel columns supporting W24 girders and either W14 beams at the building core or W16 beams towards the perimeter of the structure. Overall, sizes and span lengths vary greatly throughout the building and across every floor. The third floor includes special long span plate girders over the gymnasium space (red box, **Figure 13**). Spanning roughly 80 feet each with a flange thickness



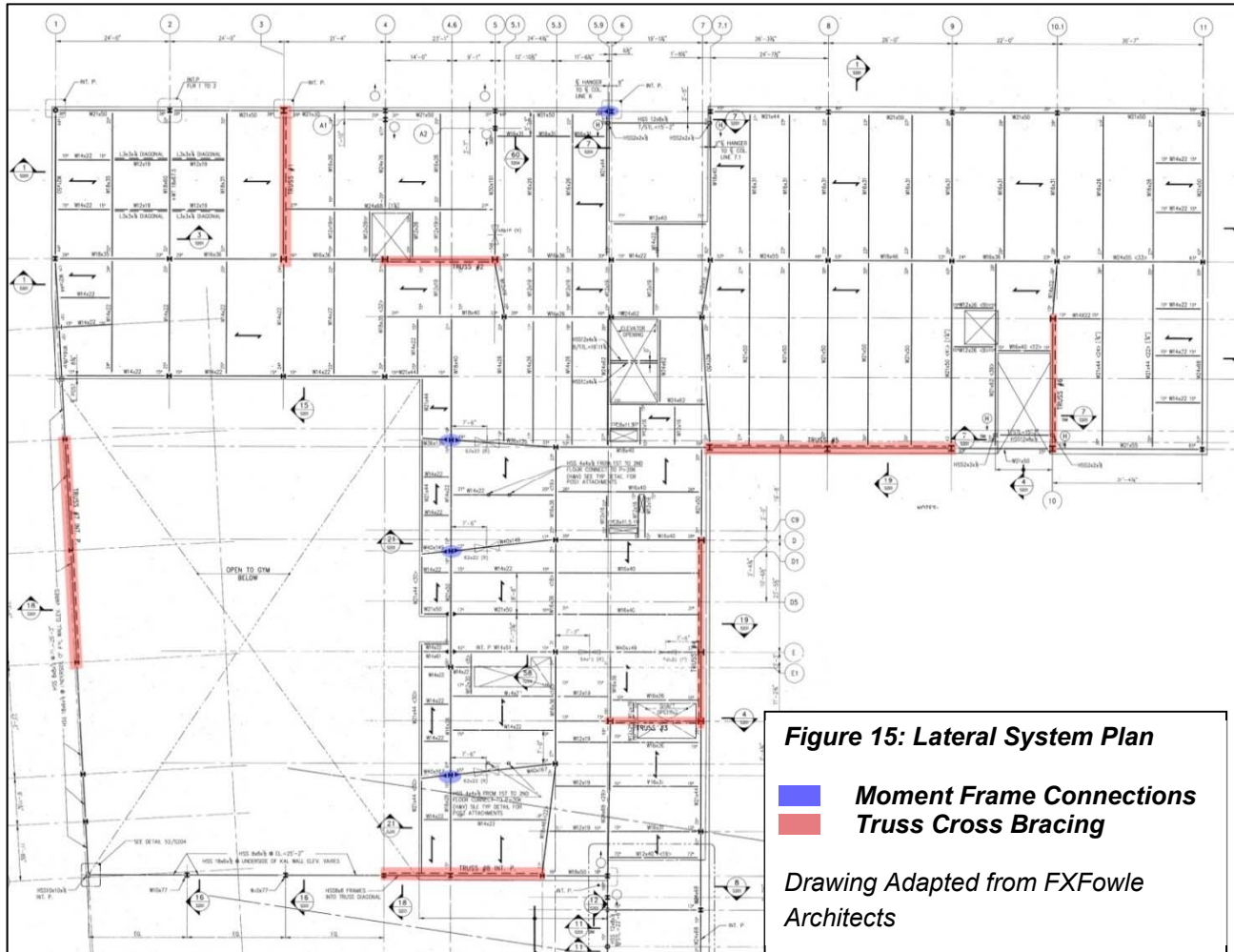
THESIS REDESIGN



Figure 14: Steel Frame Erection
Photo by SKANSKA Inc.

of 2-4 inches and overall depth of up to 3 feet, these large transfer beams allow for open gym space while adequately supporting the load transferred from the auditorium and cafeteria space in the floors directly above. Gravity loads are transferred from the floor slab to the wide flange beams then to interior and exterior columns down to the foundation system. Exterior walls and cladding transfer their weight to exterior beams.

Lateral System



The lateral force resisting system consists of both HSS and wide flange lateral truss bracing (red box, **Figure 15**), along with steel moment connections at columns around the gymnasium space (blue circles, **Figure 15**). There are six different types of truss bracing systems, two of which are shown in **Figure 16** to the right. Single bay trusses are primarily used along interior spaces, while stiffer double bay trusses are implemented along the exterior wall where there is more room. Several of

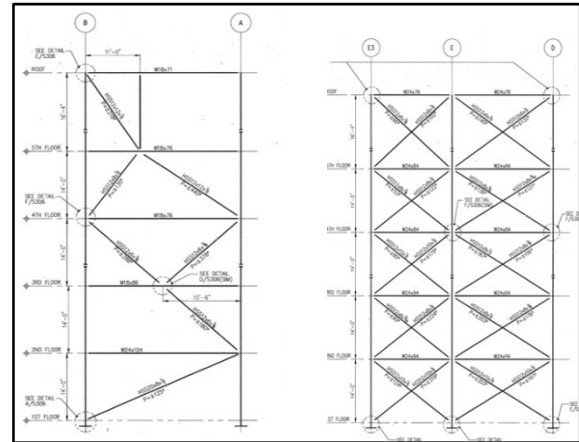


Figure 16: Two types of lateral bracing used in the design

Drawing by FXFowle Architects



Figure 17: Lateral bracing erected

Photo by SKANSKA Inc.

the truss systems allow for architectural use and have odd cross bracing, such as the left truss in **Figure 16**. Trusses run in both the N-S and E-W directions. The first floor implements lateral force resisting systems the most. This is due to the 2 story cavity formed in the framing system to allow for open gym space. A 3D model of the lateral system can be seen in **Figure 18** below.

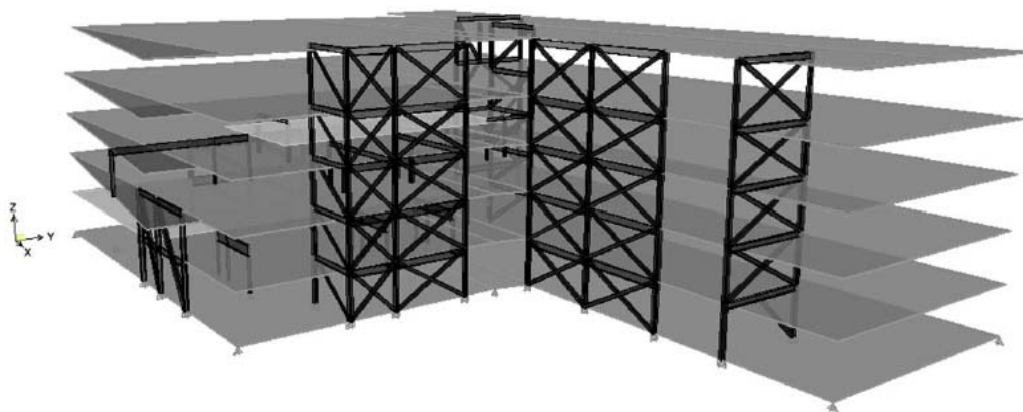


Figure 18: ETABS MODEL: Lateral Force Resisting System

DESIGN CRITERIA

This section provides data regarding codes, materials, and gravity loads for the design of Hunter's Point South. This thesis project will differ from the original design in that it will implement design criteria from ASCE7-10 and IBC 2009 rather than the NYCBC 2008 building code. There are several reasons for doing this. First of all, obtaining outdated copies of the NYCBC and other code books is not an option due to availability. The NYCBC also references the IBC and ASCE7 throughout, so much of the design will be the same. The only issue with using newer codes is that they may have different design procedures, which may change the design slightly. However, using codes up to today's standards will be most beneficial for future use and creating a code compliant redesign.

CODES & REFERENCES

Design Codes

Building Code

- New York City Building Code, NYCBC 2008, (2008)

Reference Codes

- American Concrete Institute Building Code, ACI 318-02, (2002)
- American Institute of Steel Construction, AISC 9th edition (1989)

Thesis Codes

Building Code

- International Building Code, IBC 2009 (2009)

Reference Codes

- American Institute of Steel Construction, AISC 14th edition (2011)
- American Society of Civil Engineers, ASCE 7-10 (2010)
- Seismic Provisions for Structural Steel Buildings, AISC 341-10 (2010)
- Seismic Design Manual, AISC 327-05 (2005)

MATERIAL STRENGTHS

Design Materials and strengths were found in the construction drawings on page S001 and in general notes on individual framing plans.

Table 1: Material Strengths

Material Strengths			
Material	Element	Type	Strength
Cast-in-Place Concrete	Pile Caps under Columns	Normal Weight Concrete	f'c= 5950 psi
	Grade & Strap Beams	Normal Weight Concrete	f'c= 4000 psi
	Column Pier and Buttress	Normal Weight Concrete	f'c= 4000 psi
	Slab on Grade	Normal Weight Concrete	f'c= 4000 psi
	Floor Slab	Light Weight Concrete	f'c= 3500 psi
Reinforcing Steel	Concrete Reinforcing bars		FY= 60 ksi
	Caisson Steel threadbars		Fy= 75 ksi
Structural Steel	Steel Wide Flange Members	ASTM A992	Fy= 50 ksi
	Steel HSS Tubes	ASTM A500	Fy= 46 ksi
	Steel Base Plates	ASTM A572 gr 50	Fy= 50 ksi
	Steel Deck	ASTM A653	Fy= 40 ksi
	Steel Bolts	ASTM A325	Fu= 120 ksi
ASTM A490		Fu= 150 ksi	

REDESIGN PROPOSAL

PROBLEM STATEMENT

The in-depth lateral system analysis performed in Technical Report III showed that Hunter's Point South was adequate at supporting the controlling seismic load case. As an academic exercise, the structure will be moved to a site in a higher seismic zone on the west coast and be analyzed to determine if the lateral system will withstand the increased lateral seismic forces.

Redding, California is chosen as the new building site. This site is chosen because it is a city with almost the same latitude (40.7°), elevation (400 feet), and climate (wind/precipitation/temperature) as the current location. The only main *design* difference is Redding's increased spectral response accelerations prescribed by ASCE7-10 Figures 22-1 and 22-2 for seismic design. The existing lateral system will need to be reevaluated, and perhaps redesigned, to resist the increased earthquake loading.

This redesign will be analyzed to determine if the integrated school building can feasibly be constructed in an area with more rigorous code requirements. The redesign will be designed using two different seismic design methods prescribed by ASCE7-10, and the results will be compared.

REDESIGN PROPOSAL

PROPOSED SOLUTION

The redesign of Hunter's Point South will be a steel design with eccentrically braced lateral load resisting frames. The new lateral system will be modeled in ETABS, and be analyzed under two separate seismic design methods. The first will be the Equivalent Lateral Force Analysis (ELFA), and the second will be the Modal Response Spectrum Analysis (MRSA).

The alternate floor system analysis performed in Technical Report II proved that the original steel deck on steel frame system was one of the most economic for this structure. Therefore, this thesis redesign will implement the original system. Due to the increased response accelerations found in ASCE7-10, the new site will most likely fall under seismic design category (SDC) D rather than SDC C as it was originally designed for (ASCE7-10 Table 11.6-1). This SDC does not permit the use of the original lateral system, which was comprised of ordinary steel moment frames around the gymnasium and auditorium spaces and concentrically braced frames located throughout the rest of the building. Therefore, to comply with code, eccentrically braced frames will be implemented in place of the original lateral system.

The placement and number of eccentrically braced frames must also be reconsidered in the redesign. This will differ between the two design methods. The original lateral design created an overall torsional irregularity in the structure. Though this was acceptable in SDC C, ASCE7-10 SDC D requires that no such irregularity exists if the Equivalent Lateral Force Analysis is to be used to design the structure for seismic loads. However, if the Modal Response Spectrum Analysis is used, no such requirement exists. Therefore, there is a possibility that the lateral system will not have to be as oversized.

The new lateral system will have an effect on the foundation design. Therefore, localized pile type and pile location may change to function as a suitable foundation for the axial forces caused by the eccentric bracing under seismic loading. No other structural systems should be greatly affected by the lateral system redesign.

REDESIGN PROPOSAL

M.A.E. GRADUATE COURSE INTEGRATION

The redesign of Hunter's Point South School will implement material from several courses that are part of the Master of Architectural Engineering program. The redesigned structure will be modeled in ETABS using knowledge gained in AE597A (Computer Modeling). The design of eccentric braced frames to resist seismic loads will reference material taught in AE538 (Earthquake Design). Material learned in AE534 (Steel Connections) will be used to design typical steel connection details included in the redesign.

BREADTH STUDY 1: ARCHITECTURAL IMPACT

The increase in lateral load will require more lateral support in the building. By adding new braced frames, changing moment frames to braced frames, and moving frame locations to prevent building torsion, the redesign of Hunter's Point South can have an impact on the architectural layout of the building. An architectural breadth study will be completed to see if the new lateral system designs will work with the current building layout (both functionally and visually), or if changes must occur. This analysis will mainly focus on the locations of the gymnasium and auditorium spaces, as well as new locations of eccentrically braced frames. A redesign of the exterior façade and interior spaces will be implemented as needed and presented through revised floor plans, elevations, and section cuts.

BREADTH STUDY 2: CONSTRUCTION AND COST IMPACT

The impact of the redesign on the cost and construction schedule of the Hunter's Point project will be analyzed in the second breadth study. First, the current schedule and cost estimate will be evaluated against each new redesign to see the effect seismic zoning has on the structure. Along with changes in such things as location factors, each new design will create a new critical path schedule in the construction of the structure that will ultimately change both the construction time and overall construction costs. Then, a comparison between the ELFA and MRSA redesigns will be done to establish whether the MRSA process is worth the extra design time in saving cost and construction time.

REDESIGN PROPOSAL

SUMMARY

The structural depth for this thesis is an academic exercise that will be to redesign the lateral force resisting system of Hunter's Point South School after moving the building site to a higher seismic zone in Redding, California. To comply with more stringent code requirements, the Equivalent Lateral Force Analysis (ELFA) and Modal Response Spectrum Analysis (MRSA) found in ASCE7-10 Section 12 will be used to design two new lateral systems using only eccentrically braced frames. Each new redesign will be analyzed to determine its effectiveness, and be compared to the current design (which is not for high seismic zones) to determine the practicality of implementing the overall structural design on a more universal level. This depth study will also look at the advantages of using a more in-depth seismic response analysis (MRSA) when developing a lateral system in a high seismic zone.

An architectural breadth study will be performed to determine if the new lateral system will obstruct the architectural layout in either a functional or visual manner. Solutions will be suggested if any such obstructions exist. A second breadth study will be developed to analyze the construction impact each redesign will have. Both new designs will be compared to each other, and to the current design, to determine the effect each has on the schedule and cost estimate of the project.

STRUCTURAL DEPTH

INTRODUCTION

For the structural depth study of this thesis project, the building is relocated from New York City to Redding, California, and it is redesigned to withstand the increased loading caused by the higher seismic zone. The lateral system of Hunter's Point South is redesigned to incorporate eccentrically braced frames (EBF's) rather than the concentrically braced vertical trusses (CBF's) and moment frames the original design used. The redesign is necessary due to code requirements set by the IBC that state that Ordinary CBF systems are not allowed in high seismic zones (Seismic Design Category (SDC) D or higher from ASCE 7-10). An EBF system was chosen to replace the original design because of its high ductility and resistance to seismic loading, and because an EBF system will work best with the current structural layout to prevent excessive and costly changes to the rest of the building design.

The original design for the lateral system can be seen in **Figure 19**. This layout uses CBF cross bracing in 4 different locations in both directions, as well as several moment frames around the gymnasium space. A 3D model of the lateral system was created in ETABS for analytical purposes, and can be seen in **Figure 20**. After running an analysis on this design under the original loading for New York City, it was found that the system was adequate in supporting the current lateral seismic load while keeping story drift to a minimum, but a torsional irregularity was present. That is, at least one corner of a floor rotated under seismic loading and exceeded a limit of story drift set by ASCE 7-10 as 1.2 times the average story drift for two ends of a floor. Analysis of the original design can be found in the Appendix of this report.

Using ASCE 7-10 Section 12, it was determined that Hunter's Point South would be a SDC D building in its new location, and it would require a more sophisticated lateral system than the original design to withstand the increased seismic design load. Greater strength and ductility were going to be necessary for proper strength and serviceability requirements. Once the building design was moved to Redding, California, an EBF system had to be designed to create a new lateral system that would pass code standards.

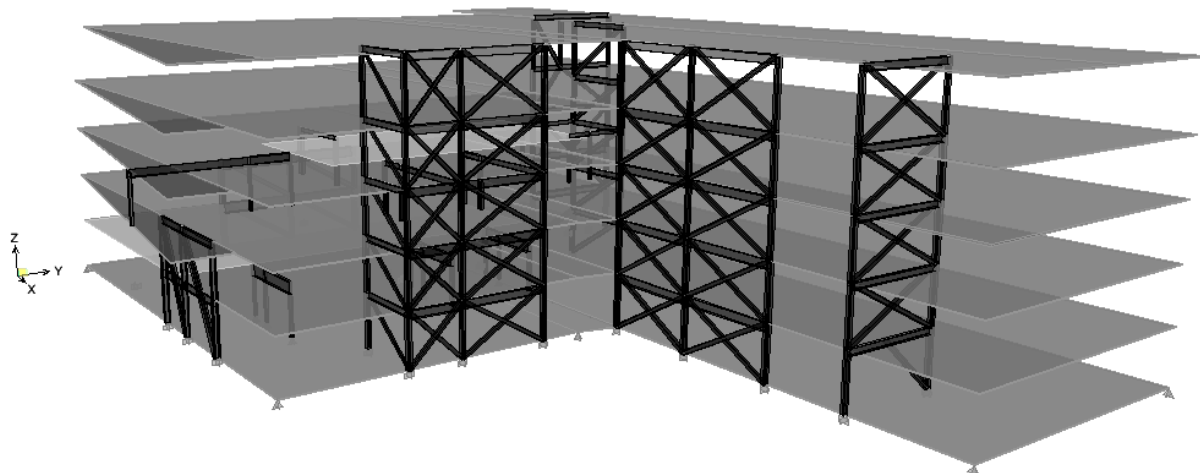
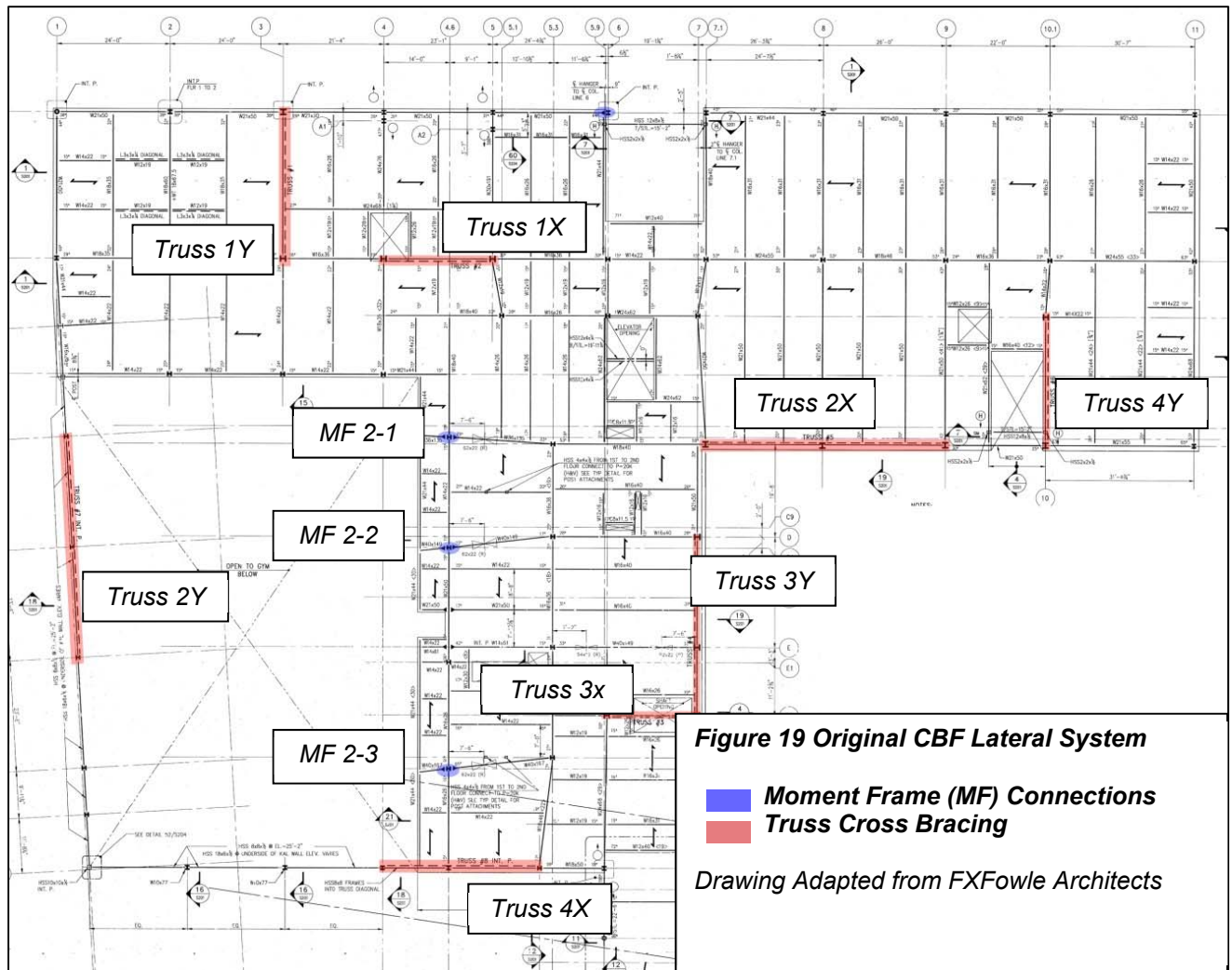


Figure 20: Lateral System - Original Design

ECCENTRICALLY BRACED FRAMES

Research included in this section references information gathered from AISC, ASCE, and “Seismic Design Practice for Eccentrically Braced Frames” by the Structural Steel Education Council (SSEC). Eccentrically braced steel frames, or EBF's, are similar to concentrically braced frames (CBF) in that they use some type of lateral bracing between steel frames to take the lateral load of a building. There are also several different patterns of bracing, some of which include 1 or two braces. One of the more common patterns in EBF design is chevron bracing (upside down V). This is what will be used in the redesign to replace the cross, or X type, bracing the CBF system used. That is really where the similarities end, and the differences begin between these two lateral systems. EBF systems stay true to their name, such that the bracing is no longer concentrically braced to the supporting frame. That is, there is an eccentricity, or gap, between two brace connections or a connection and column. This eccentricity causes shear and moment forces to develop in the short portion of the beam between the bracing. This small beam portion is referred to as the link.

This link is the primary support for the lateral load. Stiffness is controlled by changing the size of the eccentricity. Shorter link lengths create a stiffer frame (similar to CBF) and longer links create a more flexible frame (like moment frames). To work properly, the link must deform inelastically under loading, while the rest of the system stays elastic. For this to happen, outside members and connections must be designed and detailed accordingly, and the link must exhibit significant ductility and energy dissipation (SSEC). Outside members usually are given an amplification factor on the normal design loads to allow for an overstrength factor that creates elastic response.

Link length is very important to design. The longer it is, the more it is affected by bending, while shorter members are governed more by shear forces. AISC 327-05 referenced code states the following for link design:

$e < 1.6 M_s/V_s$ = Shear yield controls design

$e > 2.6 M_s/V_s$ = Flexure yield controls design

$e = 2.0 M_s/V_s$ = Balanced design (AISC 327-05.3)

Since shear yield is more reliable than yielding due to flexure, it is recommended that link length be designed to the first equation above. AISC also warns the designer not to go below 1.3 times the ratio, but rather stay close to the upper limit to promote minor link rotation (which can be difficult to design for and cause failure easier). Countless testing and research has proven that this value for link length creates a very successful

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frame that shows good ductility and suitable hysteretic response. This paper will not cover any of this research, as it focuses more on the overall building response and not the individual frames. However, links will be designed to fit the criteria for a shear controlled system.

In "Seismic Design Practice for Eccentrically Braced Frames" by the SSEC, design of EBF's is broken down into 5 main steps to create a quick way to iterate to final design:

- 1. Establish the design criteria**
- 2. Identify a bracing configuration**
- 3. Select link length**
- 4. Choose appropriate link section**
- 5. Design braces, column and other components of the frame**

These steps are used during the design of the EBF systems for this project. To design an EBF system, it is recommended that a structural analysis program be used for quick iterations (SSEC). To design properly, a 3D building model of the lateral system will need to be enhanced as frame location, building period, force distribution, and link properties change to fulfill the design requirements. ETABS will be used for the designs.

The first step uses design load analysis covered in each redesign section that follows to come up with proper design criteria. As stated before, a chevron type bracing is chosen for the EBF frames. This is done because it will allow for the most usable architectural space in between the frames, such that redesign of any architectural aspects due to new bracing locations will be kept to a minimum. Analysis of bracing location is covered in each of the redesign sections that follow.

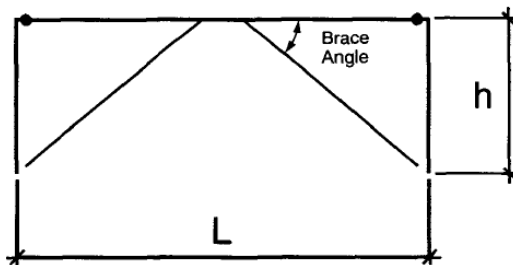


Figure 21: Eccentric Chevron Brace
Adapted from SSEC Figure 1 Intro to EBFs

SSEC tells designers to start link design by choosing a link length of 15% the beam length. This was done for the initial EBF design for this paper. Once all frames had an initial length, a uniform length was chosen and used on all frames for simplicity. Each frame was then checked to make sure link length was adequate to have shear yielding

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control. This thesis uses a link length, $e = 48$ inches. SSEC also tells designers to choose a link length that will create a brace angle between 35° - 60° to prevent unwanted axial loads in the link and other issues (See **Figure 21**). Checking this geometry for each EBF design showed that all but a few frames fell in this range (just below the minimum). Further analysis showed that axial forces were not an issue in any frames in either redesign, and the initial link length was kept.

Finally, using ETABS steel design function, hand calculations, and AISC 327-05.3, link section properties and other member sizes are found. The program designs all members at once, and spot checks are used to confirm the accuracy of the programs assumptions (which many were manually inserted into the program before design began). Iterations are done until a suitable system is found and all member sizes are adequate at taking the load and remaining elastic while the link is able to deform plastically and give the system the ductility required for high seismic loading.

ELFP DESIGN

The first redesign uses the Equivalent Lateral Force Procedure (ELFP) from ASCE 7-10 Section 12.8 to find the design lateral loads caused by seismic loading. This procedure is first chosen because it is the same procedure prescribed in the original design. This helps to understand the direct difference location and system design have on the overall lateral system performance as compared to the original design. To use ELFP in a high seismic region (SDC D), however, torsional irregularity must be eradicated from the system. Taking this into account with the increased seismic zone creates the issue that the ELFP redesign could become an inefficiently expensive design. This is dependent on how much ductility can be developed from the EBF design, which allows the design loads to be decreased dramatically.

After using ELFP, the design loads were found in the form of story shear forces and overturning moments due to seismic forces in both E-W and N-S directions. This data can be seen in **Table 2** and **Table 3**. The total shear was 850 kips and the max overturning moment was 7550 kip-feet. This shear is about 85% of the original design. This can be attributed to the ductility of an EBF system which divides the forces by a Response Modification Factor (R-Factor) of 8. This factor is explained more fully in the connection design section of this report. Like the original design analysis, these forces were put into ETABS to simulate forces in all four cardinal directions.

Table 2: E-W Design Loads for ELFP Design

East-West Direction Loading (ELFP)											
										T= 1.042 s k= 1.271 V _b = 849 kips	
i	h _i	h	w	w*h ^k	C _{vx}	f _i	v _i	B _y	5%B _y	A _x	M _z
	ft	ft	kips			kips	kips	ft	ft		k-ft
6	16.33	72.33	2945	679089	0.407	346	346	178	9	1	3075
5	14	56	2563	426996	0.256	217	563	178	9	1	1933
4	14	42	2277	263185	0.158	134	697	178	9	1	1192
3	14	28	3500	241647	0.145	123	820	178	9	1	1094
2	14	14	1978	56595	0.034	29	849	178	9	1	256
1											
			Σ	13263	1667511		849 =V				7550

Table 3: N-S Design Loads for ELFP Design

North-South Direction Loading (ELFP)											
										T= 1.042 s k= 1.271 V _b = 849 kips	
i	h _i	h	w	w*h ^k	C _{vx}	f _i	v _i	B _x	5%B _y	A _x	M _z
	ft	ft	kips			kips	kips	ft	ft		k-ft
6	16.33	72.33	2945	679089	0.407	346	346	131	7	1	2263
5	14	56	2563	426996	0.256	217	563	131	7	1	1423
4	14	42	2277	263185	0.158	134	697	131	7	1	877
3	14	28	3500	241647	0.145	123	820	131	7	1	805
2	14	14	1978	56595	0.034	29	849	131	7	1	189
1											
			Σ	13263	1667511		849 =V				5557

Once the forces were placed in the program, design could begin. Design using ETABS was done through iteration. To start, the original bracing layout was used. This design yielded appropriate strength, but failed in torsional irregularity (which was expected). Several new layouts were chosen that would work with the current architectural layout

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and help to prevent torsional irregularity, and they were tested using ETABS. Note that iterations are not shown in this report. Once a general layout was found, ETABS Steel Design was used to design lateral system member sizes. Using hand calculations to check the compatibility of the computer design, more iterations were done until a final design was found that passed strength and serviceability limits and was efficient. The layout for the ELFP EBF design can be seen in **Figure 22**. The ETABS 3D model of the ELFP design can be seen in **Figure 23**. Elevations of each individual bracing frame can be found in the Appendix of this report.

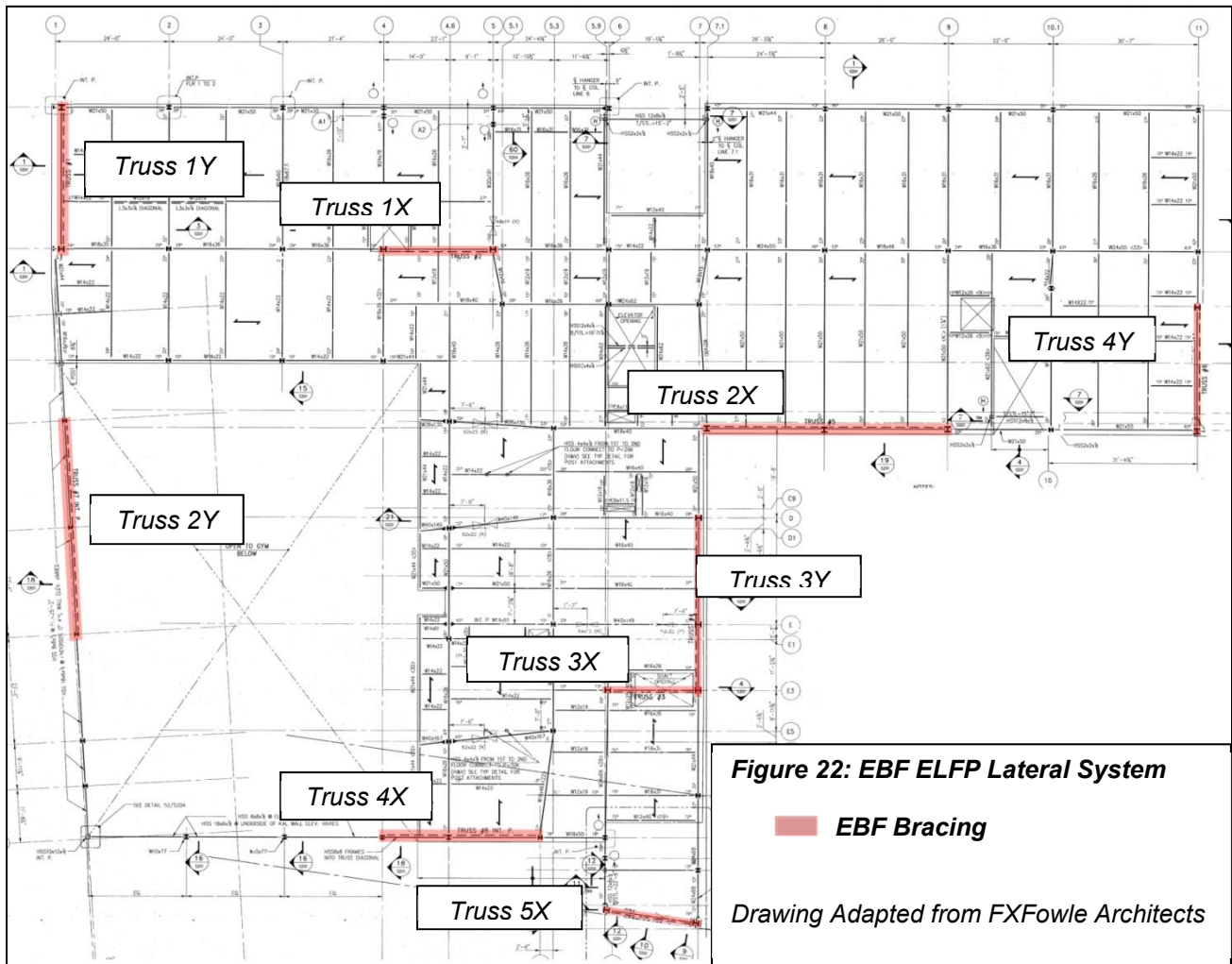


Figure 22: EBF ELFP Lateral System

EBF Bracing

Drawing Adapted from FXFowle Architects

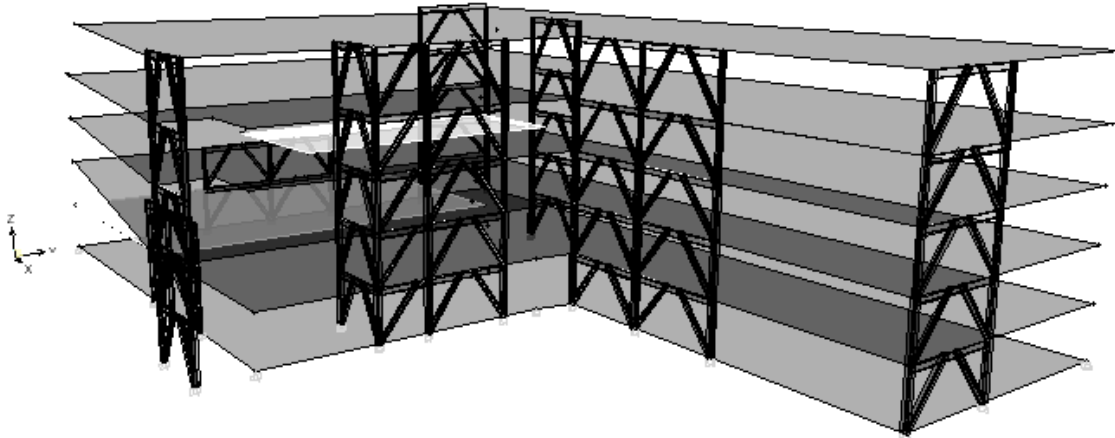


Figure 23: Lateral System – ELFP EBF Redesign

MRSA DESIGN

The second lateral system redesign of Hunter's Point South uses the Modal Response Spectrum Analysis (MRSA) from ASCE 7-10 Section 12.9 to find the design lateral loads caused by seismic loading. The MRSA method is a more detailed analysis than ELFP, but often gives significantly lower forces (often less conservative but more accurate). MRSA also does not require such irregularities as torsional irregularity to be prevented in high seismic regions, which could have huge implications on this specific design. This procedure is often used in high seismic regions, including much of the West coast. Many municipalities have it in their local building code that a procedure at least as accurate as MRSA must be used to design lateral systems for buildings (i.e. no ELFP). For this project, it was automatically assumed that ELFP would be acceptable in Redding California, so as to see the difference in each procedure.

MRSA uses an analysis of building modes under lateral loading to distinguish the ductility and forces each frame receives. According to ASCE7-10, enough modes to account for 90% of the building mass must be analyzed for accurate results. Looking at **Table 4**, Hunter's Point South required six modes to be analyzed.

Table 4: MRSA Modal Mass Participation from ETABS Analysis

Modal Mass Participation (MRSA)													
Mode	Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
1	0.930839	44.7871	4.569	0	44.7871	4.569	0	6.0038	55.9326	27.1556	6.0038	55.9326	27.1556
2	0.881182	24.3712	39.084	0	69.1582	43.653	0	50.8151	31.9584	9.2281	56.8189	87.891	36.3836
3	0.806855	8.2734	34.1372	0	77.4316	77.7903	0	42.755	11.4781	33.2076	99.5739	99.3691	69.5912
4	0.365144	4.0036	3.3429	0	81.4352	81.1331	0	0.1729	0.0034	9.8794	99.7469	99.3724	79.4706
5	0.343723	10.1586	5.2326	0	91.5938	86.3657	0	0.0862	0.2831	1.9135	99.833	99.6555	81.3841
6	0.315976	2.1517	6.6411	0	93.7456	93.0068	0	0.0248	0.1739	8.1346	99.8578	99.8294	89.5187
7	0.22163	0.0317	2.0073	0	93.7772	95.0141	0	0.1012	0.0292	5.2921	99.959	99.8586	94.8108
8	0.201219	3.3601	0.046	0	97.1373	95.0601	0	0.0013	0.1246	0.1467	99.9604	99.9832	94.9575

Next, different design parameters, such as story drift and member forces, must be computed for each mode. This is done using modal properties and referencing the design spectrum created for the building site. For this project, ETABS was used to create the simulated design spectrum and properties. ETABS also combines the modal properties through a process called square root of the sum of the squares (SRSS) to create an equivalent total force used for design. This is hand checked against ASCE7-10 12.9.4.2 to make sure that a max of 15% decrease in base shear load from ELFP is found. A scaling factor is employed into ETABS to correct for the error and final design loads are computed. These design loads can be found in **Table 5** and **Table 6**.

Table 5: E-W Design Loads for MRSA Design

East-West Direction Loading (MRSA)								
Floor	Story Height	Story Weight	Story Shear	Story Force	B _y	5%B _y	A _x	M _z
X	ft	kip	kip	kip	ft	ft		kip-ft
6	72.33	2945	282.20	282.20	178	9	1	2510
5	56	2563	448.54	166.34	178	9	1	1480
4	42	2277	572.44	123.90	178	9	1	1102
3	28	3500	684.63	112.19	178	9	1	998
2	14	1978	721.86	37.23	178	9	1	331
				Base Shear=	721.86	Overturning Moment=		6421

Table 6: N-S Design Loads for MRSA Design

North-South Direction Loading (MRSA)								
Floor	Story Height	Story Weight	Story Shear	Story Force	B _y	5%B _y	A _x	M _z
Y	ft	kip	kip	kip	ft	ft		kip-ft
6	72.33	2945	284.41	284.41	131	7	1	1862
5	56	2563	452.8	168.39	131	7	1	1102
4	42	2277	577.99	125.19	131	7	1	820
3	28	3500	689.62	111.63	131	7	1	731
2	14	1978	721.55	31.93	131	7	1	209
				Base Shear=	721.55	Overturning Moment=		4723

Once again, forces were found in the form of story shear forces and overturning moments due to seismic loading in both the E-W and N-S directions. For simplicity, the maximum values from the tables above were used, and story forces were set equal in both directions. This is slightly more conservative, but should not make much of a difference due to the closeness of reported load values. Therefore, the total design

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shear will be 721 Kips and the max overturning moment will be 6420 kip-feet. As expected, this is roughly 85% that of the ELFP redesign forces, or 73% of the original design forces.

Like the process in the other design analysis, these forces were put into ETABS to simulate forces in all four cardinal directions. Once the forces were placed in the program, design could begin as was done before. As was stated before, torsional irregularity no longer is a code issue when designing with MRSA. However, it should be noted that irregularities can still be problematic, and should be avoided if possible. Design using ETABS was done through iteration until a viable solution was found that yielded appropriate strength and deflection. Hand calculations (seen in Appendix) were then used to check member design. Once it was determined that the design was sufficient, further analysis could be completed. The layout for the MRSA EBF design can be seen in **Figure 24**. The ETABS 3D model of the MRSA design can be seen in **Figure 25**. Elevations of each individual bracing frame can be found in the Appendix of this report.

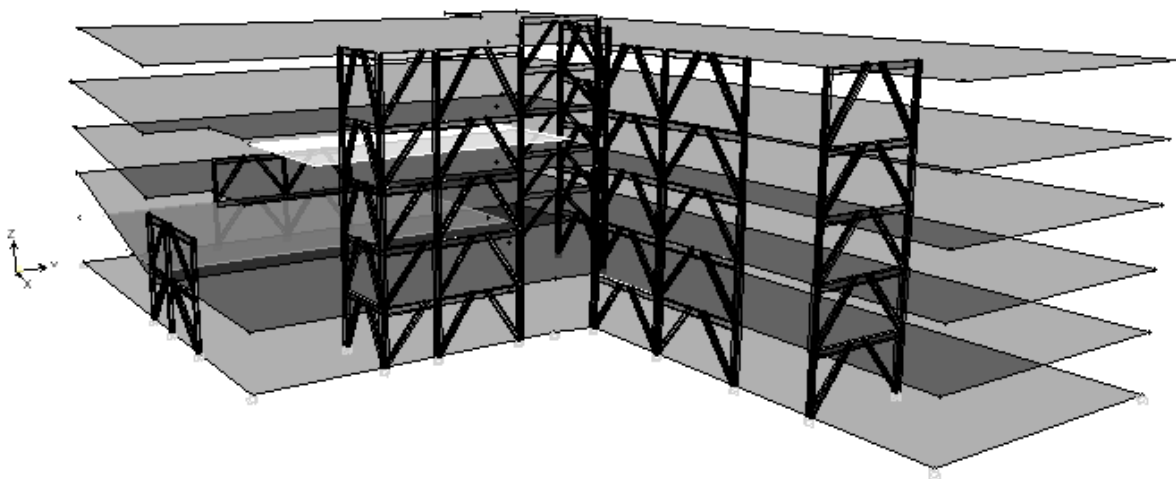
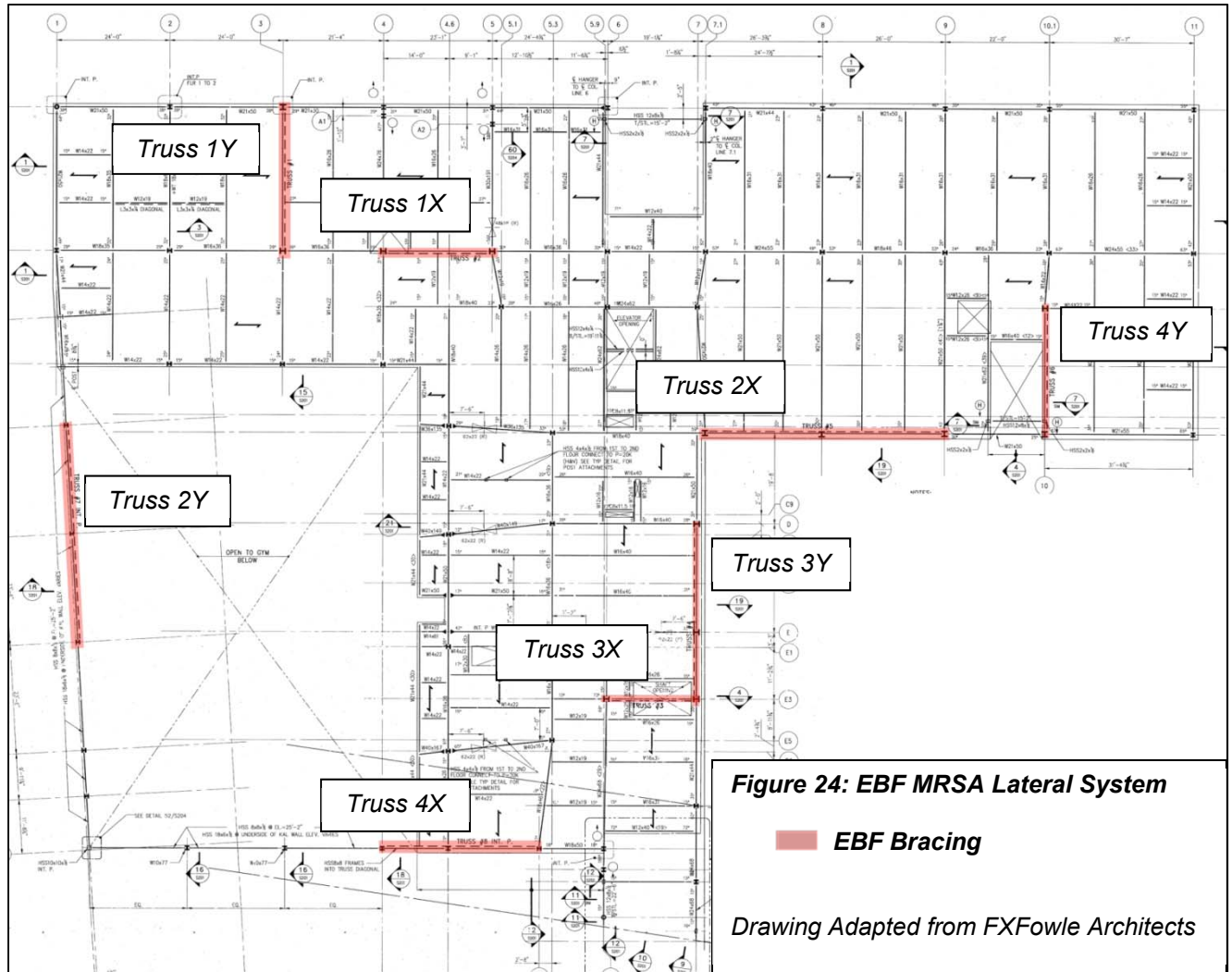


Figure 25: Lateral System – MRSA EBF Redesign

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COMPARISON BETWEEN CBF, ELFP, AND MRSA

After both the ELFP and MRSA redesigns are completed and analyzed, a comparison between the three different lateral systems can be done. This section will only focus on the overall design of each system, while the next section goes into detail about the connections of each design.

Stiffness/Deflection

As can be seen in **Table 7**, the original design had a very uneven contribution to stiffness by the lateral frames. Truss 2X, 4X, 2Y, and 3Y took the majority of the load. This was due in part by the frame size and individual member stiffness of each frame. In each of the redesigns (**Table 8** and **9**), this contribution shifted, as EBF 4X and 2Y take the majority of the load themselves. This changes the torsional movement of the building (which turns out to be good in this case), but can change the stresses of the building and floor as well. Floor stresses were checked quickly in ETABS to make sure no critical stresses formed in either redesign, and designs were deemed adequate.

Table 7: Relative Stiffness of Frames (Original CBF Design)

Relative Stiffness (Original)				
Truss	Load (P)	Displacement (Δ)	Stiffness (K)	% Contribution
East-West	Kip (k)	Inches (in)	(k/in)	in Lateral System
Truss 1X	100.0	1.11400	90	6.27
Truss 2X	100.0	0.19300	518	36.20
Truss 3X	100.0	1.90400	53	3.67
Truss 4X	100.0	0.26800	373	26.07
Moment Frame 2-1	100.0	1.67400	60	4.17
Moment Frame 2-2	100.0	1.66800	60	4.19
Moment Frame 2-3	100.0	1.66500	60	4.20
Moment Frame 4-3	100.0	1.52000	66	4.60
Moment Frame 4-4	100.0	1.03000	97	6.78
Moment Frame 4-6	100.0	1.81400	55	3.85
		$\Sigma=$	1431	100.00
North-South	Load (P)	Displacement (Δ)	Stiffness (K)	% Contribution
Truss 1Y	100.0	0.99600	100	10.65
Truss 3Y	100.0	0.28300	353	37.49
Truss 4Y	100.0	0.72800	137	14.57
Truss 2Y	100.0	0.06450	205	21.75
Moment Frame 4-1	100.0	2.06600	48	5.14
Moment Frame 4-2	100.0	1.80400	55	5.88
Moment Frame 4-5	100.0	2.34600	43	4.52
		$\Sigma=$	943	100.00

Table 8: Relative Stiffness of Frames (ELFP EBF Design)

Relative Stiffness (ELFP)				
EBF	Load (P)	Displacement (Δ)	Stiffness (K)	% Contribution
X- Direction	Kip (k)	Inches (in)	(k/in)	in Lateral System
EBF 1X	100.0	0.8948	112	14.65
EBF 2X	100.0	0.946539	106	13.85
EBF 3X	100.0	1.0251	98	12.79
EBF 4X	100.0	0.2809	356	46.66
EBF 5X	100.0	1.0868	92	12.06
		$\Sigma=$	763	100.00
Y-Direction	Load (P)	Displacement (Δ)	Stiffness (K)	% Contribution
EBF 1Y	100.0	0.749	134	16.94
EBF 2Y	100.0	0.2424	413	52.35
EBF 3Y	100.0	0.8051	124	15.76
EBF 4Y	100.0	0.8486	118	14.95
		$\Sigma=$	788	100.00

Table 9: Relative Stiffness of Frames (MRSA EBF Design)

Relative Stiffness (MRSA)				
EBF	Load (P)	Displacement (Δ)	Stiffness (K)	% Contribution
X- Direction	Kip (k)	Inches (in)	(k/in)	in Lateral System
EBF 1X	100.0	0.560843	178	15.82
EBF 2X	100.0	0.581146	172	15.27
EBF 3X	100.0	0.611979	163	14.50
EBF 4X	100.0	0.163075	613	54.41
		$\Sigma=$	1127	100.00
Y-Direction	Load (P)	Displacement (Δ)	Stiffness (K)	% Contribution
EBF 1Y	100.0	0.458787	218	17.87
EBF 2Y	100.0	0.151114	662	54.24
EBF 3Y	100.0	0.551219	181	14.87
EBF 4Y	100.0	0.629635	159	13.02
		$\Sigma=$	1220	100.00

Then, torsional irregularity was checked in all three designs. 3 identically placed corner points were placed at the roof level of each design to come up with building displacement. Using ASCE7-10 as a reference, analysis was done to see if a point deflected more than 120% of the average deflection of the story under any of the normal loadings. If this occurred, the story was marked as torsionally irregular. Results for this test can be found in **Table 10**. Values in red indicate that the point had torsional irregularity under that specific loading.

Table 10: Torsional Irregularity Check for All 3 Designs

Torsional Irregularity Check												
LOAD	Original Design				ELFP Final Design				MRSA Final Design			
x	point 100	point 60	point 61	$\bar{\delta}_{avg} * 1.2$	point 100	point 60	point 61	$\bar{\delta}_{avg} * 1.2$	point 100	point 60	point 61	$\bar{\delta}_{avg} * 1.2$
5a +	0.790	1.190	1.190	1.188	0.800	1.200	1.200	1.200	0.636	0.931	0.931	0.940
5b +	0.300	0.122	0.122	0.253	0.094	0.130	0.130	0.134	0.024	0.042	0.042	0.040
5a -	0.820	1.170	1.170	1.194	0.812	1.200	1.200	1.207	0.630	0.930	0.940	0.936
5b -	0.270	0.090	0.090	0.216	0.089	0.130	0.130	0.131	0.024	0.033	0.033	0.034
y	point 100	point 60	point 61	$\bar{\delta}_{avg} * 1.2$	point 100	point 60	point 61	$\bar{\delta}_{avg} * 1.2$	point 100	point 60	point 61	$\bar{\delta}_{avg} * 1.2$
5a +	0.220	0.249	0.350	0.359	0.220	0.250	0.370	0.372	0.167	0.189	0.250	0.263
5b +	1.340	1.370	0.750	1.272	0.950	0.940	1.210	1.290	0.700	0.698	0.737	0.861
5a -	0.168	0.194	0.323	0.310	0.198	0.285	0.420	0.423	0.157	0.180	0.280	0.276
5b -	1.300	1.320	0.780	1.260	0.933	0.920	1.170	1.254	0.690	0.690	0.700	0.834

The original CBF design was found to be irregular (matching the analysis from before). As the original design is not in a high seismic region, this irregularity passes code. However, it is still not good to have irregularity because torsion can become a huge issue under strong loads and members can fail easier.

The ELFP design was found to have no irregularity in torsion. This is good, because code requires no irregularity if ELFP is to be used in high seismic regions. The MRSA design was found to have very minimal irregularity, and only in certain loadings. ASCE7-10 does not require that MRSA use regular buildings, but, as said before, having minimal torsion irregularity helps strengthen the design of the lateral system and prevent failure of connecting parts. The ELFP system clearly wins this comparison, but at what price? This system also has added frames and costs quite a bit more to erect than the other two systems.

Story Shear Forces

Story shears for each frame can be found for their respective systems in **Table 11, 12,** and **13**. Looking at these tables, it can be seen how the forces acting through the building change with each design. For example, the original design shows the max X-direction force acting on Truss 2X, while the redesigns have max X-direction forces on EBF 4X. The force each lateral system frame sees is proportional to the stiffness of that frame, which can be found above.

Table 11: Story Shear Forces per Frame (Original CBF Design)

Story Shear Forces (kips) per Frame (Original)						
Frame	% Contribution	Roof Load	5th Floor Load	4th Floor Load	3rd Floor Load	2nd Floor Load
Truss 1X	6.27	-26.53	-43.53	-54.25	-64.41	-66.92
Truss 2X	36.20	-153.13	-251.23	-313.13	-371.77	-386.25
Truss 3X	3.67	-15.52	-25.47	-31.74	-37.69	-39.15
Truss 4X	26.07	-110.27	-180.92	-225.50	-267.73	-278.16
Moment Frame 2-1	4.17	-17.65	-28.96	-36.10	-42.86	-44.53
Moment Frame 2-2	4.19	-17.72	-29.07	-36.23	-43.02	-44.69
Moment Frame 2-3	4.20	-17.75	-29.12	-36.30	-43.09	-44.77
Moment Frame 4-3	4.60	-19.44	-31.90	-39.76	-47.21	-49.04
Moment Frame 4-4	6.78	-28.69	-47.07	-58.67	-69.66	-72.38
Moment Frame 4-6	3.85	-16.29	-26.73	-33.32	-39.55	-41.10
Frame	% Contribution	Roof Load	5th Fl Load	4th Floor Load	3rd Floor Load	2nd Floor Load
Truss 1Y	10.65	-45.06	-73.92	-92.14	-109.39	-113.65
Truss 3Y	37.49	-158.58	-260.17	-324.27	-385.00	-400.00
Truss 4Y	14.57	-61.64	-101.14	-126.06	-149.66	-155.49
Truss 2Y	21.75	-92.00	-150.94	-188.13	-223.36	-232.06
Moment Frame 4-1	5.14	-21.72	-35.64	-44.42	-52.74	-54.79
Moment Frame 4-2	5.88	-24.88	-40.81	-50.87	-60.40	-62.75
Moment Frame 4-5	4.52	-19.13	-31.38	-39.12	-46.44	-48.25

Table 12: Story Shear Forces per Frame (ELFP EBF Design)

Story Shear Forces (kips) per EBF (ELFP)						
EBF	% Contribution	Roof Load	5th Floor Load	4th Floor Load	3rd Floor Load	2nd Floor Load
1X	14.65	-50.68	-82.47	-102.09	-120.11	-124.36
2X	13.85	-47.91	-77.96	-96.51	-113.55	-117.56
3X	12.79	-44.24	-71.98	-89.12	-104.84	-108.55
4X	46.66	-161.44	-262.69	-325.22	-382.61	-396.14
5X	12.06	-41.73	-67.90	-84.06	-98.89	-102.39
EBF	% Contribution	Roof Load	5th Fl Load	4th Floor Load	3rd Floor Load	2nd Floor Load
1Y	16.94	-58.62	-95.38	-118.08	-138.92	-143.83
2Y	52.35	-181.12	-294.71	-364.85	-429.24	-444.42
3Y	15.76	-54.53	-88.73	-109.85	-129.24	-133.81
4Y	14.95	-51.74	-84.18	-104.22	-122.61	-126.95

Table 13: Story Shear Forces per Frame (MRSA EBF Design)

Story Shear Forces (kips) per EBF (MRSA)						
EBF	% Contribution	Roof Load	5th Floor Load	4th Floor Load	3rd Floor Load	2nd Floor Load
1X	15.8	44.93	71.51	91.29	109.01	114.86
2X	15.3	43.36	69.01	88.10	105.20	110.85
3X	14.5	41.18	65.54	83.66	99.90	105.26
4X	54.4	154.53	245.94	313.95	374.89	395.03
EBF	% Contribution	Roof Load	5th Fl Load	4th Floor Load	3rd Floor Load	2nd Floor Load
1Y	17.87	50.74	80.76	103.09	123.10	129.71
2Y	54.24	154.05	245.18	312.99	373.74	393.81
3Y	14.87	42.23	67.22	85.80	102.46	107.96
4Y	13.02	36.97	58.84	75.12	89.70	94.52

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The difference in lateral frame forces ultimately changes the size/strength that particular frame needs to be. For example, looking at **Figure 26** below, the 1X lateral frame can be seen for each design (original, ELFP, and MRSA). The two redesigns have EBF 1X taking more load than the original design, so it only makes sense that the frames are a bit stockier than the CBF system. The ELFP design requires just slightly more strength than the MRSA design, and members for EBF 1X are just slightly larger for ELFP.

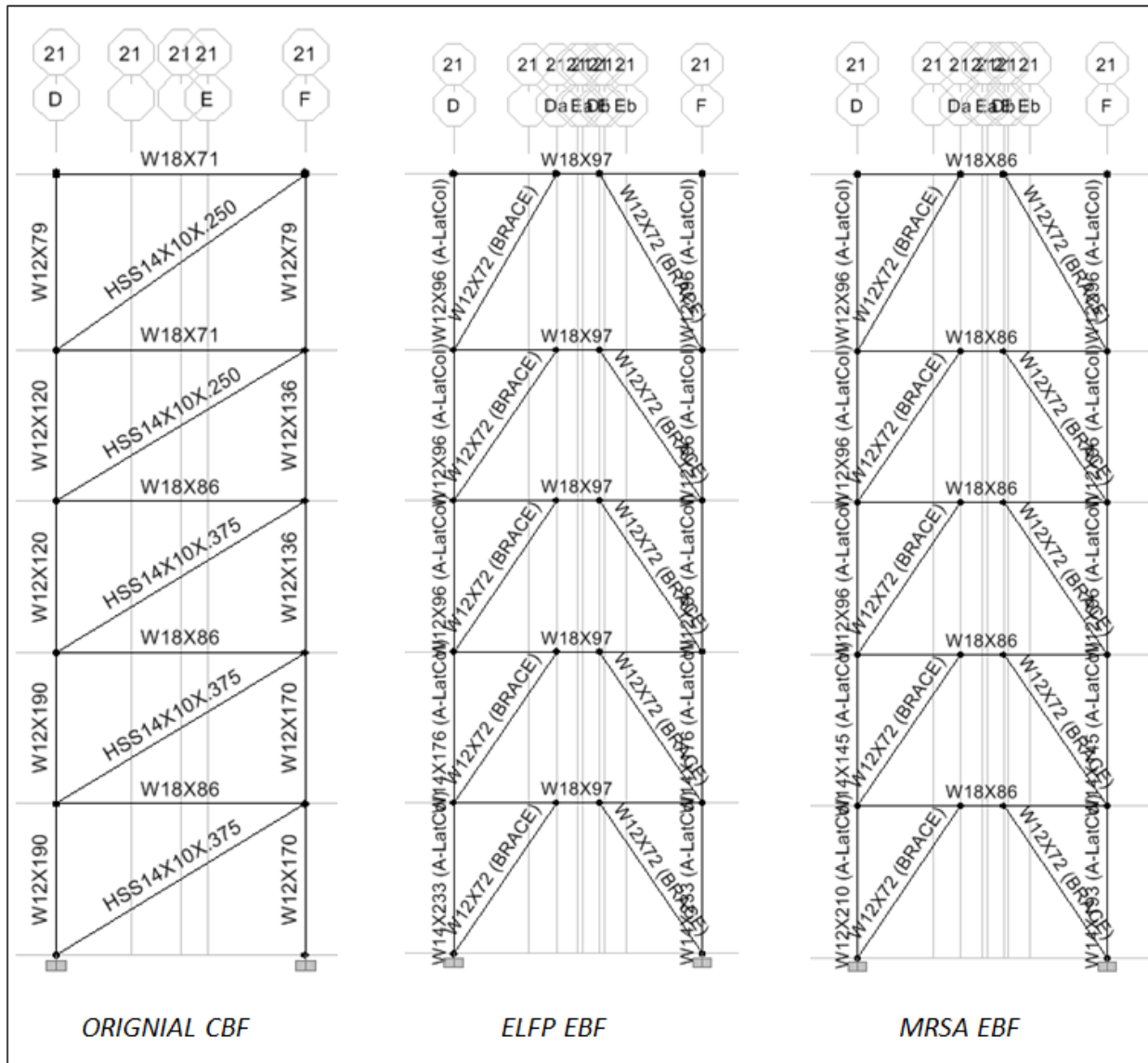


Figure 26: Lateral Frame 1X Member Sizes

Allowable Drift

Next, building deflection and story drifts were analyzed for each design. Looking at the “Torsional Irregularity Check” table from before, the greatest value for each is the max roof drift. This also becomes the structures max deflection. Therefore, max deflections are as follows:

- Original CBF → **1.37 inches**
- ELFP EBF → **1.21 inches**
- MRSA EBF → **0.940 inches**
- Max allowable → $(C_d * \sigma_{tot} / I_e) = 4 * 1.09 / 1.25 = \mathbf{3.5 \text{ inches}}$ (ASCE7-10 12.8-15)

Comparing each design to the max allowable, all of these deflections are well within the maximum deflection prescribed by ASCE7.

Each design must also be checked for inter-story drifts to make sure floor to floor deflection is within code limits set by ASCE7-10 12.12. Results for this can be found for each design in the following tables. **Table 14**, **Table 15**, and **Table 16** shows allowable drift for the original design, ELFP design, and MRSA design respectively. As can be seen in the tables, all designs were well within the code limit for inter story drift under seismic loading.

Table 14: Allowable Seismic Story Drift (Original CBF Design)

Allowable Seismic Drift (Original)						
Floor	Story Height	Story Displ.	Story Drift	Design Drift	Allowable Story Drift	
X-Dir.	(ft)	(in)	(in)	(in)	$\Delta_{EQ} \text{ (in)} = 0.015h_{sx}$	Acceptable
6	72.3	1.016	0.23600	0.75520	2.93940	Yes
5	56.0	0.780	0.23100	0.73920	2.52000	Yes
4	42.0	0.549	0.22500	0.72000	2.52000	Yes
3	28.0	0.324	0.18200	0.58240	2.52000	Yes
2	14.0	0.142	0.14200	0.45440	2.52000	Yes
Y-Dir	(ft)		(in)	(in)	$\Delta_{EQ} \text{ (in)} = 0.015h_{sx}$	Acceptable
6	72.3	1.100	0.32800	1.04960	2.93940	Yes
5	56.0	0.772	0.23200	0.74240	2.52000	Yes
4	42.0	0.540	0.25500	0.81600	2.52000	Yes
3	28.0	0.285	0.16900	0.54080	2.52000	Yes
2	14.0	0.116	0.11600	0.37120	2.52000	Yes

Table 15: Allowable Seismic Story Drift (ELFP EBF Design)

Allowable Seismic Drift (ELFP)						
Floor	Story Height	Story Displ.	Story Drift	Design Drift	Allowable Story Drift	
X-Dir.	(ft)	(in)	(in)	(in)	Δ_{EQ} (in)=0.015h _{sx}	Acceptable
6	72.3	1.035	0.21730	0.69536	2.93940	Yes
5	56.0	0.818	0.22240	0.71168	2.52000	Yes
4	42.0	0.596	0.24730	0.79136	2.52000	Yes
3	28.0	0.348	0.19680	0.62976	2.52000	Yes
2	14.0	0.152	0.15160	0.48512	2.52000	Yes
Y-Dir	(ft)	(in)	(in)	(in)	Δ_{EQ} (in)=0.015h _{sx}	Acceptable
6	72.3	1.061	0.23410	0.74912	2.93940	Yes
5	56.0	0.827	0.22890	0.73248	2.52000	Yes
4	42.0	0.598	0.25580	0.81856	2.52000	Yes
3	28.0	0.342	0.19270	0.61664	2.52000	Yes
2	14.0	0.150	0.14970	0.47904	2.52000	Yes

Table 16: Allowable Seismic Story Drift (MRSA EBF Design)

Allowable Seismic Drift (MRSA)						
Floor	Story Height	Story Displ.	Story Drift	Design Drift	Allowable Story Drift	
X-Dir.	(ft)	(in)	(in)	(in)	Δ_{EQ} (in)=0.015h _{sx}	Acceptable
6	72.3	0.800	0.17120	0.54784	2.93940	Yes
5	56.0	0.629	0.16800	0.53760	2.52000	Yes
4	42.0	0.461	0.18530	0.59296	2.52000	Yes
3	28.0	0.276	0.15520	0.49664	2.52000	Yes
2	14.0	0.120	0.12040	0.38528	2.52000	Yes
Y-Dir	(ft)	(in)	(in)	(in)	Δ_{EQ} (in)=0.015h _{sx}	Acceptable
6	72.3	0.714	0.15730	0.50336	2.93940	Yes
5	56.0	0.557	0.15810	0.50592	2.52000	Yes
4	42.0	0.399	0.16490	0.52768	2.52000	Yes
3	28.0	0.234	0.13040	0.41728	2.52000	Yes
2	14.0	0.104	0.10350	0.33120	2.52000	Yes

Overtuning Moment- Foundation Impact

The last comparison between the three systems is in the foundation system analysis. Foundation caissons under each lateral frame column must be able to withstand both the compressive and tensile forces caused by seismic lateral movement. Because the caissons are weakest in tension loading, each design will be checked to make sure foundations are capable of supporting the uplift forces caused by the lateral frames.

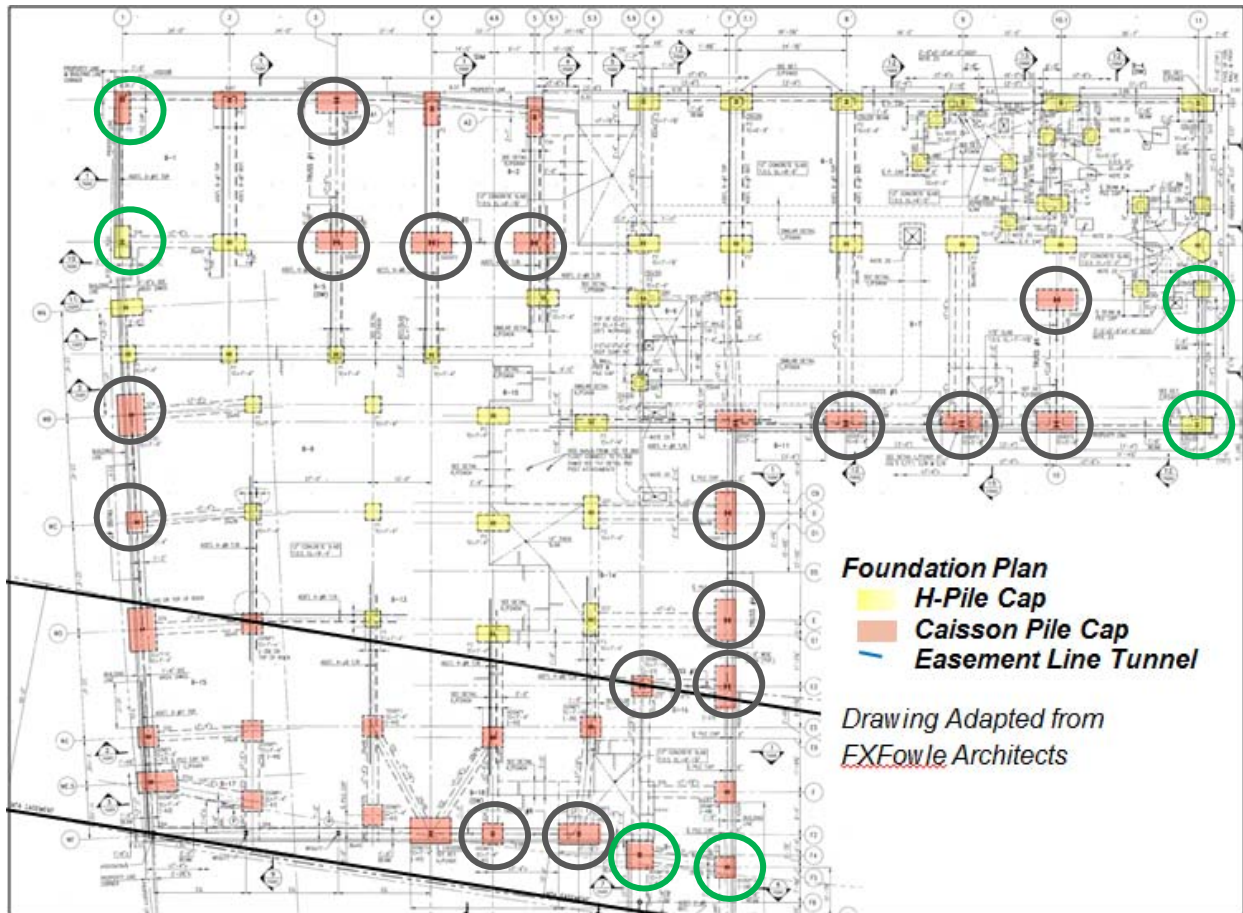


Figure 27: Lateral Frame Caissons

In **Figure 27** above, all the effected caissons in lateral loading are shown. The green circles represent the new locations for the ELFP design, while the black circles represent the caissons affected by all three designs. **Table 17** shows the tensile capacity of each of these caissons, along with the forces related to each of the three designs. According to the ETABS models, it was found that the original design actually was inadequate in several spots. The large frames that took the majority of the load are over the tension limit (compression is fine). Because it was close, it is assumed that some assumptions may have been slightly un-conservative for this analysis. The same assumptions were made for all designs, however, so the comparison should be accurate.

The ELFP design shows an increase in uplift forces in many of the caissons, which makes sense considering they take a greater percentage of the seismic load. The caissons that were over in the original design are now adequate. Overall, all caissons

THESIS REDESIGN

are adequate in this design. The MRSA design has lower forces in all lateral frames, so uplift forces should be smaller even though there are fewer frames than in the ELFP design. Looking at the base reaction table, the MRSA design is, in fact adequate with lower uplift forces.

Table 17: Base Reactions and Foundation Capacity

Base Reactions and Foundation Capacity													
N-S Loading Direction							E-W Loading Direction						
Point #	Fz (Orig.) (k)	Fz (ELFP) (k)	Fz (MRSA) (k)	Pile Cap	Axial Capacity (k)	Adequate? Y/N	Point #	Fz (Orig.) (k)	Fz (ELFP) (k)	Fz (MRSA) (k)	Pile Cap	Axial Capacity (k)	Adequate? Y/N
28	-301	-318	-267	300DP2	600	Y	17	-219	-	-182	300DP2	600	Y
29	-309	-321	-270	300DP2	600	Y	27	-203	-	-174	300DP2	600	Y
30	-168	-241	-221	300MP1A	300	Y	30	-17	-19	-16	300MP1A	300	Y
31	-126	-183	-148	300MP2	600	Y	31	-625	-273	-193	300MP2	600	Y
46	-613	-301	-258	300DP2	600	Y	45	-634	-347	-247	200DP2	400	Y
48	-612	-287	-262	200DP2	400	Y	49	-239	-	-237	300DP2	600	Y
49	-21	183	-17	300DP2	600	Y	50	-228	-	-223	200DP2	400	Y
50	-9	-11	-3	200DP2	400	Y	52	-132	-114	-87	300MP2C	600	Y
52	-48	-38	-37	300MP2C	600	Y	55	-26	-10	-9	300MP2A	600	Y
55	-135	-111	-108	300MP2A	600	Y	56	-29	-4	-4	300MP2C	600	Y
56	-145	-106	-106	300MP2C	600	Y							

STRUCTURAL DEPTH

CONNECTION DETAILS (MAE)

The new lateral system of Hunter's Point South uses eccentrically braced frames with a Response Modification Factor (R) of 8 to withstand seismic loads. The R factor represents the "inherent overstrength and global ductility capacity of structural components" (Lindeburg 2008). That is, it signifies how likely the system will create a plastic hinge to initially reduce the lateral load and how ductile a specific system is in taking further lateral loading, such that the system has enough strength to withstand the loading without requiring fully elastic response (elastic response is not economical). This ductility and overstrength is important because it allows the system to dissipate seismic energy by yielding components.

To make this dissipation happen (and create a ductile system) it is imperative to have proper detailing of the system so it is constructed exactly as designed. This seismic detailing is crucial to the effectiveness of lateral systems with high R values. In EBF design, the detailing is focused on the design of the link, and the connections of the steel members. If the connections are not designed/ constructed properly, the system will not behave as intended, and unwanted failure could occur during lateral loading.

To seismically detail the connections of the EBF systems, two methods were used. First, AISC 327-05 was used in hand calculations to design two separate connections for EBF 1X in the ELFP design. A brace-beam-column connection was detailed, and then a brace-link connection. Then, to check the design and economize the design procedure, two separate spreadsheets were created to design all steel connections in the lateral system. Once it was determined that the spreadsheets were accurate, connections were designed for EBF 1X MRSA design as well. Due to time constraints in this project, no further connection details were found. However, as mentioned, the formulated spreadsheets will allow for easy design of all other connections. It should be noted that further detailing may be required in other parts of the structural system (i.e. beam/ column connections and column splice connections) to fully adhere to a ductility found in an R-8 system, but are also not included in this design due to time constraints. Instead this paper focuses entirely on the lateral system bracing details.

The connection details that follow are the brace-beam-column connection and brace-link connection of the third floor portion of EBF 1X in both the ELFP and MRSA designs. All hand calculations and design spreadsheets can be found in the appendix of this report.

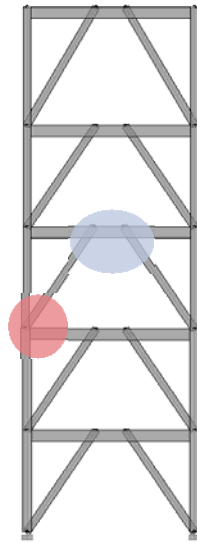


Figure 28: EBF 1X Connection Locations

- Brace-Beam-Column Connection
- Brace-Link Connection

Brace-link connections were designed using welds and a shear plate to connect the brace to the link. Stiffeners are used at the connection point (as well as down the beam) to prevent buckling of the beam during plastic action of the link. Brace-link connections for ELFP can be seen in **Figure 29** and for MRSA in **Figure 30**.

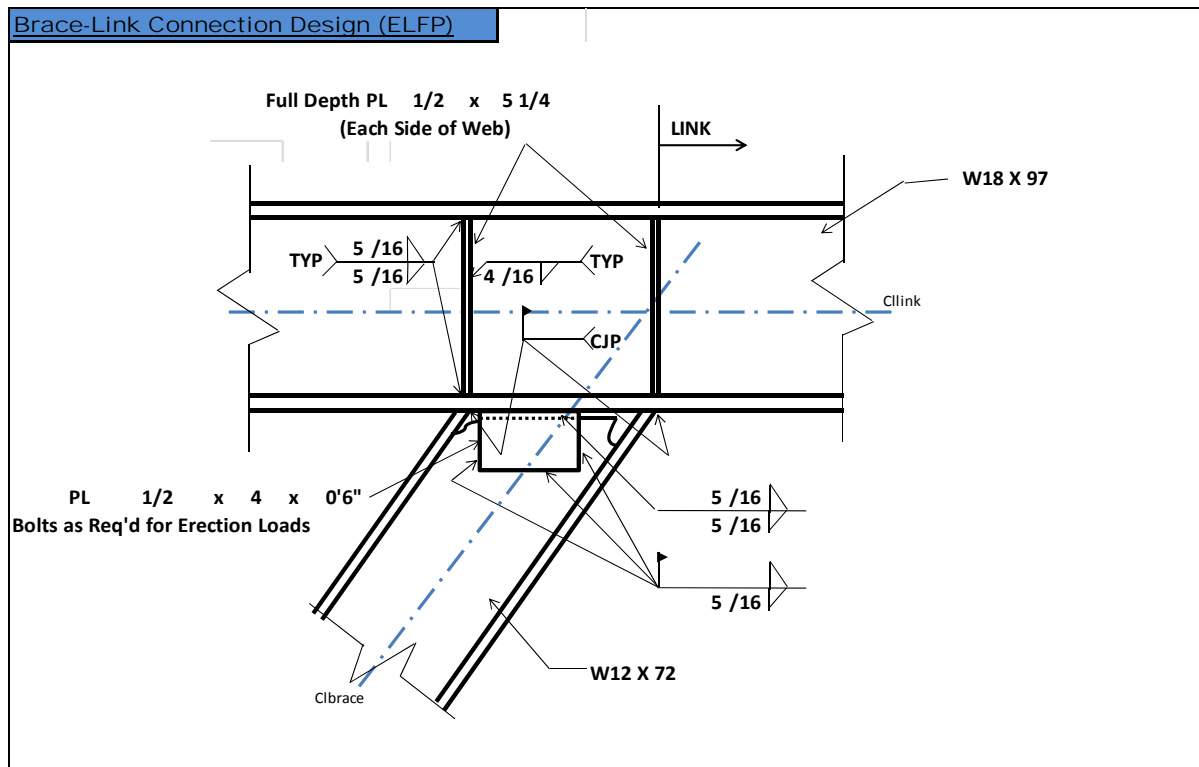


Figure 29: Brace-Link Connection Detail EBF 1X ELFP

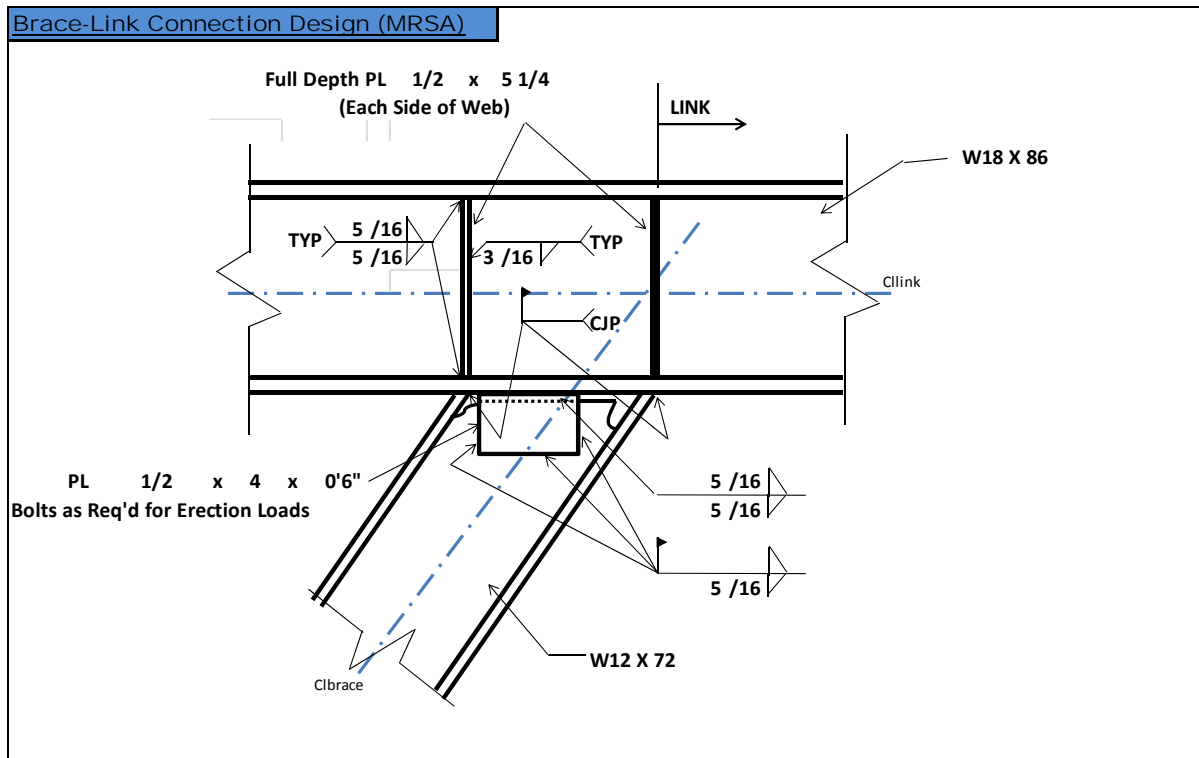


Figure 30: Brace-Link Connection Detail EBF 1X MRSA

Both redesigns used a brace-beam-column connection that had the brace connecting to a gusset plate with a bolted T flange connection. The gusset is then welded to the beam, and the beam and gusset are attached to the column using a bolted end plate welded to the end of the gusset and beam. Detail is imperative in these connections so as to create a functional moment connection that prevents rotation and allows the link design to serve its purpose. Brace-beam-column connections for ELFP can be seen in **Figure 31** and for MRSA in **Figure 32**. Note that some weld sizes were increased so as to make construction simpler and help prevent any mistakes during welding. Also, drawing is not to scale.

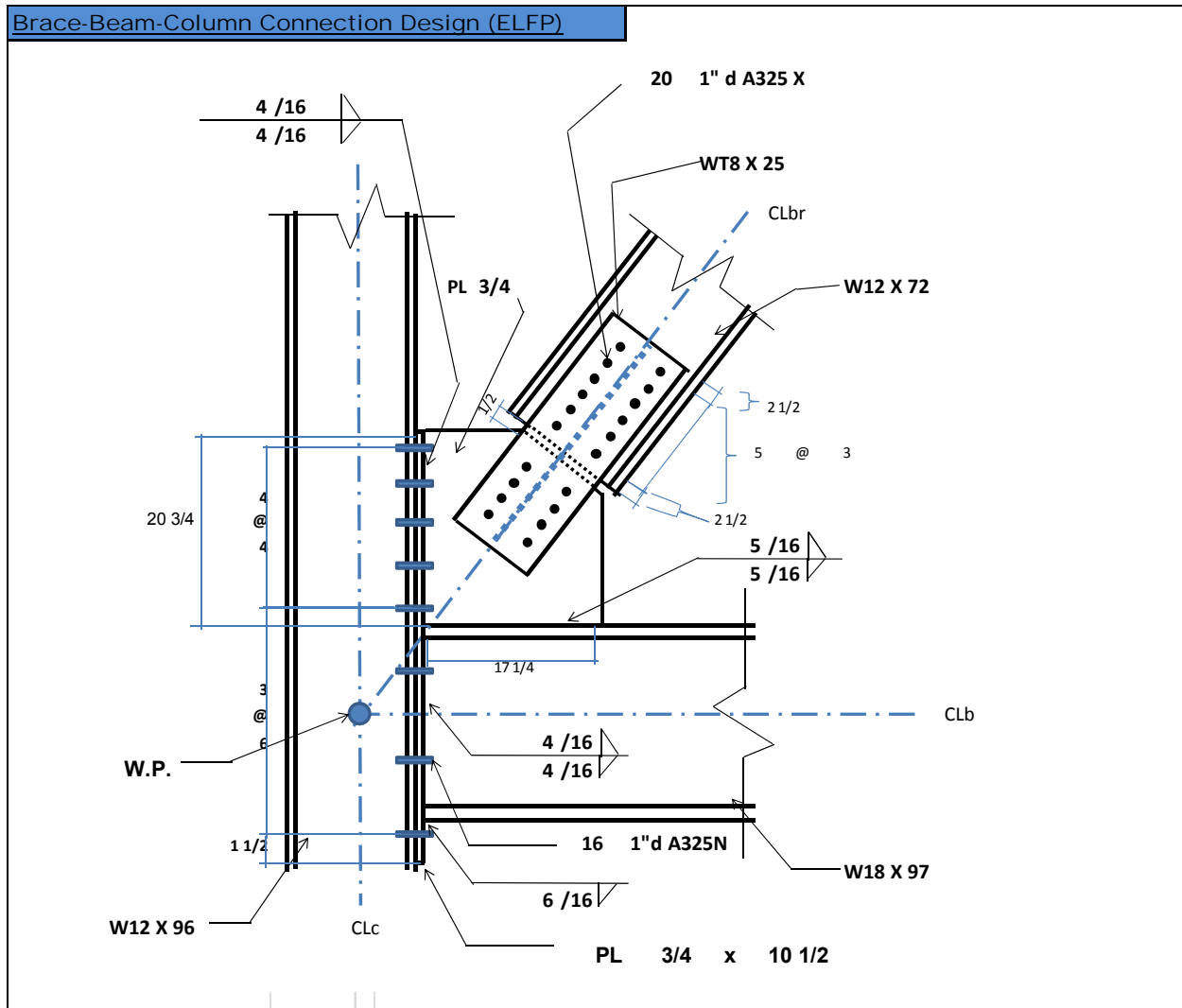


Figure 31: Brace-Beam-Column Connection Detail EBF 1X ELFP

As stated before, the EBF design works such that the link goes through plastic action while the rest of the system remains elastic during a seismic event. Therefore, it is very important to detail the connections correctly to allow for elastic behavior to remain under heavy seismic loading. All connection details are at least to code minimum requirements to allow for such behavior.

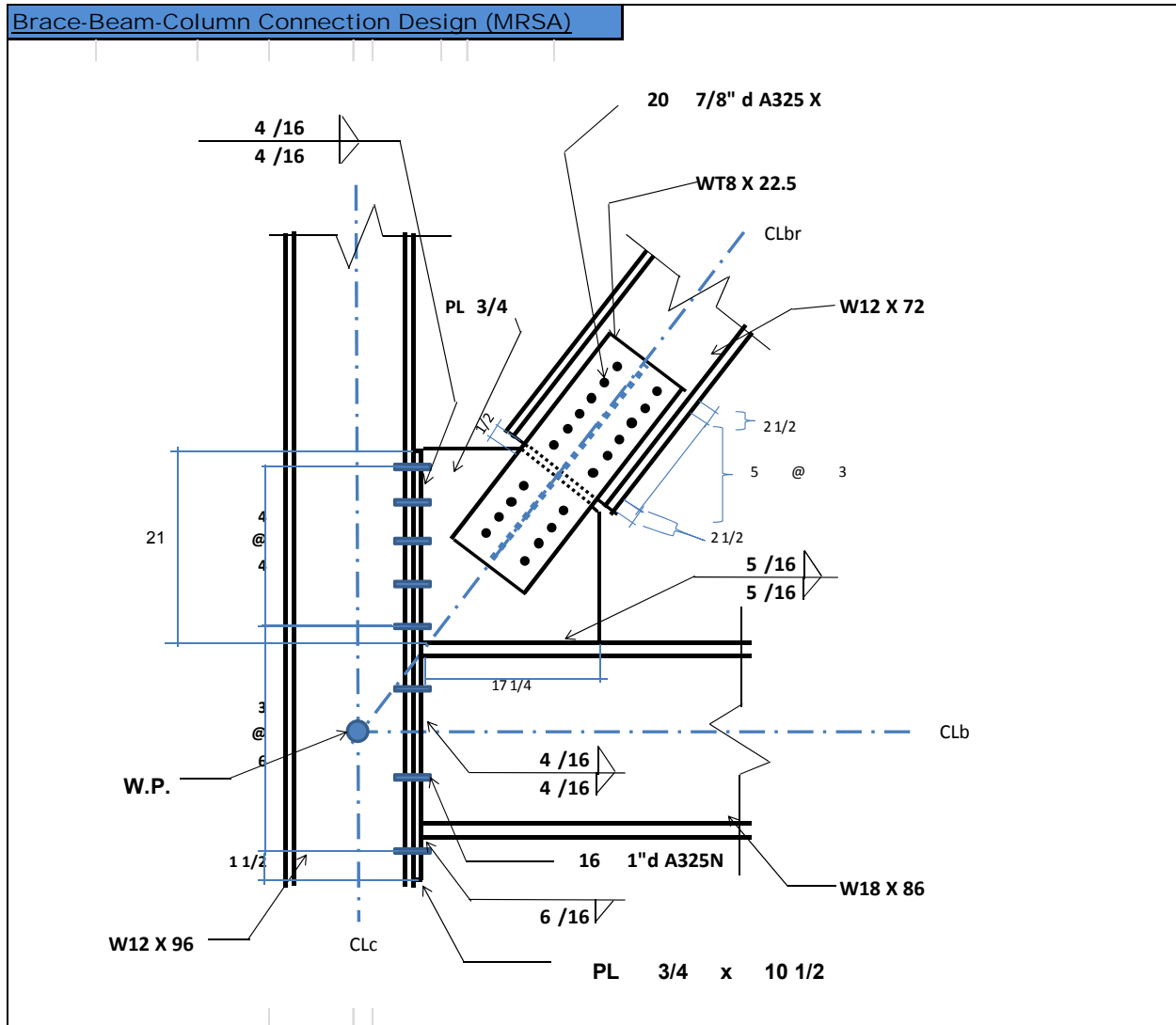


Figure 32: Brace-Beam-Column Connection Detail EBF 1X MRSA

As can be seen from the diagrams, the connection details for both redesigns are quite similar. This can be attributed to the fact that the MRSA redesign has a lower design load applied to it, but has less strength due to having fewer braced frames. However, let it be noted that it is coincidental that the designs are so alike. Overall, the MRSA connection requires less strength as compared to the ELFP design. This means that MRSA design is further proved to be the more practical design. On a final note, the new connections have a much higher level of detailing compared to the original design, thus fitting well with the higher level of ductility that an R value of 8 requires.

STRUCTURAL DEPTH

SUMMARY

After analysis is completed on both lateral system redesigns for Hunter's Point South School, conclusions on the strength and serviceability of each system are made and the better design is chosen. All the while, each system is compared to the original system to determine whether improvements exist and to what extent.

The ELFP design showed the extent of the strength increase of an EBF system. The design load process was similar to the original, but garnered forces that were approximately 85% less in magnitude. This allowed for a system that had smaller story drifts and smaller member sizes. Also, to comply with code requirements, the ELFP design had a focus on preventing lateral torsional irregularity under seismic loading. To fulfill this need, the new system required several frames to be relocated and a new frame be added to the layout. This change successfully eradicated any irregularity in building torsion.

The MRSA design added more design time, but came up with effective results. Design Forces for this method ended up becoming about 85% that of the ELFP method, and 73% that of the original design. This large decrease in design loads is due to the more accurate and less conservative design method of MRSA. This allowed for a more streamlined system overall as compared to the ELFP design. Also, the original lateral frame layout could be kept the same, with the exception of losing the moment frames. Therefore, this system is also more efficient than the original design. Because torsional irregularity was not an issue, the system did not have to be oversized in places to prevent torsion.

When comparing the connection details of the ELFP and MRSA designs, it can be seen that there is little difference. The MRSA design allows for slightly smaller members and connection hardware in places, but it is not significant. Comparing both designs to the original design, it can be seen that the drawings are much more detailed. This is necessary to comply with code that states that sufficient detailing be included to the design to allow for proper construction by the CM and to make sure the lateral system behaves correctly under seismic loading.

After comparisons are made between the two redesign systems and the original lateral system, it is clear that the MRSA EBF design is the best choice to replace the original system in the higher seismic zone. It is the more effective and efficient system in both strength and serviceability. However, the impact each redesign has on the architectural layout and construction process must be analyzed before a final design is chosen.

ARCHITECTURAL IMPACT BREADTH

The architectural breadth study for this redesign project focuses on the effects that the new lateral system has on the architecture of Hunter's Point South School. It is always important to check the structural design against the architecture to make sure that there are no issues that will come up during construction. If there *are* any issues, it is imperative that they be discussed with the architect and building teams, and a solution to the design issues is implemented as quickly as possible so as to prevent delay, change orders, and other unneeded problems.

Therefore, both the MRSA and ELFP eccentrically braced frame redesigns are analyzed against the original architectural layout to determine if there will be any issues with the new bracing (and if so how to fix it). This architectural impact analysis focuses on room/space layout of the building, as well as elevation and section analysis. It will determine if the braces get in the way of such things as hallways, doors, windows, or general occupancy; and determine if there are any visual discrepancies due to the new designs.

First, the MRSA redesign is looked at. This design kept all the original locations of bracing, but the eccentric bracing is more architecturally friendly than the cross bracing originally used. The reason for this is that the bracing allows for more area in between each frame to place such things as doors, windows, and other wall cuts. After a quick look at the building sections, it was decided that the MRSA redesign had absolutely no architectural impact on the structure.

Then, the ELFP redesign is looked at. This design had serious potential for architectural issues, because two brace locations were changed from the original CBF design, and an addition brace was included. Although positioning of these braces took architectural impact into account during design, it was near impossible to find locations that would help prevent torsional irregularity in the structure without obstructing some architecture.

After inspecting the architectural plans, elevations, and sections, it was determined that EBF 1X, 2X, 3X, 4X, 2Y, and 3Y, had no effect on the architectural layout. The location of these braces did not change from the original design. The eccentric bracing gave further room between braces for the placement of windows, doors, and other objects as compared to the concentric truss bracing. However, EBF 5X, 1Y, and 4Y did create issues.

Figure 33 and Figure 34 show the second floor plan of the original design as compared to the ELFP EBF design. New locations of bracing are highlighted to help show where the EBF braces were placed along the exterior. The second floor plan was chosen because it ends up being the only floor that has actual floor plan layout changes on it.

THESIS REDESIGN

This can be seen in the east wing of the building, which has been outlined with a blue box in **Figure 34** for simplicity. Later diagrams show a blown up plan for further detail. This plan only indicates problematic bracing in the structure. Please refer to the structural depth section for full bracing location plans.

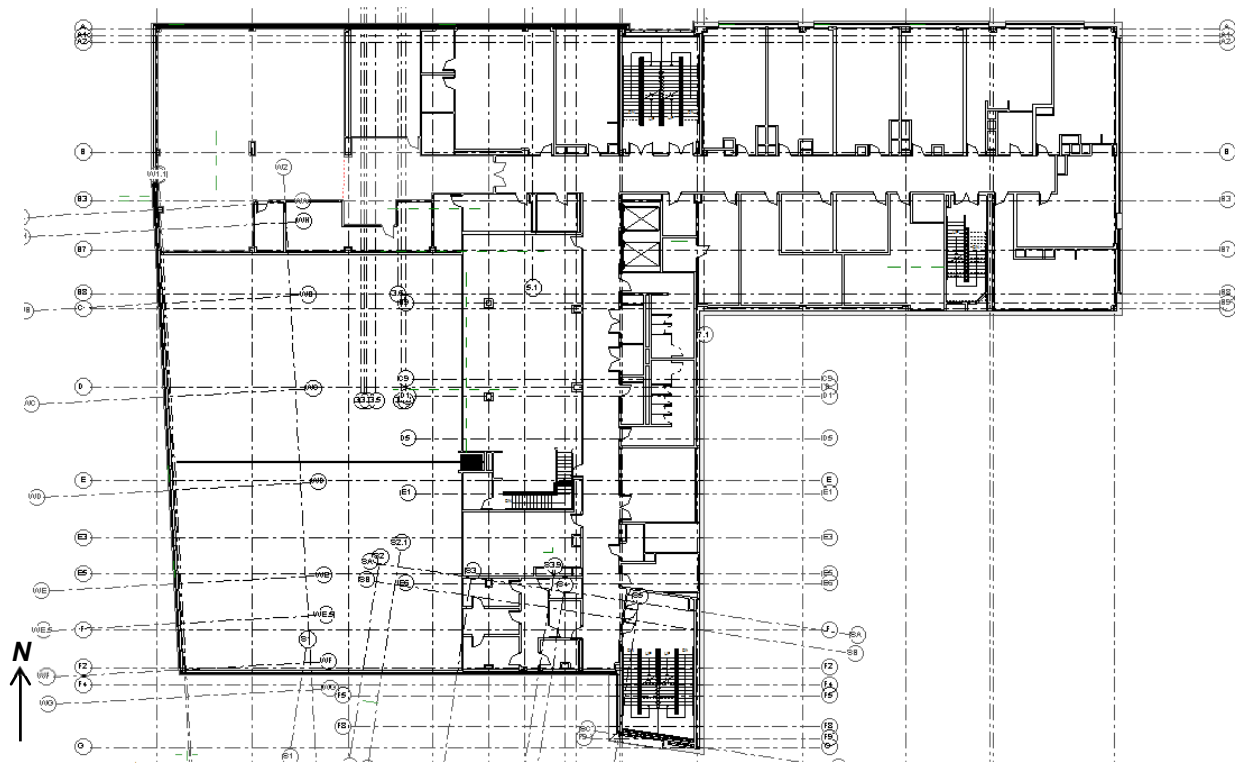


Figure 33: Floor 2 Floor Plan- Original CBF Design

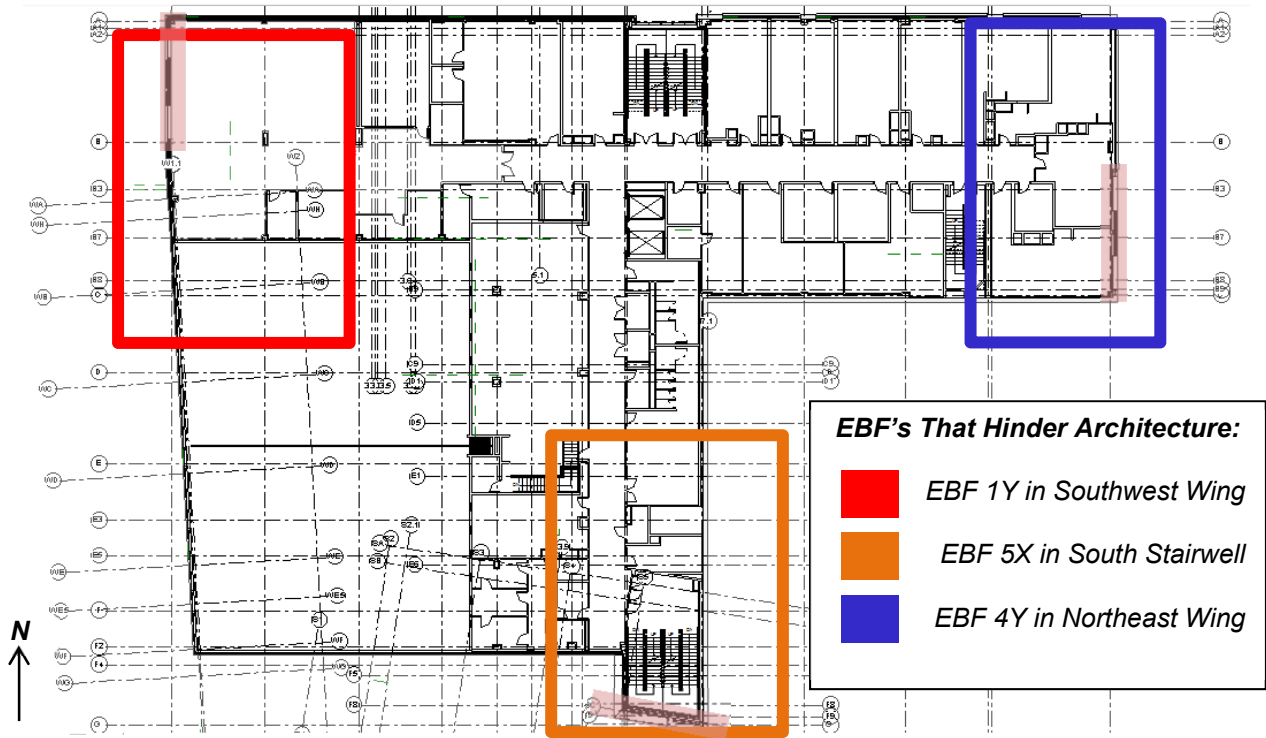


Figure 34: Floor 2 Floor Plan- EBF ELFP Design

THESIS REDESIGN

In **Figure 35**, the south stairwell plan is shown with bracing EBF 5X shown on the exterior wall. This is a blown up plan from the previous floor plan in **Figure 34** (orange box). Due to allowable space on the interior, the placement of the new bracing fit without any interior obstruction. The minimum code requirement for stair landing width was maintained, though a slight decrease in floor area was created. No other issues were found in the interior of south stairwell.

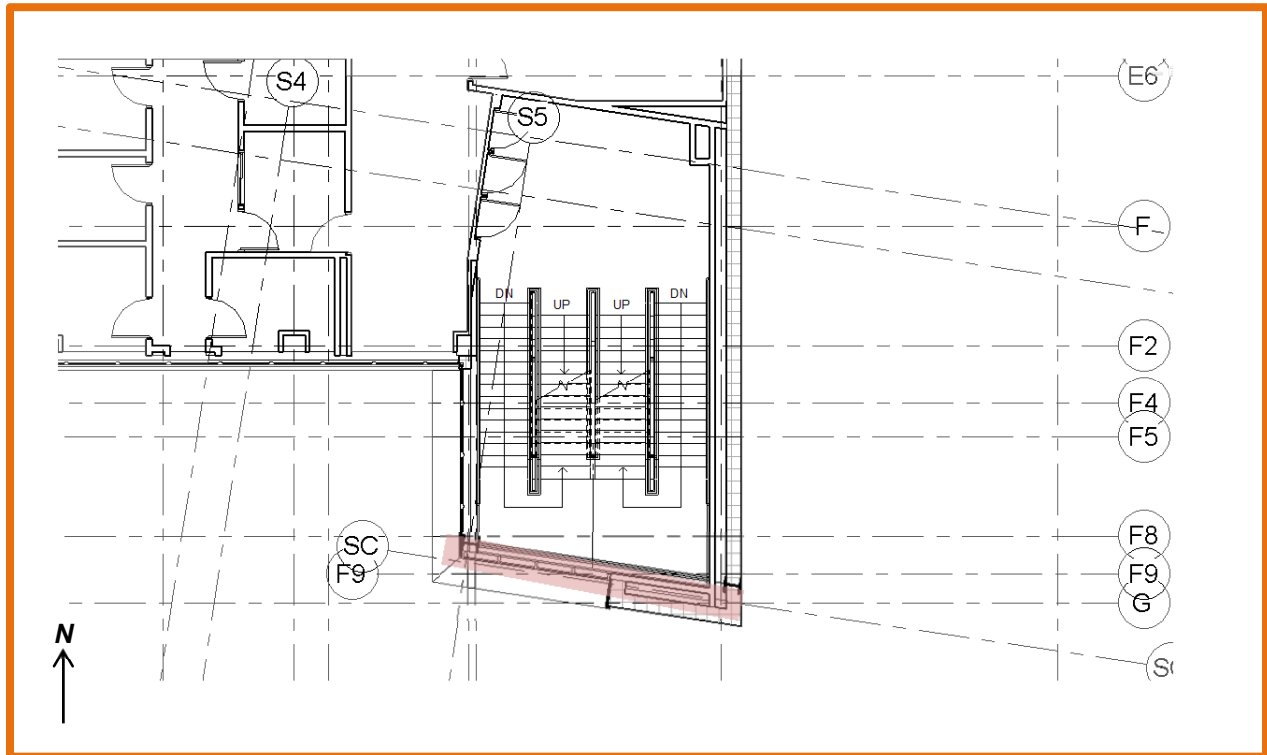


Figure 35: South Stairwell - Floor 2 Floor Plan- EBF ELFP Design

In **Figure 36** and **Figure 37** on the next page, the south elevation is shown with bracing EBF 5X shown in the south stair well (framed by orange) in **Figure 37**. As can be seen in **Figure 37**, the bracing obstructs the curtain wall façade of the south stair well. This could potentially be an issue with the architect. However, it is the analyst's opinion that leaving the bracing unhidden actually improves upon the design. The stair well curtain wall creates a slanted vertical break in the horizontal design of the exterior as it is. By keeping the EBF visible, the slanted bracing helps strengthen this architectural disruption, and creates a more unique exterior.



Figure 36: South Elevation- Original CBF Design



Figure 37: South Elevation- EBF ELFP Design

THESIS REDESIGN

In **Figure 38** below, the Library in the west wing of the 2nd floor plan is shown with bracing EBF 1Y shown on the exterior wall. This is a blown up plan from the floor plan in **Figure 34** (Red box). Due to allowable space on the interior, the placement of the new bracing fit without any interior obstruction. The architectural plans show more than adequate space in the existing exterior wall to house the new bracing frame.

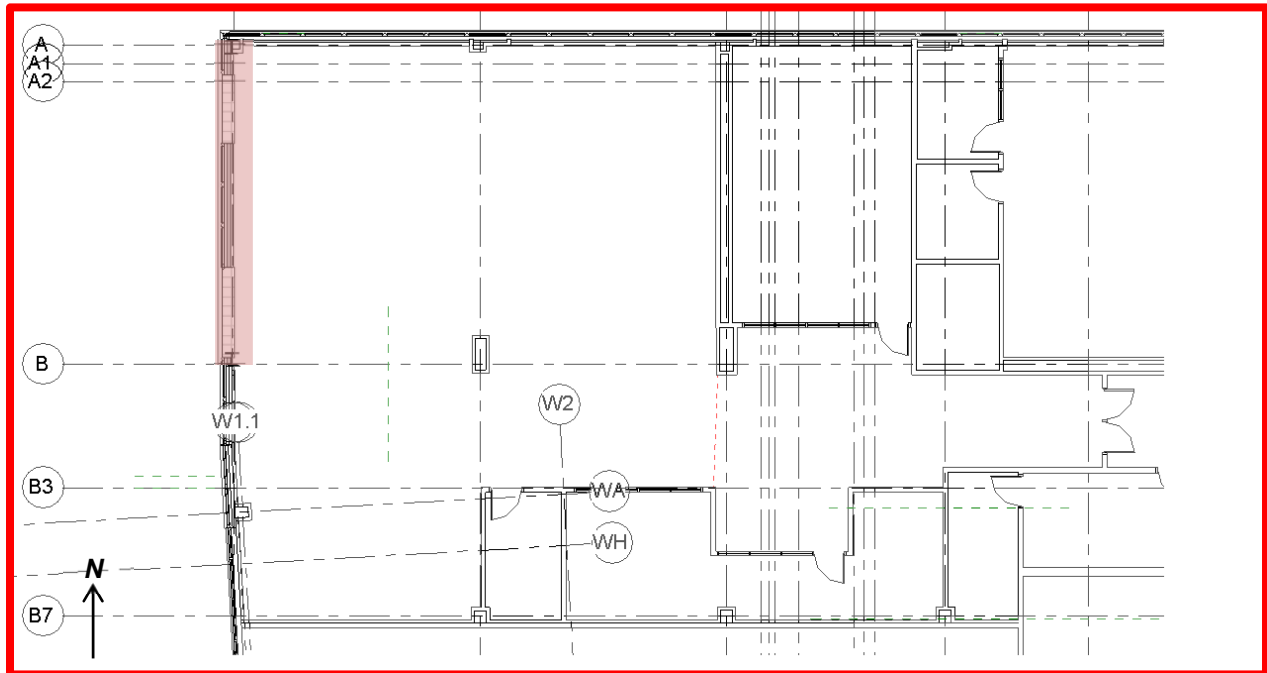


Figure 38: Library (West Wing)- Floor 2 Floor Plan- EBF ELFP Design

In **Figure 39** and **Figure 40** on the next page, the west elevation is shown with bracing EBF 1Y affecting the area framed by red in the second elevation. As can be seen in **Figure 40**, the bracing obstructed the curtain wall on the bottom two floors, and also the window placement on the top floor. Because having the bracing visible would potentially be unsightly in this situation, the exterior façade was redesigned.

For the bottom curtain wall, the solution lied with other parts of the curtain wall, which wraps to the back (north) side of the building. To hide obstructions in the original design such as walls, aluminum panels were placed intermittently along the curtain wall to

THESIS REDESIGN

cover these areas up. This was replicated in the redesign of the west façade. Although it is dissimilar from the other panel location in that two panels are blocked rather than just one, the consistence in this location helps ease the difference. It may not be exactly what the architect had in mind, but it is an effective, quick fix.



Figure 39: West Elevation- Original CBF Design



Figure 40: West Elevation- EBF ELFP Design

As for the windows placed on the top floor, the EBF required that 3 of the 6 windows be removed to hide the EBF. It is the analyst's opinion that the 5th floor windows already looked out of place due to the lack of other windows on the other floors, and deleting windows does no further harm.

When getting rid of windows or blocking curtain walls, it must be made certain that the areas inside the exterior walls are still getting sufficient day lighting. This is important because schools are required to give plenty of natural light to classrooms to help student performance and mental health. In all the floors effected in the architectural redesign, it was determined that openings on the north elevation gave plenty of natural light to allow for the reduction of openings on the west elevation.

THESIS REDESIGN

In **Figure 41** and **Figure 42**, several Special Education Classrooms in the east wing of the 2nd floor plan are shown with bracing EBF 4Y shown on the exterior wall in **Figure 42**. The second figure is a blown up plan from the floor plan in **Figure 34** (Blue box). Due to allowable space on the interior, the placement of the new bracing fit without any interior obstruction. The architectural plans show more than adequate space in the existing exterior wall to house the new bracing frame.

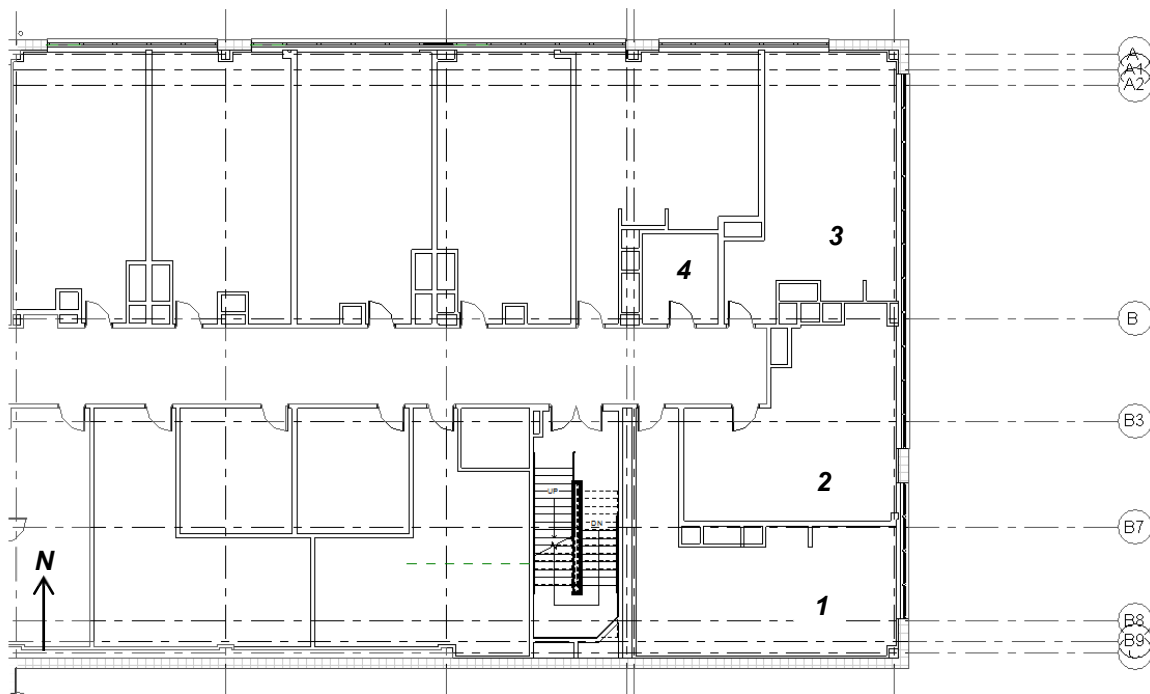


Figure 41: Classrooms (East Wing) - Floor 2 Floor Plan- Original CB Design

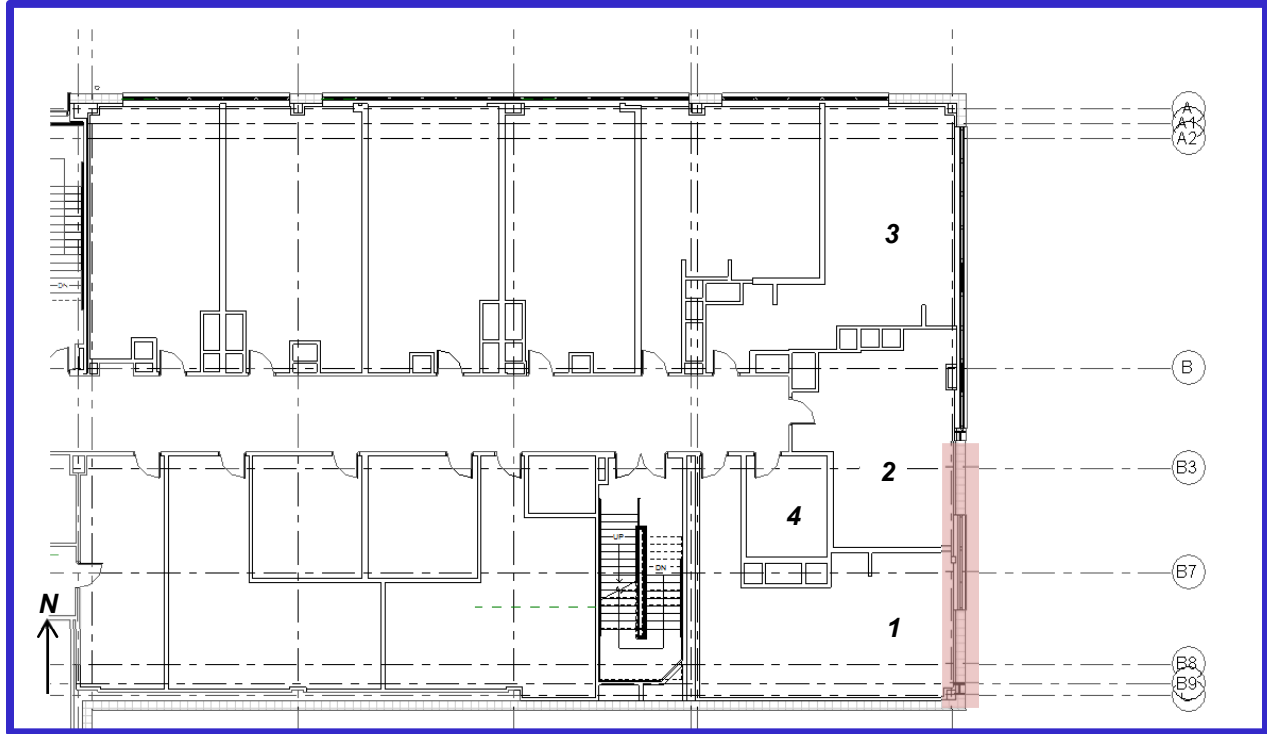


Figure 42: Classrooms (East Wing) - Floor 2 Floor Plan- EBF ELFP Design

In **Figure 43** and **Figure 44** on the next page, the east elevation is shown with bracing EBF 4Y affecting the area framed by blue in the second elevation. As can be seen in **Figure 44**, the bracing obstructed the window placement on floors 2 through 5. Because having the bracing visible would potentially be unsightly in this situation, the exterior façade was redesigned. Three windows were removed from floors 2 and 4, and four windows were removed from floors 3 and 5. The first floor contained no windows in the location of the EBF frame, and did not need to be redesigned. Though the new window design of the east façade decreases natural lighting, it does not hurt the exterior design that the architect set up for this building.

Once again, day lighting issues had to be analyzed to determine whether the new window design worked ok with the current room layout. Unfortunately, the original classroom layout was not going to work with the new window design. Day lighting was decreased to one window in Classroom 2 and to two windows in Classroom 1. This was deemed inadequate. Therefore, as can be seen in **Figure 42** the layout of the rooms on the second floor of the east wing were changed.

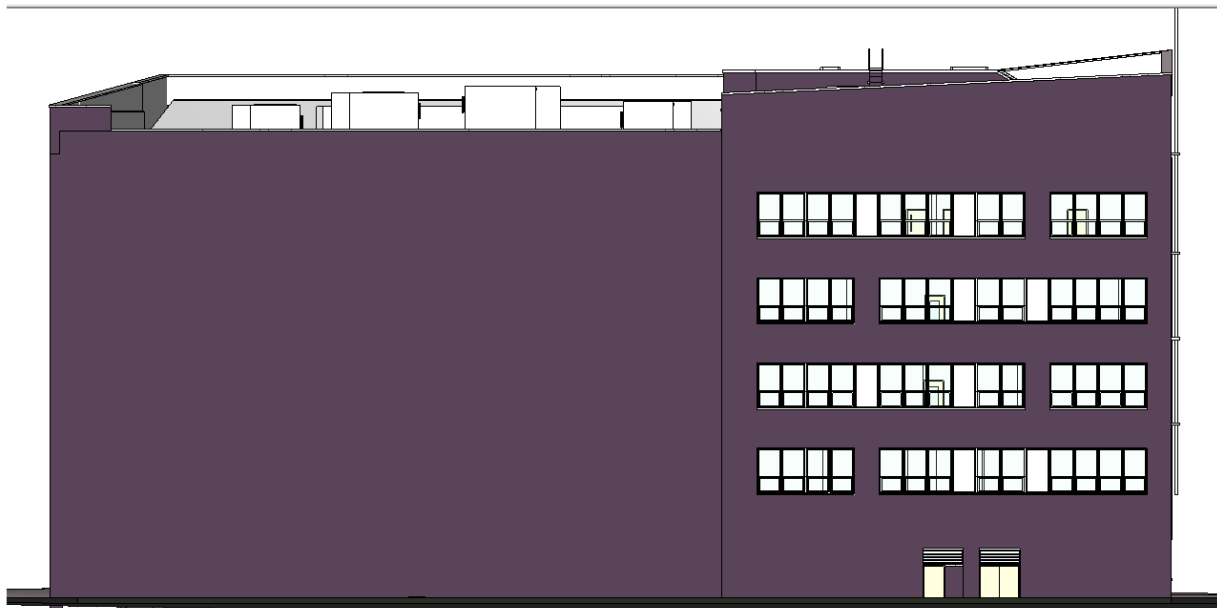


Figure 43: East Elevation- Original CBF Design

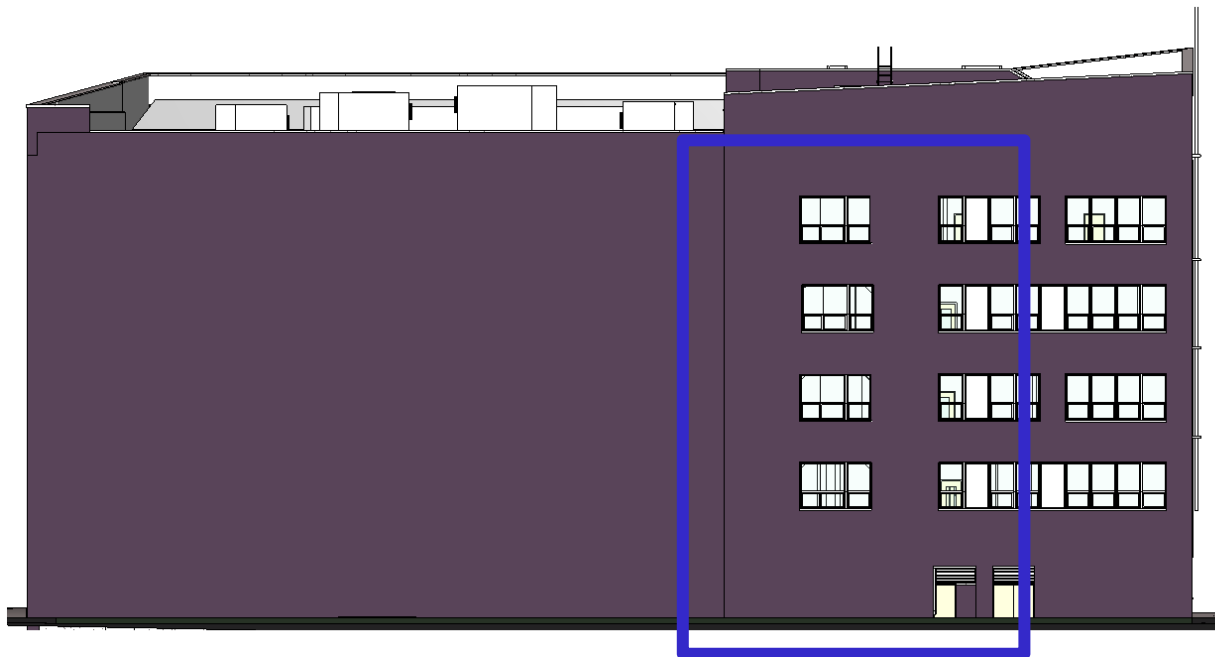


Figure 44: East Elevation- EBF ELFP Design

THESIS REDESIGN

The original design had three special education classrooms at the end of the east wing, in addition to Storage Closet 4 outside of Classroom 3. A layout redesign was implemented for the ELFP redesign to move several walls and rearrange these four spaces to allow for more daylight in two of the rooms. All walls moved were non-structural, non-loadbearing walls, so movement of these walls were easily done with no issues. Classroom 2 had its north wall extended into Classroom 3 and its south wall brought in to extend Classroom 1. The lower west wall of Classroom 2 was brought in to make room for a new placement of Storage Closet 4 and the upper west wall was brought out into dead space of the hallway. Classroom 3 had its west wall extended to where the Storage Closet 4 walls originally were to allow for a new entrance (due to the extended west wall of Classroom 2 covering the original door location). Several closets and computer stations were moved in several rooms to allow for better classroom layout.

The resulting architectural redesign creates rooms with slightly different layout shapes, but equal areas. It is decided that the shape difference is not enough to affect the use of the classrooms or closet. The final design allows for one added window in both Classroom 1 and Classroom 2. Though it is still less window area than the original design, it is deemed adequate. The redesign is complete.

SUMMARY

In conclusion, it is determined that the ELFP redesign does, in fact, create several issues with the building's architectural layout. After analysis, it is proven that layout changes would only be required in one location. Though changes are made for the second floor east wing, the layout is successfully changed by moving only interior non-loadbearing walls and does nothing to affect the uses of the rooms involved in the layout change. Exterior changes only involve the removal/ movement of several windows in two locations to prevent viewing of the added structure. It was determined that the final exterior bracing location actually improved upon the façade design and, therefore, was kept unhidden.

No additional obstruction was caused by the EBF design. It is unknown if the architect on this project is willing to budge easily on the design, but this analysis shows the simplicity of the solutions. It is the analyst's opinion that the architectural impact is small enough that the design *would* be successfully implemented, and no further issues would arise.

THESIS REDESIGN

In the MRSA redesign, no architectural impact was found. This is due to the fact that this EBF redesign kept all the original bracing locations and did not cause any interior wall/ space conflicts throughout the building.

CONSTRUCTION & COST IMPACT BREADTH

CONSTRUCTION SCHEDULE

The construction breadth study of this research project focuses on the impact that the redesign of the lateral system of Hunter's Point South has on both the construction schedule and the overall construction cost of the building.

First, the construction schedule is analyzed to determine the effects that designing both types of EBF systems in higher seismic zones has on the completion time of the overall building project. To begin, the original construction schedule was acquired from SKANSKA Construction. This schedule included a breakdown of all building parts, and most importantly a breakdown of the structural system construction sequence. Also included was a summarized critical path schedule that was used later to calculate the final schedule completion date. This analysis assumes that the only major changes in schedule duration occur in construction of the superstructure (specifically due mainly only to the new seismically detailed steel lateral system.) Research that included such sources as RS Means was used to help develop an accurate schedule for the new designs of the lateral system.

A summarized schedule created using MS Project for the original design can be seen in **Figure 45**. As can be seen from the figure, the steel erection is broken up into two phases of work to help speed up the process of construction. This method will stay unchanged in the redesigns to continue the efficiency of the build. Overall, steel erection was expected to last from August 12, 2011 to November 11th, 2011; or 63 days. The final completion date for the overall project was expected to be October 7th, 2013.

The specific time for erection of the original lateral system was unknown, but was estimated using RS Means and the original structural plans. Assuming the majority of welds to be prefabricated, an estimated time was found using the erection time for 2 crews to construct the amount of steel and bolts the structural drawings specified. Though this may not be a perfectly accurate representation of the lateral system erection time, it will not matter because the same assumptions will be made in the redesign schedules and the difference will be factored into the original design to come up with the new erection time.

Following this assumption, it was found that the original design of the CBF lateral system would take roughly 22 days to complete erection (about 1/3 of the steel erection time). Though this seemed slightly high, as reasoned before, it will not matter in the final schedule process.

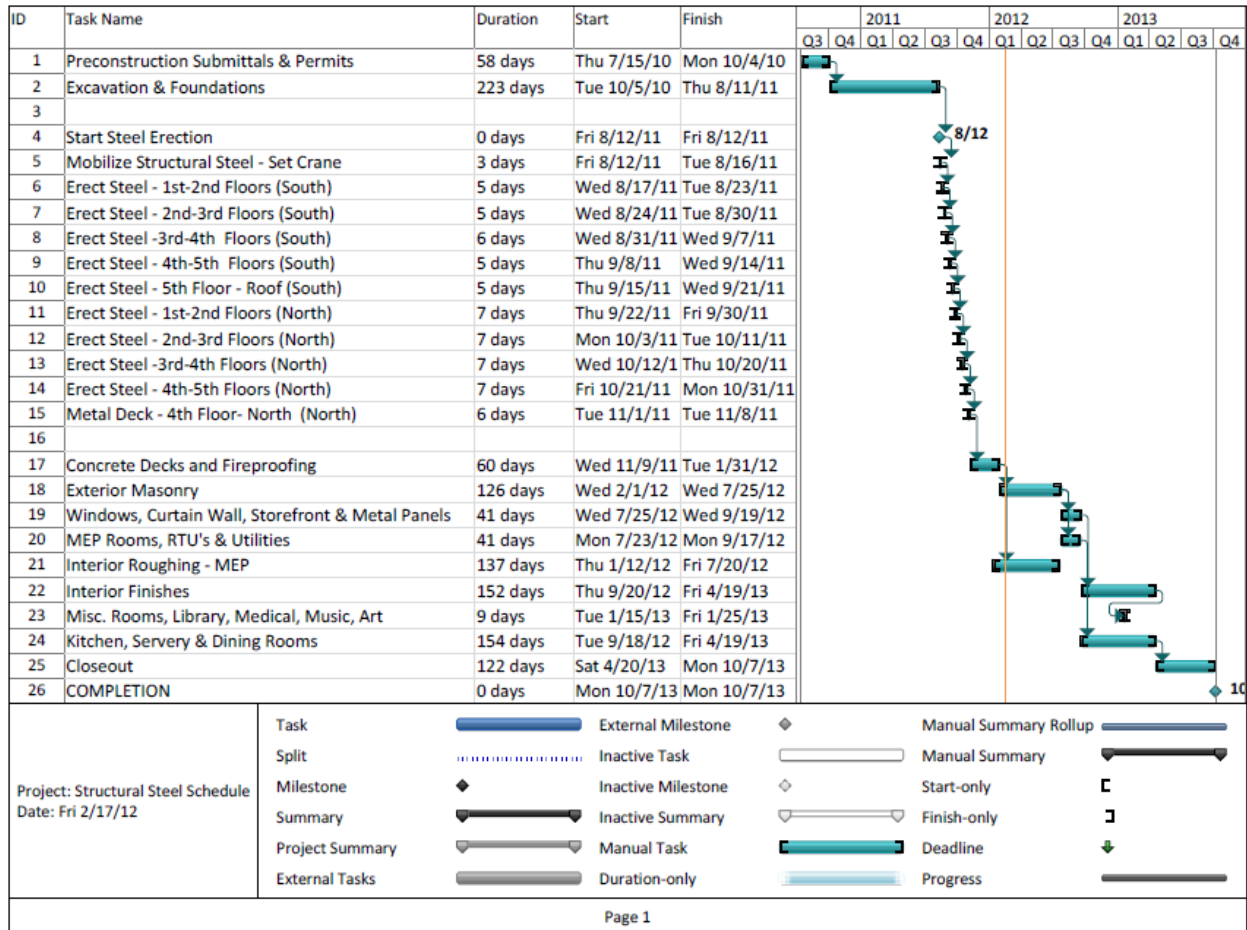


Figure 45: Summarized Schedule: Original Design

Note, because the schedule is lengthened in each redesign, some systems may be further delayed due to bad weather conditions not suitable for the construction of that system (i.e. extreme cold and concrete pouring) or other unforeseen issues. This analysis will ignore these effects and assume that the only difference in construction time occurs due to the changes in the structural system.

Once the original schedule was analyzed for the lateral system, the two redesign schedules could be created. The first redesign focused on the ELFP EBF lateral system. Like the original design, the steel and bolt erection time were analyzed using an average found from the bracing and connection design done in this report. Analysis showed that the ELFP design would take roughly 29 days to complete the lateral system erection. The change in time can be attributed to several factors. The EFLP design included an additional bracing frame which would increase time due to steel and

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connection erection. Also, the number of connections and complexity of the erections increased.

The ELFP design had an increase in welded connections substituting for bolts, which actually decreased construction time due to the fact that they were all mostly prefabricated welds and not done in the field. However, due to the seismic detailing, an additional inspection time was included. This additional time was factored in with other seismic detailing concerns and added a 5% increase of time to the erection, creating a 30 day erection period. This 5% increase is only a rough estimate, but is often used in design cost and design time to compensate for the added detailing required by the code for the high R value system. (ATC).

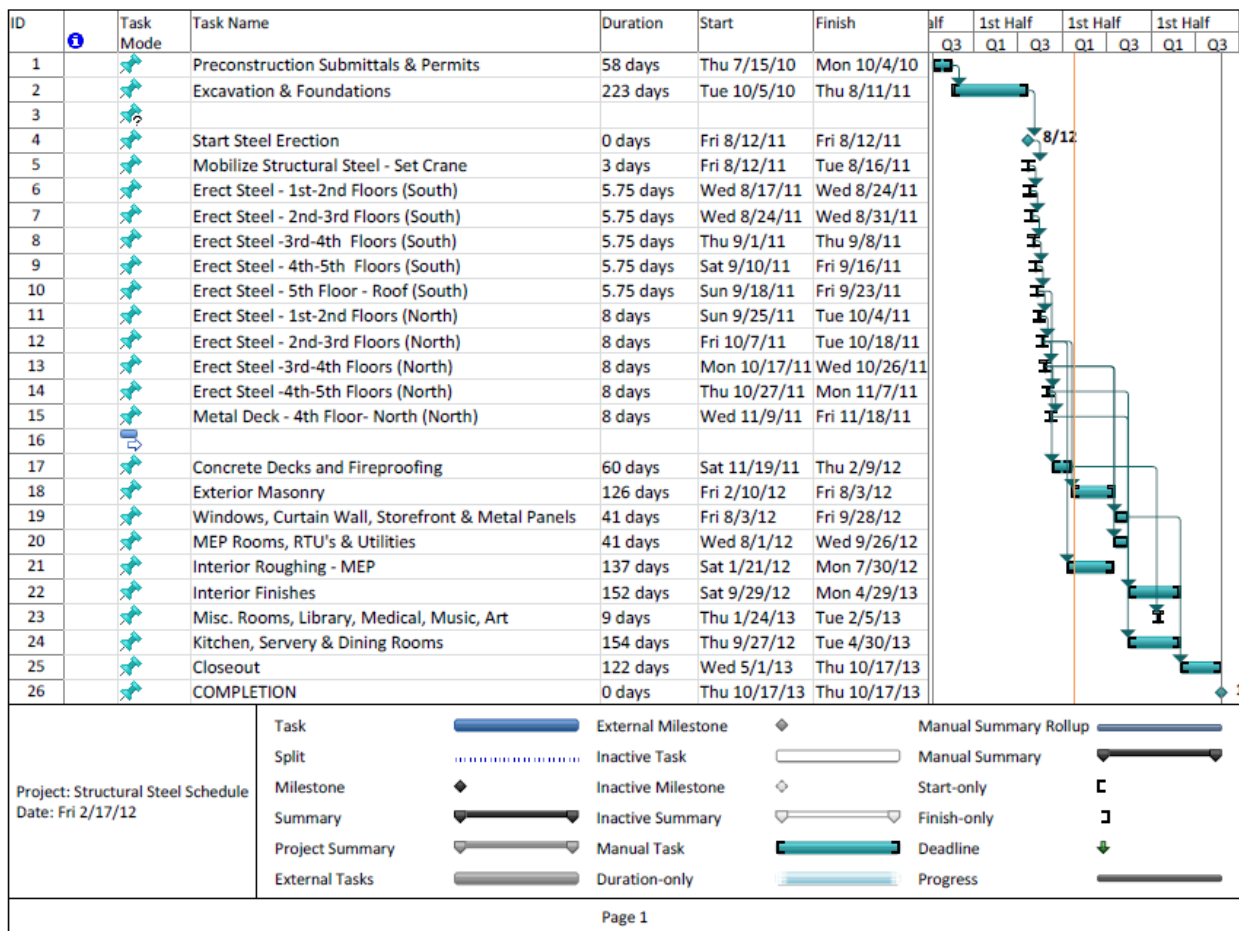


Figure 46: Summarized Schedule: ELFP EBF Design

By comparing the ELFP schedule, which can be seen in **Figure 46**, to the original schedule, the steel erection time increases by ¾ of a day on average for each floor for the south construction phase, and 1 day for each floor in the north construction phase. This increase causes the steel erection to be completed on November 18th, 2011 (an

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erection time of 72 days; a 9 day increase from CBF design). Referring to the critical path schedule obtained from SKANSKA, the overall project will be affected by the lateral system change with an increase of 11 days, causing an overall completion date of October 17th, 2013.

The second redesign focused on the MRSA EBF lateral system. Like the other two system designs, the steel and bolt erection time were analyzed using an average found from the bracing and connection design done in this report. Analysis showed that the MRSA design would take roughly 25 days to complete the lateral system erection. The change in time can be attributed to several factors. The MRSA design included additional bracing members in several bracing frames, which would increase time due to steel and connection erection. Also, the number of connections and complexity of the erections increased.

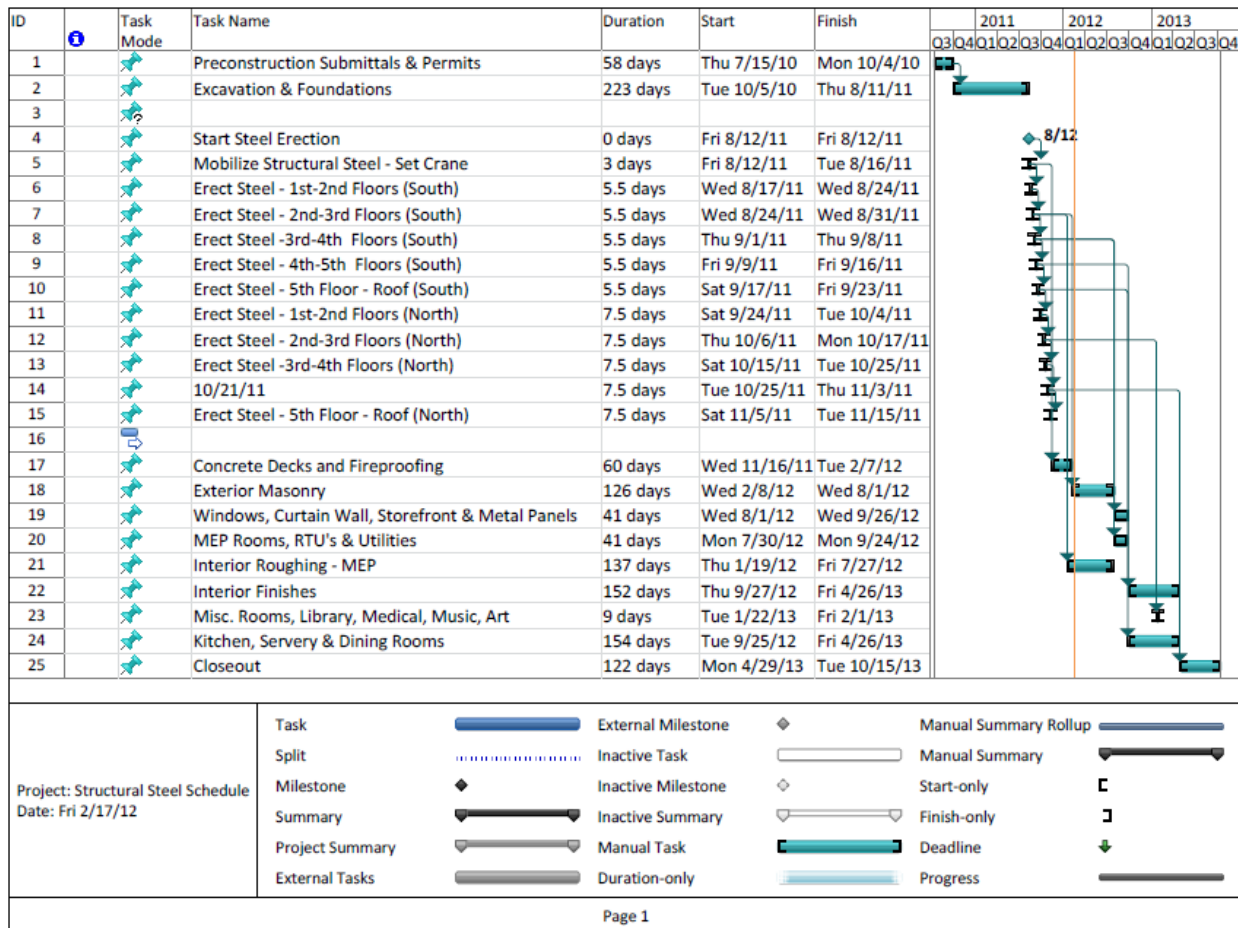


Figure 47: Summarized Schedule: MRSA EBF Design

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The MRSA design also had an increase in welded connections substituting for bolts, which again decreased construction time due to the fact that they were all mostly prefabricated welds and not done in the field. However, due to the seismic detailing, an additional inspection time was included with other factors in a 5% increase in construction time and was included in the new schedule. This additional time was factored in and a 26 day erection period was found.

By comparing the MRSA schedule, which can be seen in **Figure 47** on the previous page, to the original schedule, the steel erection time increases by $\frac{1}{2}$ of a day on average for each floor for the south construction phase, and $\frac{1}{2}$ of a day for each floor in the north construction phase. This increase causes the steel erection to be completed on November 15th, 2011 (an erection time of 68 days; a 5 day increase from CBF design). Referring to the critical path schedule obtained from SKANSKA, the overall project will be affected by the lateral system change with an increase of 8 days, causing an overall completion date of October 15th, 2013.

COST ESTIMATE

The second part of this construction breadth study focuses on the cost impact the new redesigns of Hunter's Point South have on the overall construction process. This section will focus on the both the material and erection costs of the ELFP and MRSA designs and compare them to the original design costs. Information from RS Means Construction Cost Data was used to calculate costs.

Design factors that were taken into account included the historical cost factor that takes into account the change in construction costs from now and the beginning of the actual start day of the original design, and the location factor that takes into account material and construction costs differences between different regions in the United States. As was expected, the cost difference due to time difference was very small and had little to no effect on the overall cost of each redesign. The location factor, on the other hand, was not expected to change as much as it did between New York and California. As seen in **Table 18**, the original design has a location factor that is 0.22 higher than the redesigned models. To show the difference that location plays in overall cost, each redesign shows the overall cost with *and* without the location factor included.

Table 18: Steel Cost Factors

Steel Cost Factors			
	ORIGINAL	ELFP	MRSA
Location Factor	1.30	1.08	1.08
Historical Cost Factor	0.99	1.00	1.01
Steel Weight (lbs)	391960	446632	398573
Cost/Pound	1.73	1.73	1.73

To figure out cost, total steel member weight was calculated for each design. Once the weight of the first was found, RS Means and a purchase order for steel fabrication from the original design were used to find an average cost per pound for steel. The cost for lateral steel members is compared to the total cost in **Table 23** and broken up into floor costs by multiplying by the steel weight per floor. Then all three designs were analyzed to find total steel member cost for each floor. This cost, which includes material and construction costs can be seen as the member cost in **Table 22**.

Then, using the connection information from the original and new design details, cost per connection was found for each design. These details are found in **Table 19, 20, and 21**. This was found using the assumption that each connection was a typical connection. Once again, RS Means was used to find costs for line items such as welds, bolts, and other connection details, and all costs were added together to find a total cost per connection. Then, the number of connections in the overall lateral system were totaled and a final cost was found. This can be seen as the connections cost in **Table 22**. This table also shows a total for each lateral system design.

Table 19: Original Design Typical Connection

Original Design Typical Connection:					
Item	Type	#	Cost/Unit	Uunit Type	Total Cost
Bolt	1" A490	29	10.5	bolt	304.5
Weld	1/4" Br-G	5	11.26	foot	52.5
	1/4" e-C	4	11.26	foot	48.8
	1/4" e/B	8	11.26	foot	90.1
Plate	3/4" Guss	8.6	38.5	sqft	333.0
	3/4" end	1.6	38.5	sqft	62.8
	3/4" end	1.8	38.5	sqft	69.8
Total					961.5

Table 20: ELFP Design Typical Connection

ELFP Redesign					
Typical Connection:					
Item	Type	#	Cost/Unit	Uunit Type	Total Cost
Bolt	1" a325x	36	10.16	bolt	365.76
Weld	5/16 B-G	3	14.84	ft	42.7
	5/16 E-G	3	14.84	ft	51.3
	1/4" B-E	3.1	11.26	ft	34.5
Plate	3/4" Guss	2.5	38.5	sqft	95.7
	3/4" end	2.8	38.5	sqft	106.0
T member	WT8x25	72.9	1.73	lbs	126.1
Total					822.2

Table 21: MRSA Design Typical Connection

MRSA Redesign					
Typical Connection:					
Item	Type	#	Cost/Unit	Uunit Type	Total Cost
Bolt	7/8" a325x	20	9.06	bolt	181.2
	1" a325x	16	10.16		162.6
Weld	5/16 B-G	3	14.84	ft	42.7
	1/4" E-G	3.5	11.26	ft	38.9
	1/4" B-E	3.1	11.26	ft	34.5
Plate	3/4" Guss	2.5	38.5	sqft	95.7
	3/4" end	2.8	38.5	sqft	106.0
T member	WT8x22.5	65.6	1.73	lbs	113.5
Total					775.2

Table 22: Lateral System Material and Construction Costs

Material and Construction Cost				
		Original	ELFP	MRSA
Floor 2	Members	\$190,534	\$226,363	\$205,265
	Connections	\$ 28,560	\$ 29,598	\$ 25,581
Floor 3	Members	\$187,724	\$196,224	\$179,277
	Connections	\$ 29,512	\$ 29,598	\$ 25,581
Floor 4	Members	\$105,713	\$128,044	\$111,991
	Connections	\$ 26,656	\$ 21,376	\$ 17,829
Floor 5	Members	\$ 93,485	\$109,118	\$ 93,810
	Connections	\$ 25,704	\$ 21,376	\$ 17,829
Roof	Members	\$ 94,709	\$113,833	\$ 99,998
	Connections	\$ 25,704	\$ 21,376	\$ 17,829
Total		\$808,304	\$896,907	\$794,989

After lateral system costs were calculated for each design, a revised table from the steel fabricator purchase order shows total costs due to total steel erection. This uses the difference in lateral system cost for each design to calculate overall cost in a breakdown of floor erection, steel material, and construction costs. It was decided that this was the best way to accurately show the final total cost for each steel erection process. As was done in the schedule section of this breadth study, a 5% increase of cost was added to items like Admin & Project Management to account for any issues that come up due to seismic detailing the R=8 system (ATC). This 5% increase assumes that seismically detailed structures will cost roughly 5% more than conventionally designed structures. That being said, it was assumed that the total cost difference between the original design and two redesigns would be around 5%.

When looking at the cost breakdown tables of each system, the original steel system was expected to cost roughly \$5,502,247 (**Table 23**). This cost includes material, construction, administration, and design costs through the steel fabricator. Looking at the ELFP cost breakdown table (**Table 24**), it can be seen that, without including location factor, the cost for the system is \$5,812,473 (a 5.64% increase as expected). Looking at the MRSA cost breakdown table (**Table 25**), it can be seen that, without including location factor, the cost for the system is \$5,627,315 (a 2.27% increase which is lower than expected). If location factor is included into design, the redesign costs drop dramatically. The ELFP design changes to a 0.36% increase from the original design, and the MRSA design actually decreases in overall cost by 3.25%! As stated

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before, the cost differences are attributed to change in overall lateral system steel weight, change in field work required, and number of connections.

Table 23: Original Design Lateral System Cost Breakdown

	ORIGINAL DESIGN	
1	Administration & Project Mgmt.	\$161,640
2	Structural Steel Material	\$1,697,220
3	Drawings & Engineering	\$323,280
4	Structural Steel Fabrication 2nd Floor	\$140,088
5	Structural Steel Fabrication 3rd Floor	\$910,575
6	Structural Steel Fabrication 4th Floor	\$126,080
7	Structural Steel Fabrication 5th Floor	\$112,070
8	Roof	\$112,070
9	All Other Expenses	\$1,919,224
	TOTAL	\$5,502,247
	Total with Location Factor	\$5,502,247

Table 24: ELFP Design Lateral System Cost Breakdown

ELFP DESIGN		
1	Administration & Project Mgmt. Seismic Design =5% increase (ATC)	\$169,722
2	Structural Steel Material	\$1,798,636
3	Drawings & Engineering. Seismic Design =5% increase (ATC)	\$339,444
4	Structural Steel Fabrication 2nd Floor	\$176,954
5	Structural Steel Fabrication 3rd Floor	\$919,161
6	Structural Steel Fabrication 4th Floor	\$143,131
7	Structural Steel Fabrication 5th Floor	\$123,374
8	Roof	\$126,865
9	All Other Expenses 5%increase	\$2,015,185
TOTAL		\$5,812,473
		% change 5.64%
Total with Location Factor		\$5,522,158
		% change 0.36%

Table 25: MRSA Design Lateral System Cost Breakdown

MRSA DESIGN		
1	Administration & Project Mgmt. Seismic Design =5% increase (ATC)	\$169,722
2	Structural Steel Material	\$1,715,395
3	Drawings & Engineering. Seismic Design =5% increase (ATC)	\$339,444
4	Structural Steel Fabrication 2nd Floor	\$151,839
5	Structural Steel Fabrication 3rd Floor	\$898,196
6	Structural Steel Fabrication 4th Floor	\$123,530
7	Structural Steel Fabrication 5th Floor	\$104,519
8	Roof	\$109,483
9	All Other Expenses 5%increase	\$2,015,185
TOTAL		\$5,627,315
% change		2.27%
Total with Location Factor		\$5,323,682
% change		-3.25%

CONSTRUCTION & COST IMPACT BREADTH

SUMMARY

Analysis showed that the original system would take about 22 days to erect, allowing for a steel completion time of 63 days, and an overall completion date of Oct. 7th, 2013. The first redesign using ELFP forces created a system that would take 29 days. This 7 day increase would push back steel completion by 9 days, and push the overall project completion date back to Oct. 17th, 2013. Considering there is additional framing to go up, and a 5% increase in time was attributed to seismic detailing/inspection, this additional time is not very much for this length of project. There is a higher chance that weather delays more days than the added work for the ELFP EBF system. The MRSA EBF system was found to increase the lateral steel erection 4 days, leading to a 5 day increase in steel erection and an 8 day delay in building completion (Oct. 15th, 2013). This method produces are more accurate comparison against the original design because the frame layout is similar. It can be assumed that steel detailing and inspection time make up most of this delay. Once again, this is a small delay considering the length of the project. MRSA would only save 2 more days than ELFP, so added design time may not be worth it when looking at it in a schedule standpoint.

Documents acquired from the contractor show that the cost for the original design is \$5,502,247. A breakdown shows that roughly \$810,000 goes to constructing the lateral frames. Research done prior to cost analysis of the two redesigns suggested that a more ductile system for high seismic region, such as an EBF system, would increase costs by roughly 5%. Comparing the overall costs of the redesigns without inclusion of location cost factors; this turns out to be an accurate approximation. The ELFP design creates a lateral system that costs about \$90,000 more, and creates an overall building cost increase of 5.6% (\$5,812,473). The MRSA design actually shows a \$15,000 decrease in lateral system costs, but a total building cost increase of 2.3% (\$5,627,315). This amounts to a \$200,000 savings as compared to the ELFP method. It seems clear that MRSA is well worth the added work.

Cost differences between the three designs end up being a non-issue when accounting for location factor. The new location was chosen because of its similarities to the original location (minus the seismic load intensity), but it turns out cost factors were hugely underestimated. The original design was built in New York, which costs more to construct steel buildings than Redding, California. The ELFP design costs roughly the same as the original, while the MRSA design *decreases* in cost by 3.25%! In conclusion, it is determined that cost is not a big issue when trying to move the school structure and create an adequate *and* economic design in a higher seismic region.

SUMMARY AND CONCLUSION

After analysis was completed, it was determined that the school could be moved to a higher seismic zone and a new lateral system could be designed to effectively and efficiently take the increased seismic loading while abiding to code. Both new lateral system designs were successfully designed and implemented into Hunter's Point South. Each system had its own advantages and disadvantages, but one had to be chosen as the best overall choice to redesign the school.

The Equivalent Lateral Force Procedure (ELFP) design is the quicker, simpler process, but has its drawbacks. To prevent lateral torsional irregularity, this system had to be oversized and frames needed to be moved and added. This design created a stronger, more effective system than the original CBF system, but required a lot of changes to do so.

The Modal Response Spectrum Analysis (MRSA) design is the more in-depth process, but the extra work seems to be worth the time and effort. This system creates a very efficient design due to the 15% decrease in design loads as compared to the ELFP design. Allowed to ignore torsional irregular issues, this design was able to keep the original layout of the lateral system CBF's. Though some frames required larger members than in the ELFP design, less steel had to be used overall.

When comparing the two designs as they affected the architectural layout, it was clear which one was better. The MRSA design had absolutely no impact on the architecture. The ELFP design created several architectural issues. Because the frames were moved to the exterior walls, the exterior façade (i.e. windows) had to be changed to hide the structure. This led to insufficient day-lighting in classrooms, which created the need to redesign the layout of the 2nd floor special needs classrooms in the east wing.

The cost of each system is the most important factor in the construction industry. The cost increase of the two redesigns must be small enough (or negligible) for the redesign to be an effective substitute. When including the location factor of the new and old locations, both redesigns end up costing the same or less than the original! The ELFP method was found to increase the system cost by less than 1% and delay the entire construction project by 11 days. The MRSA design was found to take only 8 days more than the original design to construct; but had an overall cost savings of 3%!

Overall, it was determined redesigning Hunter's Point South using the MRSA design prescribed in this report would be the best design choice, and would adequately and efficiently support the increased seismic loads in the higher seismic zone.

APPENDIX

APPENDIX A

ELFP EBF DESIGN AND ANALYSIS

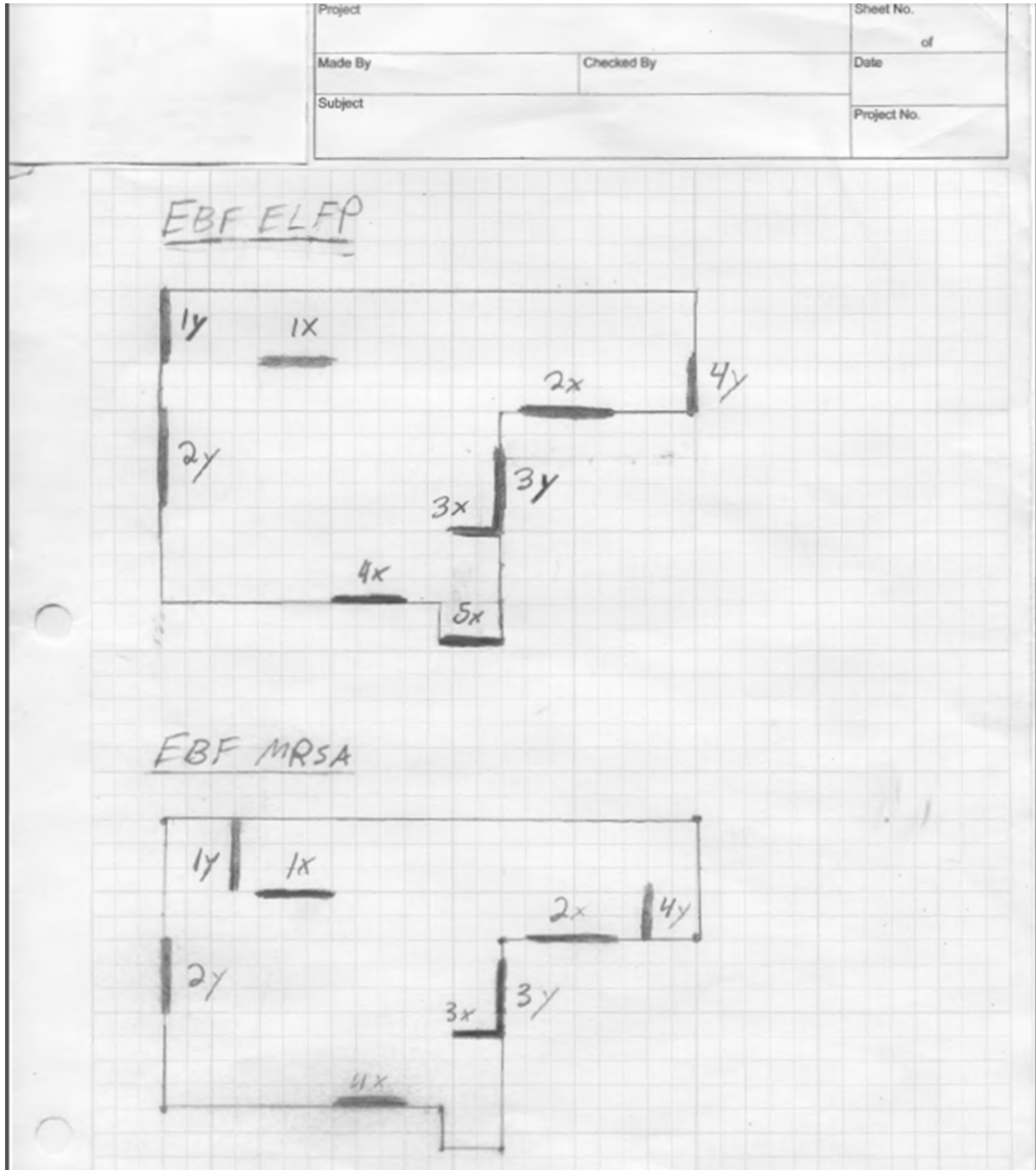


Figure 48: EBF Layout for Both Redesigns

THESIS REDESIGN

<small>Project</small>	Hunter's Point South Redesign	<small>Sheet No.</small>	1 of 2
<small>Standard</small>	ASCE7-10	<small>Category</small>	Seismic Load Calc.
<small>Subject</small>	Equivalent Lateral Force Procedure	<small>Date</small>	1/11/12
		<small>Project No.</small>	

Follow Procedure prescribed in Section 12.8 of ASCE7-10
 To calculate building seismic loads.

[5 story Steel Frame building placed in Redding, Cal. Formica]

Fig 22-1 : $S_s = 90$	Fig 22-12 : $T_L = 16 \text{ sec}$
Fig 22-2 : $S_1 = 40$	Fig 22-17 : $C_{RS} = 0.9$
Fig 22-7 : $MCE6: P6A = 30\%g$	Fig 22-18 : $C_{R1} = 0.9$

Table 12.2.1 : $R = 8$ For Eccentrically Braced Frames
 Table 20.3.1 : Site class = E (assume same as original location)
 table 1.5.1 : Risk Category = III
 table 1.5.2 : Importance Factor (C_{I2}) = 1.25

Table 11.4.1 : $F_a = 1.14$ (From interpolation)
 Table 11.4.2 : $F_v = 1.60$

$$\left\{ \begin{array}{l} S_{m5} = F_a S_s = (1.14)(90) = 1.026 \\ S_{m1} = F_v S_1 = (1.60)(40) = 0.64 \end{array} \right\}$$

$$\left\{ \begin{array}{l} S_{D5} = 2/3 S_{m5} = 2/3(1.026) = 0.684 \\ S_{D1} = 2/3 S_{m1} = 2/3(0.64) = 0.4267 \end{array} \right\}$$

$$T_0 = 0.2 \frac{S_{D1}}{S_{D5}} = (0.2) \frac{(0.4267)}{(0.684)} = 0.124766$$

$$T_s = \frac{S_{D1}}{S_{D5}} = \frac{(0.4267)}{(0.684)} = 0.62383$$

Table 11.6.1 \rightarrow SDC = D
 Table 11.6.2 \rightarrow SDC = D \rightarrow Seismic Design Category **D**

Figure 49: ELFP Redesign Loads Page 1

Project	Hunter's Point South Redesign	Sheet No.	2 of 2
ASCE 7-10	Seismic Load Calc's	Date	1/11/12
Subject	Equivalent Lateral Force Procedure		

Base Shear (V) : $V = C_s W$

$$C_s = \begin{cases} SDS/[R/I_e] = 0.684/[8/1.25] = 0.106875 \\ SD1/[T(R/I_e)] = 0.4267/[1.042(8/1.25)] = 0.06400362 \\ \min \left\{ \begin{array}{l} SD1 \cdot T_L/[T^2(R/I_e)] = (0.4267)(16)/[1.042^2(8/1.25)] = 0.93367 \\ \left(T_a = C_t h_n^x \left\{ \begin{array}{l} \text{Table 12.8.2: } C_t = 0.03 \\ h_n = 72.33 \\ \text{Table 12.8.2: } x = 0.75 \end{array} \right\} (0.03)(72.33)^{0.75} = 0.7440637 \right. \\ T = C_u T_a - \text{Table 12.8.1 } C_u = 1.4 \\ = (1.4)(0.7440637) = 1.041689 \approx 1.042 \end{array} \right. \end{cases}$$

$W = 13,263 \text{ k}$ (from weight calc. analysis)

$V_{base} = C_s W = (0.06400362)(13,263)$

$V_{base} = 849.0 \text{ k}$

* See Tables for $F_x, V, \& M$

Figure 50: ELFP Redesign Loads Page 2

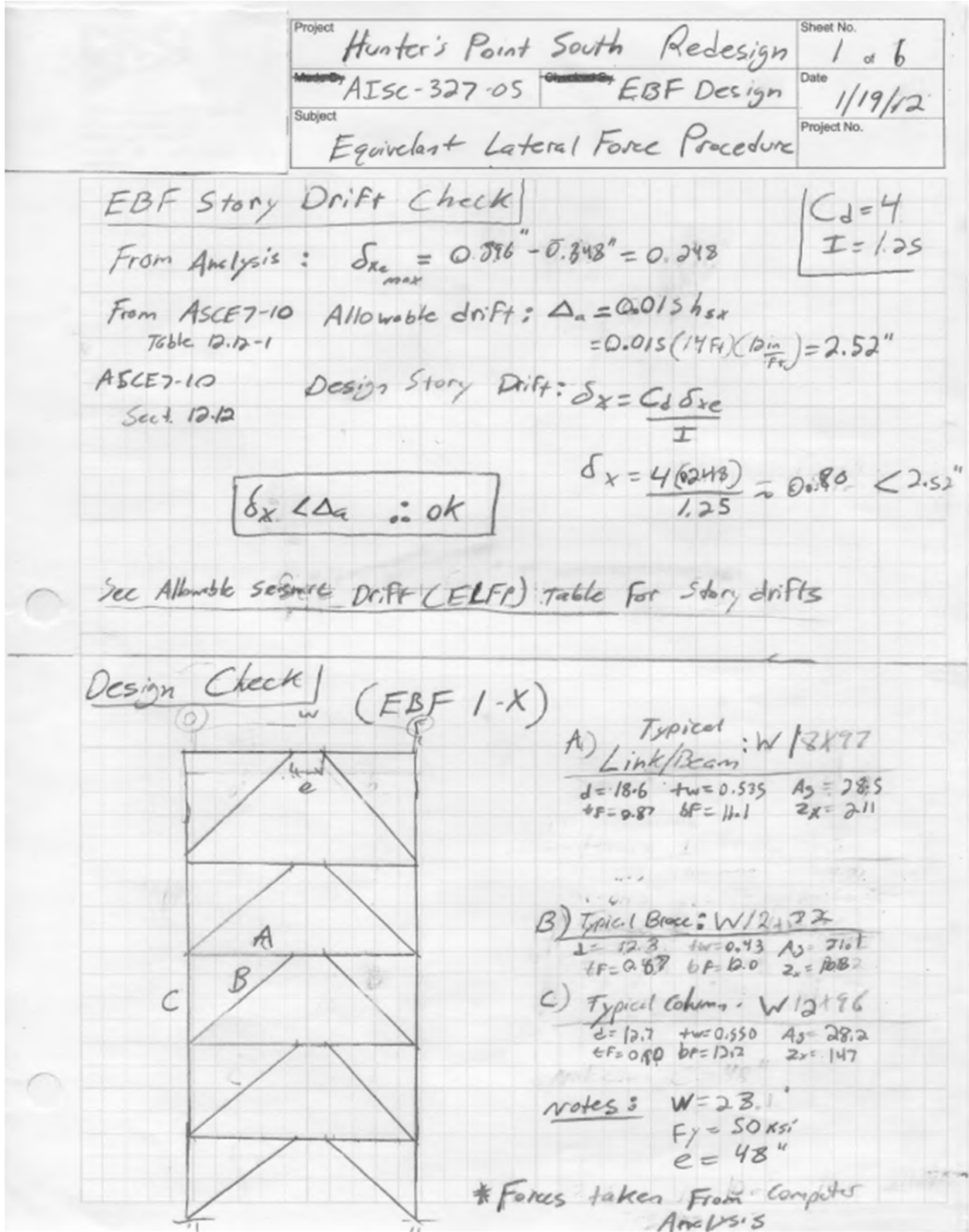


Figure 51: ELFP Redesign Page 1

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Project	Hunters' Point South Redesign	Sheet No.	2 of 6
Reference	AISC-322-05	Checked By	EBF Design
Subject	Equivalent Lateral Force Procedure	Date	1/24/12
		Project No.	

E1BF Link Design $P_u = 33.3 \text{ k}$ ^{from analysis} $V_u = 102 \text{ k}$ $M_u = 2462 \text{ k-in}$

geometry check $d_b > 16 ?$ (ok) $b_f > 10 ?$ (ok)

Slenderness check
 Per AISC 341-05 $\lambda_F = \frac{b_f}{2t_f} = \frac{11.1}{0.875} = 6.37$

Flange $\lambda_{ps} = 0.3 \sqrt{\frac{E}{F_y}} = 0.3 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 7.72 > 6.37$ (ok)

web $\lambda_w = h/t_w = 14/0.535 = 26.2$

$C_a = \frac{P_u}{\phi F_y A_g} = \frac{33.3}{0.9(50)(28.1)} = 0.026$ $P_u = 33.3 \text{ k}$

$\lambda_{ps} = 3.14 \sqrt{\frac{E}{F_y}} (1 - 1.57 C_a) = 72.6 > 26.2$ (ok)

Shear Strength

From AISC 325 Table 3-1 $0.15 P_y = 214 \text{ k}$
 AISC 341 sect 15.2b $\rightarrow P_u < 0.15 P_y$ can be ignored!
 AISC 325 Table 3-1: $V_p = 271 \text{ k}$ For shear strength
 $M_p = 10,600 \text{ k-in}$

$V = \frac{V_p}{\phi} = \frac{271}{0.9} = 301 \text{ k}$ $\phi V_n > V_u$ (ok)

$\phi V_n = \frac{0.8(271)}{0.9(2)(10600)/48} = 243.9$

Link rotation Angle AISC 341 sect 15.2b $\frac{V_p e}{M_p} = X = \frac{(271)(48)}{10600} = 1.22 < 1.6$

AISC 341 C-15.3: rotation angle = $\gamma_p = \frac{L}{e} \theta_p$ Good: dominated by shear

$= \frac{L}{e} \frac{\Delta p - \text{story drift}}{h}$

$\gamma_p = \frac{277}{48} \left(\frac{0.343}{14 \times 12} \right)$

$= 0.012 \text{ rad} < 0.08 \text{ radians} = \text{limit for shear controlled link}$

∴ W18x97 is adequate for loads in Link

Figure 52: ELFP Redesign Page 2

Project	Hunter's Point South Redesign	Sheet No.	3 of 6
Member	AISC 327-05	Classification	EBF design
Subject	Equivalent Lateral Force Procedure	Date	1/24
		Project No.	

EBF Link Design

Lateral Bracing AISC 341 15.5 $R = 0.06 R_y F_y \cdot 2 / (d - t_f)$
 $R_u = 0.06 (1.1) (50) (211) / (18.6 - 0.87) = 39.27$

∴ Top/bottom Flange bracing w/ $R_u \geq 39.3K$ will be provided @ each end of the link

End Stiffener Requirements

AISC 341 15.3 Requires double-sided, full depth stiffeners at each end of link w/ minimum width of:

$$W_{min} = \frac{b_f \cdot 2 t_w}{2} = \frac{11.7 - 2(0.535)}{2} = 5.02 \text{ in}$$

and minimum thickness of:

min $t = 0.75 b_w = 0.75 (0.875) = 0.66$
 max $t = 3/8" = 0.375$ t_{min} ≈ 1/2"

∴ Full depth 1/2 in x 5 1/4" Stiffeners will be provided on both sides on each end

Use 5/8" for ease of construction

Intermediate Stiffeners

Link depth $\geq 25"$
 ∴ required on 1 side only

Spacing: For $\rho_p \leq 0.02$, use $S_{2tw-1/8}$ in table 3-1
 Max Spacing = 24.1 in
 t_{min} ⇒ table 3-1 ⇒ 5/8"
 W_{min} ⇒ table 3-1 ⇒ 5.02"

∴ Full depth 5/8" x 5 1/4" interm. web stiff. provided along link on one side of web @ spacing of less than 24.1 inches.

Weld Check AISC 341-15.3

Stiff → web : $D = \frac{F_y A_{st}}{2(1.392)(d - 2t_f - 2(2 \cdot 3/8))}$ clip
 $A_{st} = 5/8 (5.25) = 3.28 \text{ in}^2$
 $D = \frac{(36)(3.28)}{2(1.392)(18.6 - 2(0.87) - 2(2 \cdot 3/8))} = 3.5 \text{ sixteenths}$
 table J24 min = 1/4 or 4/16"

Stiff → Flange : $D = \frac{F_y A_{st}}{4(2(1.392)(5.25 - (3/4)))}$ clip
 $D_{SF} = \frac{(36)(3.28)}{(4)(2)(1.392)(5.25 - 3/4)} = 2.356 \text{ sixteenths}$
 table J24 min = 1/4 or 4/16"

∴ Use double sided 1/4" Fillet weld to connect stiffeners to the link web & to the link flanges

Figure 53: ELFP Redesign Page 3

THESIS REDESIGN

Project	Hunters Point South Redesign	Sheet No.	4 of 6
Master By	ATSC 327-05	Checked By	EBF design
Subject	Equivalent Lateral Force Procedure	Date	1/24/12
		Project No.	

EBF outside Beam design Determine Factored loads

$V_Q \approx 95.7 \text{ k}$ → overstress; Factor: $1.1 \frac{R_y V_Q}{V_Q} = \frac{(1.1)(1.1)(95.7)}{95.7} = 3.43$

$P_E = 3.43 P_Q = 3.43(107.9) = 370 \text{ k}$

$V_E = 3.43 V_Q = 3.43(11.4) = 39 \text{ k}$

$M_E = 3.43 M_Q = 3.43(146.4) = 502 \text{ k-ft}$

Beam slenderness From Link design: $\lambda_w = 26.2$

From ATSC Table B-4.1: $\lambda_p = 2.76 \sqrt{E/F_y}$

$= 90.6 > 26.2$ (OK)

unbraced Length $L_b = \frac{L - c - 2(\frac{d_c}{2})}{2}$

$= \frac{(23 \times 12) - 48 - 2(12.7)}{2} = 131.65'' \approx 11 \text{ ft}$

2nd order effects: $B_1 = \frac{C_m}{1 - \frac{P_r}{P_c}} \geq 1$ $B_2 = 1.0$ bc ends not to move

Assuming $k=1.0$ $P_c = \frac{\pi^2 EI}{(kL)^2} = \frac{\pi^2 (29000)(1780)}{[(1.0)(132)]^2} = 28750 \text{ k}$ $P_r = 370 \text{ k}$

$C_m = 1.0$ Assumed conservative

$B_1 = 1.0 / [1 - \frac{15(370)}{28750}] = 1.01$

$M_r = 1.01(502) = 509 \text{ k-ft}$

Combined Loading $KL = 27\frac{1}{2} - 2' = 11.5 \text{ ft}$

ATSC Table 6.1: $p = 0.950 \times 10^3 \text{ k}$ $b_x = 1.17 \times 10^{-3} \text{ k-ft}$

ATSC 341 $\frac{P_r}{P_c} = \frac{P_r}{R_y} = \frac{(0.95 \times 10^3)(370)}{1.1} = 0.32 > 0.2$ $\frac{P_r}{P_c} + \frac{3}{9} \frac{M_r}{M_c} \leq 1$

Section 6 $\frac{3}{9} \frac{M_r}{M_c} = \frac{b_x M_r}{R_y} = \frac{(1.17 \times 10^{-3})(502)}{1.1} = 0.533$ $0.32 + 0.533 = 0.853 < 1$

note: additional Flange bracing not required! (OK)

$\phi_w 18 \times 97$ adequate to resist loads
 so given for the beam outside the Link

Figure 54: ELFP Redesign Page 4

THESIS REDESIGN

Project	Hunter's Point South Redesign	Sheet No.	5 of 6
Member	AISC 327-05	Connectivity	EBF design
Subject	Equivalent Lateral Force Procedure	Date	1/25/12
		Project No.	

EBF Brace Design W12x72 P=118.0 k V=4.03 k M=43.0 k-ft

Factored Loads AISC 341-15.6: Load = Load + 1.25 brk shear strength
 $1.25 R_y V_{brk} = 1.25(1.1)(271) = 372.5 \text{ k}$
 ∴ over strength Factor $\Rightarrow \frac{1.25 R_y V_{brk}}{V_a} = \frac{373}{90} = 3.16$

$P_e = 3.16(118.0) = 432 \text{ k}$
 $V_e = 3.66(4.03) = 15.0 \text{ k}$
 $M_e = 3.66(43.0) = 157 \text{ k-ft}$

AISC 325 - Table 12
 W12.72 allowed
 (bracing member)

Brace Slenderness AISC 341 15.6: $\lambda_p = \frac{b_F}{2t_F} = \frac{12.0}{(2 \times 0.67)} = 8.95$
 $\lambda_p = 0.38 \sqrt{\frac{E}{F_y}} = 0.38 \sqrt{\frac{29000}{50}} = 9.15 > 8.95$ (OK)
 $\lambda_w = \frac{h}{t_w} = \left(\frac{123 - 2 \times 0.63}{0.43} \right) = 85.48$
 table B4.1 $\lambda_p = 3.76 \sqrt{\frac{E}{F_y}} = 3.76 \sqrt{\frac{29000}{50}} = 90.6 > 85.48$ (OK)
 ∴ web & flanges meet local buckling requirements

Unbraced Length $L_b = \sqrt{11^2 + 14^2} = 17.8 \text{ Ft} = 214 \text{ in}$
 Assume k=1.0
 $P = \pi^2 E I / (kL)^2 = \pi^2 (29000)(597) / (10 \times 214)^2 = 3731 \text{ k}$ conservatively Assume $C_m = 1.0$
 $P_e = 482$
 $B_1 = 1.0 / \left[1 - \frac{1.0(482)}{3731} \right] = 1.13$ $B_2 = 1.0$ b/c ends don't translate
 ∴ $M_e = 1.13(157) = 178 \text{ k-ft}$

Combined Loading $KL = 17.8 \text{ Ft}$ Table 6.1 $P_c: 1.50 \times 10^{-3}$
 $P_r / P_c = \frac{P_r}{R_y} = \frac{1.05 \times 10^{-3} (432)}{1.1} = 0.589 > 0.2$
 $\frac{8}{9} \frac{M_e}{M_c} = \frac{b_1 M_e}{R_y} = \frac{2.42 \times 10^{-3} (178)}{1.1} = 0.348$ $\left. \begin{array}{l} P_r / P_c > \frac{M_e}{M_c} \\ \frac{8}{9} \frac{M_e}{M_c} \end{array} \right\} \frac{P_r}{P_c} + \frac{M_e}{M_c} = 0.94 < 1.0$ (OK)

Shear Strength $\frac{h}{t_w} = 85.48$ $2.24 \sqrt{\frac{E}{F_y}} = 54$ $2.24 \sqrt{54} = 54$ (OK)
 ∴ $C_v = 1.0$ $V_n = 0.6 F_y A_w C_v = 0.6(50)(12.3)(0.43)(1.0) = 160 \text{ k} > 13 \text{ (OK)}$ ∴ W12x72 is adequate to resist brace loads

Figure 55: ELFP Redesign Page 5

THESIS REDESIGN

Project	Hunters Point South Redesign	Sheet No.	6 of 6	
Reference	AISC 327-05	Case	EBF design	
Subject	Equivalent Lateral Force Procedure		Date	1/26/12
		Project No.		

EBF Column Design | W12x96 C: $P_u = 120K$ $M_u = 36.1 Kft$ $K_m = 3$ K_R
 Ends of column are fixed T: $T_u = 67$ $M_u = 83.9 = 77 Kft$

Check $P_u/\phi P_n$ | Table 4.1 $L=14 \Rightarrow \phi P_n = 1020K$ $\frac{P_u}{\phi P_n} = \frac{120}{1020} = 0.12 < 0.4$
 AISC 341 Sect 8.3 $P_u/\phi P_n < 0.4$
 ∴ do not worry about Amplified seismic Load effects.

Required Column Strength | per AISC 341 15.8
 strain hardened shear strength of links above: table 3-1 W18x97 = 271 K each
 $1.0 R_n \leq V_n = 1.1 (1.1) (271 + 271 + 271) = 984 K$
 Assume the following controls: $P_u = 984 K$ $M_{ux} = 7 Kft$ $M_{uy} = 8 Kft$

Second order effects
 $P_{e_x} = \pi^2 EI / (KL)^2 = \pi^2 (29000)(833) / (14(12))^2 = 8447 K$ $K=1$
 $P_{e_y} = I_y / I_x P_{e_x} = \frac{270}{833} (8447) = 2738 K$ $C_m = 1$
 $P_r = P_u + B_2 P_r \Rightarrow B_2 P_r = 1.0 (984) = 984 K$ $\alpha = 1$
 $B_{1x} = 1 / (1 - \frac{984}{8447}) = 1.13$ $M_r = B_1 M_{rt} + B_2 M_{rb} = B_1 M_u$
 $B_{1y} = 1 / (1 - \frac{984}{2738}) = 1.86$ $M_{rx} = 1.13 (7) = 7.91 Kft$
 $M_{ry} = 1.86 (8) = 14.88 Kft$

Combined Loading | Table 6-1 $K_1 = 14$ $p = 0.978 \times 10^{-3}$ $b_x = 1.67 \times 10^{-3}$
 $b_y = 3.51 \times 10^{-3}$
 $\frac{P_r}{P_c} = p P_r = 0.978 \times 10^{-3} (984) = 0.963 < 2$
 $\frac{8}{9} \frac{M_{rx}}{M_{cx}} = b_x M_{rx} = 1.67 \times 10^{-3} (7.91 Kft) = 0.012$
 $\frac{8}{9} \frac{M_{ry}}{M_{cy}} = b_y M_{ry} = 3.51 \times 10^{-3} (14.88) = 0.038$
 $0.963 + 0.012 + 0.038 = 0.963 < 1.0$ ✓
 ∴ W12x96 is adequate for column loads

Figure 56: ELFP Redesign Page 6

APPENDIX B

MRSA EBF DESIGN AND ANALYSIS

Project	Hunters Point South Redesign	Sheet No.	1 of
Modeling	ASCE 7-10	Classification	Seismic Load Calc
Subject	Modal Response Spectrum Analysis	Date	1/26/12
		Project No.	

Follow ASCE 7-10 12.9 using Etabs model for analysis
 Analysis will include modal mass participation of 90% or more
 → using ETABS : modal mass participation of 6 modes = 93%
 • Modal Response Spectrum is created in ETABS

Scaling Forces

ELFP Base Shear ($C_s W$) = 849 k
 MRSA Base Shear (V_r) = 11.67 k

∴ $V_r < 0.85 C_s W$
 ↳ Scale Factor = $0.85 \frac{C_s W}{V_r} = 0.85 \frac{849}{11.67} =$ 61.838

• Add scaling factor to ETABS model → New Base Shear = 721.86 (OK)
 (∵ $85\% C_s W$)

• Using Etabs, Find story shears & story forces
 (Excel Table)

Figure 57: MRSA Redesign Loads Page 1

THESIS REDESIGN

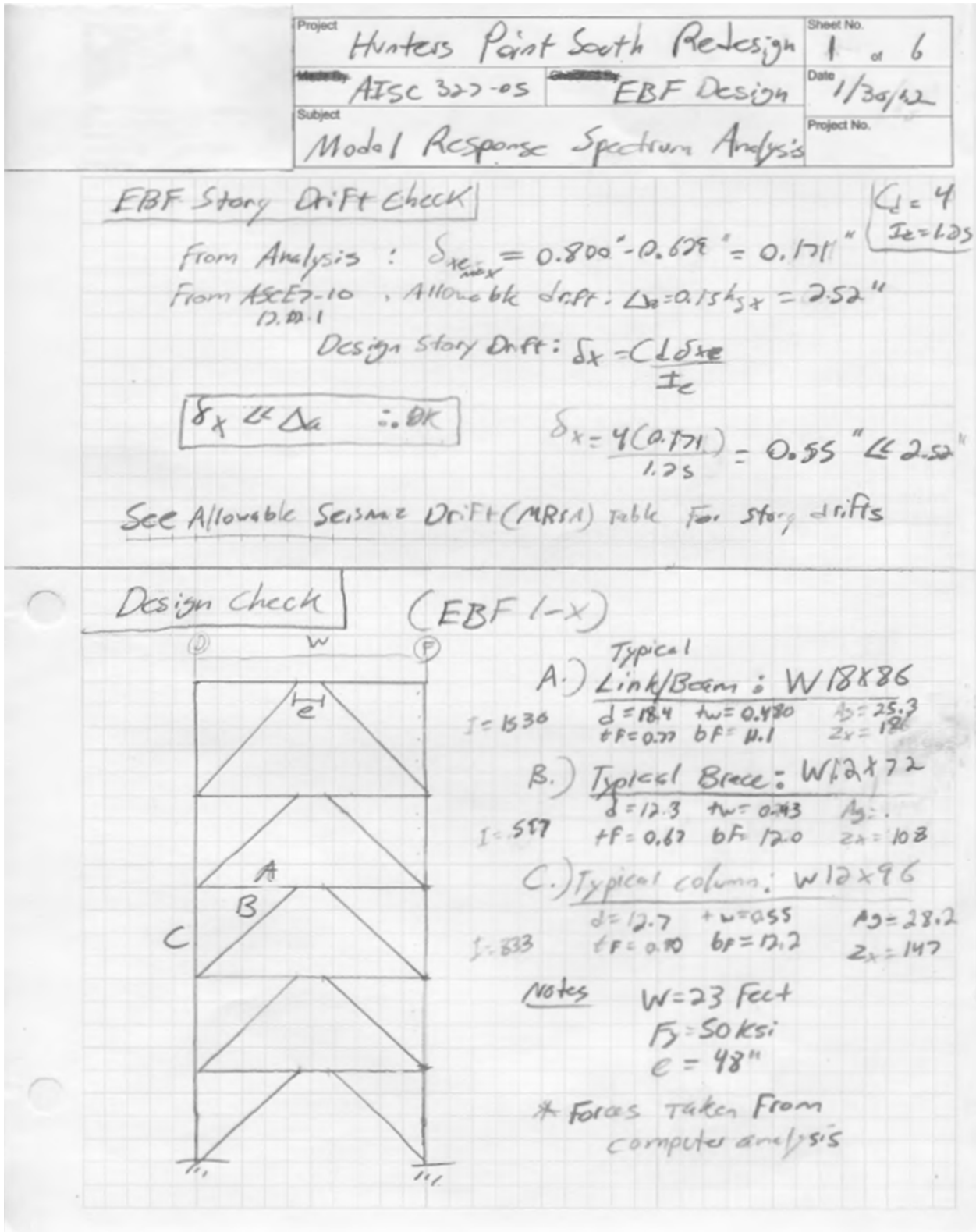


Figure 58: MRSA Redesign Page 1

THESIS REDESIGN

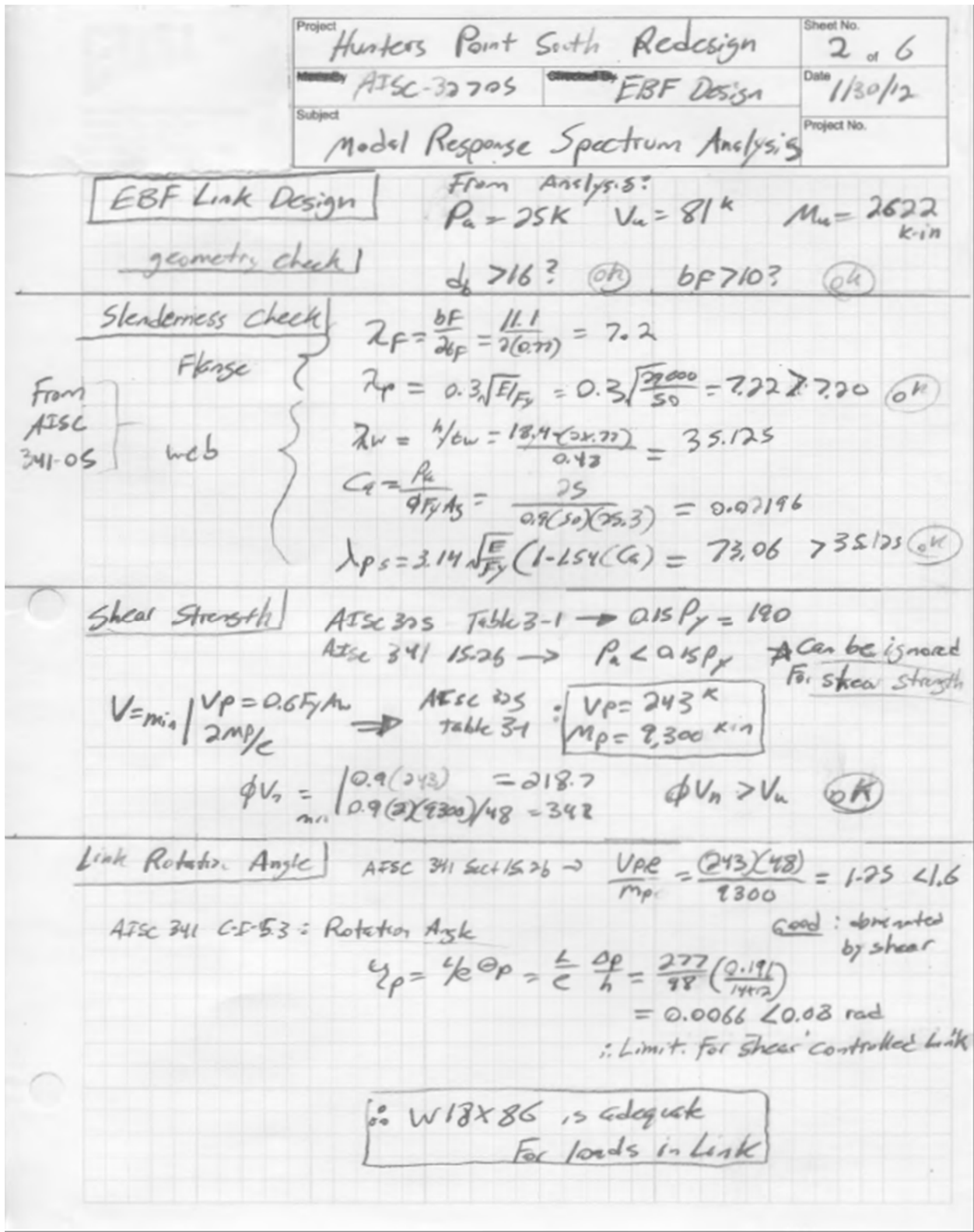


Figure 59: MRSA Redesign Page 2

THESIS REDESIGN

Project	Hunters Point South Redesign	Sheet No.	3 of 6	
Met by	AISC 327-05	Controlled by	FBF Design	
Subject	Modal Response Spectrum Analysis		Date	1/30/12
			Project No.	

FBF Link Design

Lateral Bracing

AISC 341 15.5 $\Rightarrow R = 0.06 R_y F_y Z / (d - 6t_f)$

$R_u = (0.06)(1.1)(50)(18) / (18.4 - 0.77) = 34.8^k$

\therefore Top/bottom Flange bracing w/ $R_u > 34.8^k$ will be provided @ each end of the link

End Stiffener Requirements Per AISC 341.15.3

$W_{min} = \frac{bf - 2t_w}{2} = \frac{11.1 - 2(0.48)}{2} = 5.07$

$t = \max \left\{ \begin{array}{l} 0.75 t_w = 0.75(0.48) = 0.36 \\ 3/8" = 0.375 \end{array} \right. \quad t_{min} = 3/8"$

Use 1/2" Stiffener for each of const.

\therefore Full depth 3/8" x 5 1/4" Stiffeners will be provided on both sides of the web on each end of Link

Intermediate Stiffeners

Spacing: For $\rho_p \leq 0.02 \rightarrow$ use 5.2.4.1.5 in table 3-1

$L_s \max$ spacing = 21.3 in

table 3-1 $\left\{ \begin{array}{l} t_{min} = 1/2" \\ W_{min} = 5.07" \end{array} \right.$

Link depth < 25" \therefore req. on 1 side only

\therefore Full depth 1/2" x 5 1/4" intern. web stiff. provided along link on one side of web @ spacing less than 21.3 in

Weld Check AISC 341-15.3

$A_{st} = 1/2" (5.75) = 2.625"$

Stiffener web: $D = \frac{F_y A_{st}}{2(1.392)(d - 2t_f - 2(2 \cdot 3/8))}$ clip

Stiffener Flange: $D = \frac{F_y A_{st}}{4(2.132)(w - 3/4)}$ clip

$D_{sw} = \frac{(36)(2.625)}{2(1.392)(18.4 - 0.77) - 2(2 \cdot 3/8)} = 2.81$ sixteenths

table 3.2.4 min = 1/4 or 4/16

$D_{sf} = \frac{(36)(2.625)}{4(2.132)(5.75 - 3/4)} = 2.356$ sixteenths

table 3.2.4 min = 1/4 or 4/16

\therefore use double sided 1/4" fillet weld to connect stiffeners to the link web & to the link flanges

Figure 60: MRSA Redesign Page 3



Figure 61: MRSA Redesign Page 4

THESIS REDESIGN

Project	Hunters Point South Redesign	Sheet No.	5 of 6
Reference	AISC 327-05	Category	EBF Design
Subject	Model Response Spectrum Analysis	Date	1/30/12
		Project No.	

EBF Brace Design | W 12x72 | P-104 | V=3361 k | M=37.75

Factored loads | AISC 341-15.1 → Load = Load + 1.25 link shear strength
 $1.25 R_y V_{link} = 1.25(1.1)(243) = 334.2 \text{ k}$
 over strength Factor → $\frac{1.25 R_y V_n}{V_o} = \frac{334.2}{85} = 3.91$
 $P_o = 3.91(104) = 406.6$
 $V_o = 3.91(334.2) = 1306.7$
 $M_o = 3.91(37.75) = 147.6$

Brace Slenderness | AISC 341 15.6: $\lambda_p = \frac{b_F}{2t_F} = \frac{10.0}{(2 \times 0.57)} = 8.75$
 table B4.1 $\lambda_p = 0.38 \sqrt{E/F_y} = 9.15 > 8.75$ (OK)
 $\lambda_w = \frac{h}{t_w} = \frac{(12.3 - 2 \times 0.67)}{0.43} = 25.48$
 Table B4.1 $\lambda_p = 2.76 \sqrt{E/F_y} = 90.6 > 25.48$ (OK)
 in web & flanges meet local buckling requirements

unbraced Length | $L_b = 17.8 \text{ ft}$ or 214 in | Assum $k=1, \alpha=1, C_m=1.0$
 $P = \pi^2 EI / (KL)^2 = \pi^2 (29000)(597) / (10 \times 214)^2 = 3717.0 \text{ k}$ | $P_o = 406.6$
 $B_1 = 1.0 / [1 - \frac{P_o(365)}{3717}] = 1.11$ | $B_2 = 1.0$ b/c ends don't translate
 $M_r = 1.11(147.6) = 163.8$

Combined loading | $KL = 17.8 \text{ ft}$ | Table 6.1 | $P_o = 1.50 \times 10^3$
 $P_r = 1.50 \times 10^3$
 $P_c = 242 \times 10^3$
 $\frac{P_r}{P_c} = \frac{P_o}{R_y} = \frac{(1.5 \times 10^3)(365)}{1.1} = 0.50 > 0.2$
 $\frac{8/9 M_r}{M_c} = \frac{b_1 M_r}{M_c} = \frac{2.42 \times 10^3 (146.7)}{1.1} = \frac{8}{9} (232) = 0.79$
 $\frac{P_r}{P_c} + \frac{8}{9} \frac{M_r}{M_c} = 0.80 < 1.0$ (OK)

Shear Strength | $h/t_w = 25.48$ | $2.24 \sqrt{E/F_y} = 54 > 25.48$ (OK)
 $\phi C_v = 1.0$ | $V_n = 0.6 F_y A_w C_v$
 $= 0.6(50)(12.3)(243)(1.0) = 160 \text{ k} > 12.3 \text{ k}$ (OK)
 ∴ W12x72 is adequate to carry brace loads

Figure 62: MRSA Redesign Page 5

THESIS REDESIGN

Project Hunter's Point South Redesign		Sheet No. 6 of 6
Reference ATSC 377-05	Classification EBF Design	Date 1/31/12
Subject Modal Response Spectrum Analysis		Project No.

EBF Column Design W12x96 C: 116.2 k $M_{uc} = 3.4$ k-Ft
 Ends of columns are Fixed T: 55.4 k $M_{uT} = 8.05$ k-Ft

Check P_u/P_n table 4.1 L=14 $\rightarrow \phi P_n = 1020$ $\frac{P_u}{\phi P_n} = \frac{116.3}{1020} = 0.11 < 0.4$
 AISC 341 section 8.3 $P_u/P_n < 0.4$
 \therefore Do NOT worry about amplified seismic load effects

Required Column Strength per AISC 341-15.8
 Strain hardened shear strength of Links above: Table 3-1 W18x86 $\rightarrow 243$ k each
 $1.1R_y \leq V_n = (1.1)(1.1)(243 + 243 + 243) = 882$ k
 Assume the Following Controls: $P_u = 882$ k $M_{ux} = 8.05$ $M_{uy} = 2.9$ k

Second order effects
 $P_{ex} = \pi^2 EI / kL^2 = \pi^2 (29000)(882) / (14 \times 12)^2 = 8447.5$ k $k=1$
 $P_{ey} = I_y / I_x P_{ex} = 270/833(8447.5) = 2738$ k $C_m=1$
 $\alpha=1$
 $P_r = P_u + B_2 P_{1t} \Rightarrow B_2 P_u = (1.0)(882) = 882$ k
 $B_{1x} = 1 / (1 - \frac{882}{8447.5}) = 1.12$ $M_r = B_{1x} M_u + B_{2x} M_{1t} = B_{1x} M_u$
 $B_{1y} = 1 / (1 - \frac{882}{2738}) = 1.47$ $M_{rx} = 1.12(8.05) = 9.0$
 $M_{ry} = 1.47(2.9) = 4.3$

Combined loading $K_1 = 14$ $p = 0.978 \times 10^{-3}$ $b_x = 1.67 \times 10^{-3}$
 $b_y = 3.51 \times 10^{-3}$
 $\frac{P_r}{P_c} = P/P_r = 0.978 \times 10^{-3}(882) = 0.863$
 $\frac{8}{9} \frac{M_{rx}}{M_{cx}} = b_x M_{rx} = 1.67 \times 10^{-3}(9.0) = 0.02$
 $\frac{8}{9} \frac{M_{ry}}{M_{cy}} = b_y M_{ry} = 3.51 \times 10^{-3}(4.3) = 0.02$
 $0.863 + 0.02 + 0.02 = 0.905 < 1.0$ (OK)

\therefore W12x96 is adequate for column loads

Figure 63: MRSA Redesign Page 6

APPENDIX C

CONNECTION DESIGN AND ANALYSIS

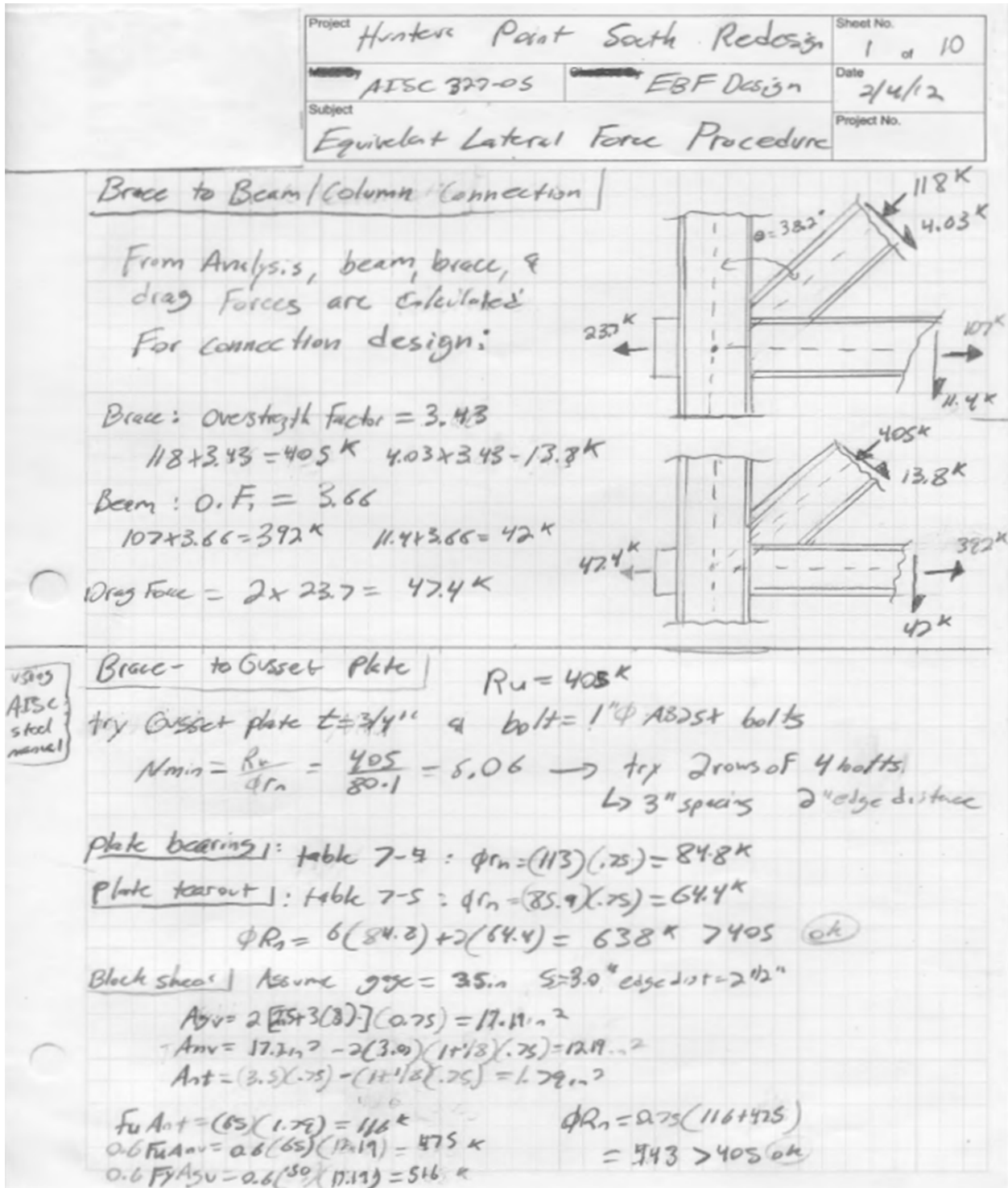


Figure 64: ELFP Connection Design Page 1

THESIS REDESIGN

Project	Hunter's Point South Redesign	Sheet No.	2 of 10
Master No.	ASCE 3705	Checked By	EBF design
Subject	Equiweight lateral Force Resistor	Date	2/4/12
		Project No.	

Whitmore Section: $L_w = 3\frac{1}{2} \text{ in} + 2(3(3.0)) + 30 = 13.9 \text{ in}$
 $L = \frac{8\frac{9}{8} + 2(19\frac{1}{2})}{3} = 3.58 \text{ in}$
 $K_L = (0.05)(3.58) = 2.33$
 using table 1-8 (ASCE 370) $\rightarrow \phi R_n = 33.8 \text{ k/in}$
 $\phi R_n = 33.8 \text{ kips/in}(13.9) = 469.9 \text{ k} > R_u$ (OK)
 USE A $\frac{3}{4}$ " gusset plate

Gusset-brace connection Brace = $12 \times 72 \rightarrow T = 9\frac{1}{8}$
 choose WT w/ bf \angle brace \rightarrow WT 8 \times 25 $b_f = 9.07 < 9.125$ (OK)
 \rightarrow (2) WT 8 \times 25

Tension yield $R_u = 405 \text{ k}$ $\phi R_n = 0.9 F_y A_g$
 $= 0.9(50)(2)(25.7) = 663 \text{ k} > 405$ (OK)

Tension Rupture $A_n = 2(A_g - 2d h_{bf})$
 $= 2(7.37 - 2(1\frac{1}{8})(6.30)) = 11.91 \text{ in}^2$
 $U = 1 - \frac{x}{L} = 1 - \frac{1.89}{2.5} = 0.82 \rightarrow A_e = U A_n = 0.85 A_g$
 $= 0.82(11.91) = 0.85(2)(7.37) = 9.8 \leq 12.6$
 $\phi R_n = 0.75 F_u A_e = 0.75(65)(9.8) = 477 > 405$ (OK)

Slenderness
 Flange) $\lambda_r = 0.45 \sqrt{E/F_y} = 10.8$ $\frac{b_f}{2t_f} = \frac{7.07}{2(6.3)} = 5.6 < 10.8$ (OK)
 web) $\lambda_r = 0.75 \sqrt{E/F_y} = 18.1$ $\frac{d}{t_w} = \frac{8.13}{0.38} = 21.4 > 18.1$
 \therefore local buckling reduces compressive strength

Table B34-1

Figure 65: ELFP Connection Design Page 2

THESIS REDESIGN

Project	Hunters Point South Redesign	Sheet No.	3 of 10
Master By	AISC-327-05	Checked By	EBF Design
Subject	Eq.ivalent Lateral Force Procedure	Date	2/4
		Project No.	

Compressive Strength | $\frac{KL}{r} = \frac{0.65(S.S)}{1.5F} = 2.25$

Table 1-8 (AISC 322) $\rightarrow Q_s = 0.824$ $F_{cr} = 0.824(50 \text{ ksi}) = 41.2 \text{ ksi}$

$\phi P_n = \phi F_{cr} A_g$
 $= (0.9)(41.2)(2)(237) = 546 > 405$ (OK)

$\frac{bF}{d} = \frac{7.07}{8.13} = 0.87$ $\left. \begin{array}{l} bF/d > 0.5 \\ tF/tw > 1.0 \end{array} \right\}$ Flats torsional buckling need not be checked (table C-E9.2)

$\frac{tF}{tw} = \frac{0.630}{0.38} = 1.66$

Bearing / Tearout | ΣWT Flange thickness = $2 \times 0.63 > 0.75$ " (Gusset)
 by inspection WT will pass

Block shear | By inspection, Gusset will control so WT will pass

use (2) T8x25 to connect Brace web to gusset

Bearing / T.O brace web | $B_1 \phi R_n = (113)(0.43) = 49$ $8 \times 49 + 2 \times 37 = 466 > 405$ (OK)
 table 7-4/5 T.O $\phi R_n = 85.9(0.43) = 37$

Block Shear

$A_{gv} = 2[2.5 + 5(3)](0.43) = 18.05$ $F_u A_{nt} = 65(1.03) = 67$
 $A_{nv} = 18.05 - 2(3)(1/8)(0.43) = 12.0$ $0.6 F_u A_{nv} = 0.6(65)(12.0) = 475.8$
 $A_{nt} = (3.5)(0.43) + (11/8)(0.43) = 1.03$ $0.6 F_y A_{gv} = 0.6(50)(18.05) = 541.5$

$\phi R_n = (0.75)(67 + 475.8) = 407 > 405$ (OK)

Shear rupture | $A_n = A_g - 2(dh + 1/16)tw = 21.1 - 2(1/8)(0.43) = 20.2 \text{ in}$
 $U = 1 - \frac{x}{L}$ or 0.7 conservatively
 $A_e = (0.7)(20.2) = 14.15$ $\phi P_n = 0.75(65)(14.15) = 689 > 405$ (OK)

use (12) 1" ϕ A325X bolts to connect brace web to WT section connectors - use 3" spaces, 2.5" edge, 3.5" gage

Figure 66: ELFP Connection Design Page 3

THESIS REDESIGN

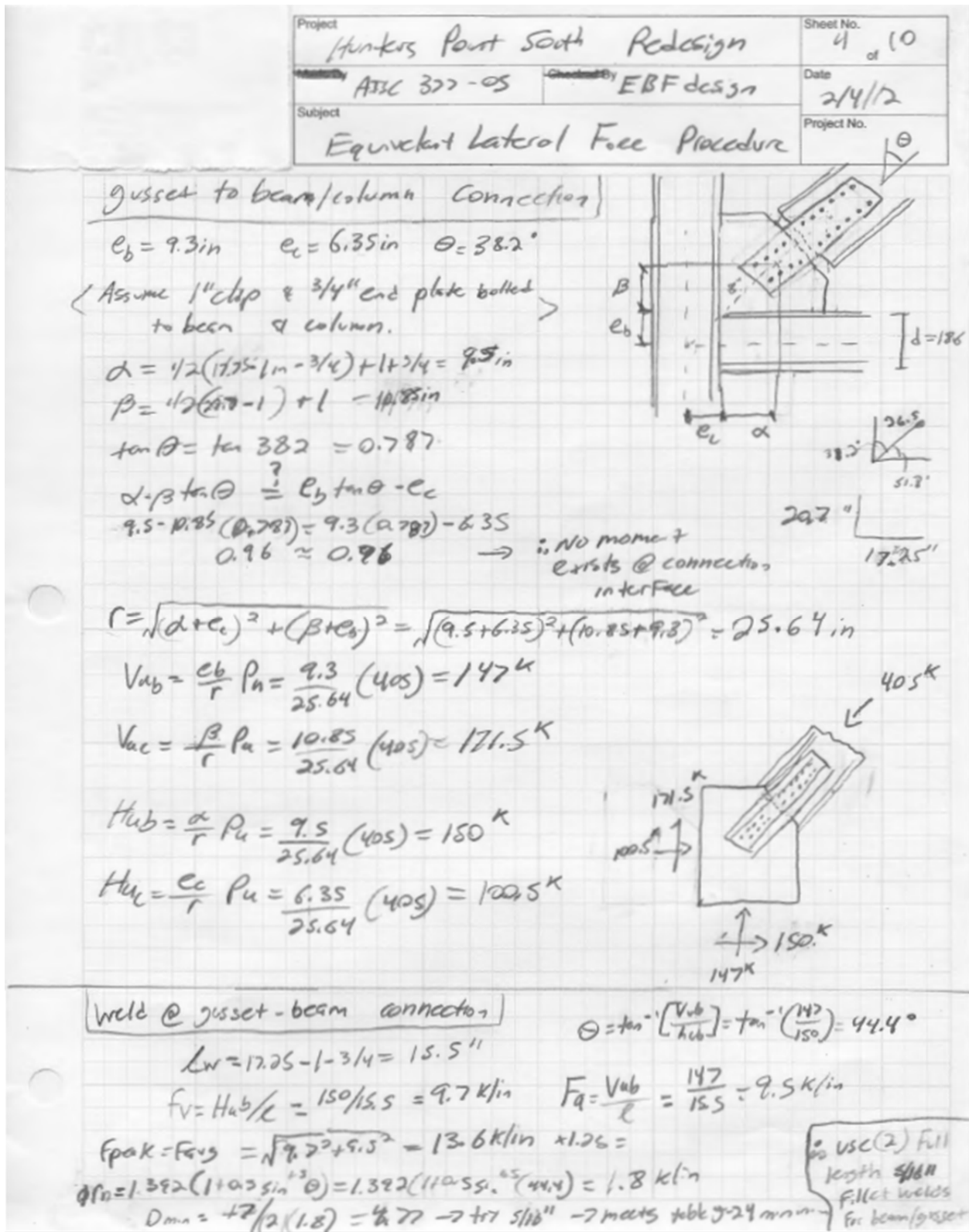


Figure 67: ELFP Connection Design Page 4

THESIS REDESIGN

Project	Hunter's Point South Redesign	Sheet No.	5 of 10
Made By	AISC 327-05	Checked By	EBF Design
Subject	Equivalent Lateral Force Procedure		Date 2/4/12
		Project No.	

Gusset yielding $\phi R_n = \phi(0.6)(F_y)(t)(L_w)$ $\sqrt{147^2 + 150^2}$
 $= 0.9(0.6)(50)(0.75)(15.5) = 314k > 210k$ (OK)

Beam web local yielding
 (Force applied L \leq 6 from the end) $\phi R_n = \phi(2.5k)(L_w)F_y + t_w$
 $= 1.0(2.5)(1.77 + 15.5)(50)(0.535) = 500k \gg 147k$ (OK)

Beam web crippling
 $L_w/d = 15.5/18.6 = 0.833$
 Force applied > 6 from the end
 $R_n = 0.8t_w^2 \left[1 + 3 \left(\frac{L_w}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_y t_w t_f}{L_w}} = 0.8(0.535)^2 \left[1 + 3(0.833) \left(\frac{0.535}{0.87} \right)^{1.5} \right] \sqrt{\frac{29000(50)(0.87)}{15.5}}$
 $R_n = 775k \gg 147k$ (OK)

Weld between Gusset & end plate $L_w = 20.7 - 1 = 19.75 \text{ in}$
 $F_v = \frac{V_{uc}}{L} = \frac{171.5}{19.75} = 8.7 \text{ k/in}$ $F_c = \frac{t_{uc}}{L} = \frac{100.5}{19.75} = 5.1 \text{ k/in}$
 $F_r = \sqrt{F_v^2 + F_c^2} = \sqrt{8.7^2 + 5.1^2} = 10.1 \text{ k/in} \times 1.25 = 12.6 \text{ k/in}$
 $\phi = \tan^{-1} \left(\frac{t_{uc}}{V_{uc}} \right) = \tan^{-1} \left(\frac{100.5}{171.5} \right) = 30.4^\circ$
 $\phi R_n = 1.392 \text{ k/in} (1 + 0.5 \sin^{1.5}(\phi))$
 $= 1.392 (1 + 0.5 \sin^{1.5}(30.4)) = 1.64 \text{ k/in}$
 $D = 12.6 / 2(1.64) = 3.84 \rightarrow \text{Use } 5/16'' \text{ For construction ease}$
 (meets minimum weld size)

Use (2) 5/16" Full length Fillet welds For gusset-end plate conn.

Gusset yielding $\phi R_n = \phi(0.6)F_y t L_w$ $\sqrt{147^2 + 150^2}$
 $= 0.9(0.6)(50)(0.75)(19.75) = 400k > 210k$ (OK)

Figure 68: ELFP Connection Design Page 5

THESIS REDESIGN

Project	Hunter's Point South Redesign	Sheet No.	8 of 10
Reference	AISC 327-05	Given By	EBF Design
Subject	Equivalent Lateral Force Procedure	Date	2/14/12
		Project No.	

Weld between beam-end plate $V_{ub} - V_{ub, beam} = 147 - 42 = 105 \text{ K}$

$$D \geq \frac{105 \text{ K}}{2(1.392)(15.125)} = 2.5 \text{ sixteenths}$$

minimum weld (5.24 = 1/4")

∴ use double sided 1/4" Fillet weld to connect the beam web to end plate

horizontal Force component: $H = 47.4 \text{ K}$

$$H = 1.25R_y V_n - H_{ub} = 392 - 150 = 242$$

$$H = H_{uc} = 109.5 \text{ K}$$

∴ horizontal strength of beam-column conn. = 242 K
 - Assume it's split between both beam flanges

$$R_{uf} = \frac{H_{uc}}{2} = \frac{109.5}{2} = 54.75 \text{ K}$$

$$D_{min} = \frac{121}{1.5(1.392)(11.1)} = 5.2 \rightarrow \text{use 7/16 Fillet (meets min weld)}$$

∴ use single-sided 7/16 Fillet weld to connect beam flange - end plate

Beam web rupture @ weld

$$\phi R_n = 0.75(0.6)(0.5)(0.535)(15.125) = 237 > 105 \text{ (OK)}$$

Beam Flange rupture @ weld

$$\phi R_n = 0.75(65)(0.87)(9.125) = 387 > 121 \text{ (OK)}$$

End plate Bolts | Try 8 rows of 2 7/8" A325N bolts @ 5 1/2" gage. use 4 bolts adjacent to beam flanges & use 4 more on each side of gusset plate

$$V_u = \frac{V_{uc} + V_{ub} - V_{ub, beam}}{N_b} = \frac{171.5 + 105}{16} = 17.18 \text{ k/bolt}$$

Table 7.1 = 7-2 ∴ Shear strength/bolt = 31.8
 Tensile strength/bolt = 53.0

Figure 69: ELFP Connection Design Page 6

THESIS REDESIGN

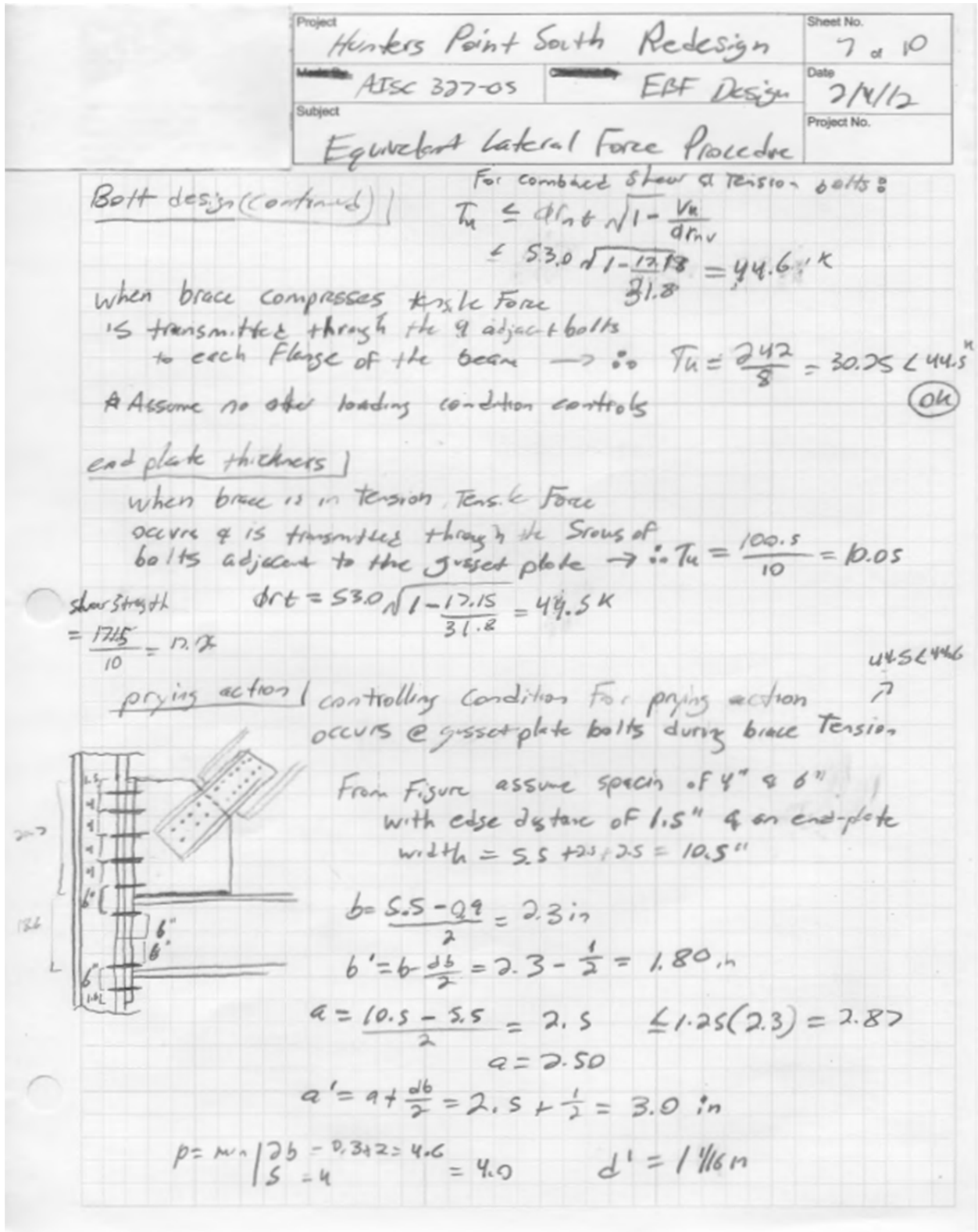


Figure 70: ELFP Connection Design Page 7

THESIS REDESIGN

Project	Hunters Point South Redesign	Sheet No.	8 of 10
Subject	AISC 327-05 EBF design	Date	2/4/12
Equivelant Lateral Force Procedure		Project No.	

Pring action (cont) $\delta = 1 - \frac{d'}{d} = 1 - \frac{11/16}{4.0} = 0.734$

$\rho = \frac{b'}{a'} = \frac{6.8}{3} = 0.6$

$\beta = \frac{1}{\rho} \left(\frac{d_{rn}}{r_{ut}} - 1 \right) = \frac{1}{0.6} \left(\frac{44.5}{10.05} - 1 \right) = 5.7 > 1 \therefore \alpha' = 1.0$

$t_{min} = \sqrt{\frac{4b' r_{ut}}{\phi_p F_u (1 + \delta \alpha')}} = \sqrt{\frac{4(1.8)(10.05)}{(0.9)(4)(65)(1 + 0.734(1.0))}} = 0.423 \text{ in}$

\therefore the 3/4" end plate thickness is acceptable

end-plate bearing strength

From table 7-4 $\phi R_n = (117 \text{ k/in}) (3/4 \text{ in}) = 87.8 \text{ k} > 17.18 \text{ k}$

From table 7-5 $\phi R_n = (42.0) (3/4) = 31.5 \text{ k/ps} > 17.18 \text{ k}$
 (Assume $L_c = 1/4 = \text{conservative!}$)

bearing strength of column flange

$t_{F \text{ column}} > 3/4 \text{ plate} \therefore$ want control = adequate

\therefore use (8) Rows of (2) 1" ϕ A325N bolts @ 5.5" gage.
 use (4) bolts adjacent to beam flange & 4 additional bolts on each side of the 3/4" gusset plate.

Shear yield of end plate

$\phi R_n = 0.9 (2) (0.6) (50) (3/4) = 40.5 \text{ k/in} > 12.6 \text{ k/in}$ (Gusset to beam)

end plate Fracture @ beam web welds

$\phi R_n = 0.75 (2) (0.6) (65) (3/4) (15.125) = 664 > 105 \text{ k}$ ok

end plate Fracture @ beam flange welds

$\phi R_n = 0.75 (0.6) (65) (3/4) (2.125) = 200 \text{ k} > 121 \text{ k}$ ok

Figure 71: ELFP Connection Design Page 8

THESIS REDESIGN

Project Hunters Point South Redesign	Sheet No. 9 of 10
Reference AISC 327-05	Date 2/4/12
Subject Equiv. Lateral Force Procedure	Project No.

end plate shear fracture @ bolt line $A_n = 2(0.75)(20.5 - 5(1 + 1/2))$
 $\phi R_n = 0.6 F_u A_n = 0.75(0.6)(65)(22.3) = 22.3 \text{ in}^2$
 $= 653 \text{ k}$
 $R_u = \sqrt{V_{uc}^2 + H_{uc}^2} = \sqrt{171^2 + 100.5^2} = 199 \text{ k} < 653 \text{ (ok)}$
 $\therefore 3/4" \times 10.5" \text{ end plate is adequate}$

Column web local yielding Adjacent to gusset plate:
 Spec J10.2 $\phi R_n = \phi (5k + l_a) F_y t_w$
 $= 1.0(5(1.5) + 19.5)(50)(0.55) = 743 \text{ k} > H_{uc} \text{ (ok)}$
 Adjacent to beam:
 $\phi R_n = 1.0[5(1.5) + 19.5](50)(0.55) = 730 \text{ k} > 121 \text{ k (ok)}$

column web crippling $l > d_c/2$
 adjacent to Gusset: $R_n = 0.8 t_w^2 \left[1 + 3 \frac{l}{d} \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_y t_w}{t_w}}$
 $= 0.8(0.55)^2 \left[1 + 3 \frac{12.5}{12.7} \left(\frac{0.55}{0.9} \right)^{1.5} \right] \sqrt{\frac{29000(50)(0.9)}{0.55}} = 1193$
 $\phi R_n = 1193 \times 0.75 > H_{uc} \text{ (ok)}$
 adjacent to Beam: $R_n = 0.8(0.55)^2 \left[1 + 3 \frac{0.87}{12.7} \left(\frac{0.55}{0.9} \right)^{1.5} \right] \sqrt{\frac{29000(50)(0.9)}{0.55}}$
 $= 409.4 \rightarrow \phi R_n = 409.4 \times 0.75 = 307 > 121 \text{ (ok)}$

column local Flange bending $t_f \geq t_{ep} \text{ plate} \rightarrow \text{End plate sufficient in Bending}$
 $\therefore \text{column Flange sufficient w/ } t = 0.9 \text{ in}$

check column shear $R_u = H_{uc} = 100.5 \text{ k}$ From column design $P_u = 984 \text{ k}$
 $\frac{P_u}{P_c} = \frac{984}{50 \text{ ksi}(282)} = 0.70 \rightarrow \text{J10.6: } \phi R_n = 0.9(0.9)(F_y)(d_c)(t_w) \left[1.4 \frac{A}{P_c} \right]$
 $(P_u > 0.1 P_c) = 0.9(0.9)(60)(12.7)(0.55)(1.4 \cdot 0.7)$
 $= 132 \text{ k} > 100.5 \text{ k (ok)}$

Figure 72: ELFP Connection Design Page 9

THESIS REDESIGN

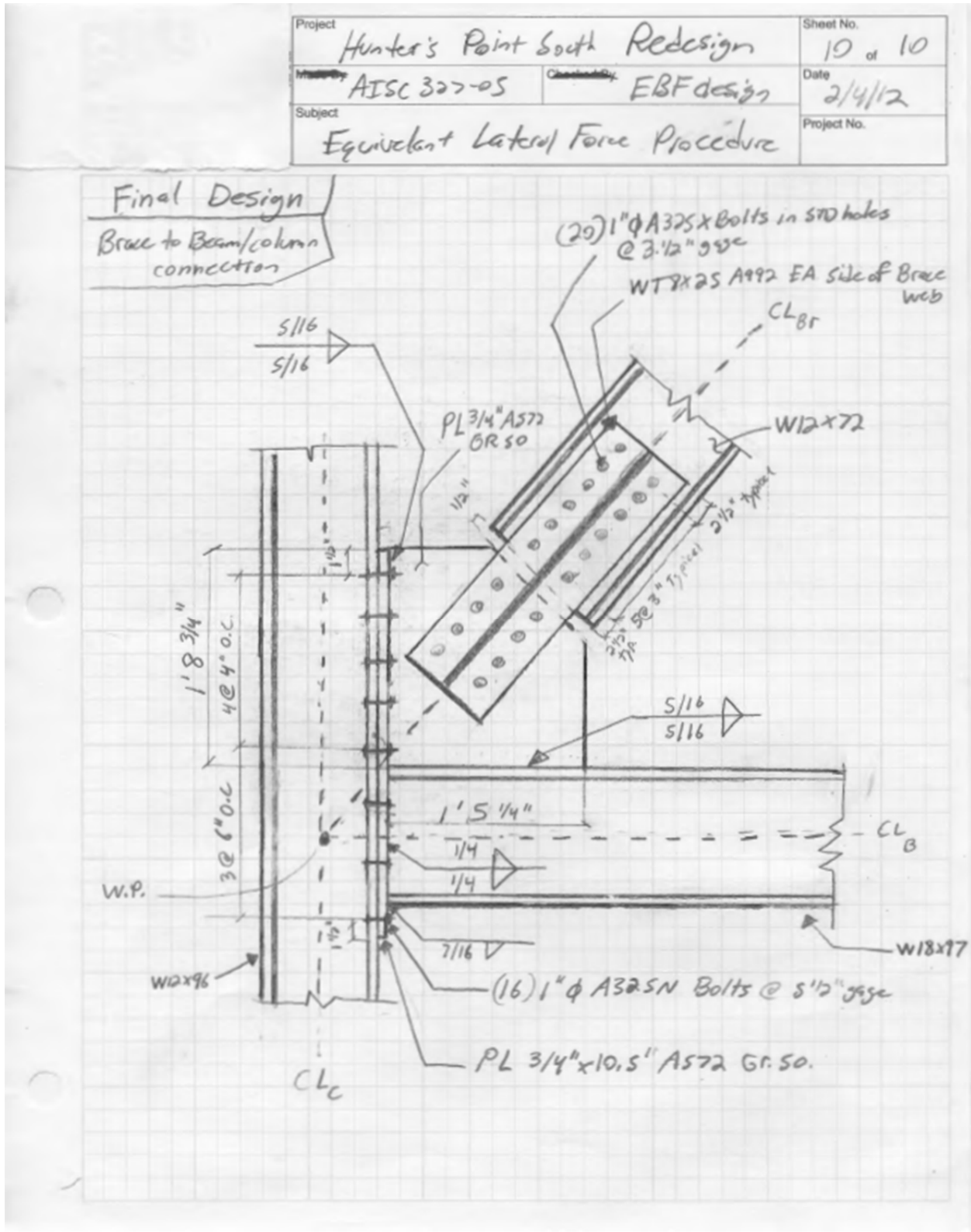


Figure 73: ELFP Connection Design Page 10

THESIS REDESIGN

BRACE TO COLUMN-BEAM CONNECTION												
Beam Section		= >	W18 X 97		A	d	tw	tf	T			
Unbraced length of the brace		$L_b =$	14	ft	28.50	18.60	0.54	0.87	15.13			
					bf	k	I	Z				
					11.10	1.27	1750.00	211.00				
Column Section		= >	W12 X 96		A	d	tw	tf	T			
Unbraced length of the brace		$L_b =$	11	ft	28.20	12.70	0.55	0.90	9.13			
					bf	k	I	Z				
					12.20	1.50	833.00	147.00				
Brace Section		= >	W12 X 72		A	d	tw	tf	T			
Unbraced length of the brace		$L_b =$	17.8	ft	21.10	12.30	0.43	0.67	9.13			
					bf	k	I	Z				
					12.00	1.27	597.00	108.00				
Factored Loads From Analysis (in kips)												
Brace	$P_u =$	118.00	$V_u =$	4.03	Overstrength Factor:	3.43	$P_u =$	404.7	$V_u =$	13.8		
Beam	$P_u =$	107.00	$V_u =$	11.40	Overstrength Factor:	3.66	$P_u =$	391.6	$V_u =$	41.7		
Drag Force	$P_u =$	23.70			Overstrength Factor:	2.00	$P_u =$	47.4				
BRACE TO GUSSET PLATE												
Gusset Plate Thickness:		$E_s =$	29000.0	ksi	Bolt:	1" d A325 X	db	dh	dh +1/16"	S		
		$F_y =$	50.0	ksi			1.00	1.06	1.13	3.00		
		$F_u =$	65.0	ksi			ϕ_{rn} shear	ϕ_{rn} Ten	gage	Edge Dis.		
		$t =$	0.75	in			80.10	53.00	3.50	2.50		
No. of Bolts:	$R_u / \phi_{rn} =$	5.05			Try	8.00	bolts					
					(2 Rows of	4.00	bolts)					
Plate Bearing:	ϕ_{rn}	113.0	X	0.75	=	84.8	$\phi_{rn} =$	128.9	+	508.5	=	637.4
Plate Tearout:	ϕ_{rn}	85.9	X	0.75	=	64.4						
Block Shear:	$F_u * A_{nt}$	115.8					$\phi_{rn} =$	86.8	+	356.5	=	443.3
	$0.6F_u * A_{nv}$	475.3										
	$0.6F_y * A_g$	517.5										
					THEREFORE:	8.00	1" d A325 X	for WT-Gusset connection				
Gusset-Brace Connection		Section: WT8 X 25			A	d	tw	tf	bf	\bar{x}	Q_s	
Tension Yield:	ϕ_{rn}	663.3			7.37	8.13	0.38	0.63	7.07	1.89	0.82	
Tension Rupture:	ϕ_{rn}	475.9										
Slenderness		Flange = $0.45 (E_s / F_y)^{0.5}$	10.8	>	$bf/2tf =$	5.6	** IF Slenderness is an issue, must reduce compressive strength due to local buckling					
		Web = $0.75 (E_s / F_y)^{0.5}$	18.1	>	$d/tw =$	21.4						
		ϕ_{cPn}	546.6		$bf/d =$	0.9	> 0.5?	**Flexural Torsional bulking need not be checked (AISC Table C-E4.2)				
					$tf/tw =$	1.7	> 1.0?					
Bearing/T.O: WT will not control--> OK												
Block Shear: WT will not control--> OK												
					THEREFORE: Use (2)	WT8 X 25	for connection					
Brace Web Bearing/T.O		ϕ_{rn}	48.6		Try	12.00	bolts					
		ϕ_{rn}	36.9		(2 Rows of	6.00	bolts)					
Block Shear:	$F_u * A_{nt}$	68.4			$\phi_{rn} =$	485.9	+	73.9	=	559.8		
	$0.6F_u * A_{nv}$	475.8			$\phi_{rn} =$	51.3	+	355.6	=	406.8		
	$0.6F_y * A_g$	474.1										
Shear Rupture	ϕ_{rn}	687.0										
					THEREFORE:	12.00	1" d A325 X	for WT-Brace connection				

Figure 74: Brace to Column-Beam Connection Design Spreadsheet

THESIS REDESIGN

Gusset-to-Beam/Column Connection			
eb	9.3	ec	6.4
α	9.5	β	10.9
	1.0	=?	1.0
No moment exists @ connection interface			
		Θ	38.2
		y	20.75
		x	17.25
		endplate t	0.75
		Clip	1.00
		E	29000.00
		Fy	50.00
		Fu	65.00
r=	25.7		
Vub	146.7	Hub	149.9
Vuc	171.6	Huc	100.2
Weld @ Gusset-Beam Connection			
Θ	44.4	lw	15.5
ϕ_{rn}	1.8	Dmin	4.7
→ Fv 9.7 Fa 9.5			
Fpeak 13.5 x1.25 16.9			
Use 2 5/16" Weld			
Gusset Yielding	ϕ_{Rn} = 313.9 >	Ru	209.7
Beam Web Local Yielding	ϕ_{Rn} = 499.6 >	Vub	146.7
*Force applied < db from the end			
Beam Web Crippling	ϕ_{Rn} = 775.5 >	Vub	146.7
*Force applied > db/2 from the end			
Weld Between Gusset & End Plate			
Θ	30.3	lw	19.8
ϕ_{rn}	1.6	Dmin	3.8
→ Fv 8.7 Fa 5.1			
Fpeak 10.1 x1.25 12.6			
Use 2 4/16" Weld			
Gusset Yielding	ϕ_{rn} 399.9 >	Ru	198.7
Weld Between Beam & End Plate			
Vub-Vubeam	105.0	Dmin	2.5
Use 2 4/16" Weld			
Horizontal Force Component	H=max		
	47.4	Ruf=	120.877
	241.8		
	100.2		
		Dmin	5.2
→ Use 2 6/16" Weld			
Beam Web Rupture @ Weld	ϕ_{Rn} = 236.7 >		105.0
Beam Flange Rupture @ weld	ϕ_{Rn} = 470.8 >		120.9
End Plate Bolts Design			
		db	1.00
		dh	1.06
		dh +1/16" S(beam)	1.13
		S(Gusset)	6.00
		Edge Dis.	1.50
		Width	4.00
	Try 8 rows of 2 1"d A325N bolts @ 5.50" gage	ϕ_{rn} shear	31.80
	Use 4 bolts adjacent to beam flanges	ϕ_{rn} Ten	53.00
	Use 4 bolts on each side of gusset plate	gage	5.50
		When brace compresses, tensile force occurs and is transmitted through the bolts adjacent to each flange of the beam	
Vu=	17.3 k/bolt	Shear strength/bolt=	31.80
		Tensile Strength/bolt=	53.00
→ For combined shear and Tension Bolts:			
		Tu=	30.2
		<	35.8
		When brace is in tension, tensile force occurs and is transmitted through the bolts adjacent to each flange of the beam	
Vu=	17.2 k/bolt	Shear strength/bolt=	31.80
		Tensile Strength/bolt=	53.00
→ For combined shear and Tension Bolts:			
		Tu=	10.0
		<	35.8
Prying Action			
Controlling condition for prying action occurs @ gusset plate bolts during brace tension			
b=	2.38	b'=	1.88
a=	2.5	a'=	3
p=	4	d'=	1.06
		δ =	0.73
		ρ =	0.63
		β =	4.12
		> 1	Therefore: α '= 1.0
tmin=	0.43	Endplate Thickness	0.75 in

Figure 74: Brace to Column-Beam Connection Design Spreadsheet

End Plate Bearing Strength										
Table (7-4)	Smallest spacing =	4	>	3.06	inches for full bearing for	1"d A325N				
Plate Bearing:	ϕR_n	117.0	x	0.75	=	87.8	>	17.2		
Plate Tearout:	ϕR_n	42.0	x	0.75	=	31.5	>	17.2		(assume $L_e=1.25$; conservative)
Column Flange Bearing:		0.90	>	0.75						Therefore won't control
	Try	8	rows of	2	1"d A325N	bolts @	5.50	"	gage	
	Use	4	bolts adjacent to beam flanges							
	Use	4	bolts on each side of	0.75	inch gusset plate					
<hr/>										
End Plate Shear Yield	ϕR_n =	40.5	>	12.6						
			>	16.9						
End Plate Fracture @ Beam Web Weld	ϕR_n =	663.6	>	105.0						
End Plate Fracture @ Beam Flange Weld	ϕR_n =	200.2	>	120.9						
Endplate Shear Fracture @ Bolt Line	R_u =	198.7								
	ϕR_n =	663.6	>	198.7						
<hr/>										
Column Check										
Column Web Local Yielding	$I > d$									
Adjacent to Gusset	ϕR_n =	749.4	>	100.2						
Adjacent to Beam	ϕR_n =	230.2	>	120.9						
Column Web Crippling	$I > d/2$									
Adjacent to Gusset	ϕR_n =	902.7	>	100.2						
Adjacent to Beam	ϕR_n =	307.0	>	120.9						
Column Local Flange Bending		0.9	>	0.75						Column Flange doesn't control over end plate, and t_f is sufficient
Column Shear Check	R_u =	100.2			P_u =	984	P_r/P_c =	0.70	>	0.4
Therefore	ϕR_n =	132.4	>	100.2						

Figure 74: Brace to Column-Beam Connection Design Spreadsheet

BRACE TO COLUMN-BEAM CONNECTION									
Beam Section	= >	W18 X 97	A	d	tw	tf	T		
Unbraced length of the brace	$L_b =$	14 ft	28.50	18.60	0.54	0.87	15.13		
			bf	k	I	Z			
			11.10	1.27	1750.00	211.00			
Column Section	= >	W12 X 96	A	d	tw	tf	T		
Unbraced length of the brace	$L_b =$	11 ft	28.20	12.70	0.55	0.90	9.13		
			bf	k	I	Z			
			12.20	1.50	833.00	147.00			
Brace Section	= >	W12 X 72	A	d	tw	tf	T		
Unbraced length of the brace	$L_b =$	17.8 ft	21.10	12.30	0.43	0.67	9.13		
			bf	k	I	Z			
			12.00	1.27	597.00	108.00			
Factored Loads From Analysis (in kips)									
Brace	$P_u =$	430.00	$V_u =$	15.00	$M_u =$	155.00	$E =$	29000.00	ksi
							$F_y =$	50.00	ksi
							$F_u =$	65.00	ksi
Brace Flange Force									
	$P_{fa} =$	215.0	k	= Force in each flange due to axial load					
	$P_{ff} =$	159.9	k	= Force in each flange due to moment (assume full load taken by flanges)					
	$P_f =$	374.9	k	= Maximum resultant force ($P_{fa} + P_{ff}$)					
Brace Web Force									
	$V_w = V_u =$	15.0	k	(Assumed entire shear force taken by web)					
Design Brace Flange Connection									
			**DETAIL CONNECTION AS FIXED						
			**TRY FULLY WELDED CONNECTION						
			USE COMPLETE-JOINT-PENETRATION GROOVE WELD FOR BRACE FLANGE-TO-BEAM CONNECTION						
Yield Strength	$\phi R_n =$	402.0	>	374.9					
Check Concentrated Forces at Brace Flange Connection									
	$V_f =$	294.8							
Local Yield Strength of Beam Web @ Brace Flange Co	$\phi R_n =$	187.8	>	294.8	Beam Web Stiffeners are Required Adjacent to the Brace Flanges				
Beam Web Crippling Strength	$\phi R_n =$	277.5	>	294.8					

Figure 75: Brace to Link Connection Design Spreadsheet

THESIS REDESIGN

Size Beam Web Stiffners	
Use Stiffner on each side of the beam web	
$P_s =$	53.5
$b =$	5.28 =Max width of each stiffner
Try Stiffner Width of	5 1/4 " with 2 1.0 " Corner Clips
	> 5 1/4 From Link Design
$t_{min} =$	0.28 > 0.50 From Link Design
	<Design of Link required a larger t_{min} , therefore it will be used for design of the connection>
	<Connection design requires a larger t_{min} than the link design called for from the Seismic Provision>
Therefore, use a	1/2 x 5 1/4 Full Depth Stiffners on each side of the beam where the bracing flanges intersect the beam flanges

Design Stiffner Welds	
Minimum Double Sided Fillet Weld Size Required to Transfer the Stiffner Load From Flanges to Stiffner	
$D_{min} =$	3.0 sixteenths → Use 4 sixteenths Minimum 5 sixteenths
Length of Stiffner Adjacent to Beam Web	
$L =$	14.9 "
Minimum Single-Sided Fillet Weld Size Required to Transfer the Stiffner Load to the Web	
$D_{min} =$	2.6 sixteenths → Use 3 sixteenths Minimum 4 sixteenths
Therefore, Use	5 /16" Double-Sided Fillet Weld to Connect Stiffner to Beam Flanges
and Use	4 /16" Single-Sided Fillet Weld to Connect Stiffner to Beam Web

Design Brace Web Connection	
$\phi R_u =$	15.0 k
$E =$	29000 ksi
$F_y =$	36.00 ksi
$F_u =$	50.00 ksi
$t_{min} =$	0.3 → Try 0.5
$D_{min} =$	5 sixteenths from code minimum (by inspection)
Therefore, Use	1/2 x 4 x 0'6" Single Plate Connection with 5 /16" Fillet Weld to Connect Plate to Beam and Brace
	**BY Inspection this Connection is more than adequate to carry the load of 15.0 k

Figure 75: Brace to Link Connection Design Spreadsheet

APPENDIX D

BREADTH ANALYSIS

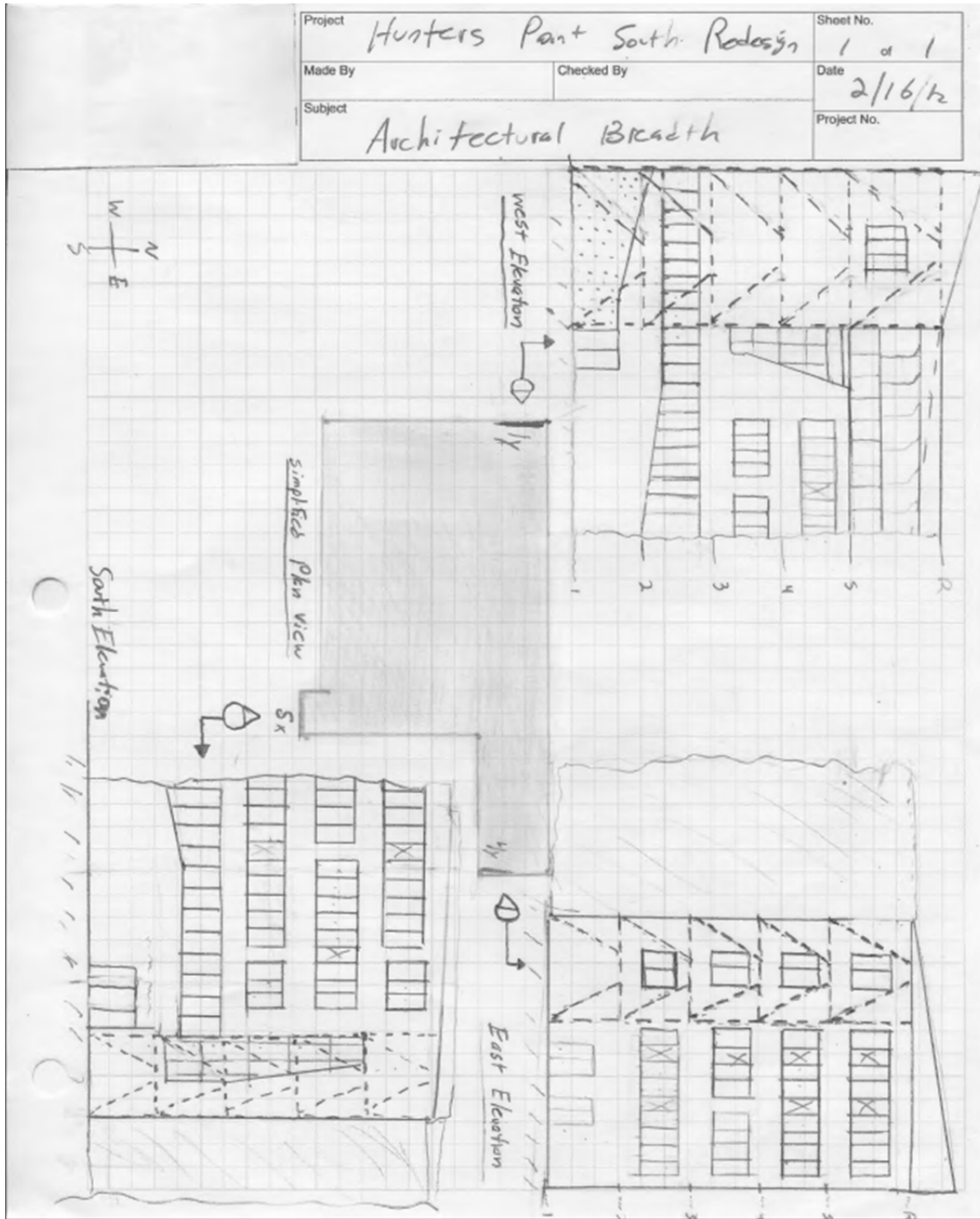


Figure 76: Architectural Breadth Sketch

APPENDIX E

WIND ANALYSIS (ORIGINAL DESIGN)

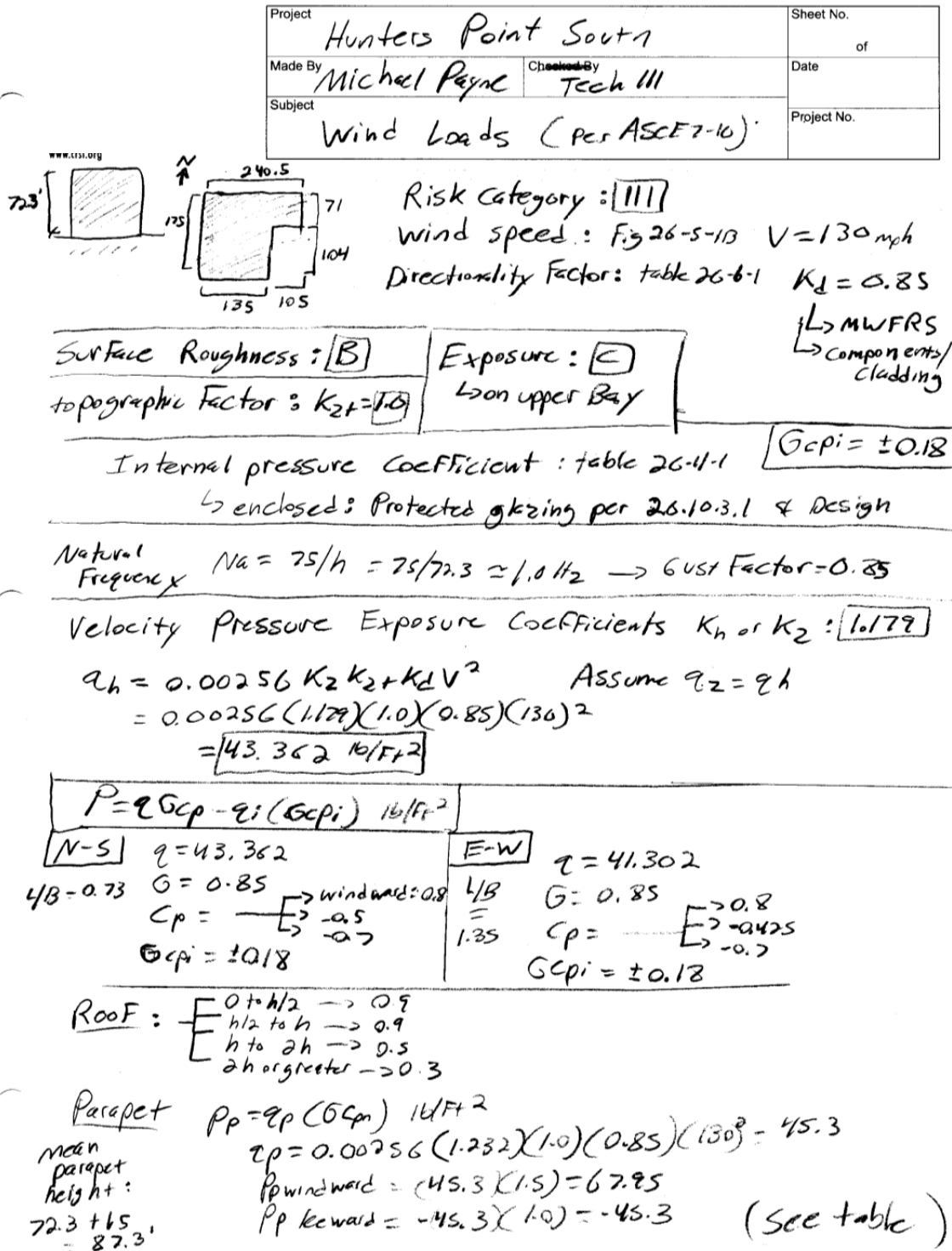


Figure 77: Wind Load Hand Calculations

Windload Design Criteria		
Per ASCE7-10	N-S	E-W
Risk Category	III	
Importance Factor	1	
Exposure	C	
Surface Roughness	B	
V	130	
K_d	0.85	
K_{zt}	1	
n_a	1.03	
G	0.85	
K_h	1.19	
h	72.3	
L	175	240.5
B	240.5	175
L/B	0.728	1.374
h/l	0.413	0.301
C_p Windward	0.8	
C_p Leeward	-0.5	-0.425
C_p Side	-0.7	
C_p Roof	0 to h/2	-0.9
	h/2 to h	-0.9
	h to 2h	-0.5
	>2h	-0.3
Reduction Factor	0.8	
GC_{pi}	+/-0.18	
K_h	1.179	
q_z	43.36	
q_p	45.30	
GC_{pn} Windward	1.5	
GC_{pn} Leeward	-1	

Velocity Pressure			
Level	Height	K_z	q_z
Parapet	87.3	1.232	45.30
Roof	72.3	1.179	43.36
5	56	1.114	40.97
4	42	1.050	38.61
3	28	0.964	35.45
2	14	0.850	31.26
1	0	0.850	31.26

Notes:

- Due to its location on the Bay, NYC Building Code requires this structure to be Risk Category Iull and Exposure C.
- Using the velocity maps in ASCE7-10, a design wind velocity of 130mph is used.
- Due to its location near the shore, the original design calls for protected glazing on the entire building. Therefore, the building is assumed to be enclosed and a P_{ip} of +/-0.18 is chosen for calculations.
- Using AISC7-10 design guide, the other factors are chosen and plugged into the story pressure equation.

Table 26: Wind Pressure: North-South Direction

Wind Pressure: North-South Direction						
Story Level	Floor to Floor Height (ft)	Story Height (ft)	Wind Pressure (psf)	Internal Pressure (psf)	Net Pressure -GCpi (psf)	Net Pressure +GCpi (psf)
Roof	15	72.3	29.488	+/- 7.80	21.68	37.29
5	16.3	56	27.857	+/- 7.80	20.05	35.66
4	14	42	26.257	+/- 7.80	18.45	34.06
3	14	28	24.106	+/- 7.80	16.30	31.91
2	14	14	21.256	+/- 7.80	13.45	29.06
1	14	0	21.256	+/- 7.80	13.45	29.06
Parapet	Windward	87.3	67.954	-	-	-
	Leeward	87.3	-45.302	-	-	-
Leeward	-	-	-18.430	+/- 7.80	-26.23	-10.62
Roof	0 to 36.15ft	-	-33.174	+/- 7.80	-40.97	-25.36
	36.15-72.3ft	-	-33.174	+/- 7.80	-40.97	-25.36
	72.3-144.6ft	-	-18.430	+/- 7.80	-26.23	-10.62
	144.6-175ft	-	-11.058	+/- 7.80	-18.86	-3.25

Table 27: Wind Loads: North-South Direction

Wind Loads: North-South Direction							
Story Level	Floor to Floor Height (ft)	Story Height (ft)	Windward (kip)	Leeward (kip)	Total Story Force (kip)	Total Story Shear (kip)	Overturning Moment (ft-k)
Parapet	15	87.3	122.6	-81.7	204.3	1322.3	16302.0
Roof	16.3	72.3	135.9	-95.6	231.5	1118.0	16735.4
5	14	56	120.1	-88.3	208.4	886.5	11671.1
4	14	42	114.7	-88.3	203.0	678.1	8527.0
3	14	28	107.4	-88.3	195.8	475.1	5481.9
2	14	14	97.8	-88.3	186.2	279.3	2606.6
1	14	0	48.9	-44.2	93.1	93.1	0.0
Σ						1322.3	61323.9

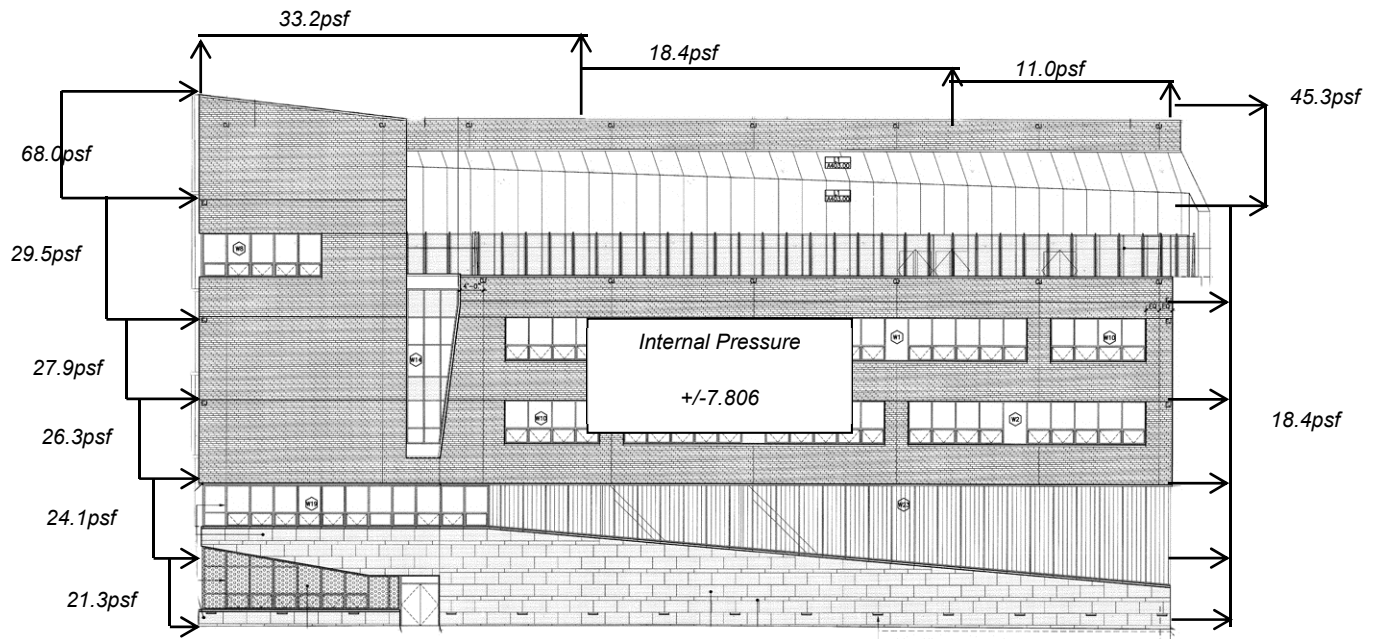


Figure 78: Wind Pressures, N-S Direction Drawing Adapted from FXFowle Architects

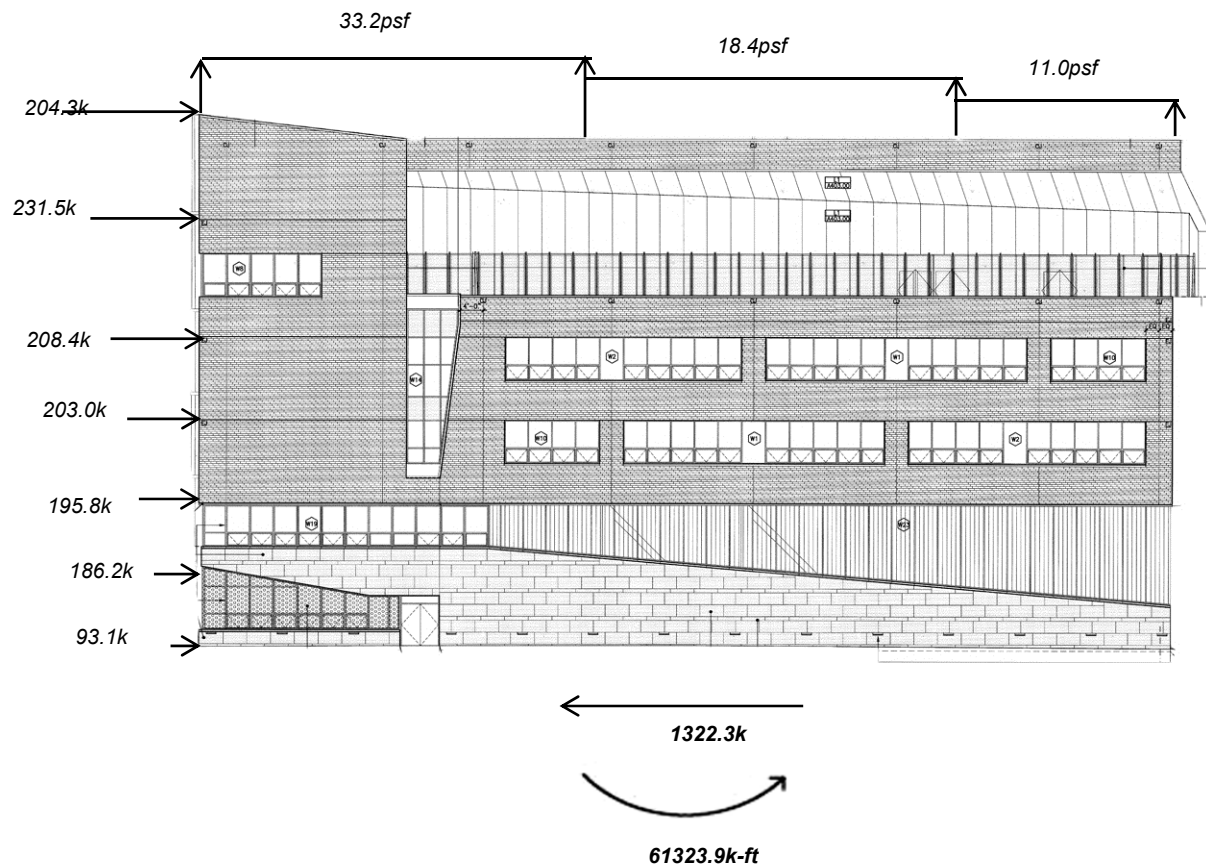


Figure 79: Wind Forces, N-S Direction
 Drawing Adapted from FXFowle Architects

Table 28: Wind Pressure: East-West Direction

Wind Pressure: East-West Direction						
Story Level	Floor to Floor Height (ft)	Story Height (ft)	Wind Pressure (psf)	Internal Pressure (psf)	Net Pressure -GCpi (psf)	Net Pressure +GCpi (psf)
Roof	15	72.3	29.488	+/- 7.806	21.682	37.293
5	16.3	56	27.857	+/- 7.80	20.05	35.66
4	14	42	26.257	+/- 7.806	18.451	34.063
3	14	28	24.106	+/- 7.80	16.30	31.91
2	14	14	21.256	+/- 7.806	13.450	29.061
1	14	0	21.256	+/- 7.80	13.45	29.06
Parapet	Windward	87.3	67.954	-	-	-
	Leeward	87.3	-45.302	-	-	-
Leeward	-	-	-15.665	+/- 7.807	-23.471	-7.860
Roof	0 to 36.15ft	-	-33.174	+/- 7.80	-40.97	-25.36
	36.15-72.3ft	-	-33.174	+/- 7.807	-40.979	-25.368
	72.3-144.6ft	-	-18.430	+/- 7.80	-26.23	-10.62
	144.6-240.5ft	-	-11.058	+/- 7.807	-18.864	-3.252

Table 29: Wind Loads: East-West Direction

Wind Loads: East-West Direction							
Story Level	Floor to Floor Height (ft)	Story Height (ft)	Windward (kip)	Leeward (kip)	Total Story Force (kip)	Total Story Shear (kip)	Overtopping Moment (ft-k)
Parapet	15	87.3	89.2	-59.5	148.6	924.3	12977.0
Roof	16.3	72.3	98.9	-62.2	161.1	775.7	11647.6
5	14	56	87.4	-57.5	144.9	614.6	8113.2
4	14	42	83.5	-57.5	141.0	469.7	5920.2
3	14	28	78.2	-57.5	135.7	328.7	3799.3
2	14	14	71.2	-57.5	128.7	193.1	1801.9
1	14	0	35.6	-28.8	64.4	64.4	0.0
Σ						924.3	44259.1

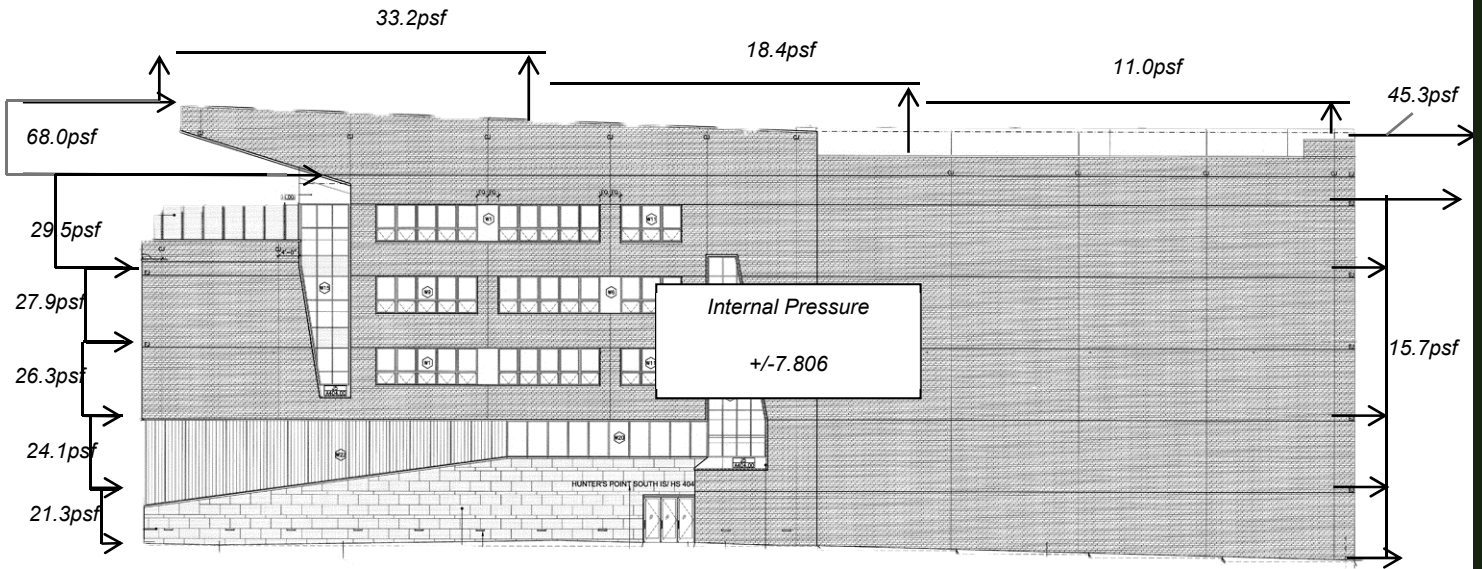


Figure 80: Wind Pressures, E-W Direction
 Drawing Adapted from FXFowle Architects

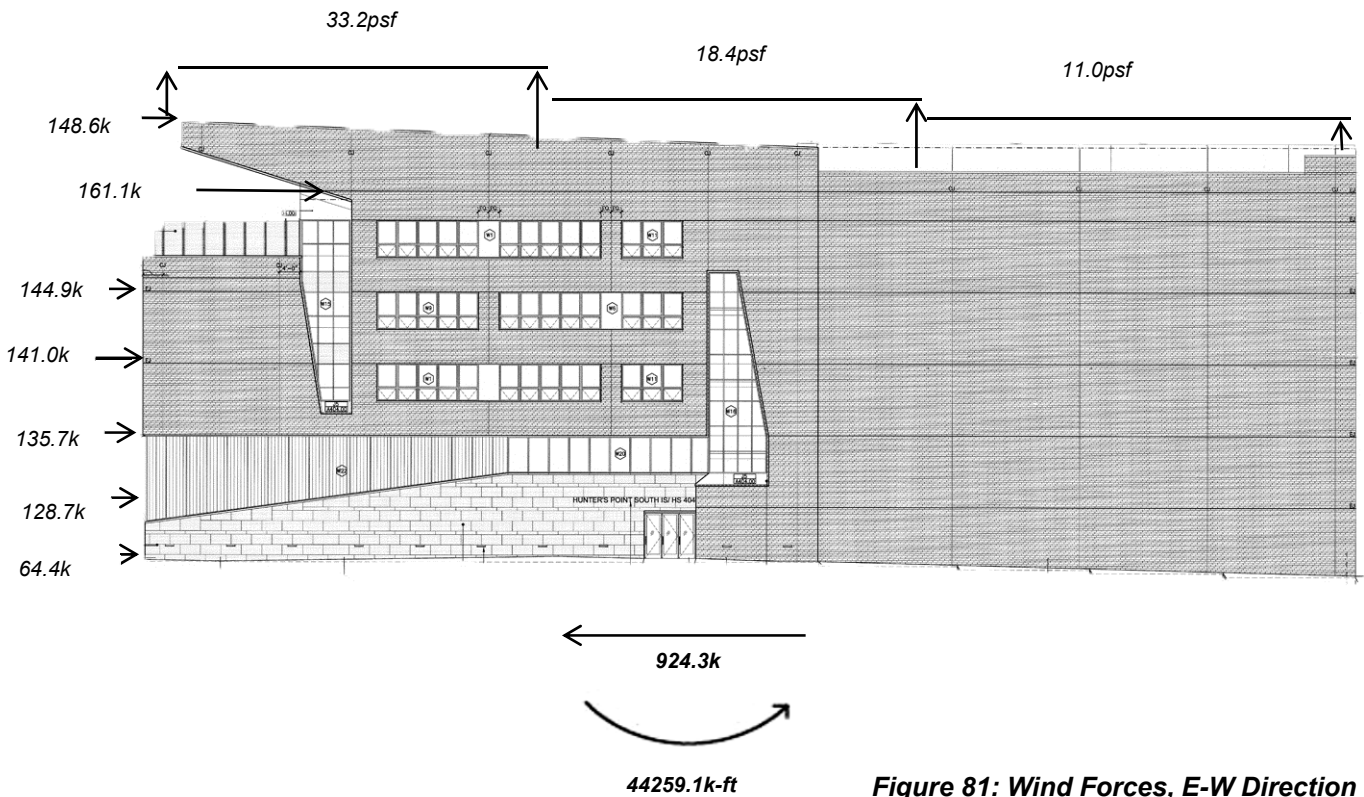


Figure 81: Wind Forces, E-W Direction
 Drawing Adapted from FXFowle Architects

APPENDIX F

SEISMIC ANALYSIS (ORIGINAL DESIGN)

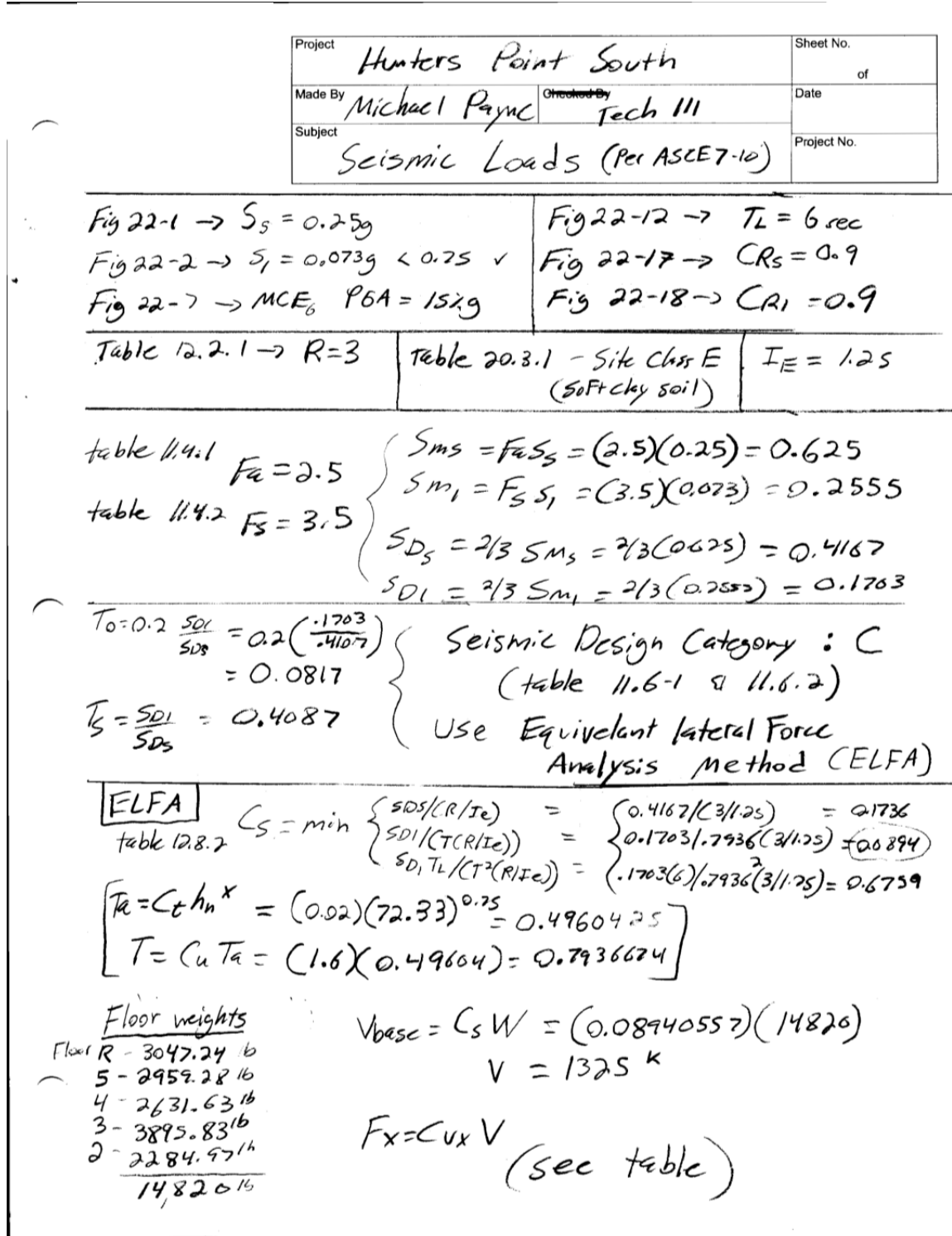


Figure 82: Seismic Load Hand Calculations

THESIS REDESIGN

Roof	weight/ft	length	weight	weight/ft	length	weight	Area	DL	LL	SL	Tot	weight		
Column				Beam										
							Floor							
10 X	49	17	833	24 X 76	24	1824	31.1 X	232.45	7229.195	85	45	22	85	614481.6
10 X	54	17	918	24 X 76	24	1824	39.25 X	198.45	7789.163	85				662078.8
12 X	96	17	1632	24 X 68	21.3	1448.4	101.75 X	104.66	10649.16	85				905178.2
10 X	54	17	918	24 X 68	23.08333	1569.667	TOTAL							2181739
10 X	54	17	918	24 X 68	24.39583	1658.917								2181.739
12 X	96	17	1632	24 X 68	19.10417	1299.083								
10 X	68	14	952	24 X 68	26.3125	1789.25								
10 X	54	14	756	24 X 68	26	1768	PERIMETER							
10 X	54	14	756	24 X 68	22	1496	19 X	592	11248	20				224960
10 X	54	17	918	30 X 99	30.58333	3027.75	11 X	172	1892	20				37840
12 X	53	17	901	14 X 22	12	264	X	0						262800
12 X	79	7	553	12 X 26	12	312								262.8
10 X	54	17	918	12 X 26	10.65	276.9								
12 X	40	17	680	14 X 22	10.19444	224.2778								
12 X	79	7	553	14 X 22	12	264								
12 X	79	7	553	12 X 26	12	312								
12 X	79	7	553	12 X 26	10.65	276.9	TOTAL	2944.57						
10 X	33	7	231	14 X 22	10.19444	224.2778								
10 X	33	7	231	12 X 26	11.54165	300.0829								
12 X	40	7	280	12 X 26	8.133333	211.4667								
12 X	40	7	280	14 X 22	11.72917	258.0417								
10 X	33	7	231	24 X 76	24	1824								
12 X	50	17	850	21 X 101	24	2424								
10 X	33	7	231	14 X 233	21.3	4962.9								
10 X	33	7	231	16 X 36	23.08333	831								
10 X	33	7	231	16 X 36	24.39583	878.25								
10 X	33	7	231	16 X 36	19.10417	687.75								
12 X	79	7	553	21 X 50	26.3125	1315.625								
10 X	33	7	231	21 X 50	26	1300								
12 X	50	7	350	21 X 50	22	1100								
12 X	79	7	553	24 X 62	30.58333	1896.167								
12 X	79	7	553	4 X 13	8	104								
12 X	79	7	553	4 X 13	8.5	110.5								
12 X	79	7	553	4 X 13	9	117								
14 X	53	15	795	4 X 13	10	130								
10 X	33	7	231	4 X 13	10.5	136.5								
12 X	40	7	280	4 X 13	11	143								
12 X	79	7	553	4 X 13	12	156								
10 X	33	7	231	4 X 13	12.5	162.5								
12 X	40	7	280	4 X 13	13	169								
12 X	79	7	553	4 X 13	14	182								
12 X	79	7	553	4 X 13	14.5	188.5								
12 X	79	7	553	4 X 13	15	195								
10 X	33	7	231	4 X 13	16	208								
14 X	61	7	427	4 X 13	16.5	214.5								
14 X	74	7	518	4 X 13	17	221								
HSS			7	0	4 X 13	18	234							
HSS			7	0	4 X 13	18.5	240.5							
14 X	109	14.25	1553.25	4 X 13	19	247								
14 X	193	13.5	2605.5	4 X 13	20	260								
14 X	233	12.75	2970.75	4 X 13	20.5	266.5								
14 X	283	12	3396	4 X 13	21	273								
14 X	342	11.25	3847.5	4 X 13	22	286								
14 X	342	10.75	3676.5	4 X 13	22.5	292.5								
10 X	49	7	343	4 X 13	24	312								
10 X	33	7	231	12 X 55	20	1100								
10 X	49	7	343	12 X 35	23.5	822.5								
10 X	33	14	462	12 X 35	23.5	822.5								
10 X	33	14	462	12 X 35	23.5	822.5								
10 X	33	14	462	12 X 35	20	700								
TOTAL			46874.5	12 X 35	23.5	822.5								
			46.8745	12 X 35	23.5	822.5								
				12 X 35	22.75	796.25								
				12 X 35	22.75	796.25								
				12 X 35	22.75	796.25								
				12 X 35	22.75	796.25								
				12 X 35	23.5	822.5								
				12 X 35	23.5	822.5								
				12 X 35	23.5	822.5								
				12 X 35	23.5	822.5								
				21	101	20	2020							
				21 X 44	23.5	1034								
				21 X 44	23.5	1034								
				21 X 44	23.5	1034								
				21 X 44	23.5	1034								
				21 X 44	23.5	1034								
				12 X 35	23.5	822.5								
				21 X 44	22.75	1001								
				18 X 76	22.75	1729								
				21 X 73	20	1460								
				14 X 22	30	660								
				14 X 53	30	1590								
				14 X 22	30	660								
				14 X 82	30	2460								
				16 X 31	30	930								
				14 X 90	30	2700								
				16 X 40	30	1200								
				14 X 109	28	3052								
				14 X 22	26	572								
				14 X 90	24	2160								
				14 X 22	20	440								
				14 X 82	15	1230								

Figure 83: Part of Story Weight Calculations using Microsoft Excel

THESIS REDESIGN

21 X 57	40	2280
24 X 68	10	680
24 X 62	22	1364
30 X 99	30.58333	3027.75
16 X 40	11	440
16 X 40	25	1000
24 X 55	23	1265
16 X 40	18	720
30 X 99	46	4554
30 X 99	45	4455
30 X 99	44	4356
16 X 36	21	756
16 X 36	20	720
16 X 36	20	720
16 X 36	30	1080
16 X 31	30	930
16 X 31	30	930
16 X 31	32	992
16 X 31	32	992
16 X 36	32	1152
16 X 36	32	1152
16 X 36	32	1152
21 X 50	40	2000
21 X 50	40	2000
14 X 22	20	440
14 X 22	20	440
14 X 22	20	440
14 X 22	20	440
24 X 68	20	1360
24 X 68	20	1360
24 X 68	23.5	1598
24 X 68	23.5	1598
24 X 117	45	5265
24 X 162	45	7290
24 X 117	45	5265
24 X 162	45	7290
24 X 117	42	4914
24 X 117	40	4680
14 X 22	20	440
21 X 50	23.5	1175
14 X 22	23.5	517
14 X 22	23.5	517
14 X 22	23.5	517
14 X 22	23.5	517
14 X 22	23.5	517
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
16 X 36	11.5	414
24 X 62	27	1674
12 X 50	35	1750
24 X 62	35	2170
21 X 57	35	1995
24 X 62	35	2170
24 X 55	25	1375
24 X 55	20	1100
24 X 55	28	1540
24 X 55	12	660
24 X 55	22	1210
24 X 55	35	1925
24 X 68	25	1700
14 X 22	12	264
14 X 68	12	816
24 X 68	15	1020
24 X 68	20	1360
24 X 76	20	1520
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190

	Weight
Misc	
AHU1	37200
AHU2	39600
AHU3	39600
AHU4	34900
AHU5	21400
AHU6	20700
	193400
	193.4

12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
12 X 19	10	190
30 X 99	31	3069
16 X 40	31	1240
16 X 40	31	1240
16 X 40	31	1240
16 X 31	31	961
16 X 31	31	961
16 X 40	31	1240
16 X 40	31	1240
16 X 40	31	1240
16 X 40	31	1240
16 X 40	31	1240
16 X 40	31	1240
16 X 40	31	1240
16 X 40	31	1240
16 X 26	31	806
16 X 26	31	806
16 X 26	31	806
16 X 26	31	806
16 X 31	31	961
16 X 31	31	961
16 X 31	31	961
16 X 31	31	961
30 X 99	31	3069
40 X 167	40	6680
18 X 35	28	980
21 X 50	40	2000
21 X 50	40	2000
21 X 50	40	2000
21 X 50	40	2000
21 X 50	40	2000
21 X 57	40	2280
21 X 57	40	2280
21 X 57	40	2280
21 X 57	40	2280
18 X 22	12	264
14 X 22	10	220
14 X 22	10	220
14 X 22	10	220
14 X 22	10	220
16 X 40	15	600
16 X 40	15	600
X		0
X		0
X		0
X		0
X		0
X		0
X		0
X		0
X		0
X		0
X		0
TOTAL		259757.2
		259.7572

Figure 84: Part of Story Weight Calculations using Microsoft Excel

THESIS REDESIGN

Project		Sheet No.	
		of	
Made By	Checked By	Date	
Subject 3rd Floor Beams		Project No.	

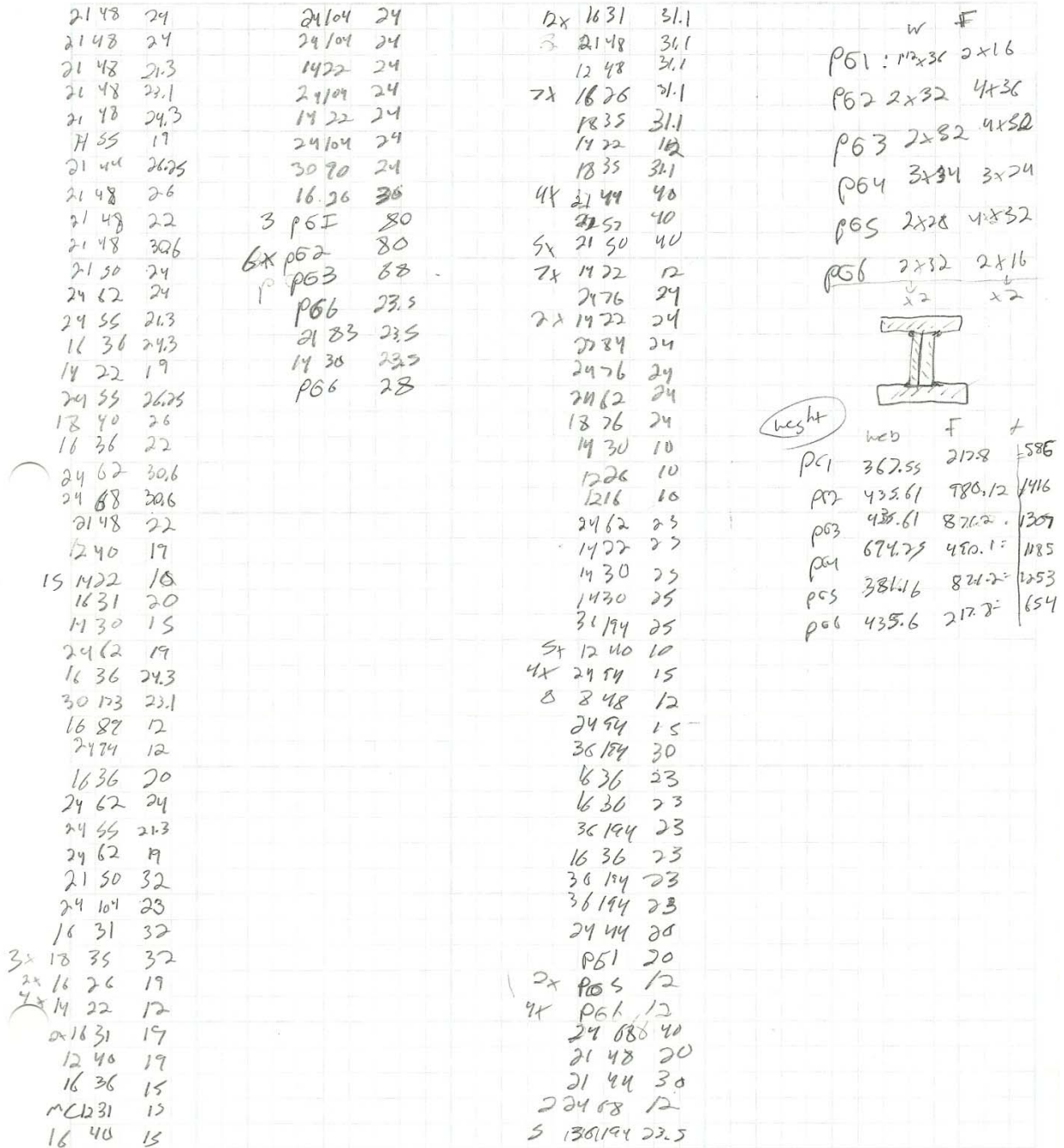


Figure 85: Part of Story Weight Hand Calculations

Table 30: North-South Direction Loading

North-South Direction Loading											
										T= 0.882 s k= 1.191 V _b = 1067 kips	
i	h _i	h	w	w*h ^k	C _{VX}	f _i	v _i	B _x	5%B _y	A _x	M _z
	ft	ft	kips			kips	kips	ft	ft		k-ft
6	16.33	72.33	2945	482573	0.396	423	423	131	7	1	2766
5	14	56	2563	309691	0.254	271	694	131	7	1	1775
4	14	42	2277	195314	0.160	171	865	131	7	1	1120
3	14	28	3500	185228	0.152	162	1027	131	7	1	1062
2	14	14	1978	45848	0.038	40	1067	131	7	1	263
1											
			Σ	13263	1218654		1067 =V				6986

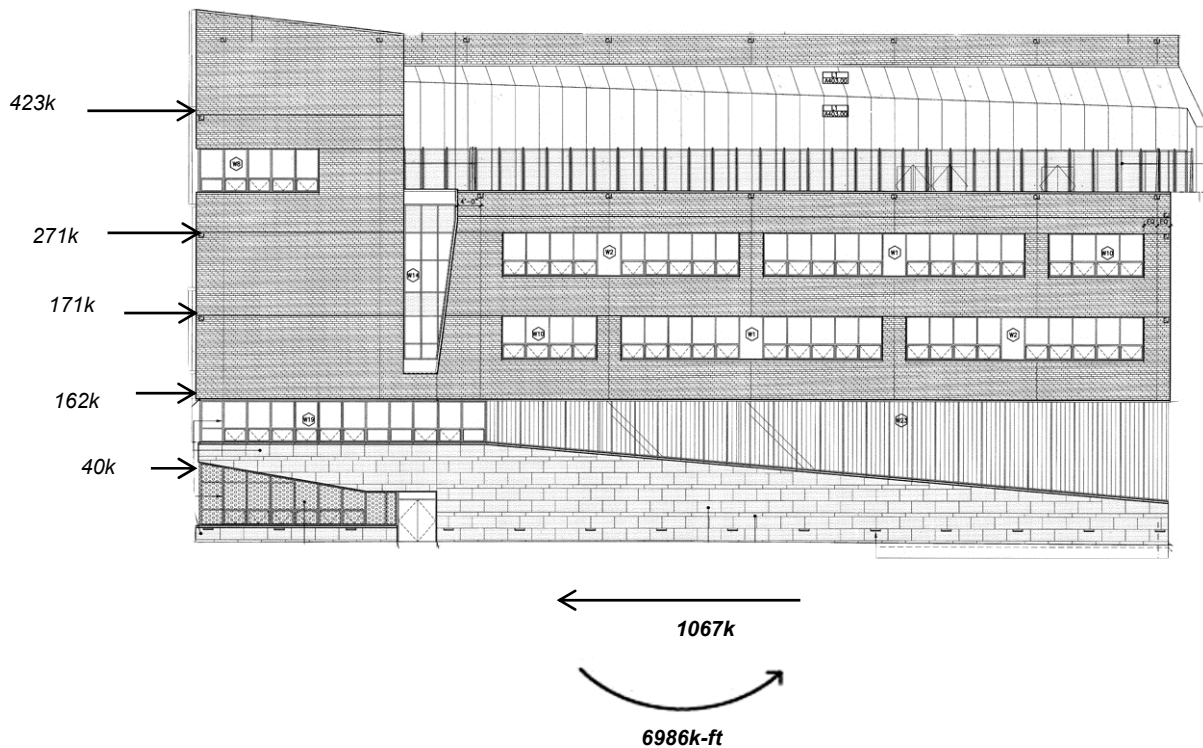


Figure 86: Seismic Forces, N-S Direction
 Drawing Adapted from FXFowle Architects

Table 31: East-West Direction Loading

East-West Direction Loading											
										T= 0.882 s k= 1.191 V _b = 1067 kips	
i	h _i	h	w	w*h ^k	C _{VX}	f _i	v _i	B _y	5%B _y	A _x	M _z
	ft	ft	kips			kips	kips	ft	ft		k-ft
6	16.33	72.33	2945	482573	0.396	423	423	178	9	1	3759
5	14	56	2563	309691	0.254	271	694	178	9	1	2412
4	14	42	2277	195314	0.160	171	865	178	9	1	1521
3	14	28	3500	185228	0.152	162	1027	178	9	1	1443
2	14	14	1978	45848	0.038	40	1067	178	9	1	357
1											
			Σ	13263	1218654		1067 =V				9491

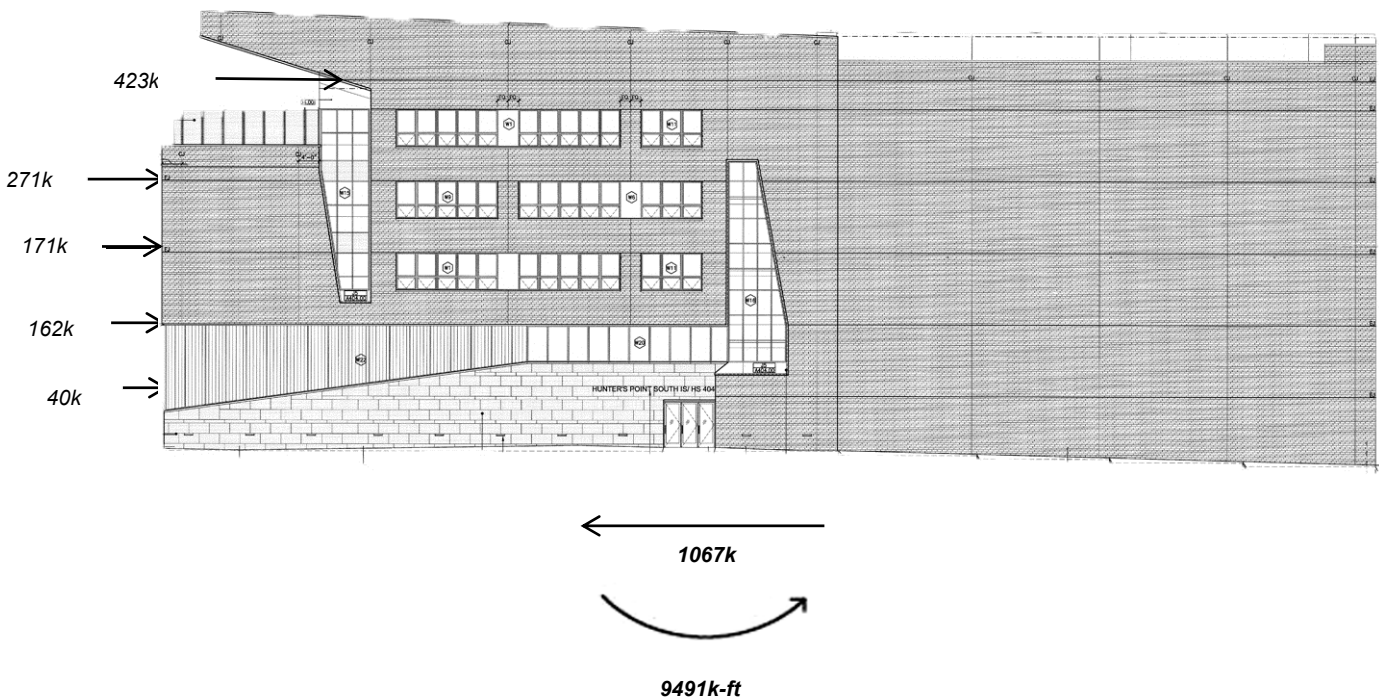


Figure 87: Seismic Forces, E-W Direction
 Drawing Adapted from FXFowle Architects

Fixed Base Assumption:

North-South Trusses

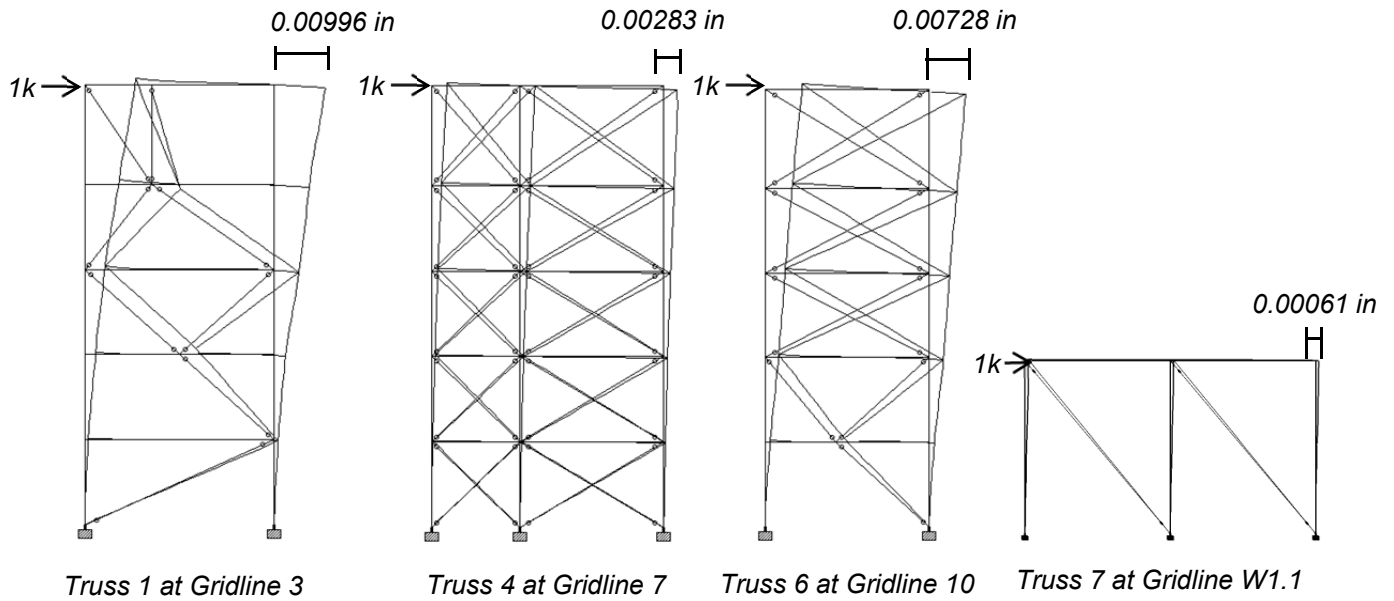


Figure 88: P & Δ: North-South Frames (fix)

**Fixed Base Assumption:
East-West Trusses**

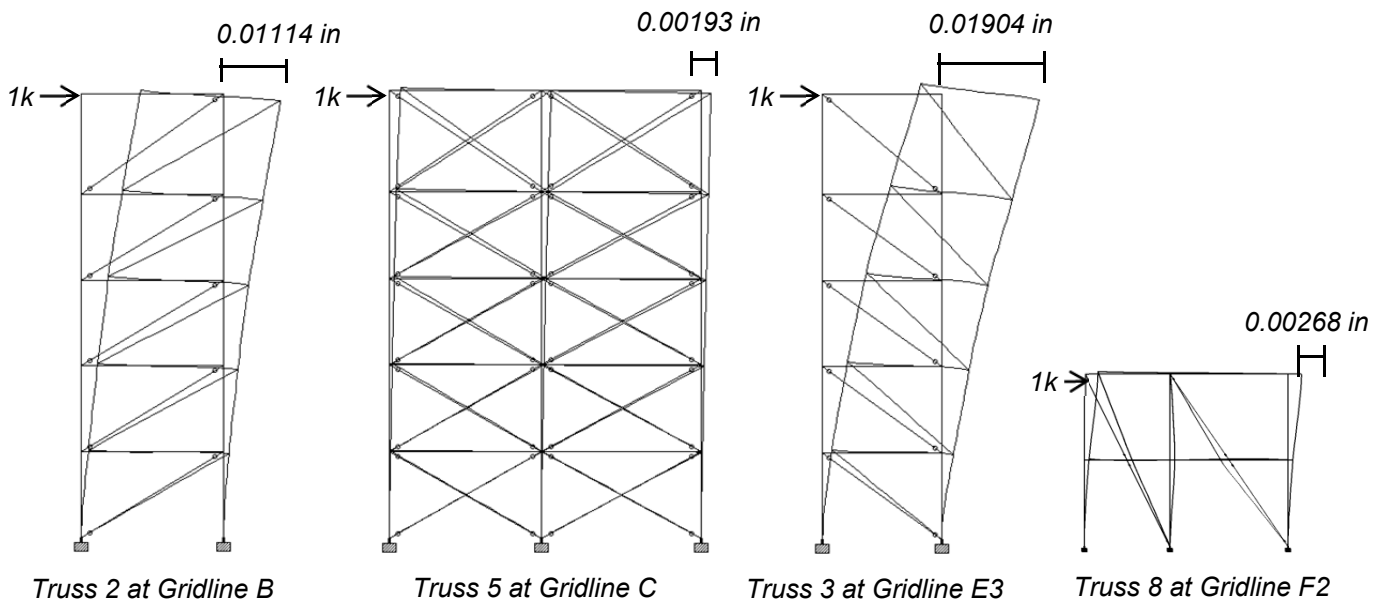


Figure 89: P & Δ: East-South Frames (fix)

LOAD COMBINATIONS

The following are the 7 basic load combinations prescribed by ASCE7-10 Chapter 2.3 for use in “combining factored loads using strength design”:

- 1.) $1.4D$
- 2.) $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
- 3.) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
- 4.) $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
- 5.) **$1.2D + 1.0E + L + 0.2S$**
- 6.) $0.9D + 1.0W$
- 7.) $0.9D + 1.0E$

(D=Dead, L=Live, L_r =Roof Live, S=Snow, R=Rain, W=Wind, E=Earthquake)

Figure 90: Load Combinations

APPENDIX G

MISCELANEOUS CHECKS FOR ORIGINAL DESIGN

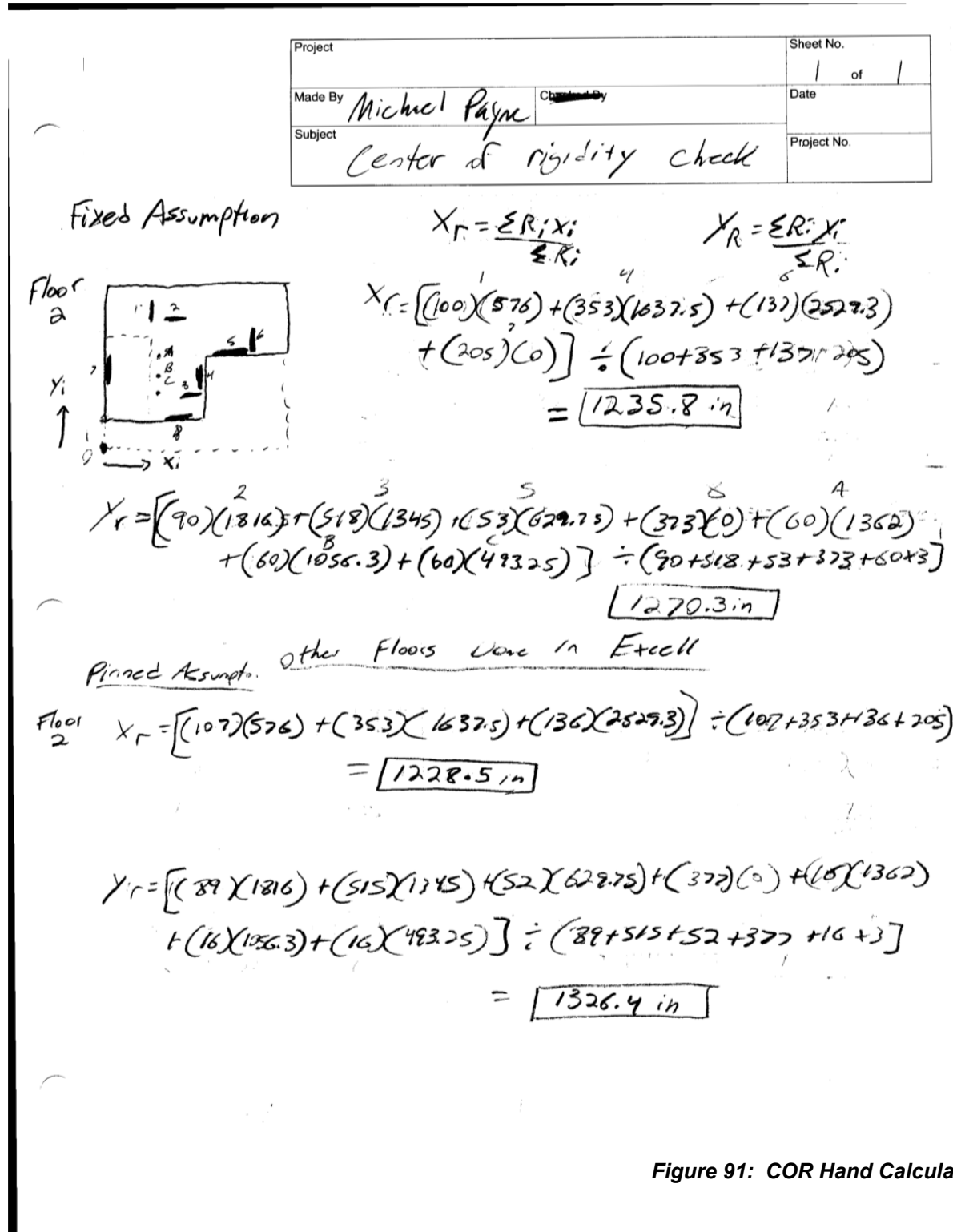


Figure 91: COR Hand Calculations

THESIS REDESIGN

LATERAL MEMBER SPOT CHECK (ORIGINAL DESIGN)

Project	Hunters Point South	Sheet No.	1 of 2
Made By	Michael Payne	Checked By	Tech Report III
Subject	Lateral Force Member Spot check	Date	
		Project No.	

★ Loading in X-Direction

1.) column member @ Truss 2 : Floor 1



W12x90 column
 $P_u = 216.2 \text{ k}$
 $M_u = 5353.7 \text{ k-in}$
 $= 446.2 \text{ k-ft}$

Use table 6-1, AISC:
 Combined Axial & Bending
 $KL = (14) \frac{r_x}{r_y} = (14)(1.78) = 25 \text{ Ft}$
 $P \times 10^{-3} = 0.742$ $b_1 \times 10^{-3} = 0.822$

$$\frac{216.2(0.742)}{1000} + \frac{446.2(0.822)}{1000} = 0.53 < 1.0 \quad \text{OK}$$

2.) column member @ Truss 2 : Floor 4

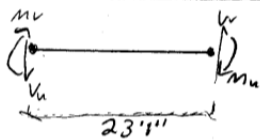


W12x120 column
 $P_u = 29.6 \text{ k}$
 $M_u = 1354.7 \text{ k-in}$
 $= 112.9 \text{ k-ft}$

From table 6-1 : combined Axial/Bend.
 $KL = (14)(1.76) = 24.65$
 $P \times 10^{-3} = 1.20$ $b_1 \times 10^{-3} = 1.44$

$$\frac{29.6(1.2)}{1000} + \frac{112.9(1.44)}{1000} = 0.20 < 1.0 \quad \text{OK}$$

3.) Beam member @ Truss 2 : Floor 3



W18x86 beam
 $V_u = 88 \text{ k}$
 $M_u = 6086.1 \text{ k-in}$
 $= 506.7 \text{ k-ft}$

Use table 3-2, AISC
 W-shapes
 $\Delta M_p = 698$ $L_b = 23 \text{ ft}$
 $BF = 13.6$ $L_p = 9.29$
 $\phi V_n = 265$

$$M_n = M_p - B_f(L_b - L_p)$$

$$= 698 - 13.6(23.08 - 9.29)$$

$$= 510 \text{ k-ft} > 506.7 \text{ k-ft} \quad \text{OK}$$

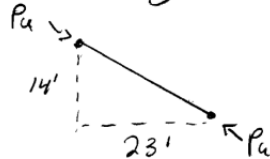
$$V_n = 265 \text{ k} \rightarrow 88 \text{ k} \quad \text{OK}$$

Figure 92: Lateral Member Spot Check

Project	Hunters Point South	Sheet No.	2 of 2
Made By	Michael Payne	Checked By	
Subject	Lateral Load Member Spot Check	Project No.	

★ Loading in x-direction (cont.)

4.) Bracing Member @ Truss 2 : Floor 4



HSS14x10x0.25

$P_u = 108.6 \text{ k}$

$L_b = 27 \text{ feet}$

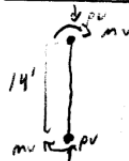
use table 4-1 in AISC

Strength in Axial compression

$\phi P_n = 276 > 108.6 \text{ (OK)}$

★ Loading in Y-Direction

5.) Column Member @ Truss 6 : Floor 2 table 6-1, AISC



W12x170 column

$P_u = 242.6 \text{ k}$

$M_u = 4288.9 \text{ k-in}$

$= 357.5 \text{ k-ft}$

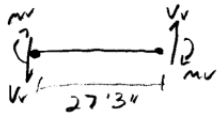
$KL = (14)(1.78) = 25$



$P_x \times 10^{-3} = 0.84 \quad b_x \times 10^{-3} = 0.938$

$\frac{242.6(0.84)}{1000} + \frac{357.5(0.938)}{1000} = 0.54 < 1.0 \text{ (OK)}$

6.) Beam member @ Truss 6 : Floor 3



W18x97 beam

$V_u = 13.8 \text{ k}$

$M_u = 2355.9 \text{ k-in}$

$= 196 \text{ k-ft}$

use table 3-2, AISC



$\phi M_p = 781$

$L_b = 27'3$

$BF = 14.1$

$L_p = 9.36$

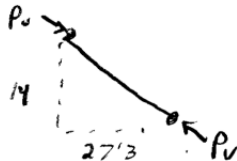
$\phi V_n = 299$

$M_n = M_p - BF(L_b - L_p)$

$= 781 - 14.1(27.25 - 9.36) = 539 > 196 \text{ (OK)}$

$\phi V_n = 299 > 13.8 \text{ (OK)}$

7.) Bracing member @ Truss 6 : Floor 4



HSS12x8x0.375

$P_u = 62.7 \text{ k}$

$L_b = 30.6'$

use table 4-1 AISC



$\phi P_n = 250 \text{ k} > 62.7 \text{ k (OK)}$

Figure 93: Lateral Member Spot Check

APPENDIX H

FLOOR PLANS

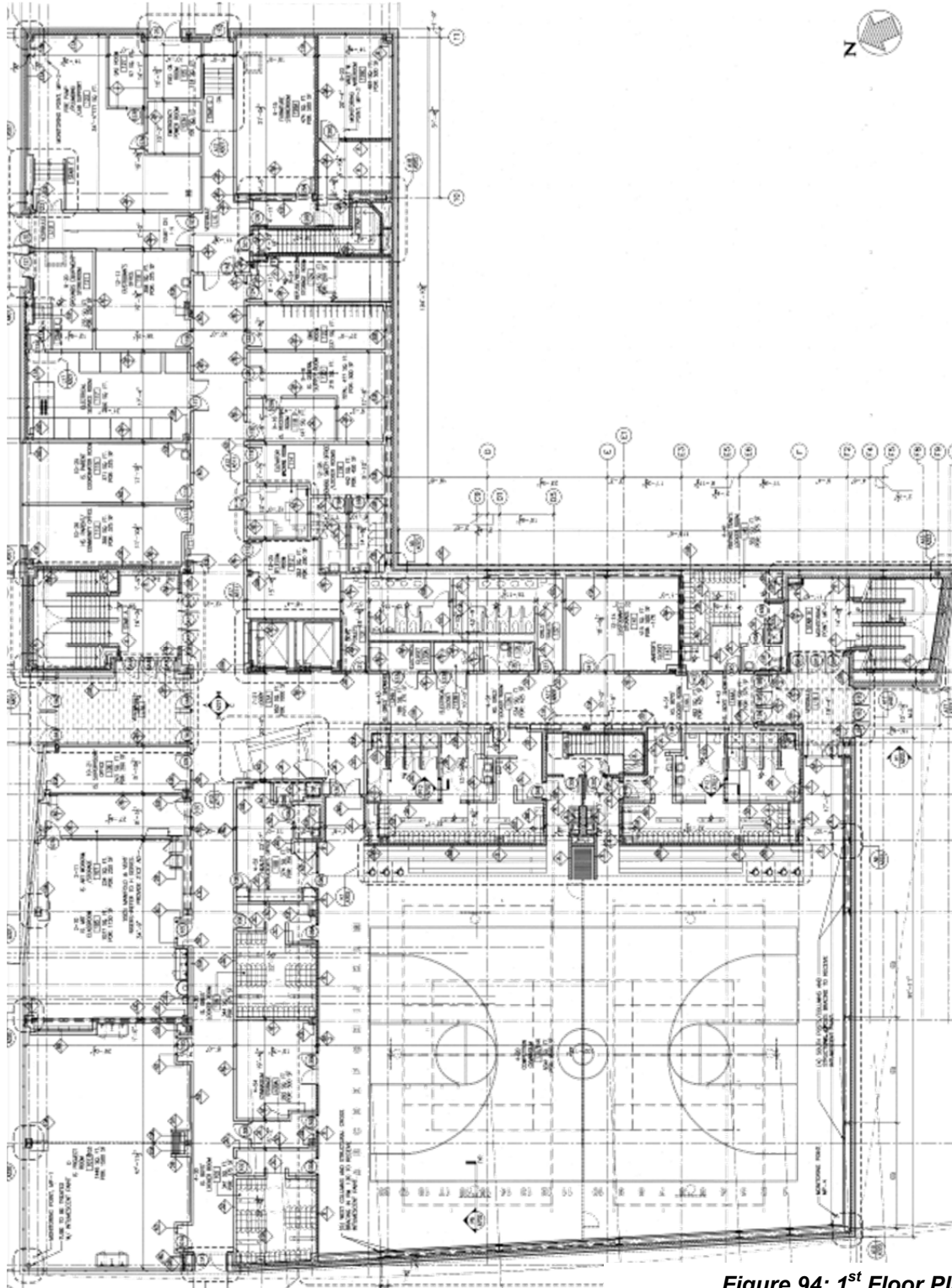


Figure 94: 1st Floor Plan

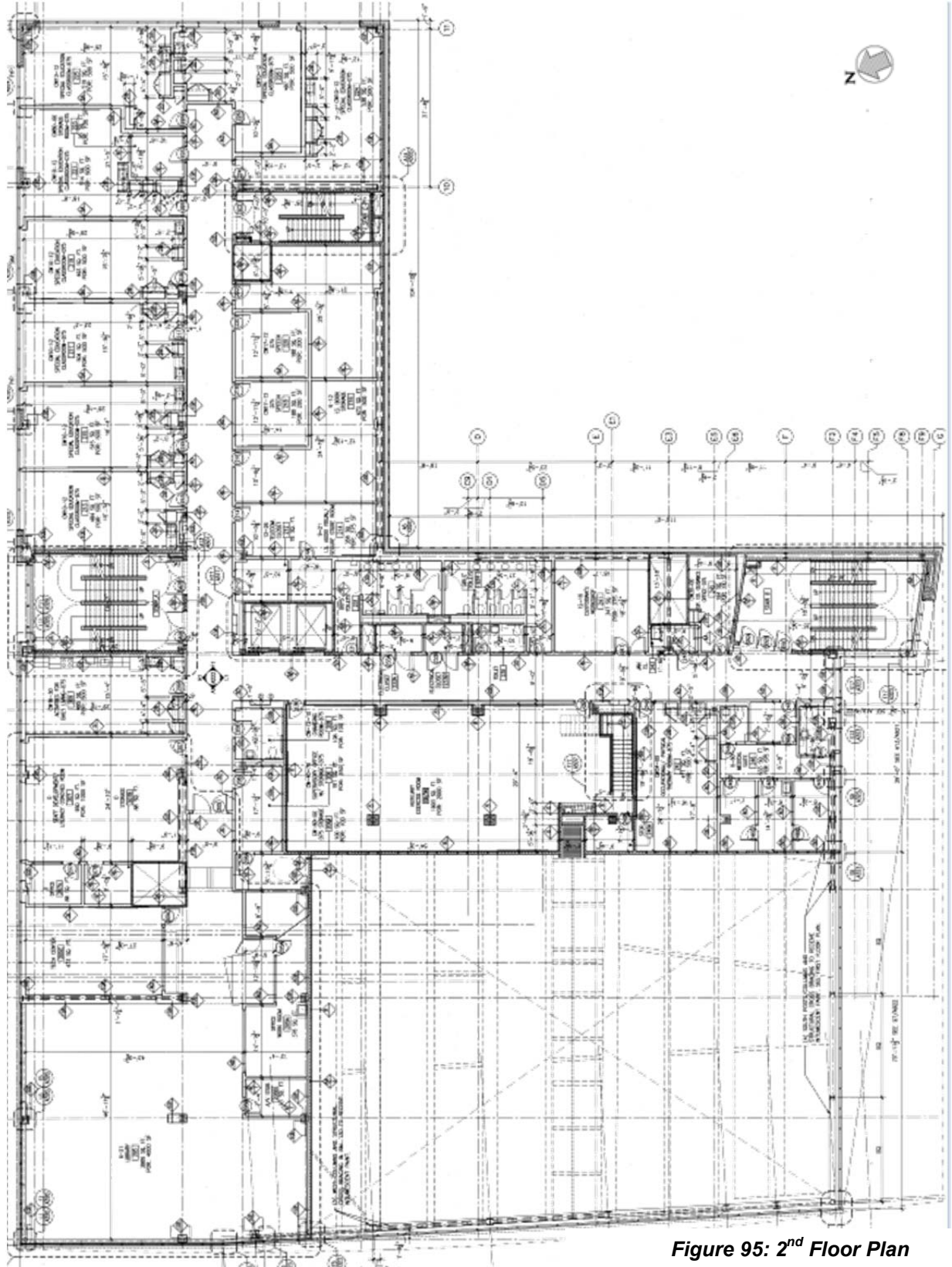


Figure 95: 2nd Floor Plan

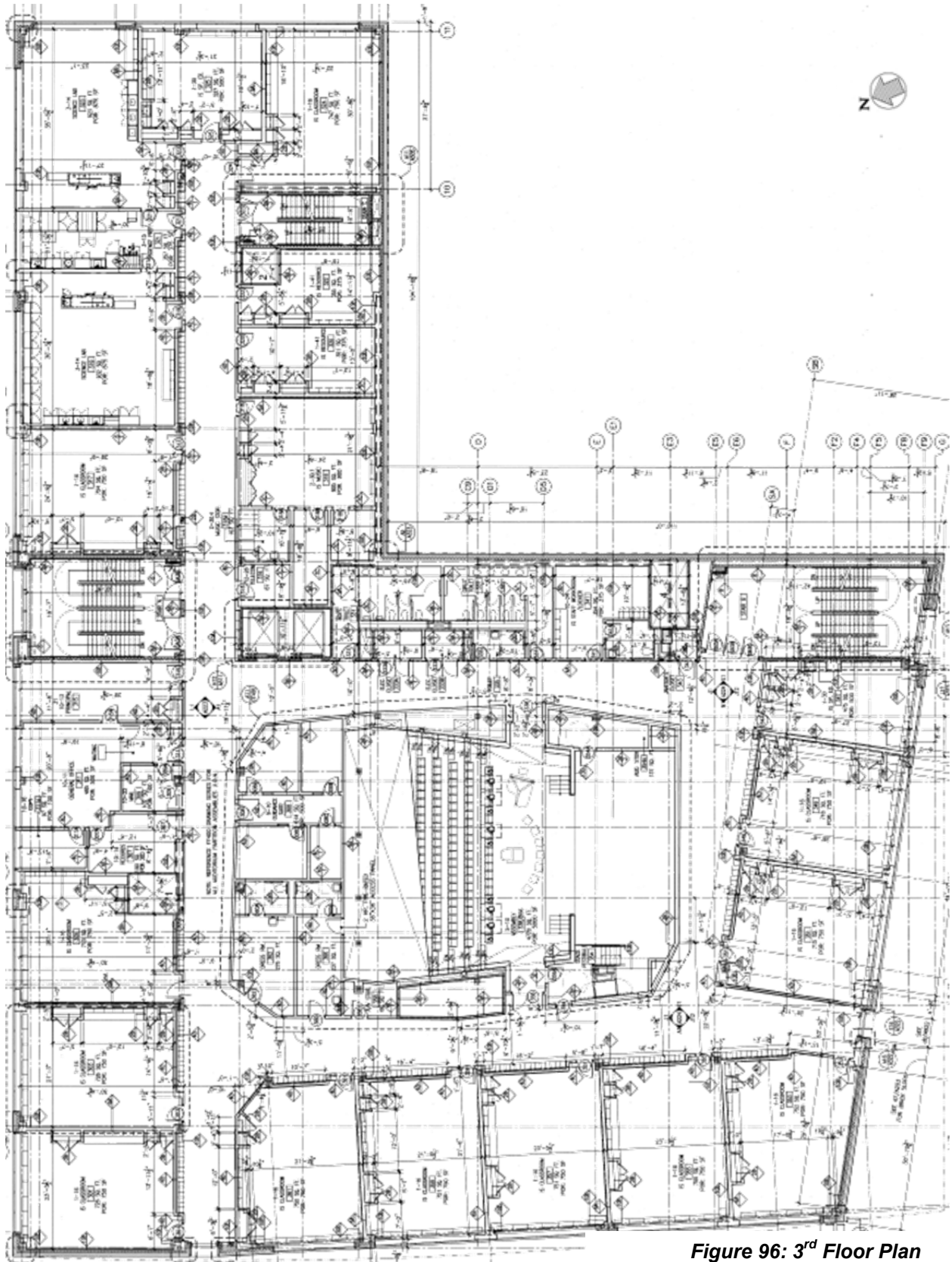


Figure 96: 3rd Floor Plan

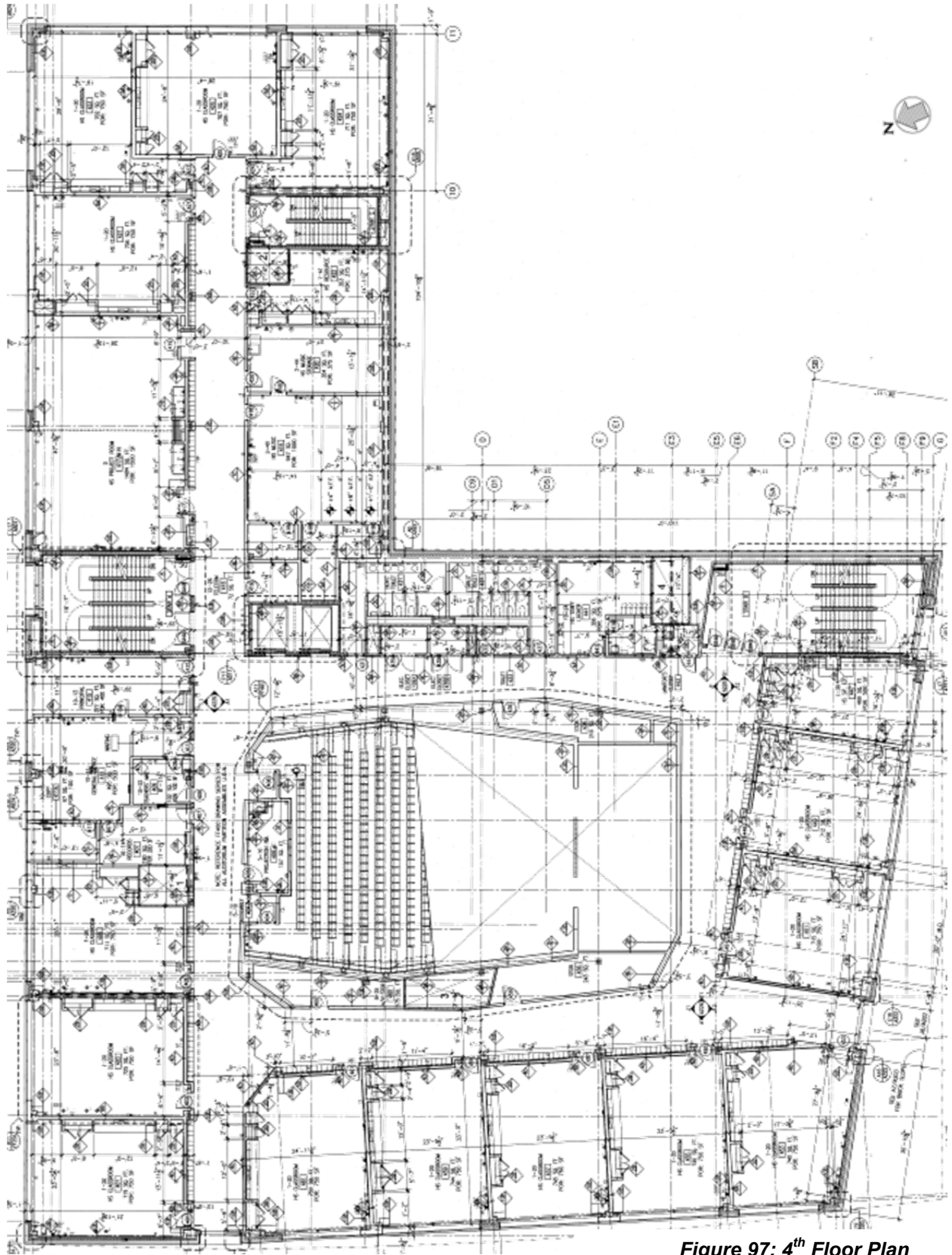


Figure 97: 4th Floor Plan

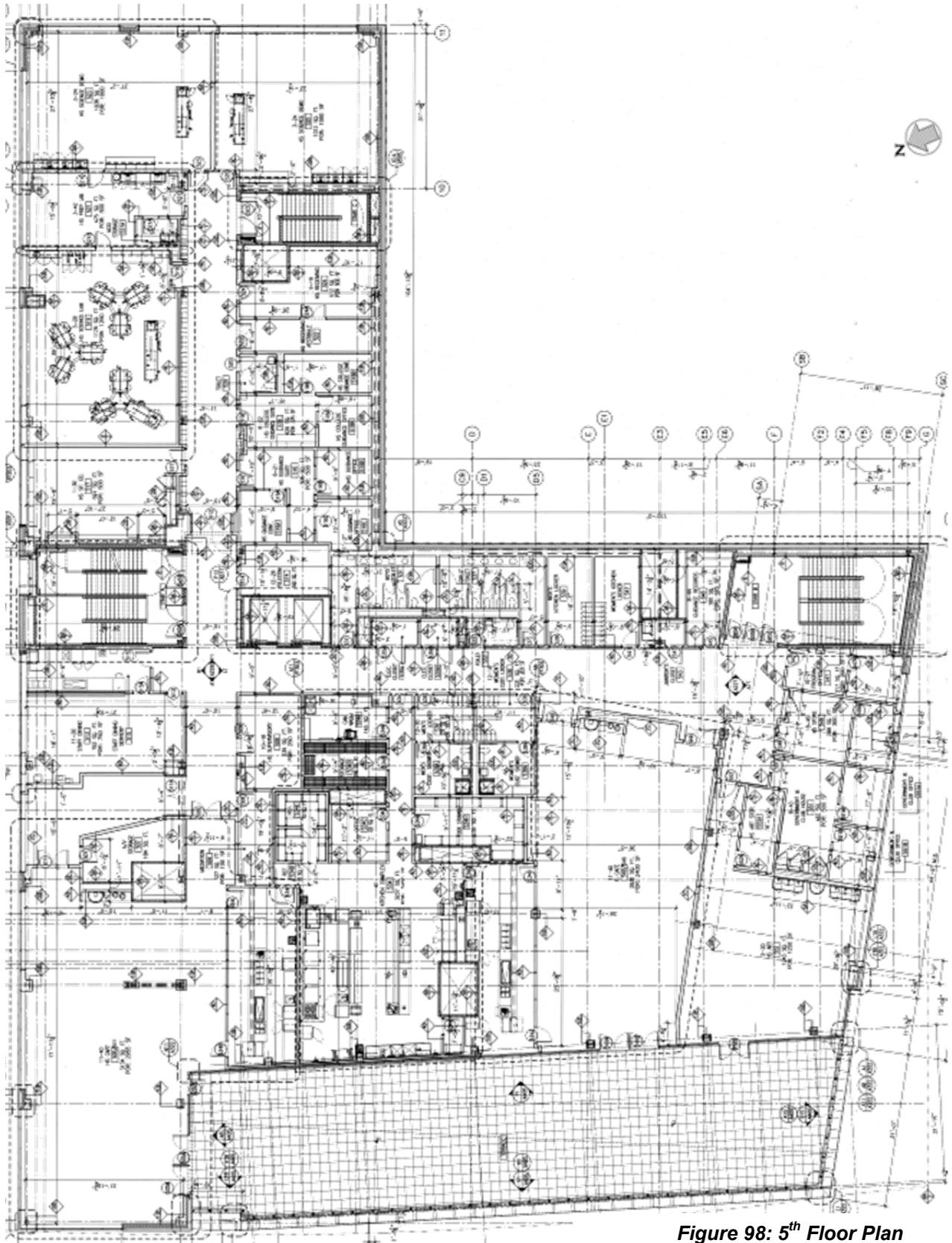


Figure 98: 5th Floor Plan

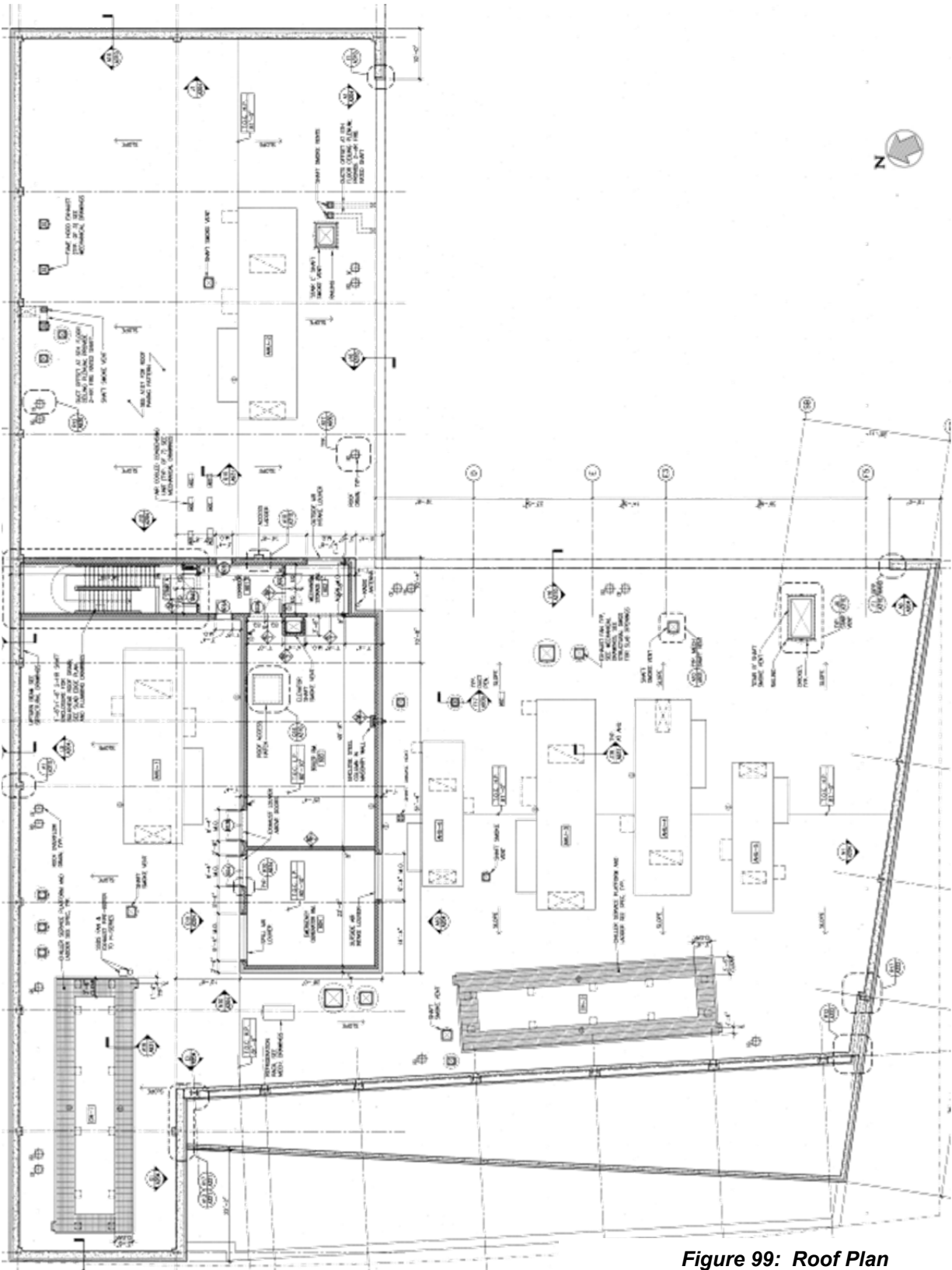


Figure 99: Roof Plan

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