SENIOR THESIS FINAL REPORT

Vibration Resistance and Lateral System Redesign

S.T.E.P.S. Building

Lehigh University

Bethlehem, PA

Joseph S. Murray

Structural Option

Faculty Advisor: Linda Hanagan

April 3, 2013

Acknowledgements and Thanks

One of my first contacts on this project was William Deegan, a family friend and architect at Spillman Farmer Architects in Bethlehem, PA. Without his connections in the building industry and personal dedication, I would never have found such a compelling building to work with. He connected me to an architect at BCJ Architects named Tom Kirk who was immensely helpful in connecting me to the owner of the building.

The final, and possibly most important, participant in my search for a building was Patricia Chase, Director of Campus Planning & Projects at Lehigh University. Ms. Chase took time out of her busy schedule to provide me with any information and documents I needed. Additionally, she personally gave me a tour of the building which provided me with a wealth of knowledge that could never have simply been read off of a construction document.

Lastly, I would like to acknowledge two people involved in the design and construction phase of S.T.E.P.S. who were quick to answer any questions I had about the structure or construction process. They are Jeff Pitchford at CVM Engineering and Eric Holland at Alvin Butz, Inc., and it is my immense gratitude to have had them as contacts on this project.

There are so many members of the AE faculty at Penn State who have helped me become a better engineer and analytical thinker. I especially want to thank my advisor on this project, Dr. Linda Hanagan. Her advice and patience both in the classes that I took and the consultation sessions that I had were essential in completing this project. I would also like to thank Professor Kevin Parfitt for answering any general structural and administrative questions I had. Lastly, I would like to thank Dr. Richard Behr for the knowledge I gained from his class on lateral loads and indeterminate structures.

Finally, I need to thank my entire immediate family: my father Sam; my mother Margie; and my sisters Emily and Maria. With their love and support, the past five years were an easier load on me than I ever expected.

S. T. E. P. S. Building

Joseph S. Murray - Structural Option

Science, Technology, Environment, Policy, and Society Building

www.engr.psu.edu/ae/thesis/portfolios/2013/jsm5242/index.html

- 135,000 square foot project
- 5 stories plus Mech. Penthouse
- Constructed 8/2008 - 8/2010
- Bethlehem, PA
- Design-Bid-Build Contract
- $62 million total cost

PROJECT TEAM:

Owner:	Lehigh University	Bethlehem, PA
CM:	Alvin H. Butz	Allentown, PA
Architect:	BCJ Architects	Philadelphia, PA
Structural:	CVM Structures	Oaks, PA
Civil:	Barry Isett & Assoc.	Trexlertown, PA
MEP/Fire:	Flack & Kurtz	New York, NY
Landscape:	Lager Raabe Skafte	Philadelphia, PA

Architectural Features:

- Classrooms, Laboratories, Faculty Offices, Atrium
- LEED Gold Certification
- Brick and stone exterior masonry walls
- Decorative wood finished and structural wood planks
- Glass curtain walls, clearstories, and glass atrium
- Fritted glass artwork and custom tiling incorporate art with science

Structural Features:

- Semi-rigid Steel Moment Frame System
- Composite Steel Deck with concrete slab
- Grade Beam and Caisson foundation

Mechanical Features:

- Custom Air Handling Units with Heat Exchangers
- Dedicated Laboratory Emergency Exhaust System

Lighting / Electrical Features:

- Automated photosensors to minimize electrical demand from lighting
- Daylighting emphasized through glass curtain walls and clearstories

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Executive Summary

The S.T.E.P.S. Building in Bethlehem, PA sits on Lehigh University's campus. It is a mixed use facility consisting of laboratories, lecture halls, and faculty offices. The building is divided into two main wings which are bridged by a central atrium.

The existing structural system of the building consists of semi-rigid moment frames and full moment frames. It uses a composite floor as a rigid diaphragm to transfer lateral loads imposed on the façade to the beams and girders. The beams and girders then transfer these loads through their moment connections to a network of mainly W14 columns. The columns finally transfer the load into the soil through a combination of spread footings and mat foundations.

The structural depth consists of two major tasks. First, the floor system were checked against vibration control tolerances and redesigned. The building contains sensitive laboratory equipment which requires vibration tolerances on the floor. The existing floor did not meet the chosen design tolerance of 2000 micro-inches/second for moderate walking. The redesigned floor included reducing the beam tributary width from 10.67' to 7.11', an increased beam section, and a W24 girder instead of a W21.

Second, the semi-rigid wind clips will be replaced with braced framed and full moment connections. The controlling wind case was found, and forces were distributed assuming a rigid diaphragm and idealized k values. They were then applies to the connections, and all applicable limit states were examined. The moment frames resulted in full depth stiffeners and doubler plates for the column. A ½" gusset plate supports an HSS 4x4x1/2 eccentric brace in the eccentrically braced frame.

The two breadths are related to the electrical and construction management disciplines. The construction management breadth consists of a detailed construction sequence with crane positioning. The construction schedule runs the length of the project in detail, and a site layout describes where the crane can safely and effectively be placed. The electrical breadth provides a typical panelboard schedule, electrical capacity estimates, and details on emergency lighting and fire alarms.

In this thesis redesign, the following goals were set and achieved:

1. Analyze the existing floor system for vibration resistance with AISC Design Guide 11
2. Redesign the floor to allow for 400x microscopes at moderate walking speeds
3. Redesign the lateral system with full moment frames and braced frames
4. Design a typical moment connection in detail
5. Design a typical braced connection in detail
6. Create a construction schedule
7. Create a sitemap with crane positioning
8. Create a panelboard schedule and estimate electrical capacity

Building Introduction

Lehigh University envisioned the Science, Technology, Environment, Policy, and Society (S.T.E.P.S.) Building as a way to attract new students and retain existing students in the science and engineering fields. A picture of the building is in Figure 1. The university lacked a modern laboratory building with all the amenities that have come with increases in technology over the years. In an interesting and experimental fashion, the departments have been intermixed by Health, Education & Research Association, Inc. They believe it will lead to increased communication and collaboration among faculty and researchers of various disciplines.

Figure 1: South Façade

Image Courtesy of Lehigh University

The building is oriented on the corner of East Packer Ave. and Vine St. as shown in Figure 2. The streets do not intersect at a 90 degree angle. The architects decided to use site lines to orient the building, which led to the nonlinear shape of the façade along Vine St.

Figure 2: Site Plan

Image Courtesy of BCJ Architects

Lehigh University slowly purchased the properties which were on the building site as they planned for a building to be put there. The location was ideal for expanding campus activities close to the campus core. This is shown in Lehigh's Campus Master Plan of 2000 in Figure 2.

Figure 2: Campus Master Plan

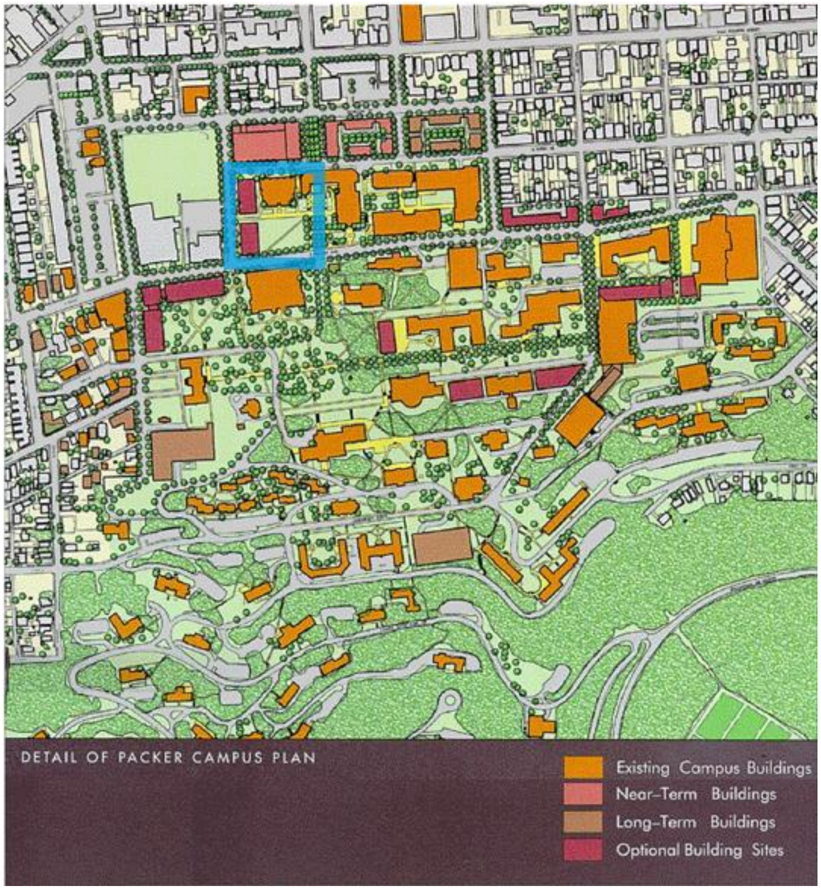


Image Courtesy of Lehigh University

The building is also connected to an existing structure through the use of a raised pathway that is enclosed. This further encourages interconnectivity between faculty, researchers, and students, because the adjoining building contains part of the College of Social Sciences. Between this adjacent building and S.T.E.P.S., there is a large open lawn. The university made a significant effort to maintain this lawn for extracurricular activities such as frisbee, croquet, and football. The S.T.E.P.S. Building is divided into three wings for the purpose of this analysis. These wings are diagramed in Figure 3.

Figure 3: Wings A, B, and C of S.T.E.P.S. Building

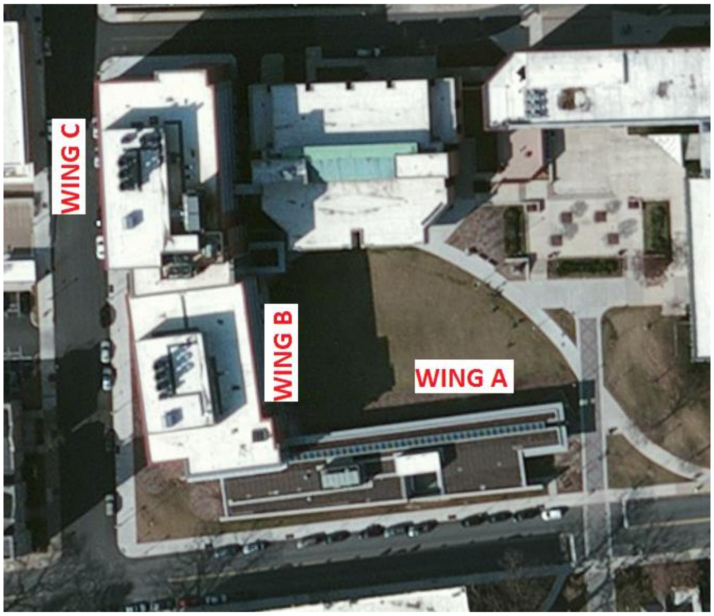


Image courtesy of Bing.com

Wing A is a one story structure with a lounge and entryway. It has raised clearstories to allow for natural daylight to illuminate the space. It also has a 12" deep green roof supported by structural wood which helped in earning LEED Certification. The building is LEED Gold certified by the United States Green Building Council (USGBC). Because of its limited building height, Wing A will not be analyzed in this report.

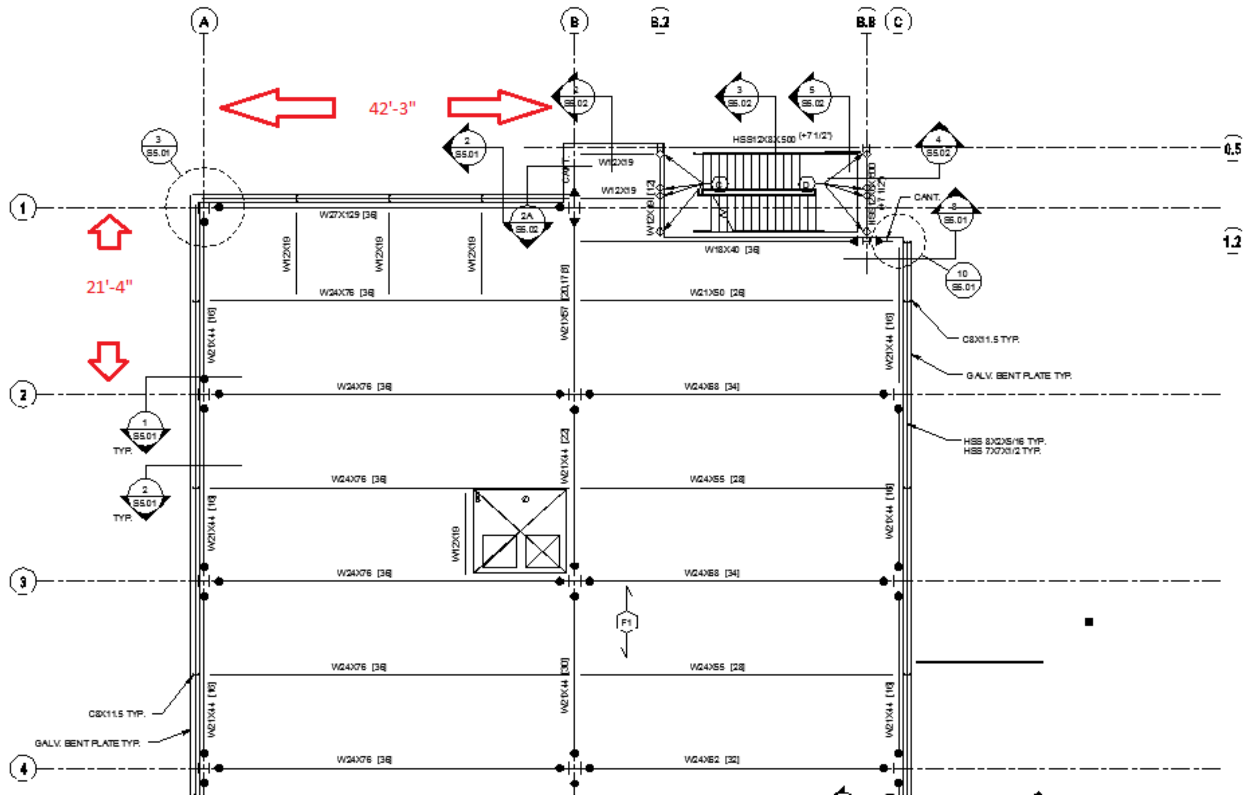
Wing B is a four story steel framed structure oriented along Packer Ave. There is a large atrium with lounge areas connecting Wing B to Wing C on each floor. Wing C is also steel framed and is 5 stories.

The gravity and lateral load resisting elements continue uninterrupted through the atrium. As a result, Wing B and Wing C will be treated as one building. The building's lateral system consists of moment connections between columns and beams throughout the building.

Sustainable features of the building include the green roof, high-efficiency glazing, sun shading, and custom mechanical systems.

Structural System

Figure 4: Typical Building Floor Plan



For a full floor plan, see Appendix A-1.

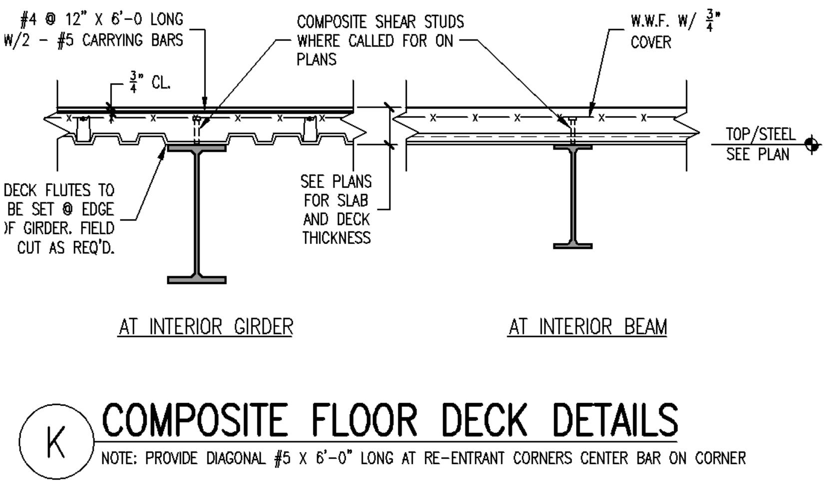
Floor System

There is a composite steel deck floor system in place for all floors in Wings B & C above grade. Basement floors are slab on grade.

Along Vine St., which will be considered the longitudinal direction of the building, typical girders have a center to center span of 21’-4” with one intersecting beam at their midpoint. The transverse beams which run parallel to Packer Ave. have a span anywhere from 36’-11” to 42’8”.

The decking is a 3” deep 18 gauge steel deck with 4-1/2” normal weight concrete topping and welded wire fabric. The bulk of the decking is run longitudinally throughout Wings B & C and has a span of 10’8” between beam centerlines. The exceptions to this are two bays to the very south of Wing B along Packer Ave. These bays are oriented transversely. The total thickness ends up being 7-1/2” with a 6x6” W2.9 x W2.9 welded wire fabric embedded ¾” from the top of the slab. Figure 5 shows a typical detail of the composite floor decking.

Figure 5: Composite Floor Deck Detail



The floor system is supported by wide flange beams designed as simply supported. A combination of full moment connections, semi-rigid moment connections, and shear connections are used. Typical sizes for transverse beams are W24x55 and W24x76. The girders are W21x44. Most beams have between 28 and 36 studs to transfer shear. Figure 5 shows a typical Full Moment Connection with field welds noted. Figure 6 shows the entirety of the first floor system for Wing B. Figure 8 shows the entirety of the first floor system for Wing C.

Vertical Members

Wide flange columns are used throughout the building for gravity loads. They are arranged for strong axis bending in the transverse direction. Most spans have a column at either end with another at the midpoint.

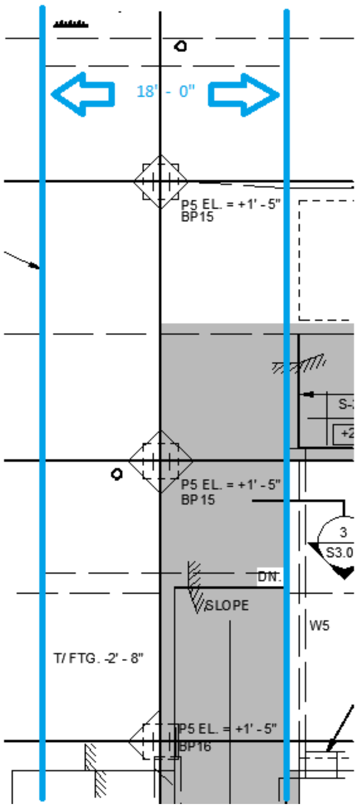
W14 is the most common section size with weights varying from W14x90 all the way up to W14x192 on the lower floors.

Foundation

Schnabel Engineering performed a geotechnical analysis of the site in 2007. This concluded that the soil had sufficient bearing capacity to support the loads from the building.

Interior columns are supported by a mat foundation 18' wide and 3'-6" deep shown in Figure 6 and Figure 7. Exterior columns bear on square footings ranging from 11'x11' to 16'x16' with depths from 1'6" to 2'. These are tied into the foundation by base plates with concrete piers.

<u>Figure 6: Mat Foundation Plan View</u>



<u>Figure 7: Mat Footing Schedule</u>

MF1	* X 18'-0" X 3'-6"	#9 @ 12" O.C. E.W. BTM #9 @ 12" O.C. LONG, TOP #7 @ 12" O.C. TRANS, TOP	SEE PLAN FOR LENGTH

The reinforced foundation walls have strip footings ranging from 2' to 6' wide with depths between 1' and 2'. These are monolithically cast with the piers for the exterior columns.

<u>Roof System</u>

The roof decking consists of a 3" 16 gauge steel roof deck with a sloped roof for drainage. Topping ranges from ¼" to 4-1/2" to achieve a ¼":1' slope. Therefore, total thickness ranges from 3-1/4" to 7-1/2". Framing is similar to floor framing with wide flanges ranging from W24x55 to W24x68.

The floor system has increased loads where the mechanical penthouses are situated. The penthouse itself is framed with square HSS tubing. Heavier W27x84 wide flange beams support this area.

Lateral System

The building resists lateral loads by moment connections at the beam to column locations. They are continuous throughout the building and beams are designed as simply supported for gravity loads. The moment connections are designed only to take lateral loads. A typical semi-rigid moment connection is shown in Figure 8. Many of these moment connections are semi-rigid connections to give the system more flexibility. An example of layout of the two types of moment connections in the floor plan is shown below in Figure 9. The triangles are full moment connections and the dots are semi-rigid.

Figure 8: Typical Semi-Rigid Moment Connection

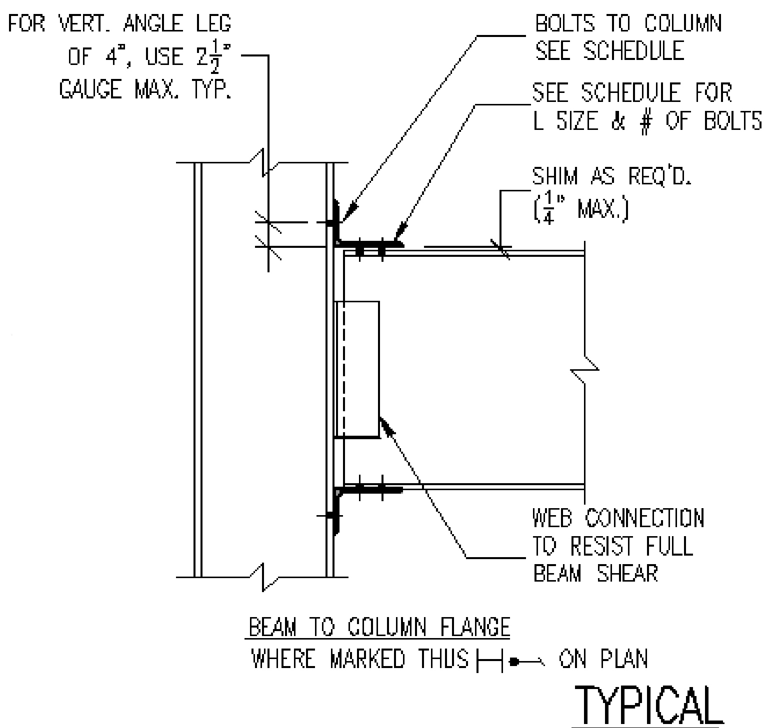
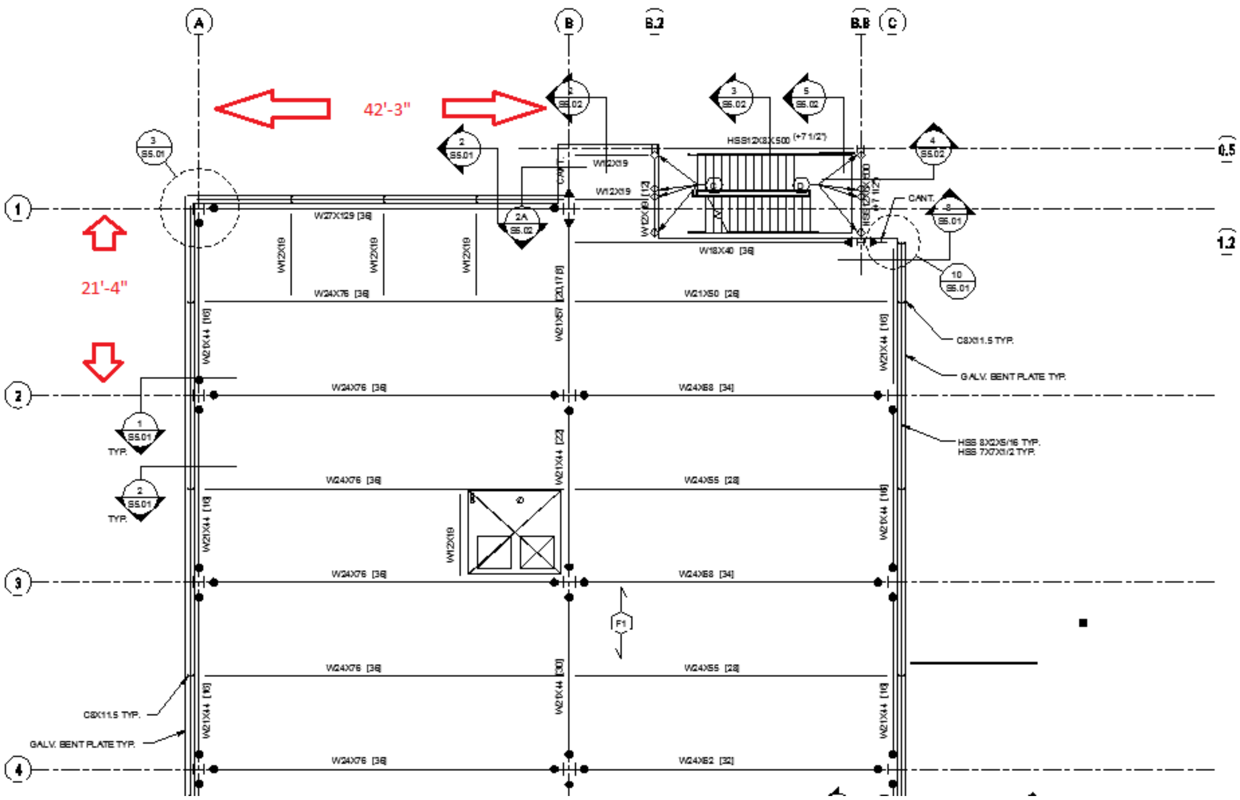


Figure 9: Typical floor plan with bay sizes



The lateral loads seen in the Penthouse are going to be the greatest based on height. At the highest Penthouse roof level, there are moment connections in the transverse direction and single angle braced frames in the longitudinal direction. The connections to the roof of the building are rigidly connected to the roof framing members. These members then transfer the load to flexible moment connections in the columns supporting the roof. These roof members are a larger W27x102 compared to adjacent members such as W24x68 or W27x84.

Design Codes

The Pennsylvania Uniform Construction Code (PUCC) is the code adopted by the city of Bethlehem, Pennsylvania. The PUCC is based on the International Code Council (ICC). When design was completed in 2008, the 2006 PUCC referenced the following codes:

2006 International Building Code

2006 International Electrical Code

2006 International Fire Code

2006 International Fuel Gas Code

2006 International Mechanical Code

ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

AISC Steel Construction Manual, 13th Edition

ACI 318-05, Building Code Requirements for Structural Concrete

ACI 530-05, Building Code Requirements for Masonry Structures

The primary codes employed in the redesign were:

AISC Steel Construction Manual, 14th Edition

ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

AISC Design Guide 11: Floor Vibrations Due to Human Activity

Design Loads

Live Loads

Table 1: Live Load Values

Occupancy	Design Load on Drawings	ASCE 7-05 Load (Tables 4-1, C4-1)
Office	50 PSF	50 PSF + 15 PSF (Partitions)
Classroom	40 PSF	40 PSF
Laboratory	100 PSF	100 PSF
Storage	125 PSF	125 PSF
Corridors/Lobbies @ Ground Level	100 PSF	100 PSF
Corridors Above Ground Level	80 PSF	80 PSF

Dead Loads

Table 2: Calculated Dead Load

	Dimension	Unit Weight	Load (PSF)
3" 18 Ga. Composite Deck			2.84
7-1/2" Concrete Slab	0.5 CF/SF	145 PCF	72.5
Self-Weight			5
MEP Allowance			10
Ceiling Allowance			5
TOTAL			**95.3 PSF**

Roof Live Load

Table 3: Roof Live Load

Occupancy	Design Load on Drawings	ASCE 7-05 Load (Tables 4-1, C4-1)	Design Load
Roof	N/A	20 PSF	**20 PSF**

Roof Dead Load

Table 4: Roof Dead Load

	Dimension	Unit Weight	Load (PSF)
3" 16 Ga. NS Roof Deck			2.46
3" Concrete Topping (Avg.)	0.290 CF/SF	150	43.5
Self-Weight			5
Roofing Allowance			10
TOTAL			**60.96 PSF**

Snow Load

Table 5: Uniform Roof Snow Load

Design Factor	ASCE 7-05	Design Value
Snow Load (Pq)	Figure 7-1	30 PSF
Roof Exposure	Table 7-2	Fully Exposed
Exposure Type	Section 6.5.6.2	B
Exposure Factor (Ce)	Table 7-2	.9
Thermal Factor (Ct)	Table 7-3	1.0
Building Type	Table 1-1	III
Importance Factor (I)	Table 7-4	1.1
Flat Roof Snow Load (Pf)	Equation 7-1	20.8 PSF
Minimum Snow Load (Pf,min)	Section 7.2	22 PSF
Design Snow Load	Section 7.2	**22 PSF**

Pf = 0.7(Ce)(Ct)(I)(Pq)

Pf = 0.7(.9)(1.0)(1.1)(30) = 20.8 PSF

20.8 < Pf,min = 22 → Use 22 PSF as the Design Snow Load

Wind Loads

Table 6: Wind Design Factors:

Design Factor	ASCE 7-05	E/W Value	N/S Value
Design Wind Speed (V)	Figure 6-1C	90 mph	90 mph
Building Type	Table 1-1	III	III
Importance Factor (I)	Table 6-1	1.15	1.15
Exposure Type	6.5.6.2	Type B	Type B
Average Height (z)	6.5.8	77'-4"	108'-4"

Table 7: Design Wind Pressure by Level (Transverse Direction)

Level	Height	kz	qz	Pz (PSF) (Windward)	Ph (PSF) (Leeward)	Ptotal (PSF)
1	0'-0"	0.57	11.55	11.7	-11.28	23
2	16'-0"	0.58	11.76	11.7	-11.28	23
3	31'-4"	0.71	14.39	13.6	-11.28	24.9
4	46'-8"	0.79	16.01	15	-11.28	26.3
Roof/5th	62'-0"	0.85	17.22	15.9	-11.28	27.2
Roof/Penthouse	77'-4"	0.92	18.65	16	-11.28	27.3

Figure 10: Elevation of Transverse Pressure Levels

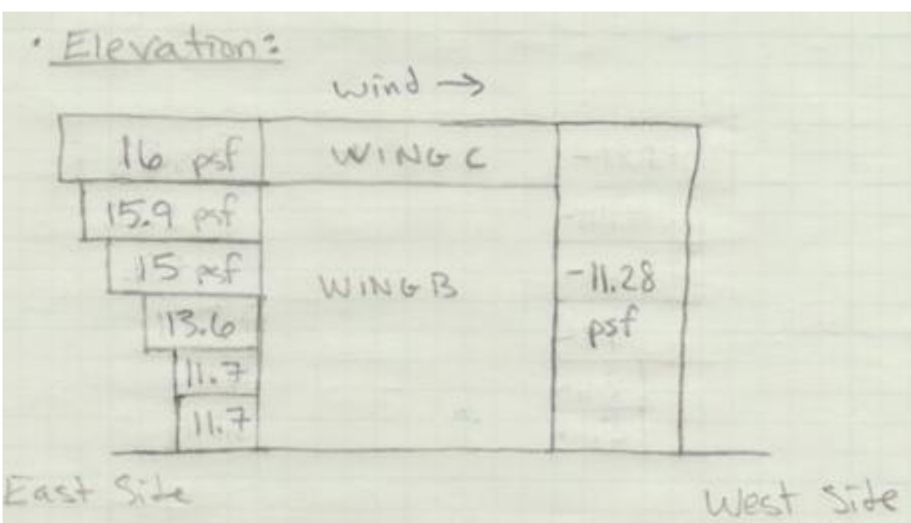


Figure 11: Elevation of Transverse Story Forces

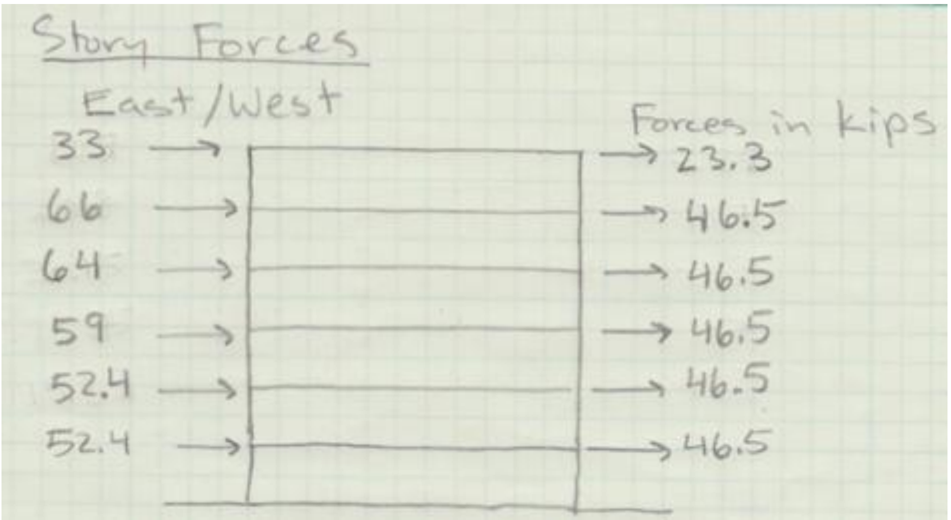


Table 8: Design Wind Pressure by Level (Longitudinal Direction)

Level	Height	kz	qz	Pz (PSF) (Windward)	Ph (PSF) (Leeward)	Ptotal (PSF)
G	0’-0”	0.57	11.55	11.7	N/A	11.7
1	16’-0”	0.58	11.76	11.7	-7.5	19.2
2	31’-4”	0.70	14.4	13.6	-7.5	21.1
3	46’-8”	0.79	16.01	14.9	-7.5	22.4
4	62’-0”	0.85	17.23	15.9	-7.5	23.4
Roof/5th	77’-4”	0.92	18.65	17.3	-7.5	24.8
Roof/Penthouse	92’-0”	0.96	19.46	17.6	-7.9	25.5

Figure 12: Elevation of Longitudinal Pressure Levels

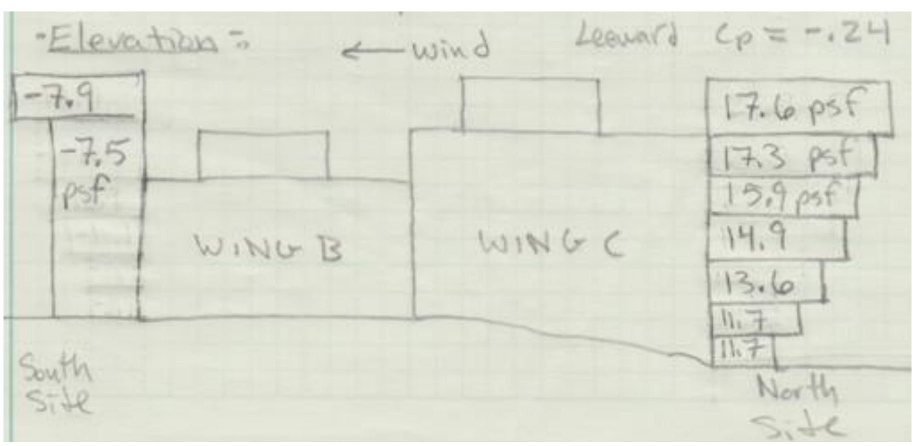
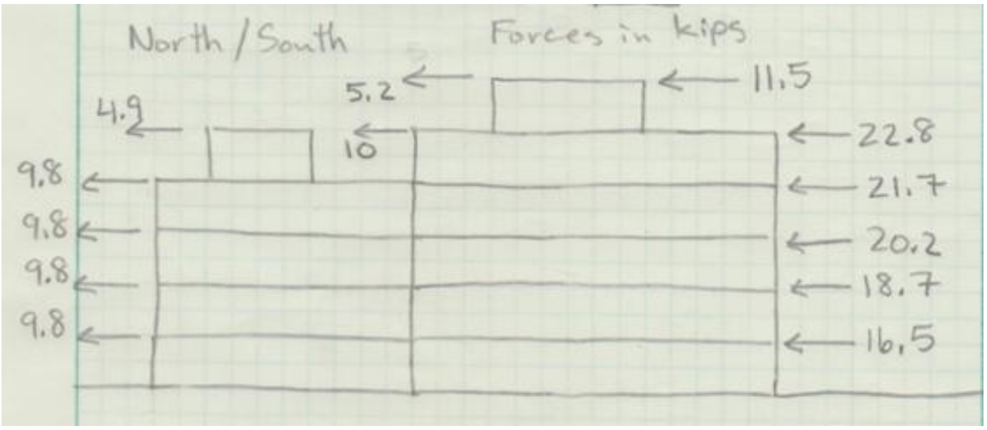


Figure 12: Elevation of Longitudinal Story Forces



Seismic Loads

Seismic loads did not control over wind loads for this building. Therefore, the main forces the lateral system should be designed for are wind and gravity loads.

Structural Proposal

The structural system of the S.T.E.P.S. building has been proven to be adequate for strength and serviceability requirements in Technical Reports 1, 2, and 3. Sections of the floor system were designed to limit floor vibrations typical of a laboratory. The existing floor typically consists of W24 beams framing into W21 girders with a 7.5" deep concrete composite decking. The floor will be checked for vibration resistance and tolerances will be adopted to design a new floor to resist greater vibrations.

The lateral force resisting system is composed of semi-rigid wind clips throughout the building. These connections are not designed for full moment capacity, and they are typically designed to simply resist the negative moment from wind loads. With a moment connection at every column connection, the erection process is likely to be increased. In order to lessen the number of moment connections in the building, a moment frame system will resist N/S wind loads, and a braced frame system will resist E/W wind loads.

The existing floor system will be analyzed for its performance in eliminating unnecessary floor vibrations with AISC Design Guide 11: Floor Vibrations Due to Human Activity. A new floor will then be designed to a certain criterion in Design Guide 11 based on sensitive equipment, possibly facility expansion, and the walking speed within a typical bay. The floor slab, beams, and girders all play a role in vibration resistance and must be assessed.

In order to design an effective alternative to the existing lateral system, the wind clips will be replaced with braced frames in the transverse direction of the building and full moment frames in the longitudinal direction. These types of lateral systems are a suitable replacement for the wind clips. Figure 13 has the floor plan revisions for Wing B, and Figure 14 has the floor plan revisions for Wing C. The braced frames are shown as red lines, and the moment frames are shown as blue triangles. All semi-rigid connections, currently marked as black dots, will be changed to shear or full moment connections as indicated.

The braced frames which will be replacing the lateral system will be designed in complete detail including gusset plate connections to the beams and columns. The new typical full moment connections will also be looked at in detail. Due to unforeseen complications, the existing connection interfaces could not be examined in the detail initially desired. Shop drawings could not be obtained from the steel fabricator, because they have since gone out of business. In addition, neither the construction team nor the structural firm was able to gain access to the shop drawings.

Figure 13: Revised Structural Floor Plan for Wing B

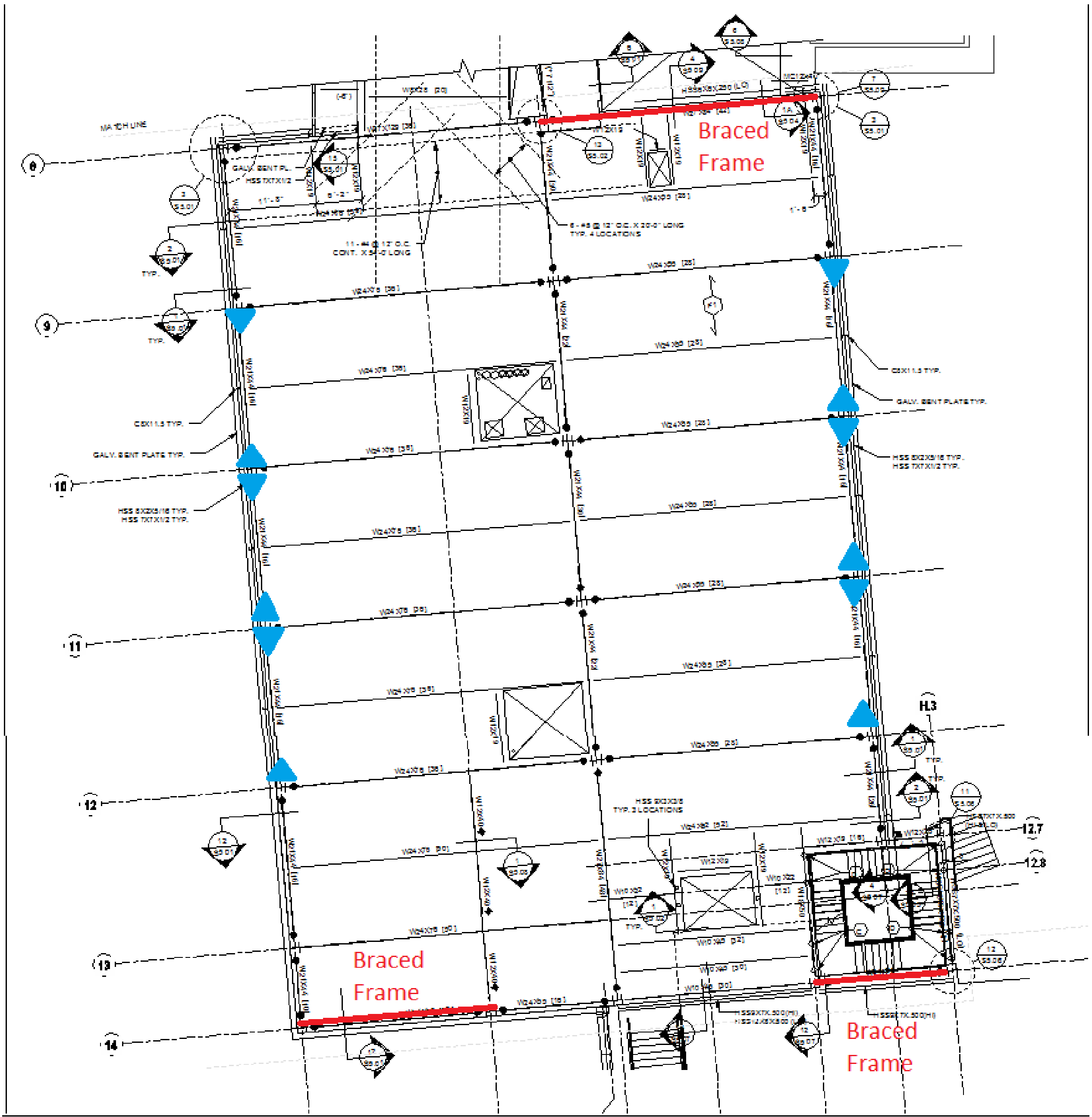
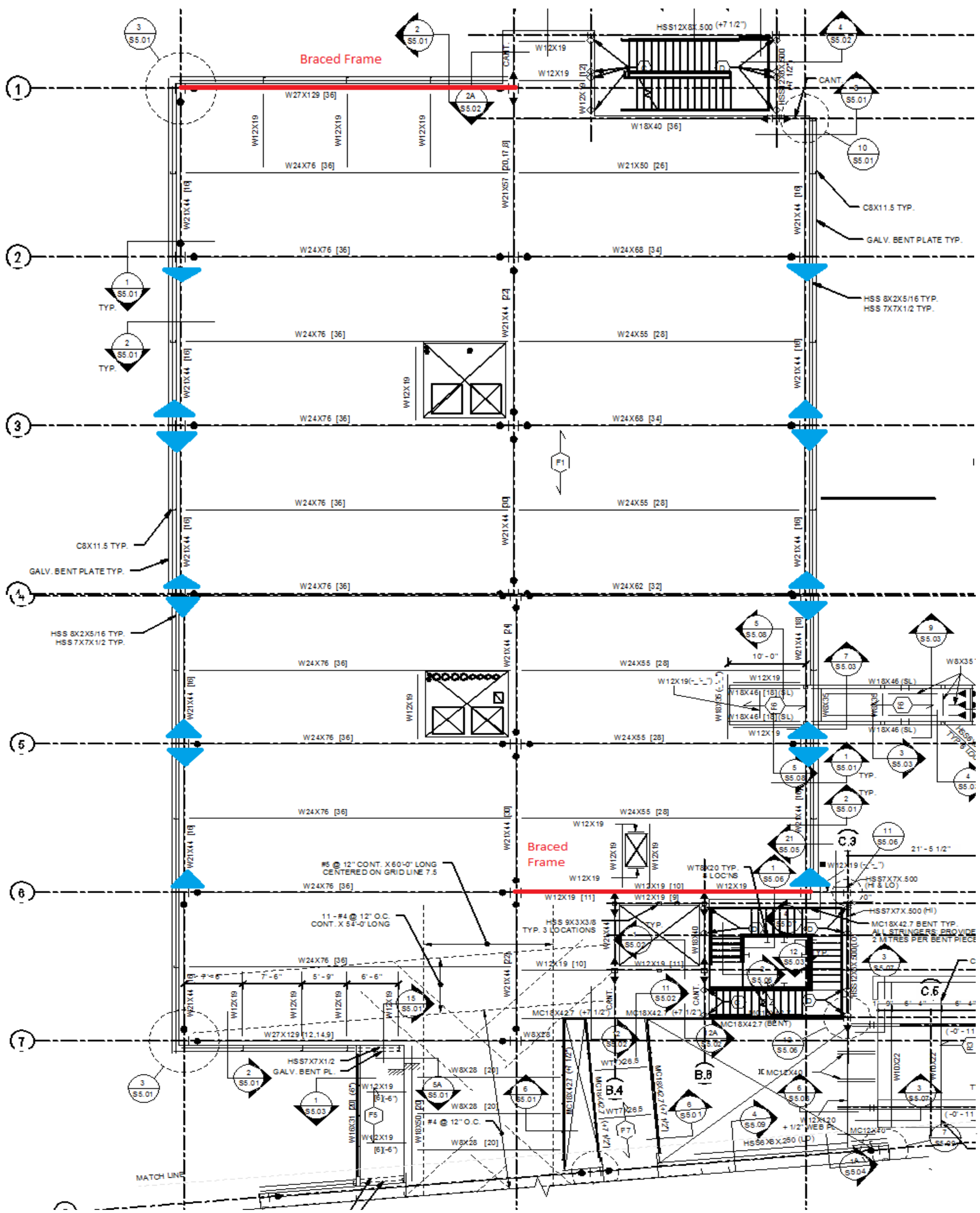


Figure 14: Revised Floor Plan for Wing C



Floor Vibrations

Vibrations occur around human beings every day, and most of these go unnoticed because they have little effect on typical activities. A car horn beeping and a person walking quickly past an office are two examples of vibrations which might be considered an annoyance. However, there are instances where vibrations are more than an annoyance and can affect people's work or activities. An imaginable example could be an office adjoining an aerobics studio. The constant bouncing up and down of people exercising will cause the floor to bounce like a trampoline. In this case, the bouncing could be more than an annoyance depending on the type of work done in the office. It's still unlikely the owner of an office would pay extra to reduce or eliminate these vibrations.

In the fields of research and development, a table moving up and down with the floor could be a bigger problem. Forces in adjoining spaces cause deflections by moving through the steel and concrete building framing. Tolerances in these fields are becoming increasingly more stringent as the scale of testing and design is becoming increasingly reduced. To avoid having an outside source affect laboratory work, measures must be taken in the design of the building. Experts in the field of acoustics have been designing concert halls and theaters to enhance sound quality for a long time. Many of these same principles about the way vibrations occur and reverberate can be applied to reduce or eliminate vibrations.

There are many ways to deal with these vibrations in structural engineering. When the criterion for design deflection is evaluated down to a micron of an inch, typical structural design can become complicated. One philosophy is to eliminate vibrations completely. This is achieved by isolating the cause of the vibrations from the rest of the structural system. Some structures have entirely separate framing systems for certain areas which require limitations on vibrations. Another option to eliminate vibrations is to use mass dampers, such as the ones shown in Figure 15.

Figure 15: Tuned Mass Damper in a Floor System

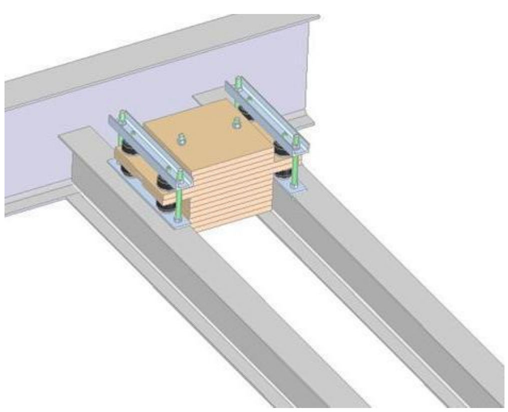


Image courtesy of www.deicon.com

The other design philosophy is to limit the amount of vibrations on the floor system. This is generally a much more economical approach, depending on the design requirements. AISC Design Guide 11 has a range of facility uses along with specific design criteria ranging from normal optical microscopes to neurological equipment used in brain surgery. A portion of this table is shown in Figure 16. Virtually any facility can be analyzed to meet the requirements set forth by the end user. Typically these structural systems are oversized in a way that severely limits their acceptable deflections. Causes of intolerable vibrations could be as small as a person walking slowly in a neighboring corridor. A range of well established universities along with their criteria for vibration are shown below. These are all modern facilities capable of meeting the research demands of the coming decades. Figure 17 contains a list of these universities.

Figure 16: Sensitive Equipment

Table 6.1
Vibration Criteria for Sensitive Equipment

Facility Equipment or Use	Vibrational Velocity*	
	(µ in./sec)	(µm/sec)
Computer systems; Operating Rooms**; Surgery; Bench microscopes at up to 100x magnification;	8,000	200
Laboratory robots	4,000	100
Bench microscopes at up to 400x magnification; Optical and other precision balances; Coordinate measuring machines; Metrology laboratories; Optical comparators; Microelectronics manufacturing equipment—Class A***	2,000	50
Micro surgery, eye surgery, neuro surgery; Bench microscopes at magnification greater than 400x; Optical equipment on isolation tables; Microelectronics manufacturing equipment—Class B***	1,000	25

Figure 17: Universities with Vibration Criteria

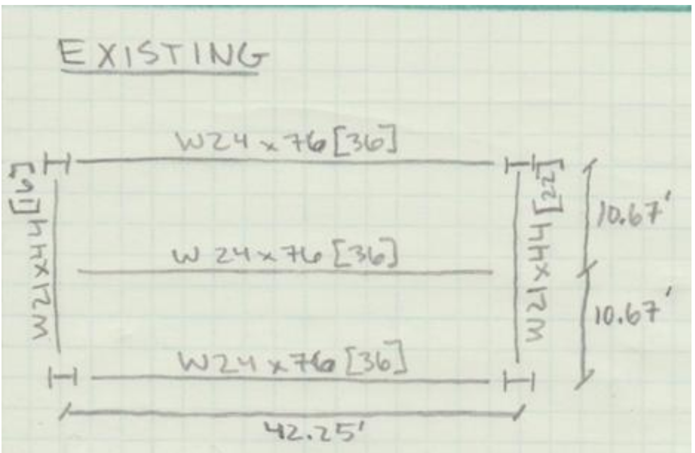
Owner	Building	Material	Criteria	Opening
Cornell	Nanotechnology Laboratory	RC	1000 µ"/s	Fall 2003
Harvard	Institute of Medicine	SS	2000 µ"/s	Summer 2005
MIT	Brain and Cognitive Science Center	SS	2000 µ"/s	Spring 2005
Duke	Science Center	RC	2000 µ"/s	Fall 2006
U. Chicago	Interdivisional Research Center	SS	750 µ"/s	Summer 2005
U. Mass Worcester	Research Institute	SS	2000 µ"/s	Fall 2000

Courtesy of "Floor System Vibration Control" by E.M. Hines

A peak velocity of 2000 α"/s (micro-inches/second) will be used as a criterion. This number was chosen for two reasons: it allows microscopes of magnifications up to 400x to be used in the laboratory, and it has been chosen as a tolerance by some of the most prestigious universities in the country.

The existing floor system was evaluated using Design Guide 11 with a criterion of 2000 α"/s. An existing laboratory bay is shown in Figure 18.

Figure 18: Existing Typical Laboratory Bay



The bay was evaluated based on concrete slab properties, steel beam and girder section properties, and bay geometry. The bay was evaluated in the following fashion:

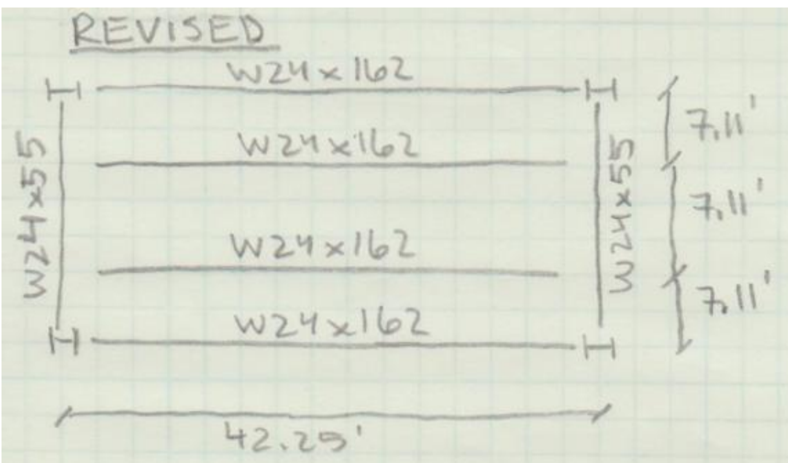
1. Determine effective width of concrete acting with the beams
2. Solve for the depth of the neutral axis
3. Transform the moment of inertia into a combined I_b value representing the beam and the effective concrete slab above
4. Calculate maximum possible service load on the beam
5. Calculate beam deflection due to load
6. Repeat steps 1-5 for girder
7. Use beam and girder deflections to determine the natural frequency of the floor
8. Evaluate deflections for a unit load on the bay
9. Determine mid-bay flexibility
10. Calculate maximum footfall velocity for a given condition (slow walking, moderate walking, fast walking)
11. Compare velocity with tabulated criteria
12. Repeat with another iteration as necessary

The existing floor system was under the proposed 2000 α"/s limit for slow walking. However it did not pass criteria for bench microscopes under fast walking or moderate walking. Based on the proximity of a corridor and the movement of people within the lab, it was redesigned for moderate walking as a minimum consideration.

The first important decision was to eliminate the shear studs and composite construction. There is no section in Design Guide 11 which permits an increase in strength or stiffness for having shear studs, because composite action is assumed. The next design decision was to limit the concrete slab to the current 7.5" thickness. This is a relatively deep slab, and adding more concrete to it will increase the dead load on the gravity system significantly.

After several iterations, it was determined that simply sizing up the members would not be the most economical decision. The beams are spanning 42.25' and carrying a tributary width of 10.67'. This amount of load was causing beam deflections to be relatively high. For architectural reasons, the span of the beam was left the same. Changing this would involve rearranging columns within the structure. So the beam spacing was reduced to 7.11' on center, framing to the girder at its triple points. A layout can be seen in Figure 19.

Figure 19: Existing Typical Laboratory Bay



This system allows for microscopes up to 400x magnification to be used in the laboratory. The floor has a maximum footfall velocity of 1764 α"/s, which is well below the criterion of 2000 α"/s. A comparison of the two systems can be seen in Table 9

Table 9: Floor System Comparison

	Existing Floor	Redesigned Floor
Natural Frequency (Hz)	3.7	5.27
Moderate Walking V (α"/s)	7021	1764
Total Floor Depth (in.)	31.5	31.5

RAM Model

RAM Structural System was used to create a 3D model of the S.T.E.P.S. Building. Gridlines were produced from AutoCAD drawings, and line elements were used to build the framework for the lateral and gravity force resisting systems. Steel sections and member properties were added to the line elements as noted on the structural drawings. The majority of the beams are W-flange members with HSS rectangular tubing used in some locations. Any columns which received a lateral beam were also modeled as part of the lateral system. The columns consist mainly of W14 sections with HSS rectangular tubing used for the elevator core. Some of the gravity beams terminated in a concrete basement wall, and an 18" thick reinforced concrete wall was modeled as shown on plan and in structural details. All exterior columns terminate in a spread footing foundation, while interior columns terminated in mat foundations.

The redesigned lateral system, consisting of moment frames in the N/S direction and braced frames in the E/W direction, was modeled in place of the existing lateral system and sized using RAM Frame. The composite floor system that exists throughout the building was modeled as a rigid diaphragm on each floor level. Weight of steel members and the floor systems was calculated by RAM, and then the weight of the wall system was added manually to each floor based on the floor's perimeter and the weight of the wall attached. Figure 20 shows the RAM model in 3D from the west direction and Figure 21 shows it from the east direction. The red color represents lateral members, and the blue color represents gravity members.

Figure 20: RAM Model (West Direction)

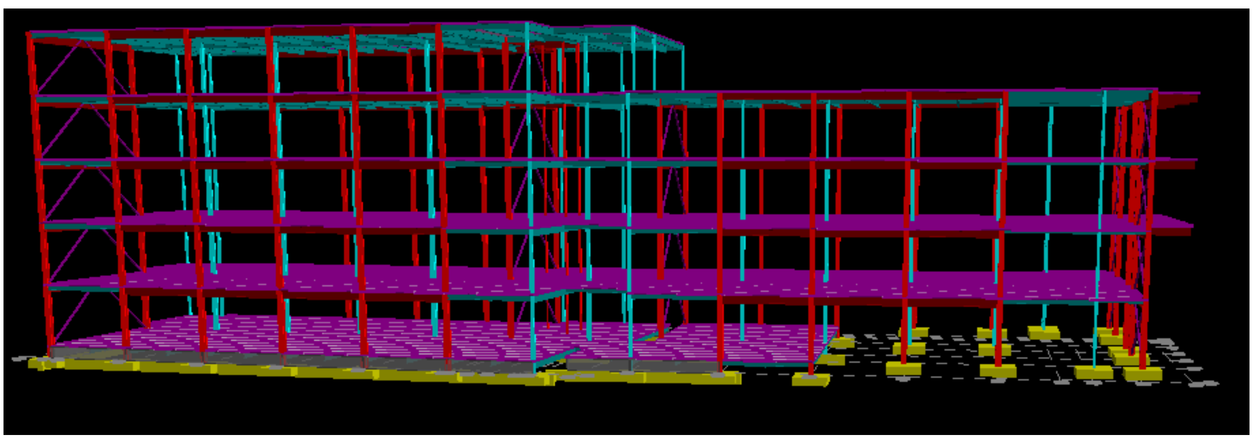
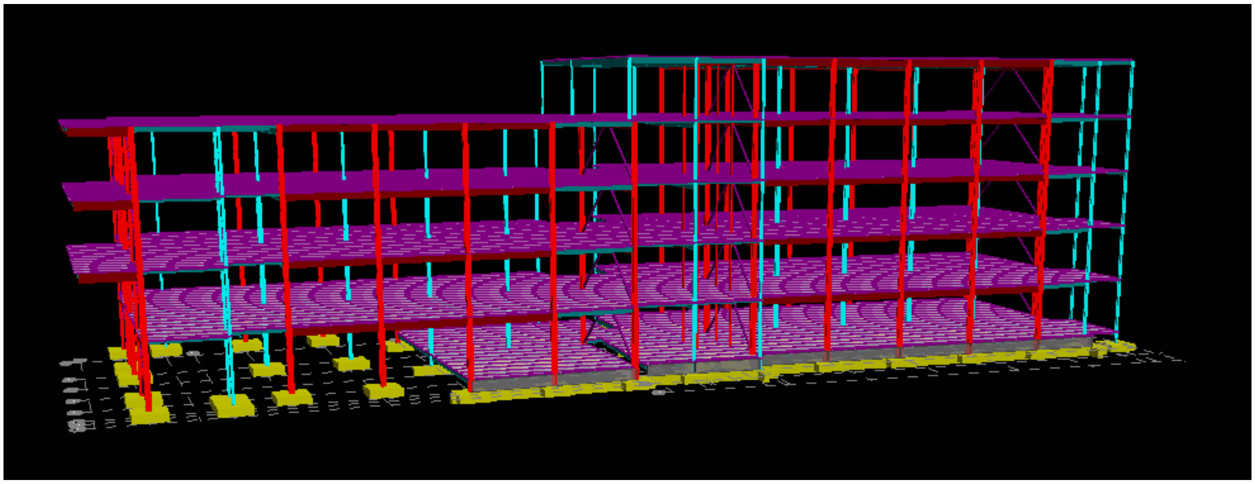


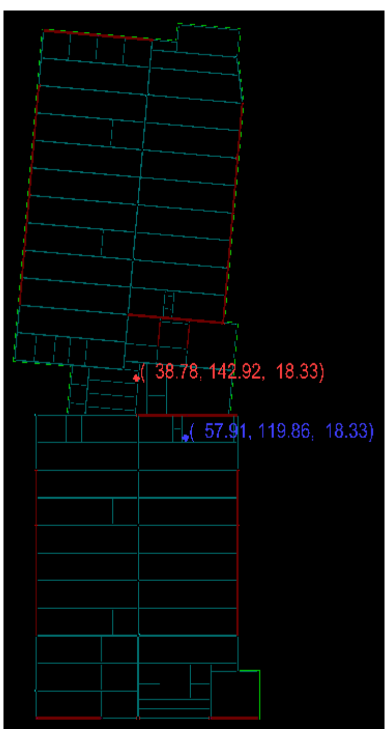
Figure 21: RAM Model (East Direction)



Center of Mass and Center of Rigidity

The center of mass (COM) and the center of rigidity (COR) were determined for each diaphragm by RAM. After visual inspection, the locations were confirmed, and analysis of the model proceeded. Figure 22 shows the center of mass and center of rigidity for the second floor. The COM is represented by a red circle (38.76, 142.92), and the COR is represented by a blue circle (57.91,119.86).

Figure 22: Center of Mass and Center of Rigidity



Load Combinations:

The load combinations in ASCE 7-05 were considered in analysis. Figure 23 shows Table 2.3.2 from ASCE. Wind load cases were considered from the ASCE 7-05 Main Wind Force Resisting System method (Method 2). These can be found in Figure 24.

Figure 23: LRFD Load Combinations

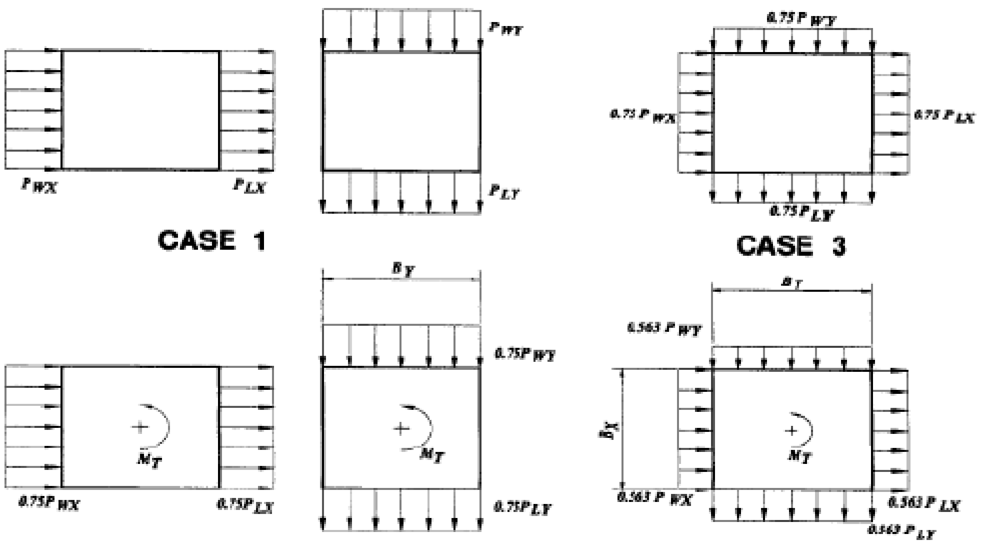
2.3.2 Basic Combinations. Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Earthquake was checked by RAM for an Sds of 0.233 as specified by the structural drawings. Wind controlled in both directions for every story.

Figure 24:

Main Wind Force Resisting System – Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



$M_T = 0.75\,(P_{WX}+P_{LX})B_X e_X$
$e_X = \pm\, 0.15\, B_X$

$M_T = 0.75\,(P_{WY}+P_{LY})B_Y e_Y$
$e_Y = \pm\, 0.15\, B_Y$

$M_T = 0.563\,(P_{WX}+P_{LX})B_X e_X + 0.563\,(P_{WY}+P_{LY})B_Y e_Y$
$e_X = \pm\, 0.15\, B_X$ $e_Y = \pm\, 0.15\, B_Y$

CASE 2

CASE 4

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   - $e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T: Torsional moment per unit height acting about a vertical axis of the building.

Gravity System Results

The gravity force resisting system was analyzed using RAM Steel Beam and RAM Steel Column. Column sizes were continuous up to the fourth floor where they were spliced. An elevation view of interior column sizes running down the middle of Wing C is shown in Figure 25. Gravity column interaction can be seen in Figure 26, which was controlled by the first floor and fourth floor. Sizes of beams in the gravity system can be seen in Figure 27.

Figure 25: Interior Column Sizes

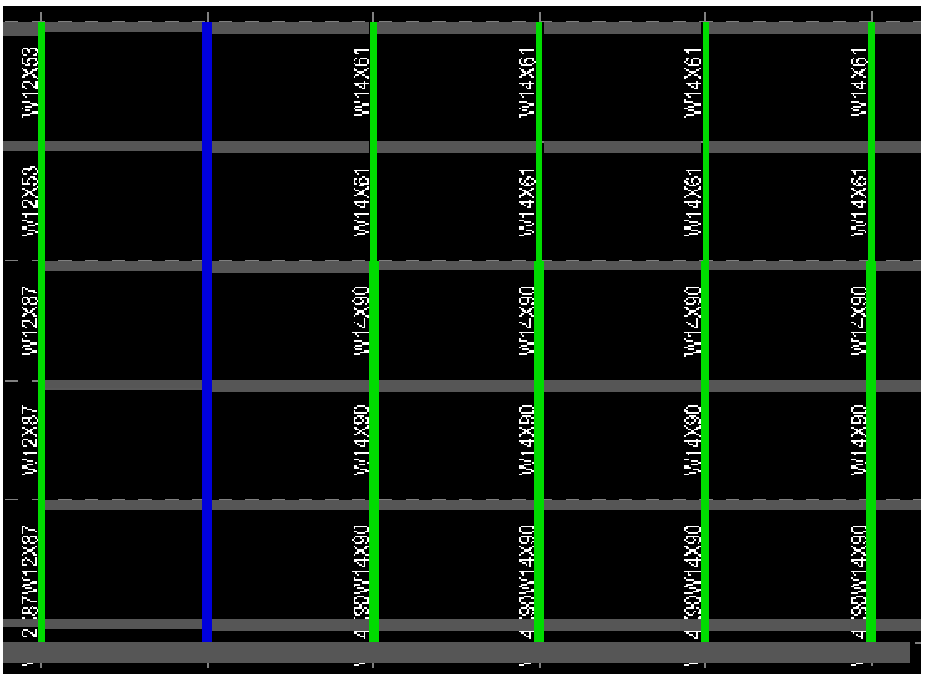


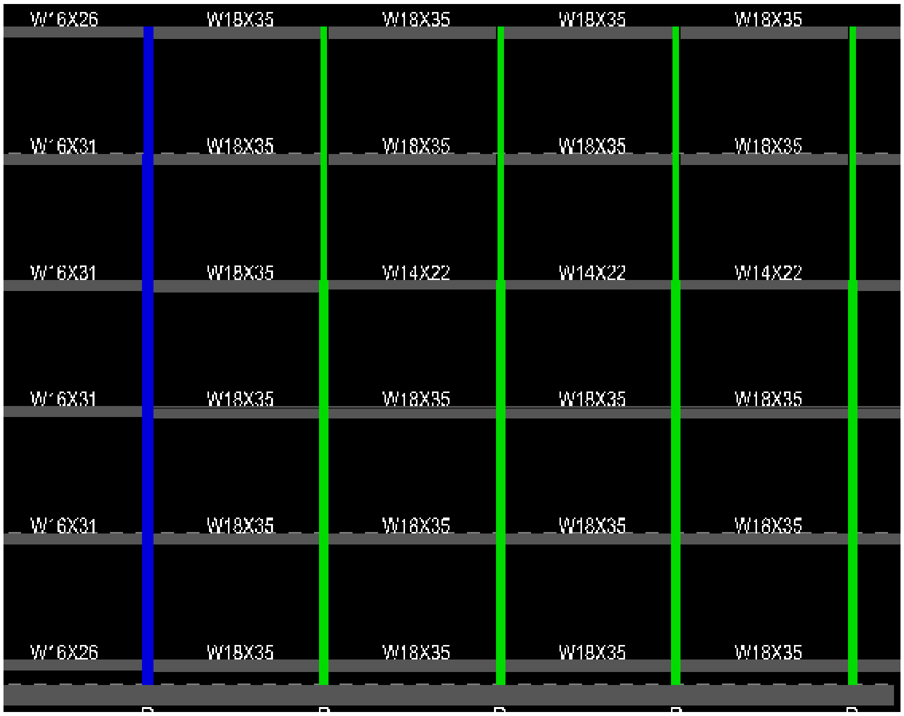
Figure 26: Gravity Column Interaction

INTERACTION EQUATION

Pa/(Pn/1.67)) = 0.338

Eq H1-1a: 0.338 + 0.020 + 0.074 = 0.432

Figure 27: Gravity System Beam Sizes



Lateral System Results

Story Shears

Story shears were checked using RAM based on the controlling wind cases. The maximum story shears in the x direction are in Figure 28, and the maximum story shears in the y direction are shown in Figure 29. It should be noted that story 1 is a section of Wing C short in height directly above the foundation wall.

Figure 28: Maximum Story Shears in the X Direction

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Story 6	19.80	19.80	0.00	0.00
Story 5	74.02	54.22	0.00	0.00
Story 4	141.45	67.43	0.00	0.00
Story 3	206.39	64.94	0.00	0.00
Story 2	264.13	57.75	0.00	-0.00
Story 1	216.42	-47.71	-3.26	-3.26

Figure 29: Maximum Story Shears in the Y Direction

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Story 6	0.00	0.00	12.05	12.05
Story 5	0.00	0.00	35.00	22.95
Story 4	0.00	0.00	57.08	22.08
Story 3	0.00	0.00	77.48	20.40
Story 2	-0.00	-0.00	96.28	18.80
Story 1	1.76	1.76	71.71	-24.57

Overturning Moment (X Direction)

The shears for each story in the X direction were multiplied by the height of each story to produce a total overturning moment of 11,471 k-ft.

The resisting moment was calculated by multiplying the weight of the building by the eccentricity of the center of mass. From previous calculations, the effective building weight is 14,143 kips. The center of mass is 38.78 feet from the edge of the building. This results in a resisting moment of 548,465 kip-ft. This is enough to handle the overturning moment produced by the controlling wind case.

Overturning Moment (Y Direction)

The shears for each story in the Y direction were multiplied by the height of each story to produce a total overturning moment of 4,473 k-ft.

The resisting moment was calculated by multiplying the weight of the building by the eccentricity of the center of mass. From previous calculations, the effective building weight is 14,143 kips. The center of mass is 143 feet from the edge of the building. This results in a resisting moment of 2,022,449kip-ft. This is enough to handle the overturning moment produced by the controlling wind case

Lateral System Response

After modeling the lateral system in RAM Frame, forces in each member were checked by the program. All applicable ASCE load cases and combinations were considered along with any applicable AISC Standard Provisions for steel design. Figure 30 shows how much capacity of each member is being utilized. With many of the members at 40% or less capacity, drift values should be well controlled. The members with the highest capacities utilized are the braces in the braced frames, with none exceeding 95%. The members closest to capacity are shown in Figure 31.

Figure 30: Lateral System Results

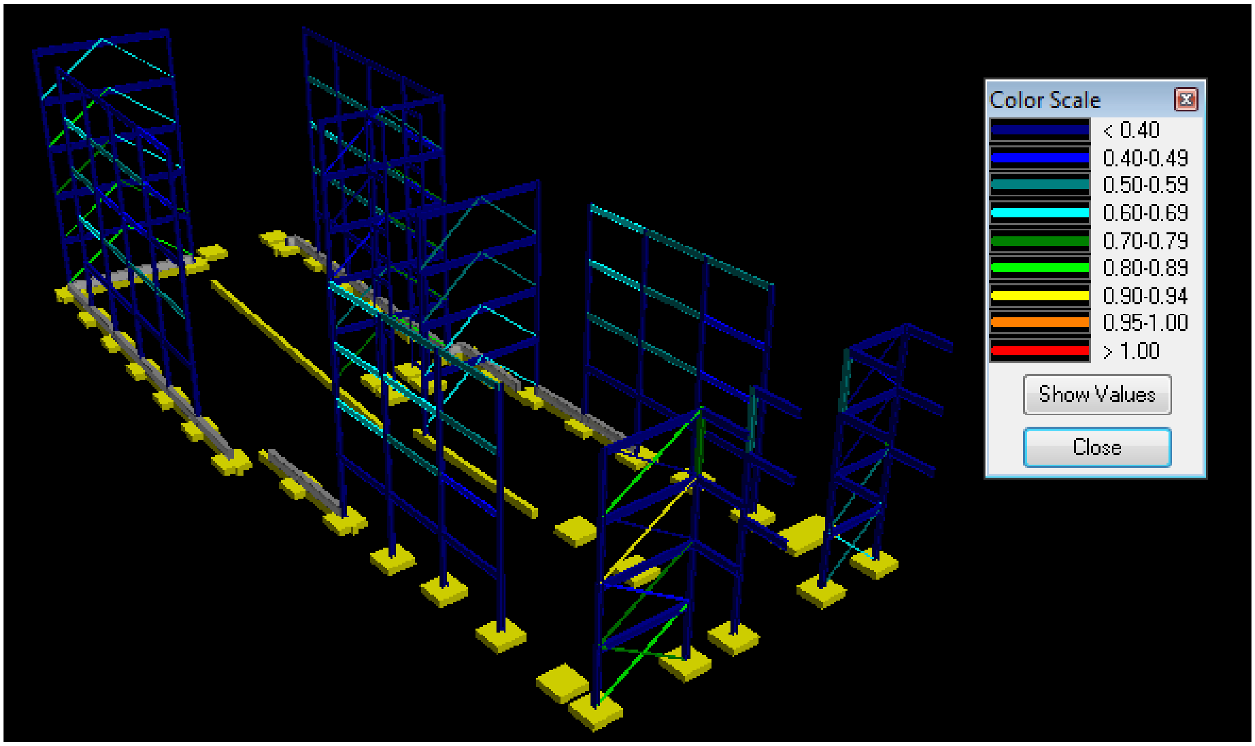
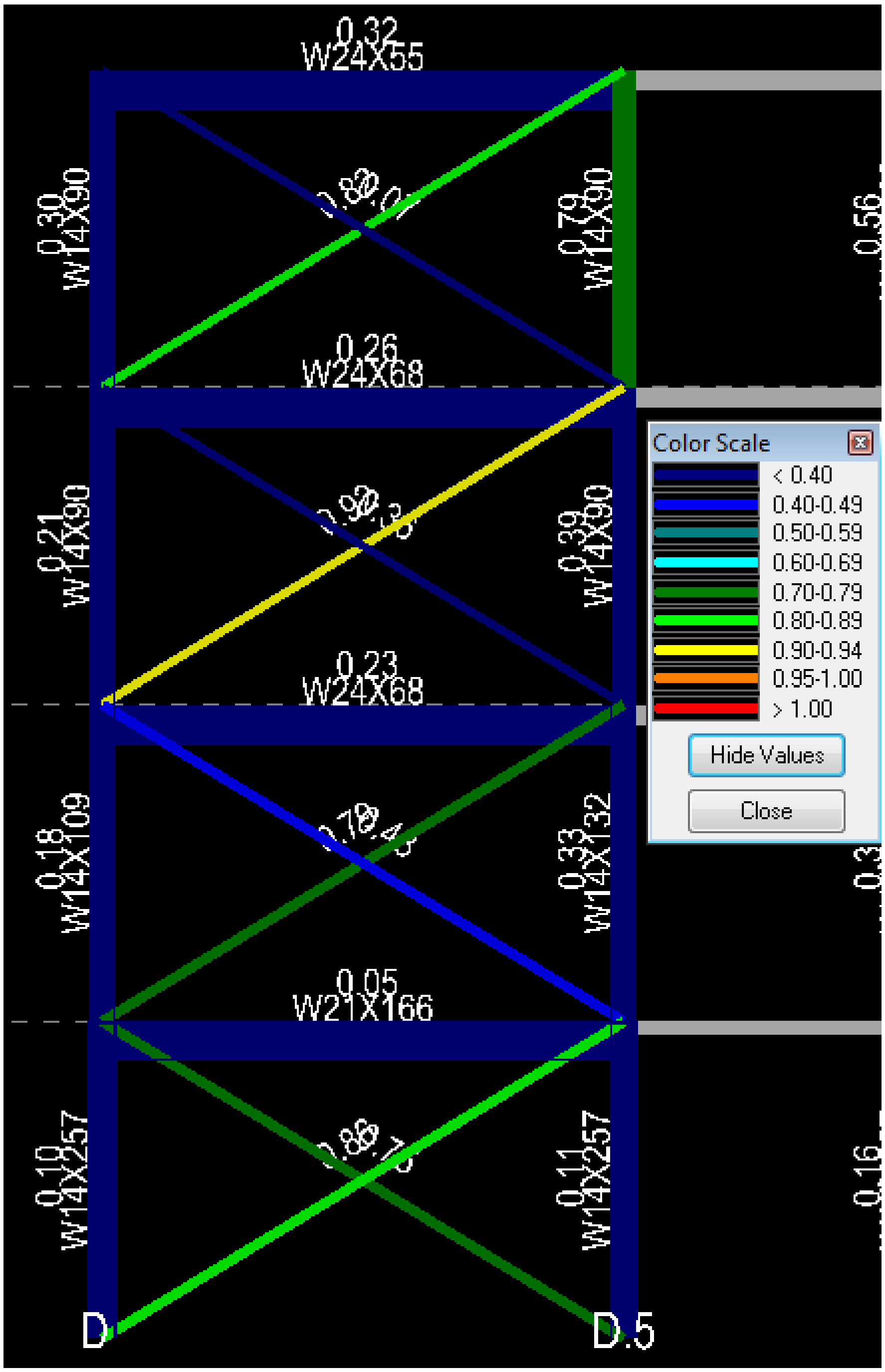


Figure 31: Most Utilized Members



Maximum Story Drifts

RAM was used to determine the story drifts based on the controlling wind cases. Four points were chosen as control points to establish displacement and drift data show in Figure 32 as blue dots. Tables 10-13 show the results and compare to allowable drifts of h/400 as per ASCE 7-05. Some of the frames do not extend to level 6 and are marked as “N/A”. The story drifts passed all acceptable drift limits based on the RAM output and an acceptable drift of h/400.

Figure 32: Location of Control Points for Drift Analysis

Table 10: Maximum Lateral Displacements and Story Drifts (Column A-1)

Maximum Wind Story Drift, N-S Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column A-1				
	6	.0567	2.32	OK
	5	-.0962	1.86	OK
	4	-.1140	1.4	OK
	3	-.1193	0.94	OK
	2	-.1119	0.48	OK
	1	.0102	0.09	OK
Maximum Wind Story Drift, E-W Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column A-1				
	6	-.2106	2.32	OK
	5	-.6055	1.86	OK
	4	-.641	1.4	OK
	3	-.518	0.94	OK
	2	-.199	0.48	OK
	1	.011	0.09	OK

Table 11: Maximum Lateral Displacements and Story Drifts (Column B-6)

Maximum Wind Story Drift, N-S Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column B-6				
	6	-.0685	2.32	OK
	5	-.070	1.86	OK
	4	-.088	1.4	OK
	3	-.0923	0.94	OK
	2	-.082	0.48	OK
	1	-.008	0.09	OK
Maximum Wind Story Drift, E-W Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column B-6				
	6	-.221	2.32	OK
	5	-.60	1.86	OK
	4	-.638	1.4	OK
	3	-.514	0.94	OK
	2	-.198	0.48	OK
	1	-.011	0.09	OK

Table 12: Maximum Lateral Displacements and Story Drifts (D.5-14)

Maximum Wind Story Drift, N-S Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column D.5-14				
	6	N/A	N/A	N/A
	5	-.0619	1.86	OK
	4	-.0767	1.4	OK
	3	-.0827	0.94	OK
	2	-.070	0.48	OK
	1	N/A	N/A	N/A
Maximum Wind Story Drift, E-W Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column D.5-14				
	6	N/A	N/A	N/A
	5	-.604	1.86	OK
	4	-.639	1.4	OK
	3	-.515	0.94	OK
	2	-.208	0.48	OK
	1	N/A	N/A	N/A

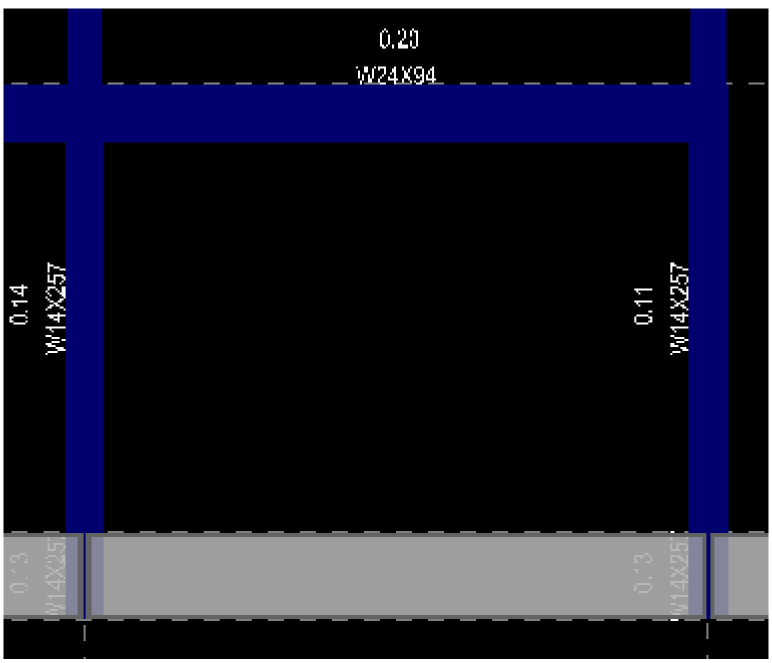
Table 13: Maximum Lateral Displacements and Story Drifts (E.5-12)

Maximum Wind Story Drift, N-S Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column E.5-12				
	6	N/A	N/A	N/A
	5	-.0607	1.86	OK
	4	-.0737	1.4	OK
	3	-.0787	0.94	OK
	2	-.0674	0.48	OK
	1	N/A	N/A	OK
Maximum Wind Story Drift, E-W Direction				
	Story	**Story Drift (in)**	**Allowable Drift (in)**	**Adequacy**
Column E.5-12				
	6	N/A	N/A	N/A
	5	-.602	1.86	OK
	4	-.636	1.4	OK
	3	-.512	0.94	OK
	2	-.207	0.48	OK
	1	N/A	N/A	OK

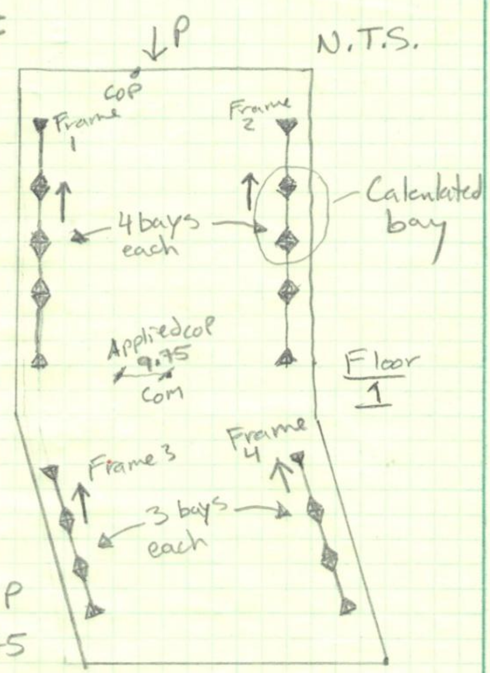
Lateral Spot Checks

Lateral spot checks were performed in a typical 1st floor moment frame, shown below in Figure 33. Capacity percentages are shown as decimal values next to their respective member sizes. These values are relatively low because the structure was designed for drift control. A braced frame is shown with similar information in Figure 34. Hand calculations show higher loads from a more conservative analysis and inclusion of the mechanical penthouses in wind calculations. Members were confirmed to meet design criteria for strength and serviceability. Columns and beams in the moment and braced frames have relatively low interaction fractions, because they were designed for drift control. Hand calculations are shown on the following pages.

Figure 33: Typical Moment Frame



Moment Frames:



- Neglected 5° turn in building
- All bays are identical lengths & stiffnesses
- Actual e = C.O.M. − C.O.R.
  = 43.34 − 38.54
  = 4.8'

15% of width
= .15(86.7) = 13'

$M_T = \begin{cases} 4.8P \\ .75(13)(P) = 9.75P \end{cases}$

larger ∴ Use e = 9.75

$$\left[2(1+2+2+1) + 2(1+2+1)\right]p = P$$

↑4 bays   ↑3 bays   ↑p for 1 bay

$$20p = P$$

Target bay takes $\frac{2}{20}P = \frac{1}{10}P$

Frame 2 takes $\frac{6}{20}P = \frac{3}{10}P$

P calculated in Tech 1 for each story.

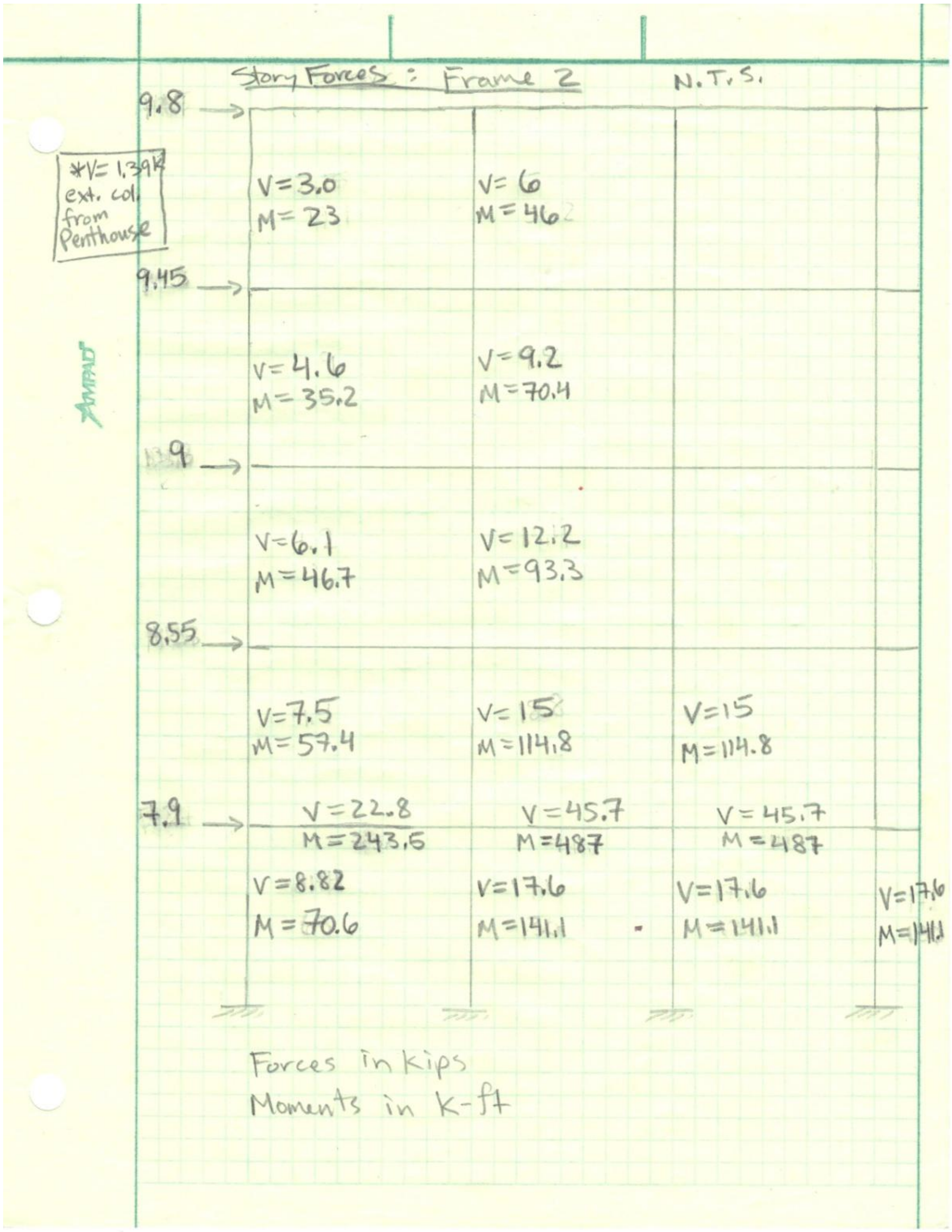
P for story 1 = 171 k

171 (9.75) = 1667 k-ft

$\Sigma d_i^2 = 1485 + 2320 = 3805$

$V_t = \frac{1667}{3805} = 0.438\,(86.7 - 38.54)$

= 21 k for Frame 2 & 4 Combined


Story Forces : Frame 2
N.T.S.
9.8
*V= 1.39k ext. col. from Penthouse
V=3.0
M=23
V=6
M=46
9.45
V=4.6
M=35.2
V=9.2
M=70.4
9
V=6.1
M=46.7
V=12.2
M=93.3
8.55
V=7.5
M=57.4
V=15
M=114.8
V=15
M=114.8
7.9
V=22.8
M=243.5
V=45.7
M=487
V=45.7
M=487
V=8.82
M=70.6
V=17.6
M=141.1
V=17.6
M=141.1
V=17.6
M=141.1
Forces in kips
Moments in k-ft

Torsional Shears

Interior col. takes $\frac{2}{10}$ or $1/5$ of 21k

$1/5(21) = 4.2$ k additive (Frame 2)

$6/10 = 3/5$ for Frame 2 total

$= 12.6$ k

Column Design: Stability Column

#stability = 9 # leaning = 5

- $\ell/400$ for drift control $\ell/400 = \frac{16(12)}{400} = 0.48''$

$$9 I_{req} = \frac{P\ell^3}{3E\Delta}$$

P = story shear for frames 2 & 4

$= (85.5 + 21)(1.6) = 170.4$ k

$$9 I_{req} = \frac{170.4(16)^3(1728)}{3(29000)(.48)}$$

$I_{req} = 3209$

Try W 14x 257 ($I_x = 3400$)

$$\Delta_{1ST} = \frac{35(16)^3(1728)}{3(29000)(3400)} = 0.84''$$

Direct Analysis Method:

Lateral Load $= (17.6 + 4.2)(1.6) = 35$ K

$y_i = 45.6(12) + 22.8(2) = 593$ k

Int Ext

$P_{nt} = 279$ k

$\Delta_{1ST} = 0.84''$ w/o reduced stiffness

$L = 16'$

* Assume $B_2 \leq 1.7$

$H_{max} = \begin{cases} 35 \text{ k} \\ N_i = .002(1)(4212) = 8.4 \text{ k} < 35 \end{cases}$

$M_{nt} = 0$
$P_{lt} = 0$
$M_{lt} = 35(16) = 560 \text{ k-ft}$

Determine B_2:

$P_{story} = 593 \text{ k}$
$P_{stability\ col.} = 9(45.6) = 410$
$R_m = 1 - .15 \frac{410}{593} = 0.9$
$P_{e\,story} = \frac{.9(35)(16)(12)}{0.84/.8} = 5760 \text{ k}$

Assumes $\alpha P_r \leq .5 P_y$

$B_2 = \frac{1}{1 - \frac{593}{5760}} = 1.11 \leq 1.7$

$P_r = 279 + 1.1(0) = 279 < .5(50)(75.6) = 1890 \text{ k}$ OK

$M_r = B_1(0) + 1.11(560) = 622 \text{ k-ft.}$

$P_c = 2900 \text{ k}$ for $K_yL_y = 16'$
M_c: $C_b = 1.67$; $L_b = 16'$
$\phi M_p = 1830 \text{ k-ft.}$ (T 3-6)

Interaction:
$\frac{279}{2900} = 0.096 < .2 \therefore$ H1-1b

$$.5\left(\frac{279}{2900}\right)+\left(\frac{622}{1830}\right)=0.39 < 1.0$$

OK

Use W 14x257 on bottom floor

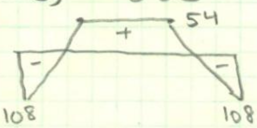
* Designed for drift control

Girder Design: Moment Frame

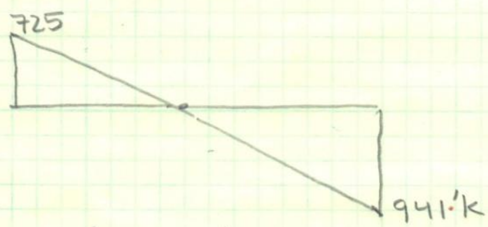
From torsion

From wind load: $1.6(487 + 33.6) = 833'k$

From gravity: $1.2D + .5L =$



Total Moment



Span = 21.33', but braced every 7.11' by beams

From (T-3-10) W24x94 has moment capacity of $\phi M_n = 951'k$ for $L_b = 7.11'$

Use W24x94 for girder in moment frames

$I_x = 2700 in^4$

Check $\Delta_{LL} = \ell/360 = 21.33(12)/360 = 0.711''$

$75 psf (21.33) = 1600 plf$; $\frac{1600(7.11)}{1000} = 11.38 k$

trib width

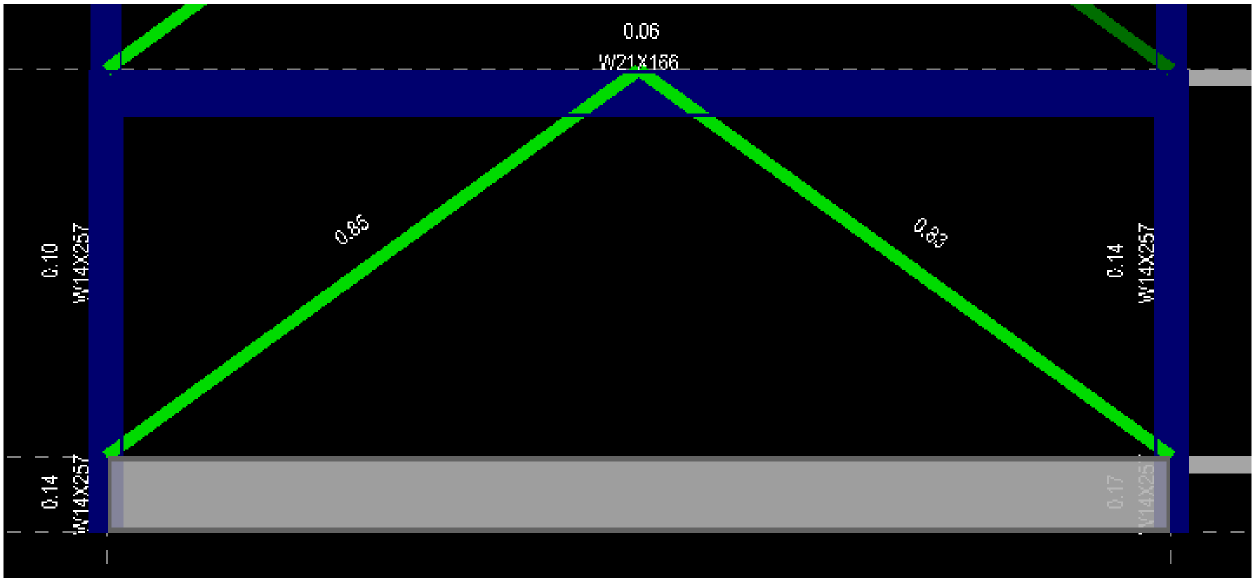
For 2 point loads, $\Delta_{max} = \frac{P\ell^3}{28EI}$

$\Delta_{max} = \frac{11.38(21.33)^3(144)}{28(29000)(2700)} = 0.007'' < .711''$ OK

Check $\Delta_{TL} = \frac{22.8(21.33)^3(144)}{28(29000)(2700)} = 0.015$

$\Delta_{TL} = .015 < \ell/240 = 1.07''$ OK

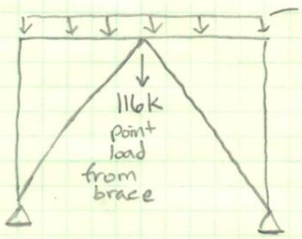
Figure 34: Braced Frame



Beam Design: Braced Frame (1.2D + .5L)

N.T.S.

$w = 152\left(\frac{7.11}{2}\right) = 540.4$ plf



① $\frac{w\ell^2}{8} = \frac{540.4\,(42.25)^2}{8\,(1000)} = 120.6'k$

② $\frac{P\ell}{4} = \frac{116\,(42.25)}{4} = 1225.3'k$

Total Moment = 1345.9'k

From (T-3-10) for $L_B = 21.125'$

$\phi M_n = 1395$ for W 21x166

$I_x = 4280\ in^4$

Check Δ_{TL} because of large wind component

$\Delta_1 = \frac{5\,(540.4)(42.25)^4(1728)}{384\,(29000)(4280)(1000)} = 0.312''$

$\ell/240 = \frac{42.25\,(12)}{240} = 2.11''$

$\Delta_2 = \frac{P\ell^3}{48EI} = \frac{116\,(42.25)^3(144)}{48(29000)(4280)} = 0.211$

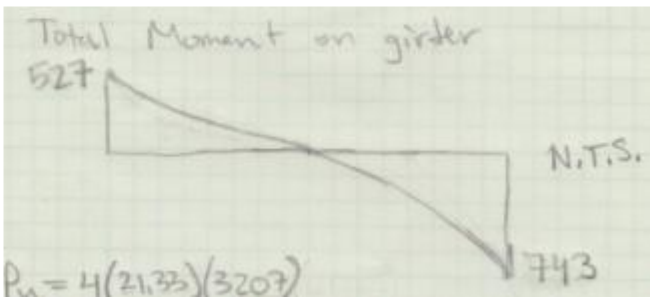
$.211 + .312 = .523'' < \ell/240$ OK

Use W 21x166 for beam in braced frames

Moment Connection

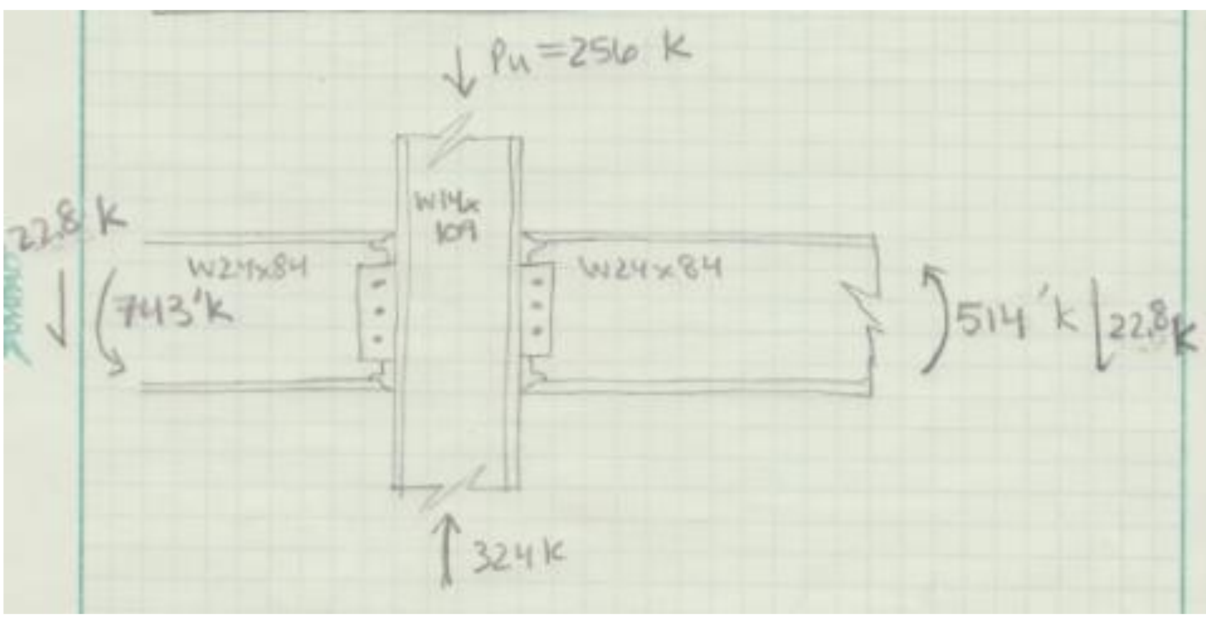
The main controlling load combination for the lateral system in the building was 1.2D +1.6W +.5L + .5S. The structure was evaluated for wind loads using the portal method of frame analysis. All calculations can be found in full in Appendix A-4. A plan view of the arrangement of moment frames can be found in Figures 13 and 14. It was quickly found by visual inspection that an interior moment frame in Wing C, the five story portion, would produce the highest load. The required minimum eccentricity of 5% for wind was exceeded by the real eccentricity of the building, so e was established to be 4.34'. After applying the eccentricity, the total negative moment on the girder was found to be -734'-k. The diagram is in Figure 35.

Figure 35: Girder Moment Diagram



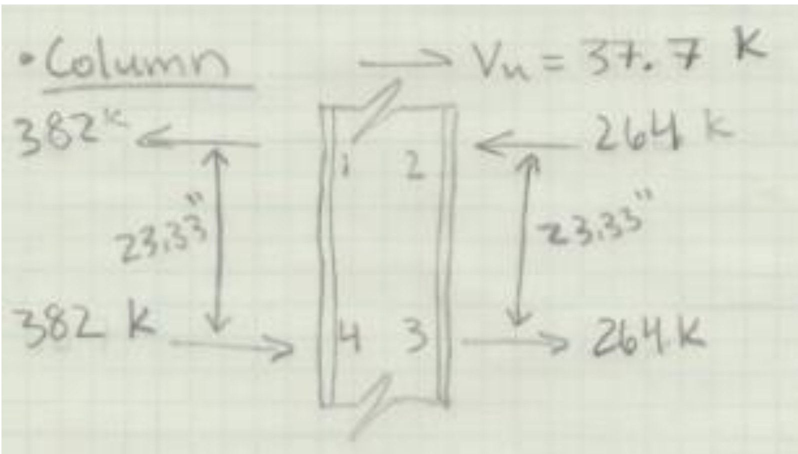
A "flange welded/ web bolted" moment connection was selected for design. The loading is shown in Figure 36.

Figure 36: Moments on Connection



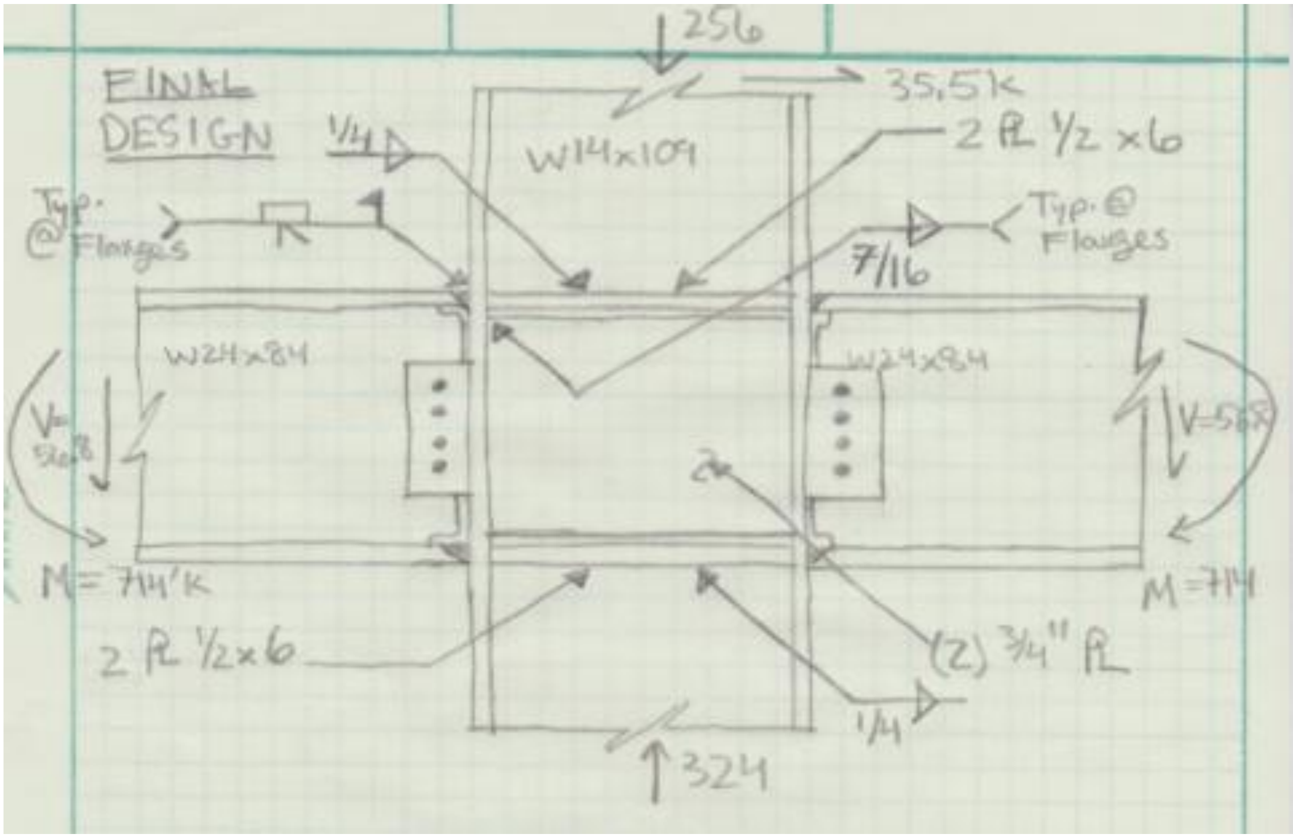
The girder flanges received full penetration welds with backing bars. These are the only field welds on this connection, and they are necessary for erection purposes. All applicable limit states were evaluated, including panel zone shear, which is shown in Figure 37.

Figure 37: Panel Zone Shear



The column required both full depth stiffeners and doubler plates. If a larger column section were selected, it might be possible to eliminate some of the column reinforcing that was necessary. The final connection is shown in Figure 38.

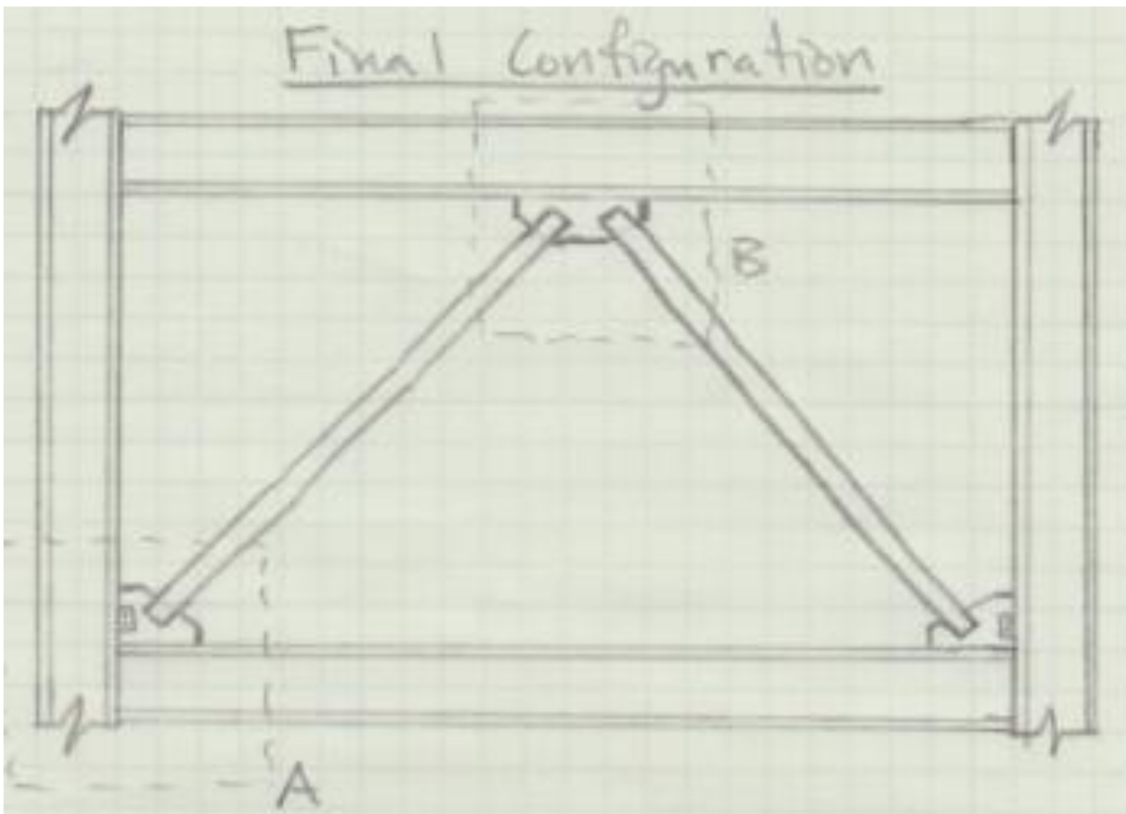
Figure 38: Moment Connection



Braced Frame Connection

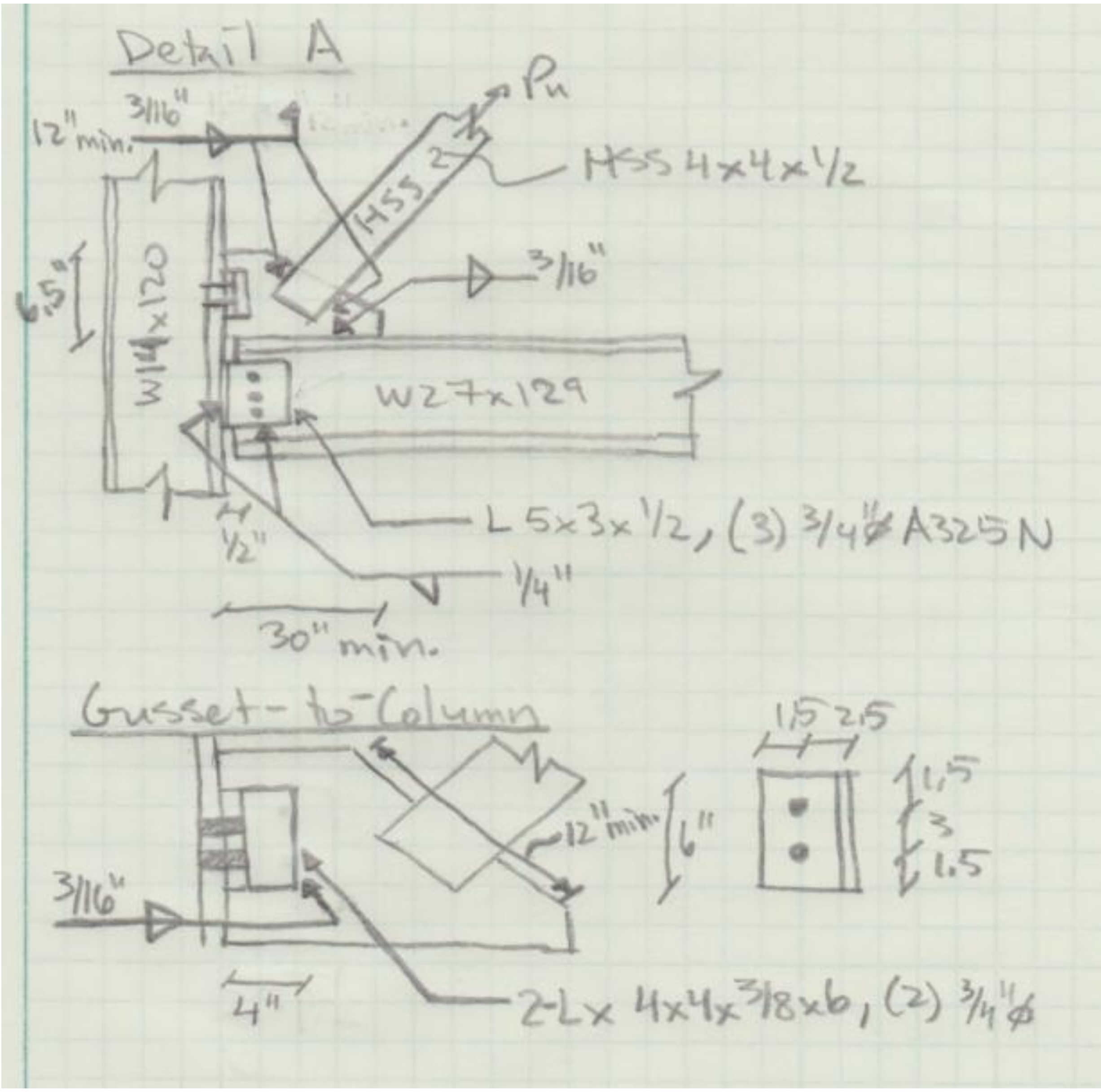
Forces were applied to the braced frame in a similar fashion as they were to the moment frame. The braces are also shown in the plan views of Figures 13 and 14. The natural eccentricity of the building was not greater than 5%, so an e_{min} was applied to the resultant wind load. Initially a concentrically braced frame was selected for design. However, this would have required one side of the gusset plate to be over 60", due to the large girder span of 42.25'. A gusset plate that large seemed impractical for this particular design, so an eccentrically braced frame was selected to change the angle of the brace. Figure 39 shows a sketch of this braced frame. Based on tension in the brace, an HSS 4x4x1/2 was selected as the bracing member.

Figure 39: Braced Frame

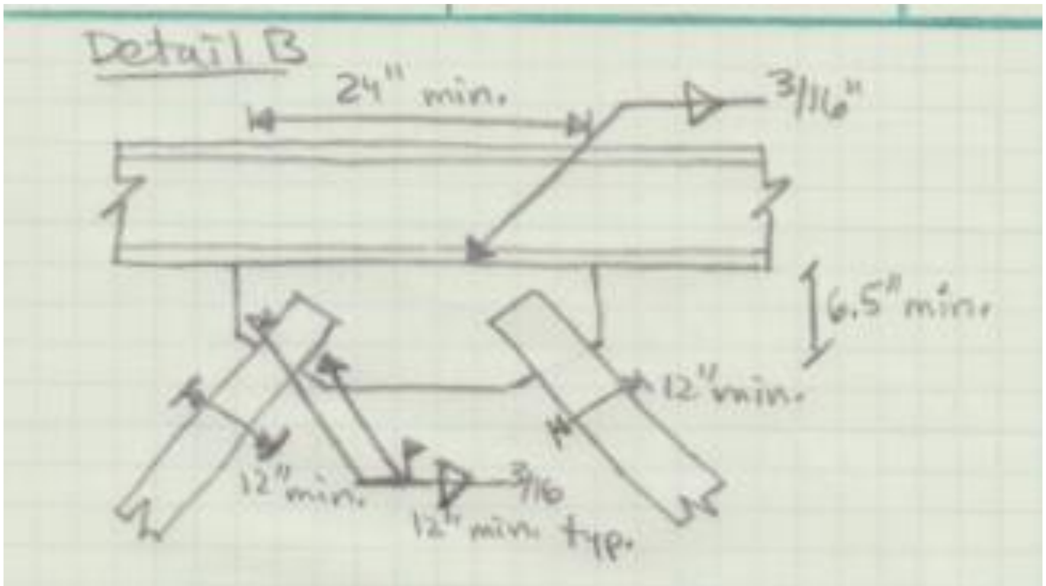


Details for the braced connection are shown in Figures 40 and 41. The corner gusset plate is ½" thick with side dimensions of 6.5" and 30" and an angled dimension of 12" to meet the HSS. The HSS is welded to the gusset plate, because there are special provisions for bolting HSS tubing that would have complicated the design. Double angles connect the gusset to the column with two rows of single bolts on either side. All other connections are welded. The HSS welds must be done in the field for erection purposes.

Figure 40: Detail A



<u>Figure 41: Detail B</u>



Results of Redesign

With the newly designed moment connections and braced connections in place, the amount of connections in the lateral system has decreased significantly. This has not eliminated an element of redundancy in the system. The building is just as capable as before in resisting lateral loads. As a consequence, the columns and beams which make up the lateral force resisting system had to be sized up for strength, serviceability, and to resist story drift. There are four braced frame systems and 2 sets of 7 moment frames. Hopefully this system is not only simpler to erect, but also more cost-effective than the previous system. Unfortunately without shop drawings or cost information from the steel fabricator, it is not possible to determine exactly what impact these connections have on the cost or schedule.

Breadth 1: Electrical

The emergency egress system includes emergency lighting and fire alarms. Emergency lighting must be provided in every-other luminaire in the corridors, as well as in lobby areas and stairwells. This will allow for a well lit means of egress in the case of a power failure. Fire alarms shall be spaced at a maximum distance of 40 ft. in corridors and shall be placed in any room which directly exits into a corridor. Emergency exit signs shall be posted at the ends of corridors or at a maximum of 50 ft. apart. Fire extinguishers shall be placed at any location where there is a fire alarm. Fire hoses shall be placed at every-other floor in the stairwell connected to the standpipe.

A typical laboratory room panelboard schedule was made through consultation with the electrical contractor. Figure 42 shows this schedule.

Figure 42: Typical Laboratory Panelboard

BRANCH CIRCUIT PANELBOARD SCHEDULE

Typical Lab	MOUNTING: SURFACE	X	MAIN LUGS ONLY	125 AMP MAIN CB
120/208V, 3 PHASE, 4 WIRE	FLUSH		SHUNT TRIP MAIN	150 AMP BUS
10 ,000MIN A.I.C. SYM	IN MCC		FEED THRU LUGS	GROUND BUS: X
NEUTRAL: 200%	NUMBER OF POLES:	42		ISOLATED GROUND BUS:

CKT No.	LOAD	TRIP (AMP)	KW / PHASE A	B	C	POLES		KW / PHASE A	B	C	TRIP (AMP)	LOAD	CKT No.
1	FUME HOOD RM 191	20	1.80			1	2	1.80			20	FUME HOOD RM 181	2
3	FUME HOOD RM 191	20		1.80		3	4		1.80		20	FUME HOOD RM 181	4
5	FUME HOOD RM 191	20			1.80	5	6			1.80	20	FUME HOOD RM 181	6
7	FUME HOOD RM 191	20	1.80			7	8	1.80			20	FUME HOOD RM 181	8
9	FUME HOOD RM 191	20		1.80		9	10		1.80		20	FUME HOOD RM 181	10
11	FUME HOOD RM 191	20			1.80	11	12			1.80	20	FUME HOOD RM 181	12
13	FUME HOOD RM 171	20	1.80			13	14	1.80			20	FUME HOOD RM 191A	14
15	FUME HOOD RM 171	20		1.80		15	16		1.50		20	ICE MAKER	16
17	FUME HOOD RM 171	20			1.80	17	18			0.50	20	SPARE	18
19	FUME HOOD RM 171	20	1.80			19	20	0.50			20	SPARE	20
21	FUME HOOD RM 171	20		1.80		21	22		1.80		20	EXT. BLUE LIGHTS	22
23	FUME HOOD RM 171	20			1.80	23	24				20	SPARE	24
25	SPARE	20	0.50			25	26	0.60			20	SPARE	26
27	SPARE	20		0.50		27	28		0.60		20	SPARE	28
29	SPARE	20			0.50	29	30			0.60	20	SPARE	30
31	SPARE	20	0.50			31	32				20	RM 181 GAS SHUT OFF	32
33	RM 191 GAS SHUT OFF	20				33	34		0.50		20	EAV (5)	34
35	SPARE	20				35	36				20	RM 171 GAS SHUT OFF	36
37	SIEMENS PANELS	20				37	38				20	SPARE	38
39	SPARE	20				39	40				20	SPARE	40
41	SPARE	20				41	42				20	SPARE	42
	SUBTOTALS		8.20	7.70	7.70			6.50	8.00	4.70		SUBTOTALS	
	TOTAL LOADS	14.7	KVA	PHASE A				DEMAND FACTOR				65%	
		15.7	KVA	PHASE B				DEMAND LOAD				27.82 KVA	
		12.4	KVA	PHASE C				LOAD X 1.25%				34.78 KVA	
	TOTAL CONN. LOAD	42.8	KVA					AMP				96.60	

Some additional estimates were made of the electrical system. These are rough estimates, but they generally reflect the capacity of the facility.

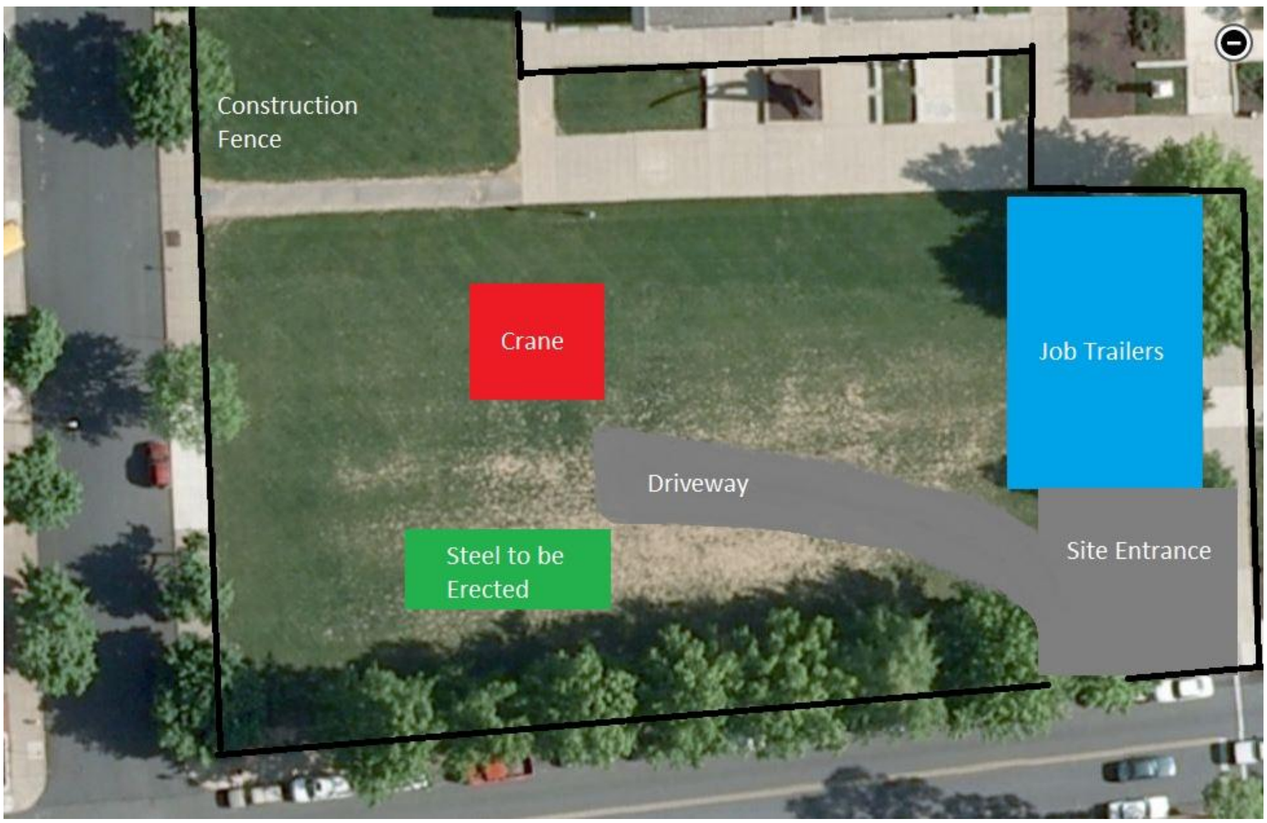
Electrical Systems:

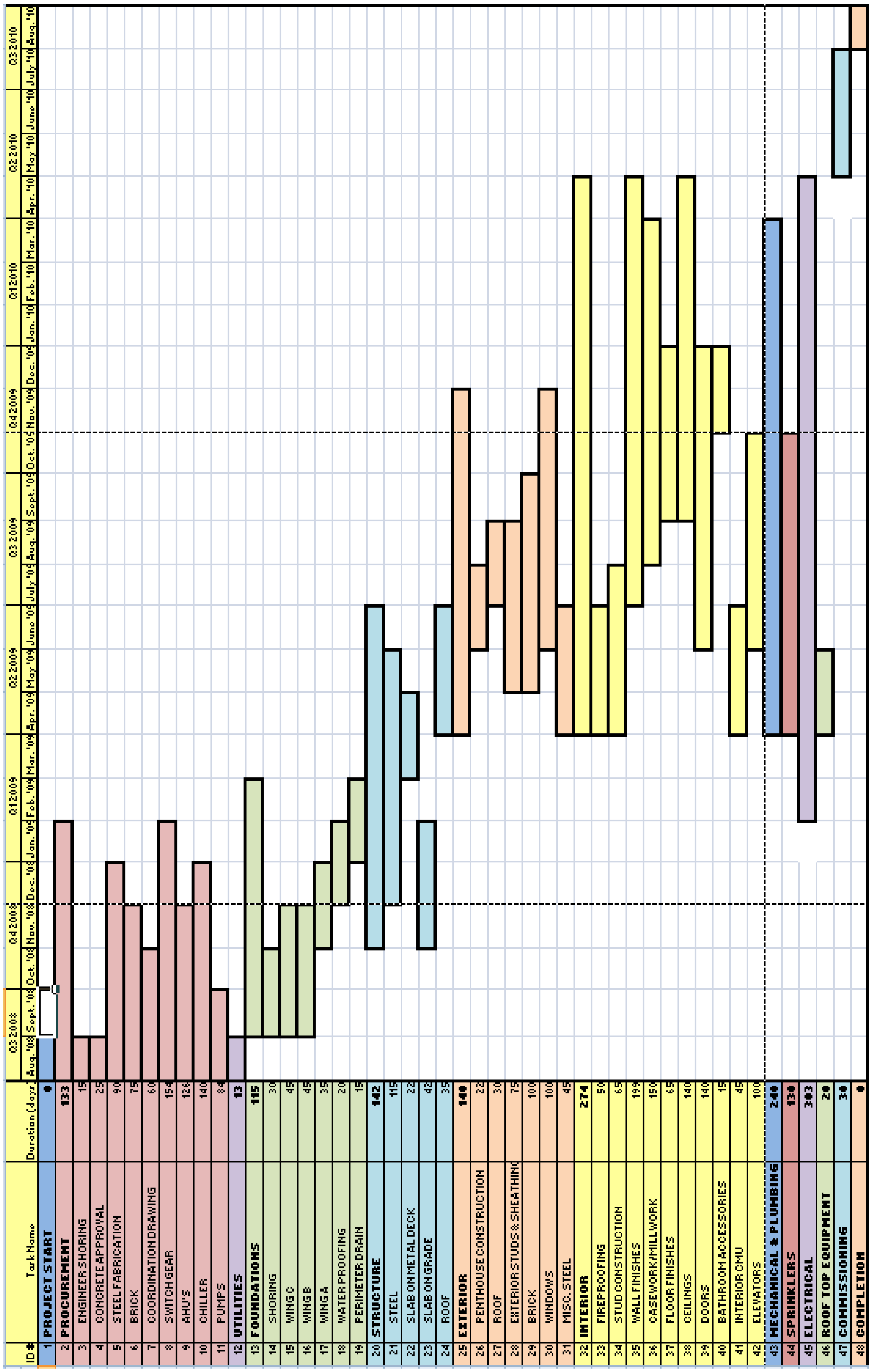
- 1500 KVA Service Transformer
- 480 / 277V 3-Phase 4-Wire Secondary Feed to 3000-amp Distribution Panel
- 2 - 150 KVA Emergency Generators
- 277V T8, T5 and Compact Fluorescent Light Sources with Ballasts

Breadth 2: Construction Management

A detailed construction schedule was produced with the help of a member of the construction team. Unfortunately the steel fabricator has gone out of business, so calculating a reduction in the cost of the project or the schedule was not possible. The schedule is located on the following page. One of the other construction management issues examined was the positioning of the crane in the site layout. Figure 43 shows this layout which will not change much over the course of the project. The site is relatively compact, and the crane should be able to reach Wing A, B, and C without moving much.

Figure 43: Crane Positioning and Site Layout





ID#
Task Name
Duration (days)
1 PROJECT START 0
2 PROCUREMENT 133
3 ENGINEER SHORING 15
4 CONCRETE APPROVAL 25
5 STEEL FABRICATION 90
6 BRICK 75
7 COORDINATION DRAWING 60
8 SWITCH GEAR 154
9 AHU'S 126
10 CHILLER 140
11 PUMPS 84
12 UTILITIES 13
13 FOUNDATIONS 115
14 SHORING 30
15 WING C 45
16 WING B 45
17 WING A 35
18 WATERPROOFING 20
19 PERIMETER DRAIN 15
20 STRUCTURE 142
21 STEEL 115
22 SLAB ON METAL DECK 22
23 SLAB ON GRADE 42
24 ROOF 35
25 EXTERIOR 140
26 PENTHOUSE CONSTRUCTION 22
27 ROOF 30
28 EXTERIOR STUDS & SHEATHING 75
29 BRICK 100
30 WINDOWS 100
31 MISC. STEEL 45
32 INTERIOR 274
33 FIREPROOFING 50
34 STUD CONSTRUCTION 65
35 WALL FINISHES 199
36 CASEWORK/MILLWORK 150
37 FLOOR FINISHES 65
38 CEILINGS 140
39 DOORS 140
40 BATHROOM ACCESSORIES 15
41 INTERIOR CMU 45
42 ELEVATORS 100
43 MECHANICAL & PLUMBING 240
44 SPRINKLERS 130
45 ELECTRICAL 303
46 ROOF TOP EQUIPMENT 20
47 COMMISSIONING 30
48 COMPLETION 0

Conclusion

This semester, a thorough undertaking examined significant changes to the lateral system, floor system, and steel connections. Research was performed on the impact of floor vibrations in a laboratory setting and ways to dampen vibratory effects. Design Guide 11 proved to be an invaluable resource for vibration design, and a floor was chosen to allow 400x magnification microscopes to be used. This floor was slightly heavier than the original floor, but it did not increase the story to story height. A major goal in designing the floor was not disrupting the architecture, and this was achieved.

A RAM model was made to size gravity and lateral force resisting members. These sizes were compared with hand calculations. Skills learned in AE 534 were used to create connections in the building for the new lateral system. All wind clips were eliminated from the system. A typical moment connection was designed and evaluated for the applied wind and gravity loads. Lastly, braced connections were designed in detail.

The two breadths were also investigated. Electrical loads were determined and then used to produce a panelboard schedule. Estimates were made on the capacity of the electrical system, and emergency lighting and fire alarm systems were specified. From a construction management perspective, a complete detailed construction schedule was created throughout the length of the project. A site plan with crane positioning was also drawn up.

The knowledge I have gained by working with a real building and real loads has been the highlight of my college career. Learning Design Guide 11 and applying it to my structure helped me learn more about vibration control. When working on a medical or laboratory building in the future, I will take a moment to consider the equipment in the building. If it is sensitive equipment, the owner may want to consider vibration control.

I have also discovered that making every column connection in the building a semi-rigid moment connection may not be the most economical or efficient option. There are other lateral systems which take longer to design, but they might result in a shorter erection schedule or a reduced cost.

Most of all, my senior thesis experience has taught me that there are options to most engineering problems. Patience and thoroughness in design and analysis can help to determine which options are most appropriate.

Appendix A-1: Existing Floor Plans

Figure 44: Structural Floor Plan of Wing B (Story 2)

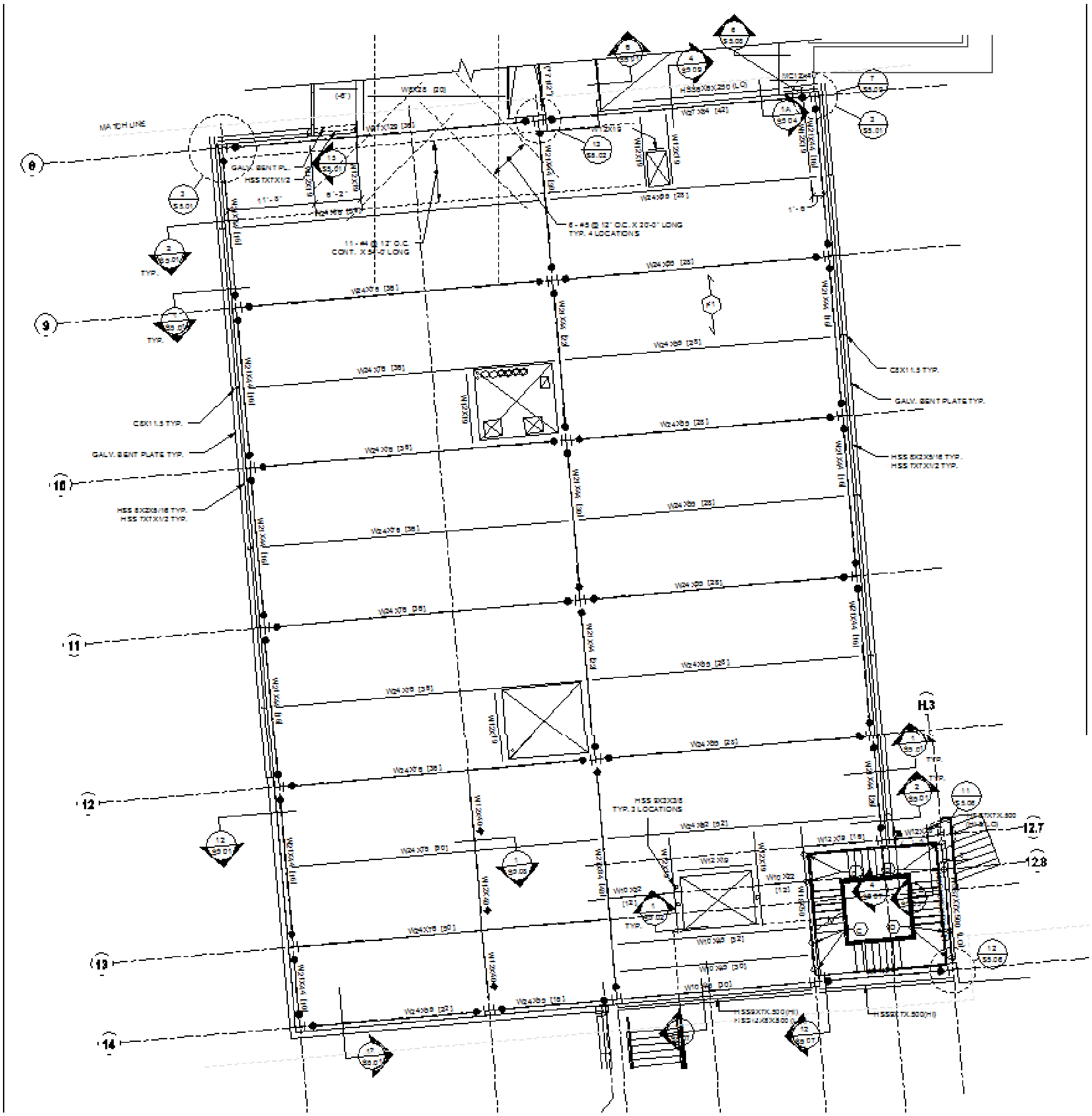


Figure 45: Structural Floor Plan of Wing C (Story 2)

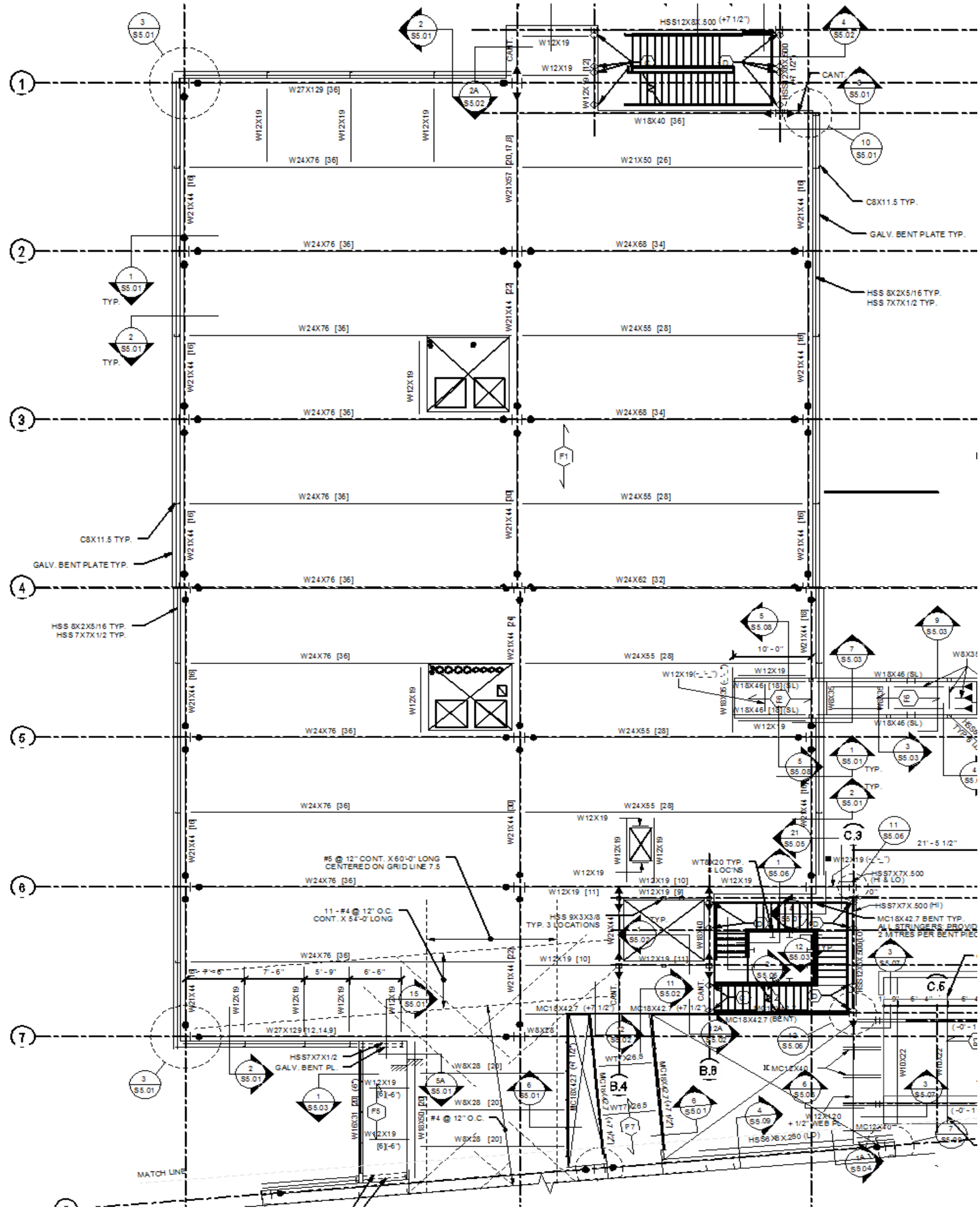


Figure 46: Typical Architectural Floor Plans

Courtesy BCJ Architects

Appendix A-2: Wind Calculations

TECH REPORT 1 | CALCULATIONS | WIND LOADS

- $V = 90$ mph (Figure 6-1b)
- Building Category = III (Table 1-1)
- Exposure = B (Urban/suburban) (6.5.6.2)

$$q_z = 0.00256\, K_z K_{zt} K_d V^2 I \qquad \text{(Eq. 6-15)}$$

- Determine z for top Level:

	E/W	N/S
$z =$	77'-4"	108'-4"

- Determine K_z for Roof Level: (Table 6-3)

	E/W	N/S	
$K_z =$	.92	1.01	
$K_{zt} =$	1.0	1.0	(6.5.7)
$K_d =$	.85	.85	

$I = 1.15$ (Category III) (Table 6-1)

	E/W	N/S
$q_z =$	16.03 psf	17.6 psf

- Since $T_o < 1$ sec $\rightarrow G = .85$ (6.5.8.1)

*Assuming #stories/10 = 6/10 = .6 = T_o

- $GC_{pi} = \pm .18$ fully enclosed buildings (Fig. 6-5)

$$p = q\,GC_p - q_i(GC_{pi})$$

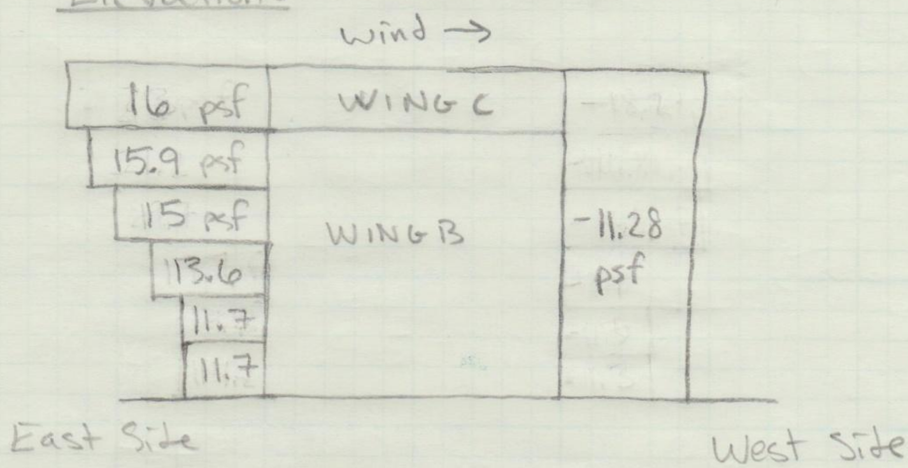
- East/West Pressures (Figure 6-6)

  Windward $C_p = .8$

  Leeward C_p is a function of L/B

  $$L/B = \frac{86.9}{275.3} = .316 \rightarrow C_p = -.5$$

- <u>Elevation:</u>

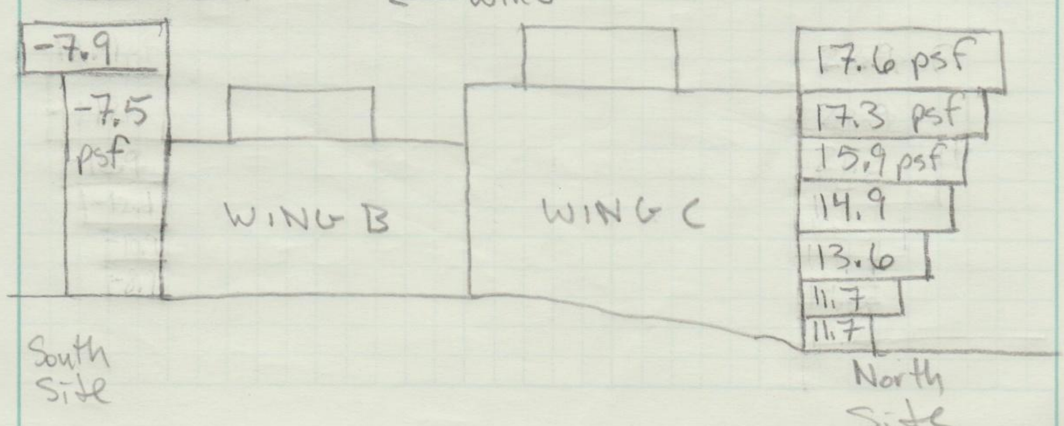


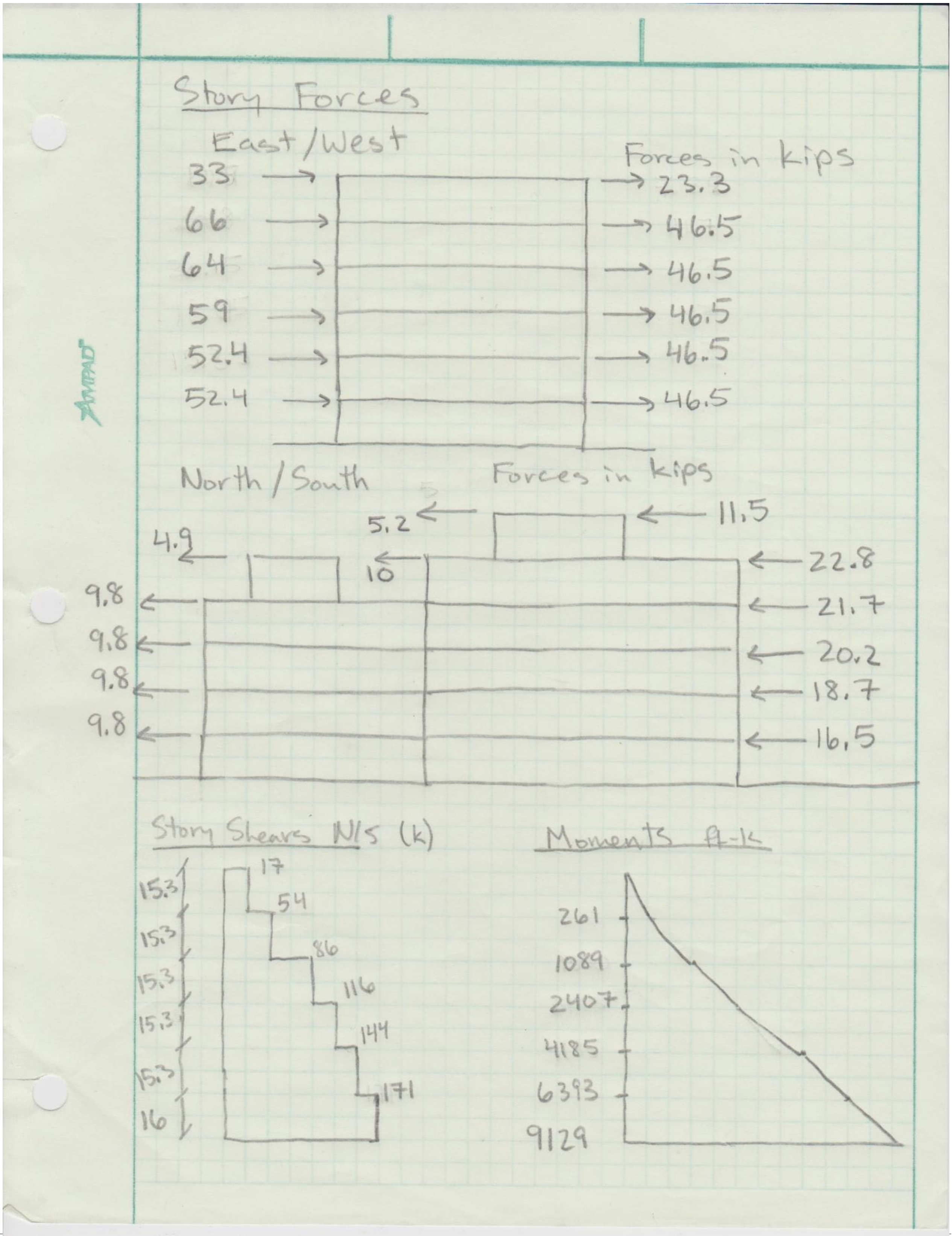
- Sample Calc (windward)

  $$p = 20.47(.85)(.8) - (20.47)(-.18)$$

  $$p = 17.6 \text{ psf}$$

- <u>Elevation:</u> ← wind Leeward $C_p = -.24$




Story Forces
East/West
Forces in kips
33
66
64
59
52.4
52.4
23.3
46.5
46.5
46.5
46.5
46.5
North/South
Forces in kips
5.2
11.5
4.9
10
22.8
9.8
9.8
9.8
9.8
21.7
20.2
18.7
16.5
Story Shears N/S (k)
15.3
15.3
15.3
15.3
15.3
16
17
54
86
116
144
171
Moments ft-k
261
1089
2407
4185
6393
9129

Appendix A-3: Vibration Resistant Floor Calculations

Figure 47: Deflection From a Footstep

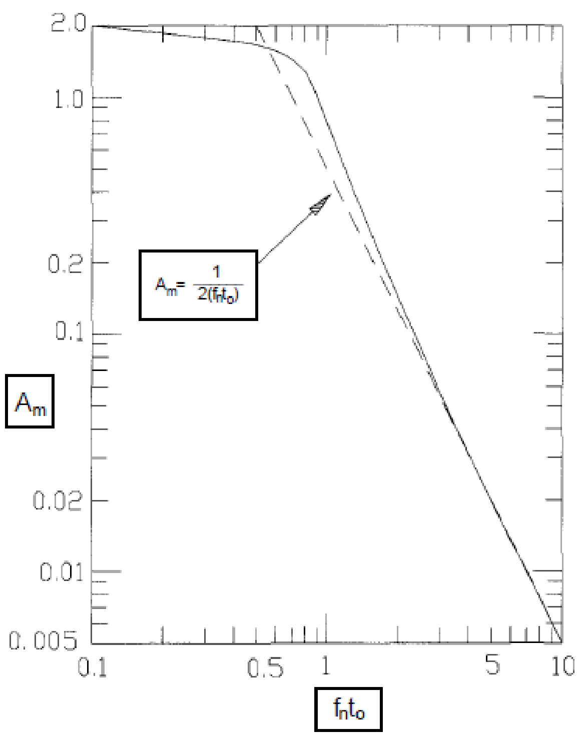


Fig. 6.5 Maximum dynamic deflection due to footstep pulse.

Courtesy AISC Design Guide 11

Figure 48: Footstep Parameters for Design

Table 6.2
Values of Footfall Impulse Parameters

Walking Pace steps/minute	F_m / W (from Figure 6.4)	F_m* kg (lb)	$f_o = 1 / t_o$, Hz (from Figure 6.4)	U_v kN·Hz² (lb·Hz²)
100 (fast)	1.7	1.4 (315)	5.0	110 (25,000)
75 (moderate)	1.5	1.25 (280)	2.5	25 (5,500)
50 (slow)	1.3	1.1 (240)	1.4	6.8 (1,500)

*For W = 84 kg (185 lb.)

Courtesy AISC Design Guide 11

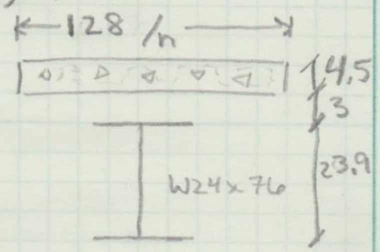
Floor Vibration Analysis

Existing Floor System: Beam

$w = 145$ pcf for NWC

$E_c = 33w^{1.5}\sqrt{f'_c} = 33(145)^{1.5}\sqrt{4000}$

$= 3644$ KSI



$b = \min \begin{cases} \text{beam spacing} = 10.67' \\ 0.4L_b = 0.4(41) = 16.4' \end{cases}$

$n = E_s / (E_c \times 1.35) = \dfrac{29{,}000}{(3644)(1.35)} = 5.89$

$\bar{y} = \dfrac{\Sigma Ay}{\Sigma A} = \dfrac{(21.7)(4.5)(2.25) + (22.4)\left(\frac{23.9}{2} + 7.5\right)}{(97.7 + 22.4)}$

$\bar{y} = 5.46''$

$I_j = \Sigma I + \Sigma Ad^2 = \dfrac{(21.7)(4.5)^3}{12} + 2100 + (21.7)(4.5)(5.46 - 2.25)^2$

$+ (22.4)(19.45 - 5.46)^2$

$I_j = 164.8 + 2100 + 1006.2 + 4384.1$

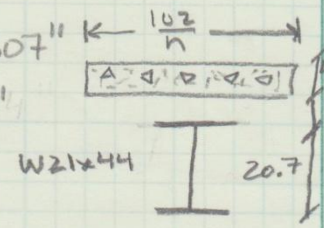
$I_j = 7655 \text{ in}^4$

$w_j = 10.67(\underbrace{75}_{LL} + \underbrace{75.3}_{slab} + \underbrace{20}_{SDL}) + 76 = 1893 \text{ plf}$

$\Delta_j = \dfrac{5(1.89)(42.25)^4(1728)}{384(29000)(7655)} = 0.61''$

Girder:

$b = \begin{cases} \text{girder spacing} = 42.25(12) = 507'' \\ 0.4L_b = 0.4(21.33)(12) = 102'' \end{cases}$ min



$b/n = 102/5.89 = 17.3$

$$\bar{y} = \frac{(17.3)(4.5)(2.25) + \left(\frac{17.3}{2}\right)(3)(6) + 13\left(7.5 + \frac{20.7}{2}\right)}{17.3(4.5) + \left(\frac{17.3}{2}\right)(3) + 13}$$

$$\bar{y} = \frac{562.9}{116.8} = 4.81''$$

$$I_g = \frac{(17.3)(4.5)^3}{12} + \frac{(8.65)(3)^3}{12} + 843 + (17.3)(4.5)(2.56)^2$$

$$+ (8.65)(3)(1.19)^2 + 13(13.04)^2$$

$$= 131.4 + 19.5 + 843 + 510.2 + 36.7 + 2211$$

$$I_g = 3752 \text{ in}^4$$

$$w_g = (1893/10.67)(38.375) + 44 = 6852 \text{ plf}$$

$$\Delta_g = \frac{5(6.852)(21.33)^4(1728)}{384(29000)(3752)} = 0.29''$$

Bay Frequency:

$$f_n = 0.18\sqrt{\frac{g}{\Delta_j + \Delta_g}} = .18\sqrt{\frac{386.4}{.61 + .29}} = 3.7 \text{ Hz}$$

Deflection for unit load:

$$\Delta_{oj} = \frac{1 \text{ lb.}(42.25)^3(1728)}{96(29\times10^6)(7655)} = 6.12\times10^{-6} \text{ in./lb.}$$

$$d_e/s = \frac{4.5 + 1.5}{(10.67)12} = 0.0469 \begin{matrix} > 0.018 \\ < 0.208 \end{matrix} \therefore \underline{OK}$$

$$L_j^4/I_t = \frac{(42.25\cdot12)^4}{7655} = 8.63\times10^6 \begin{matrix} > 4.5\times10^6 \\ < 257\times10^6 \end{matrix} \therefore OK$$

$$N_{eff} = 0.49 + 34.2\frac{d_e}{s} + (9\times10^{-9})\frac{L_j^4}{I_t}$$

$N_{eff} = .49 + 34.2(.0469) + (9\times10^{-9})(8.63\times10^{6})$
$= 2.17$

$\Delta_{jp} = \dfrac{6.12\times10^{-6}}{2.17} = 2.82\times10^{-6}$ in./lb.

$\Delta_{gp} = \dfrac{1\text{ lb.}(21.33)^3(1728)}{96(29\times10^{6})(3752)} = 1.6\times10^{-6}$ in./lb.

Mid-bay Flexibility:

$\Delta_p = \Delta_{jp} + \dfrac{\Delta_{gp}}{2} = 2.82\times10^{-6} + 0.8\times10^{-6}$

$\Delta_p = 3.62\times10^{-6}$ in/lb.

*Since f_n/f_o is not >> 0.5 for values of f_o in Table 6.2 of AISC D.G. 11, the more general approach is required, and $V = V_r\Delta_p/f_n$ cannot be used.

• From Table 6.2,

$F_m = \left(\frac{F_m}{w}\right)w = 1.7(185) = 315$ lb.

Fast walking

$f_o = 5$ Hz

$f_n f_o = f_n/f_o = 3.7/5 = 0.74$

• From Solid curve in Fig. 6.5,

$A_m = 1.2$

$X_{max} = A_m F_m \Delta_p = 1.2(315)(3.62\times10^{-6}) = 1368\times10^{-6}$ in.
$= 1,368$ μin.

$V = 2\pi f_n X_{max} = 2\pi(3.7)(1368) = 31,800$ μin./sec.

*Unacceptable for any criteria in Table 6.1 for fast walking

Check slow walking @ 50 steps/min.

$f_o = 1.4$ Hz

$F_m/W = 1.3$ from Table 6.2

$\therefore F_m = 1.3(185\text{ lb.}) = 240$ lb.

$f_n/f_o = f_n/f_o = 3.7/1.4 = 2.6$

Eq. from Fig. 6.5:

$$A_m = \frac{1}{2(f_n/f_o)^2} = \frac{1}{2(2.6)^2} = 0.074$$

$$X_{max} = A_m F_m \Delta p = (0.074)(240)(3.62 \times 10^{-6})$$
$$= 64.3 \times 10^{-6} \text{ in} = 64.3 \ \mu\text{in}$$

Eq. 6.5:

$$V = 2\pi f_n X_{max} = 2\pi(3.7)(64.3) = 1495 \ \mu\text{in/sec}$$

* Bench microscopes up to 400x magnification

Check "moderate" walking @ 75 steps/min.

$f_o = 2.5$ Hz

$F_m = 1.5(185) = 277.5$ lb. ≈ 278 lb.

$f_n/f_o = f_n/f_o = 3.7/2.5 = 1.48$

$A_m = 0.30$ from solid curve in Fig. 6.5

$$X_{max} = (.30)(278)(3.62 \times 10^{-6}) = 302 \times 10^{-6} \text{ in}$$
$$= 302 \ \mu\text{in.}$$

$$V = 2\pi(3.7)(302) = 7{,}021 \ \mu\text{in/sec.}$$

* Does not meet criteria for bench microscopes

Try W24x146 (Beam) $A = 43\ in^2$ $I = 4580\ in^4$

$d = 24.7''$

$$\bar{y} = \frac{(21.7)(4.5)(2.25) + 43\left(\frac{24.7}{2} + 7.5\right)}{97.7 + 43}$$

$$\bar{y} = 7.63''$$

$$I_j = 164.8 + 4580 + 97.7(5.38)^2 + 43(12.22)^2$$

$$= 13{,}994\ in^4$$

$$w_j = 1817 + 146 = 1963\ plf$$

$$\Delta_j = \frac{5(1.96)(42.25)^4(1728)}{384(29000)(13994)} = 0.346''$$

Switch girder to W24x55

$I = 1350 \text{ in}^4$

$A = 16.2 \text{ in}^2$

$d = 23.6''$

$$\bar{y} = \frac{17.3(4.5)(2.25) + \frac{17.3}{2}(3)(6) + 16.2\left(7.5 + \frac{23.6}{2}\right)}{17.3(4.5) + \frac{17.3}{2}(3) + 16.2}$$

$$\bar{y} = \frac{643.5}{120} = 5.36''$$

$$I_g = 131.4 + 19.5 + 1350 + (3.11)^2(17.3)(4.5) + (.64)^2(3)(8.65) + 16.2(19.3 - 5.36)^2 = 5413 \text{ in}^4$$

$$w_g = (1963/10.67)(38.375) + 55 \text{ plf} = 7115 \text{ plf}$$

$$\Delta_g = \frac{5(7.115)(21.33)^4(1728)}{384(29000)(5413)} = 0.211$$

Bay Frequency:

$$f_n = 0.18\sqrt{\frac{g}{\Delta_j + \Delta_g}} = .18\sqrt{\frac{386.4}{.346 + .211}}$$

$$= 4.74 \text{ Hz}$$

Deflection for unit load:

$$\Delta_{oj} = \frac{1(42.25)^3(1728)}{96(29\times10^6)(13994)} = 3.35\times10^{-6} \text{ in/lb.}$$

$$L_j^4/I_t = \frac{(42.25\cdot12)^4}{13994} = 4.7\times10^6 > 4.5\times10^6$$

OK

$$N_{eff} = .49 + 34.2(.0469) + (9\times10^{-9})(4.7\times10^{6})$$
$$= 2.14$$

$$\Delta_{jp} = \frac{3.06\times10^{-6}}{2.14} = 1.43\times10^{-6}\ \text{in/lb.}$$

$$\Delta_{gp} = \frac{1(21.33)^3(1728)}{96(29\times10^{6})(5413)} = 1.1\times10^{-6}\ \text{in/lb.}$$

$$\Delta_p = (1.43 + \frac{1.1}{2})\times10^{-6} = 1.98\times10^{-6}\ \text{in/lb.}$$
$$\approx 2\times10^{-6}\ \text{in/lb.}$$

From Table 6.2

$F_m = 1.7(185) = 315\ \text{lb.}$

$f_o = 5\ \text{Hz}$

$f_n/f_o = 4.74/5 = .948$

$A_m = 0.85$ from Solid curve (Fig. 6.5

$X_{max} = .85(315)(2.0\times10^{-6}) = 533\times10^{-6}\ \text{in}$
$= 533\ \mu\text{in}$

$V = 2\pi(4.74)(533) = 15874\ \mu\text{in/sec.}$

N.G.

Check "moderate" walking

$f_o = 2.5\ \text{Hz}$ $\quad f_n/f_o = 4.74/2.5 = 1.9$

$F_m = 278\ \text{lb.}$

$$A_m = \frac{1}{2(1.9)^2} = 0.139$$

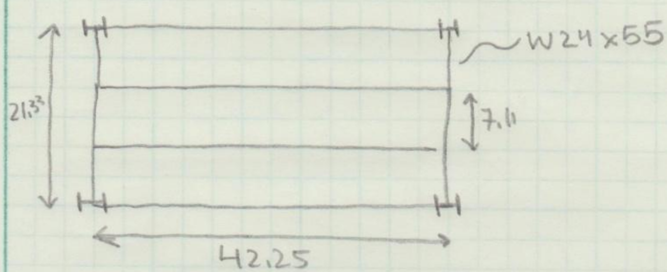
$X_{max} = .139(278)(2\times10^{-6}) = 77.3\times10^{-6}\ \text{in}$
$= 77.3\ \mu\text{in.}$

$V = 2\pi(77.3)(4.74) = 2302.2\ \mu\text{in/sec.}$

* System does not pass for "moderate" walking. Because of the lab's close proximity to the hallway, this is a requirement.

* Could make beam heavier, but try a new beam layout.

• Space beams at $\frac{21-4''}{3} = 7.11'$



• Try W24x76 (existing) to get an idea of Δ_j

$b = 7.11$

$7.11/n = 7.11(12)/5.89 = 14.5$

$$\bar{y} = \frac{14.5(4.5)(2.25) + (22.4)\left(\frac{23.9}{2} + 7.5\right)}{65.25 + 22.4}$$

$$= 6.65''$$

$$I_j = 110.12 + 2100 + 14.5(4.5)(6.65-2.25)^2 + 22.4(12.8)^2$$

$$= 7143 \text{ in}^4$$

$$w_j = 7.11(170.3) + 76 = 1287 \text{ plf}$$

$$\Delta_j = \frac{5(1.29)(42.25)^4(1728)}{384(29000)(7143)} = 0.446''$$

* Significantly lower than existing Δ_j

Try W 24x104 beam spaced @ 7.11' = 7'-1.5"
framing into W 24x55 girder

W24x104:

$A = 30.7 \text{ in}^2 \quad d = 24.1" \quad I_x = 3100 \text{ in}^4$

$$\bar{y} = \frac{65.25(2.25) + 30.7(12.05 + 7.5)}{95.95} = 7.79"$$

$$I_j = 110.12 + 3100 + 65.25(5.54)^2 + 30.7(11.8)^2$$
$$= 9487 \text{ in}^4$$

$$w_j = 1287 - 76 + 104 = 1315 \text{ plf}$$

$$\Delta_j = \frac{5(1.315)(42.25)^4(1728)}{384(29000)(9487)} = 0.34"$$

Step up to W 24x117

$A = 34.4 \text{ in}^2 \quad d = 24.3" \quad I_x = 3540 \text{ in}^4$

$$\bar{y} = 8.26"$$

$$I_j = 110.12 + 3540 + 65.25(6.01)^2 + 34.4(19.65 - 8.26)^2$$
$$= 10{,}462 \text{ in}^4$$

$$w_j = 1328 \text{ plf}$$

$$\Delta_j = \frac{5(1.33)(42.25)^4(1728)}{384(29000)(10462)} = 0.31"$$

Girder: W 24x55

$$\bar{y} = 5.36"$$

$$I_g = 5413 \text{ in}^4$$

$$w_g = \left(\frac{1328}{7.11}\right)(38.375) + 55 = 7223 \text{ plf}$$

$$\Delta_g = \frac{5(7.22)(21.33)^4(1728)}{384(29000)(5413)} = 0.21"$$

Bay Frequency

$$f_n = 0.18\sqrt{\frac{386.4}{.52}} = 4.91\,Hz$$

$$\Delta_{oj} = \frac{1(42.25)^3(1728)}{96(29\times10^6)(10462)} = 4.47\times10^{-6}\ in./lb.$$

$$\frac{L_j^4}{I_t} = \frac{(42.25\cdot12)^4}{10462} = 6.3\times10^6 > 4.5\times10^6$$
$$< 257\times10^6 \quad \underline{OK}$$

$$de/s = \frac{6}{7.11(12)} = 0.07 \quad > 0.018$$
$$< 0.208 \quad \underline{OK}$$

$$N_{eff} = .49 + 34.2(.07) + 9\times10^{-9}(6.3\times10^6)$$
$$= 2.94$$

$$\Delta_{jp} = \frac{4.47\times10^{-6}}{2.94} = 1.52\times10^{-6}\ in./lb.$$

$$\Delta_{gp} = \frac{1(21.33)^3(1728)}{96(29\times10^6)(5413)} = 1.1\times10^{-6}\ in./lb.$$

$$\Delta_p = (1.52 + .55)\times10^{-6} = 2.07\times10^{-6}\ in./lb.$$

$$f_o = 2.5\,Hz \qquad f_n/f_o = 4.9/2.5 = 1.96$$

$$F_m = 278\,lb.$$

$$A_m = \frac{1}{2(1.96)^2} = 0.13$$

$$X_{max} = .13(278)(2.07\times10^{-6}) = 75\,\mu in.$$

$$V = 2\pi(75)(4.9) = 2309\ \mu in/sec.$$

$\therefore$ Increase I_j

Try a W24 x 162

$A = 47.8 \text{ in}^2 \quad d = 25'' \quad I_x = 5170 \text{ in}^4$

$$\bar{y} = \frac{146.8 + 47.8(12.5 + 7.5)}{113.05} = 9.75''$$

$$I_j = 110.12 + 5170 + 65.25(7.5)^2 + 47.8(10.25)^2$$
$$= 13{,}972 \text{ in}^4$$

$$w_j = 1287 - 76 + 162 = 1373 \text{ plf}$$

$$\Delta_j = \frac{5(1.37)(42.25)^4(1728)}{384(29000)(13972)} = 0.24''$$

$$\Delta_g = 0.21''$$

$$f_n = 0.18\sqrt{\frac{386.4}{.45}} = 5.27 \text{ Hz}$$

$$\Delta_{oj} = \frac{1(42.25)^3(1728)}{96(29 \times 10^6)(13972)} = 3.35 \times 10^{-6} \text{ in./lb.}$$

$$\frac{L_j^4}{I_t} = \frac{(42.25 - 12)^4}{13972} = 4.7 \times 10^6 > 4.5 \times 10^6$$
$$< 257 \times 10^6 \quad \underline{\text{OK}}$$

$$d_e/s = \frac{6}{85.3} = 0.07 > 0.018$$
$$< 0.208 \quad \underline{\text{OK}}$$

$$N_{eff} = .49 + 34.2(.07) + 9 \times 10^{-9}(4.7 \times 10^6)$$
$$= 2.93$$

$$\Delta_{jp} = \frac{3.35 \times 10^{-6}}{2.93} = 1.14 \times 10^{-6} \text{ in./lb.}$$

$$\Delta_{gp} = \frac{1(21.33)^3(1728)}{96(29 \times 10^6)(5413)} = 1.1 \times 10^{-6} \text{ in./lb.}$$

$$\Delta_p = (1.14 + .55) \times 10^{-6} = 1.69 \times 10^{-6}$$

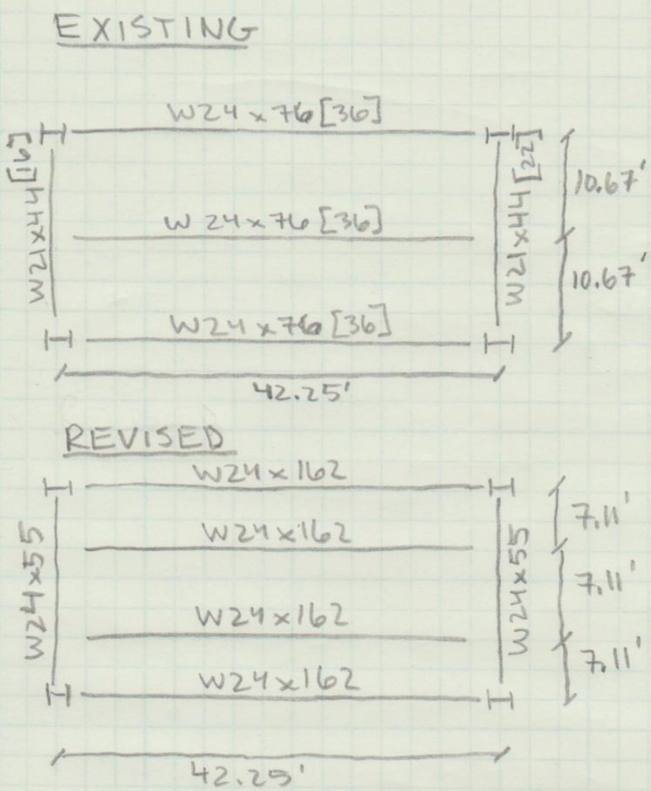
$V_V = 5500$ lb.-Hz^2

$V_p = \frac{5500(1.69)}{5.27} = 1764$ μin./sec.

$1764 < 2000$ μin./sec.

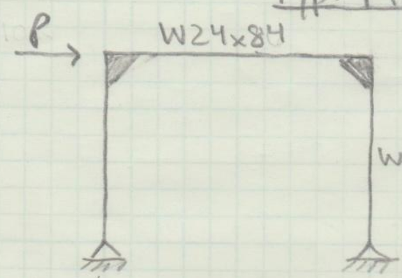
GOOD

Use: W24x162 beam
W24x55 girder
Existing 3" deck with
4.5" topping


EXISTING
W24 x 76 [36]
W 24 x 76 [36]
W24 x 76 [36]
W21x44 [16]
W21x44 [22]
10.67'
10.67'
42.25'
REVISED
W24 x 162
W24x162
W24x162
W24x162
W24x55
W24x55
7.11'
7.11'
7.11'
42.25'

Appendix A-4: Moment Frame Calculations

Typ. Moment Frame



Trib width = $\frac{42.25'}{2}$ = 21.1 ft.

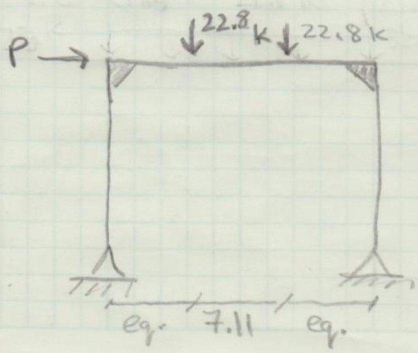
Applicable load combinations:

1.4D
1.2D + 1.6L + 0.5(Lr/S)
1.2D + 1.6W + 0.5L + 0.5(Lr/S)
0.9D + 1.6W

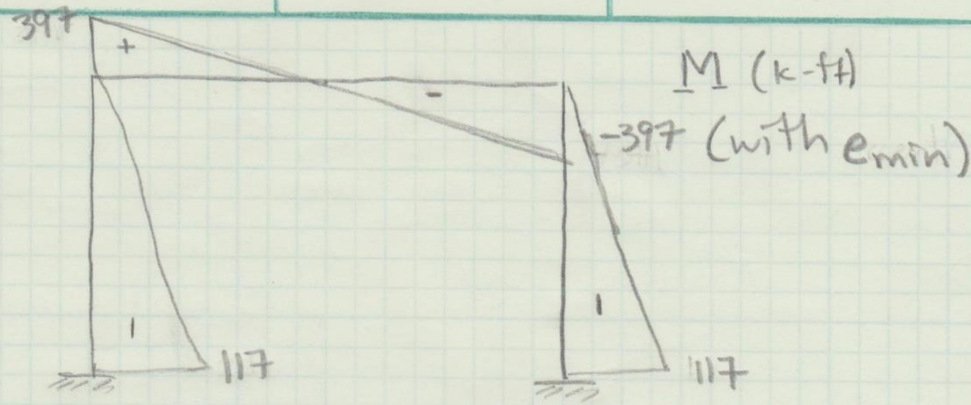
Evaluating for wind, so:

① 1.2D + 1.6W + 0.5L + 0.5S
② 0.9D + 1.6W

① 1.2(95.6) + 0.5(75) = 152 psf (21.1') = 3207 plf



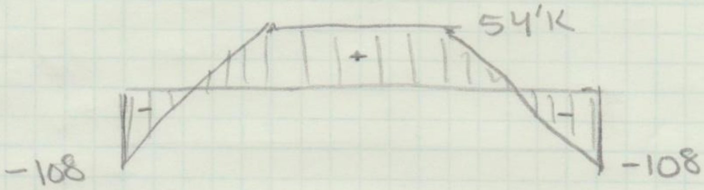
$\frac{3207(7.11)}{1000}$ = 22.8 k



Wind

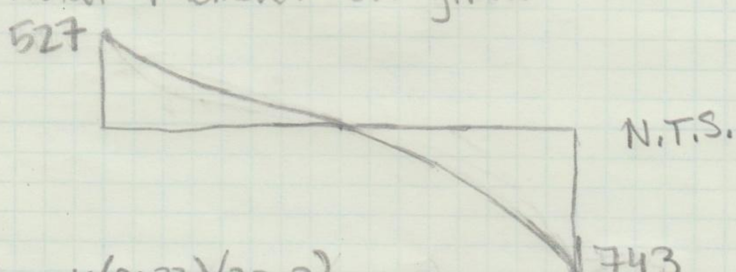
$1.6W = 635$ 'k on girder

$= 187$ 'k on column



$1.2D + .5L$

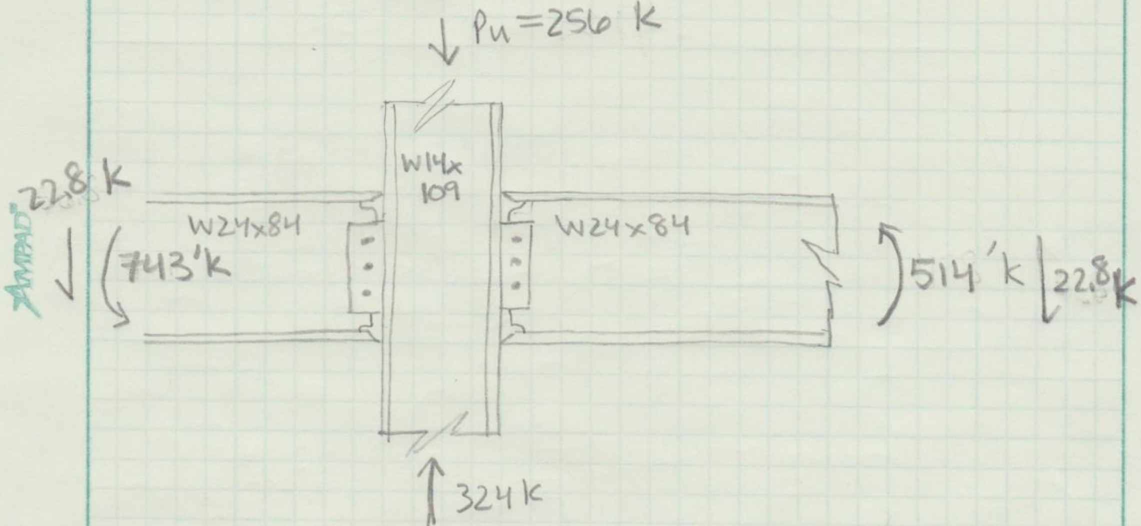
Total Moment on girder



$$P_u = 4\frac{(21.33)(3207)}{1000} + \frac{(21.33)(21.125)(11)}{1000} = 279\,k \quad + 5 = 45$$

$P_u = 324\,k$

Design a Flange Welded/Web Bolted Moment Connection:



	Girder	Column
ϕMn	840'k	
d	24.1"	14.3"
tw	0.47"	0.525"
bf	9.02"	14.6"
tf	0.77"	0.86"
k		1.46" (design)
h/tw		21.7
A		32 in²

• Girder Flange-to-Column Flange:
Full penetration welds with backing bars.

- Girder Web-to-Column Flange
  Single Plate
  *Assume beam flanges take moment
  $V_u = 22.8\ k$
- #bolts (A325-N, 3/4" ϕ) $= \frac{22.8}{17.9} = 1.27$
  $\therefore$ Use 2 bolts
- PL length: $1(3") + 2(1.25) = 5.5"$
- Determine PL thickness:
  $22.8 / [(.75)(.6)(58)(5.5 - 2(.875))] = 0.31"$
  $\therefore$ Use 5/16"
- Check Bolt Bearing:
  $\phi r_{n_{PL}} = .75(2.4)(58)(.75)(5/16) = 24.5\ k > 17.9$
  $\phi r_{n_{bm}} = .75(2.4)(65)(.75)(.47) = 41.2\ k$
  $\therefore$ Bolt Shear controls
- Block Shear:
  T 9.3a: 46.2 k ($L_{eh} = 1.5"$)
  T 9.3b: 166 k ($L_{ev} = 1.25"$)
  T 9.3c: 188 k
  $\phi r_n = 5/16(46.2 + 166) = 66.3\ k > 22.8\ k$
  OK
- Shear Yield:
  $\phi r_n = 1.0(.6)(36)(5.5)(5/16) = 37.1 > 22.8\ k$
  OK

• Shear Rupture

$\phi r_n = .75(.6)(58)(5/16)(5.5 - 2(.875))$

$= 30.6 > 22.8\text{ k}$

OK

• Weld Size = 3/16" min. for PL

• Weld Strength at Weld

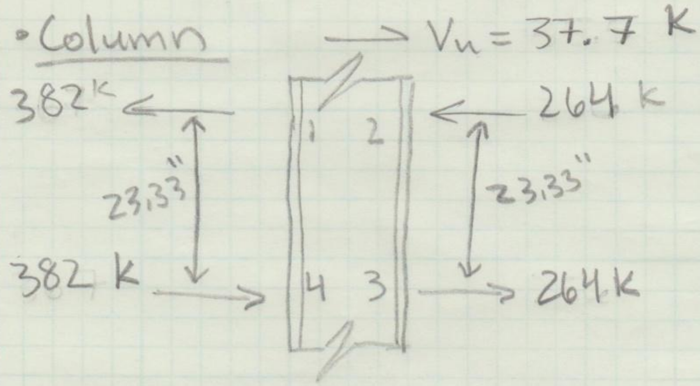
$\phi R_n = .75(.6)(65)(.47)(5.5 - 2(3/16))$

$= 70.5\text{ k} > 23$ OK

• Weld Rupture

$\phi R_n = 6(1.392)(5.125) = 42.8 > 23$ OK

• Column



$T_u = C_u = \dfrac{743(12)}{23.33} = 382\text{ K}$

$T_u = C_u = \dfrac{514(12)}{23.33} = 264\text{ K}$

• Column Flange Bending

$\phi R_n = 0.9(6.25)(.86)^2(50)$

$= 208\text{ k} < 382\text{ k}$

$< 264\text{ K}$

∴ Need 1/2 depth stiffeners @ 1 and 3

• Column Web Yielding

$$\phi R_n = 1.0(5(1.46) + .77)(50)(.525)$$
$$= 211.8\ k < 382$$
$$< 264$$

∴ Full depth stiffeners @ 1, 2, 3, 4

• Column Web Crippling

$$\phi R_n = .75(.80)(.525)^2\left[1 + 3\left(\frac{.77}{14.3}\right)\left(\frac{.525}{.77}\right)^{1.5}\right] \times \sqrt{\frac{29000(50)(.86)}{.525}}$$
$$= 0.1804(1541.2) = 278\ k > 264\ k$$
$$< 382\ k$$

∴ Half Depth Stiffener @ 4

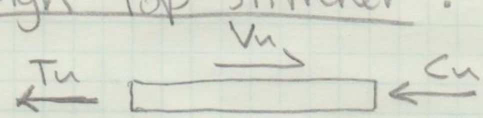
• Web Buckling

• Does not Apply

☆ Column web yielding controls:

Provide Full Depth Stiffeners

<u>Design Top Stiffener:</u>



$$T_u = 382 - \begin{vmatrix} 208 \\ 212 \end{vmatrix}_{min} = 174\ k$$

$$C_u = 264 - \begin{vmatrix} 212 \\ 278 \end{vmatrix} = 52\ k$$

✱ Same loads for bottom stiffener

$A_{st} = (6 - 3/4)(t_s)$

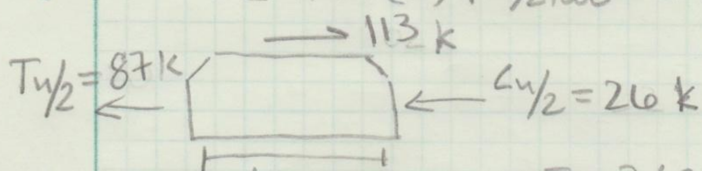
$t_s \geq \frac{.77}{2} = 0.39 \quad \therefore$ use $t_s = 0.5''$ OK

$A_{st} = (6 - 3/4)(.5) = 2.625 \text{ in}^2$

$A_{st,min} = \frac{174/2}{.9(36)} = 2.69 \text{ in}^2$

$b_s + \frac{t_{wc}}{2} = 6 + \frac{.525}{2} = 6.26 > \frac{9}{3} = 3$ OK

$t_s \geq 1.79(6)\sqrt{36/29000} = 0.378 < t_s$ OK



$L_{weld} = 14.3 - 1.5 - 2(.86) = 11.08$

Use $L_{weld} = 11''$

$D_{flange} = \frac{.9(36)(.5)}{1.5(1.392)(2)} = 3.88 \rightarrow$ Use $1/4''$ fillet

$D_{web} = \frac{113}{11(1.392)(2)} = 3.69 \rightarrow$ Use $1/4''$

* Use $1/4''$ weld throughout

Panel Zone Shear

$V_u = 382 + 264 - 37.7 = 608 \text{ k}$

$P_u = \left(\frac{256 + 324}{2}\right) = 290 \text{ k}$

$0.4P_y = .4(50)(32) = 640 \text{ k}$

$P_u < .4 P_y$

$\therefore \phi R_v = 0.6(50)(.9)(14.3)(.525) = 202.7$

$202.7 < V_u = 608\text{ k}$

$\therefore$ Need Doubler Plates

Design Doubler Plates

Assume $h/t_p \le 1.1\sqrt{\frac{K_v E}{F_y}}$

$$t_{p\,req} = \frac{608 - 202.7}{.9(.6)(36)(14.3)} = 1.46'' > 1''$$

$\therefore$ Use 2 D.P. $\frac{3}{4}''$ on each side $= 1\frac{1}{2}''$

• Plate Buckling

$$h/t_w\left(\frac{t_w}{t_p}\right) = 21.7\left(\frac{.525}{1.5}\right) = 7.6$$

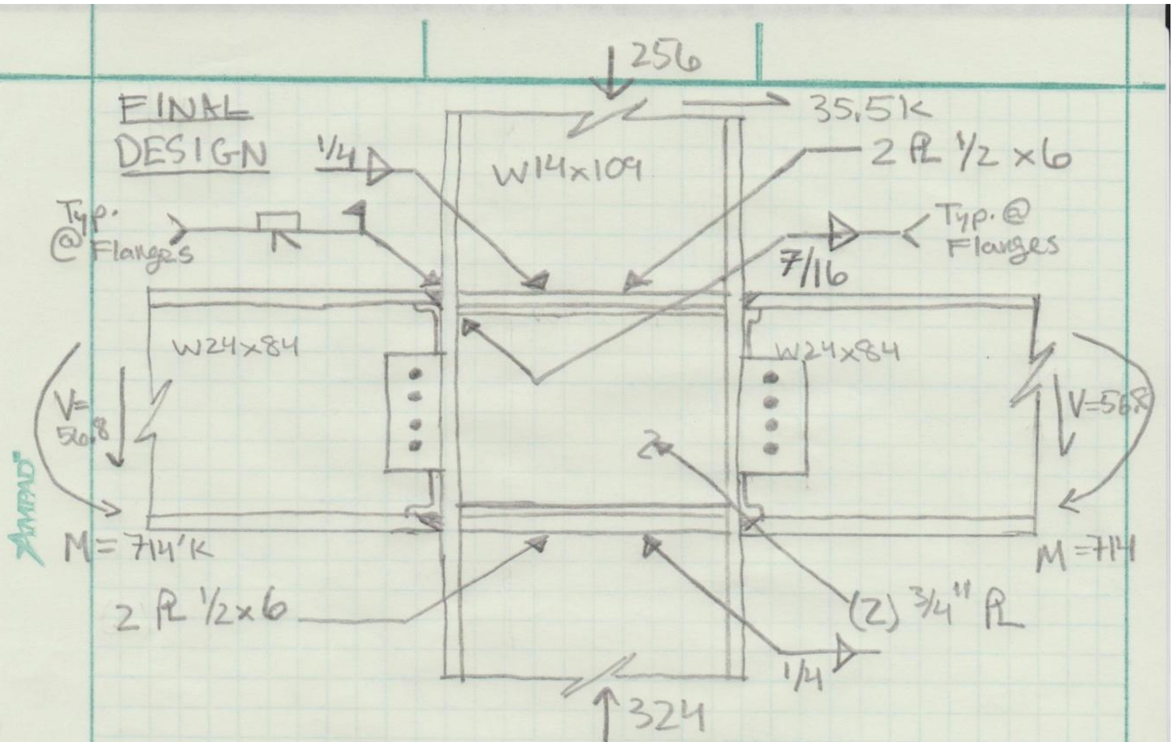
$$7.6 \le 1.1\sqrt{\frac{5(29000)}{36}} = 69.8$$

Assumption OK

Panel Zone Welds

$$V_u = \frac{608 - 202.7}{2} \approx 202.7\text{ k}$$

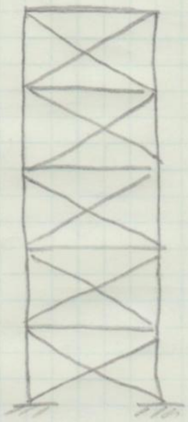
$$D_{req} = \frac{202.7}{23.33 \times 1.392} = 6.24 \;\therefore \text{Use } 7/16''$$

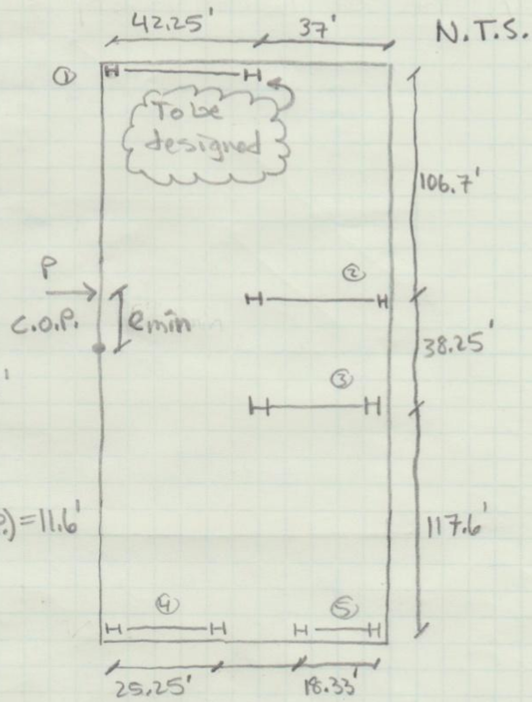

FINAL DESIGN
256
35.5K
1/4
W14x109
2 PL 1/2 x6
Typ. @ Flanges
Typ. @ Flanges
7/16
W24x84
W24x84
V=56.8
V=56.8
M=714'K
M=714
2
2 PL 1/2x6
(2) 3/4" PL
1/4
324

Appendix A-5: Braced Frame Calculations

Design a Braced Frame

* Design longer frames to take more of the applied load.





C.O.M. @ 142.9'

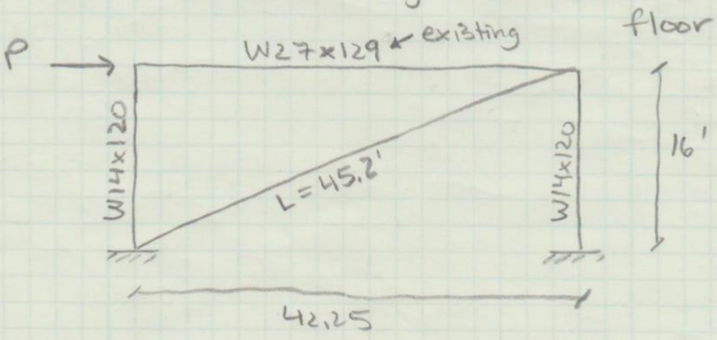
C.O.P. @ 131.3'

$0.05(262.6) = 13.1$

$13.1 > (C.O.M. - C.O.P.) = 11.6'$

$e_{min} = 13.1'$

* Assume $k_1 = k_2 = k_3 = k_{45}$ initially for trial member selection. Design for loads at lowest floor



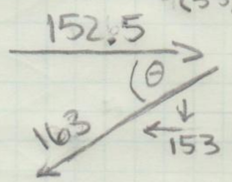
$\Sigma d_i^2 = 14316 + 167.7 + 640 + 20,450 = 35,574 \text{ ft}^2$

* Design for Story shear determined in Tech. 1. Neglect torsional resistance of mom. frames.

At floor 1, story shear = 583 k

583 / 4 braced frame systems = 146 k

$P = 146\text{ k} + \frac{(146)(13.1)}{(35,574)}(120) = 152.5\text{ k}$



$\theta = \tan^{-1}\left(\frac{16}{42.25}\right) = 20.74°$

$\cos(20.74) = \left(\frac{153}{x}\right)$

$x = 163\text{ k}$

Load on brace = 163 k

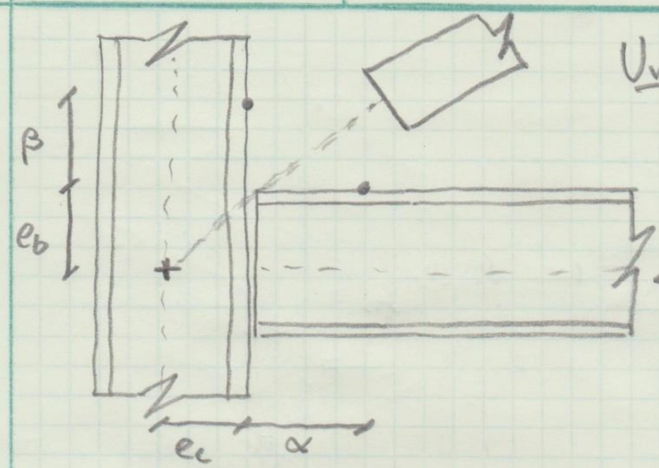
Try HSS 4x4x½

Yielding $\phi P_n = 249$ k

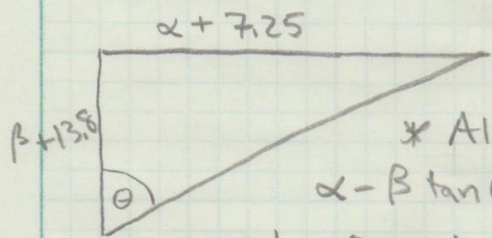
Rupture $\phi P_n = 196$ k

	Girder	Column	Brace
d	27.6"	14.5"	2.6"
t_w	.61"	0.59"	.465"
b_f	10"	14.7"	2.6"
t_f	1.1"	0.94"	.465"
A	37.8 in²	35.3 in²	6.02 in²

Uniform Force Method



$e_b = 13.8''$ $\quad e_c = 7.25''$



$\theta = 90 - 20.74 = 69.26°$

* Allow α to be 6" for trial

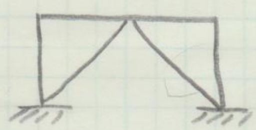
$\alpha - \beta \tan\theta = e_b \tan\theta - e_c$

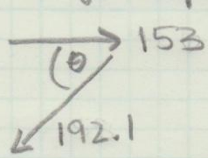
$\tan\theta = \tan(69.26) = 2.64$

$6 - \beta(2.64) = 13.8(2.64) - 7.25$

$\beta = -8.8$ N.G.

* For β to be a positive number, $\alpha \geq 29.18''$

Impractical. Try  because of large span.

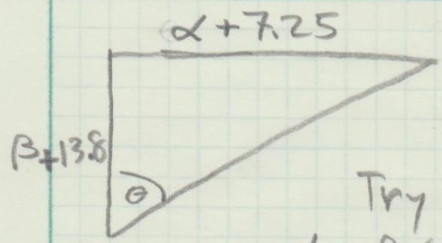


$\theta = \tan^{-1}\left(\frac{16}{21.1}\right) = 37.2°$

$\cos(37.2) = \left(\frac{153}{x}\right)$

$x = 192.1k$

* HSS 4x4x½ still ok for trial



$\theta = 90 - 37.2 = 52.8^\circ$

$\tan\theta = 1.32$

Try $\alpha = 6$ again.

$6 - \beta(1.32) = 13.8(1.32) - 7.25$

$\beta = -3.76$ N.G.

$\alpha \geq 10.97$ for β to be positive.

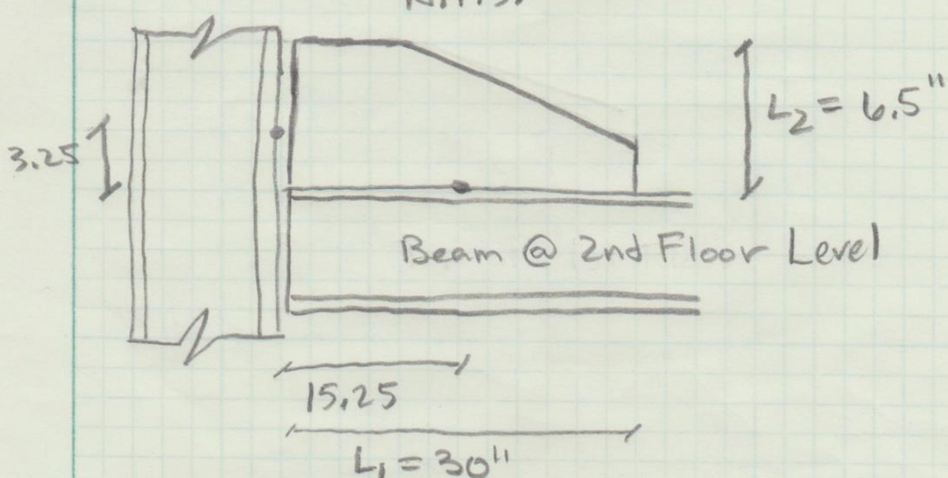
Let $\alpha = 15.25''$

$$\beta = \frac{13.8(1.32) - 7.25 - 15.25}{-1.32}$$

$= 3.25$

Gusset PL:

N.T.S.



Calculate Forces

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(22.5)^2 + (17.05)^2} = 28.2$$

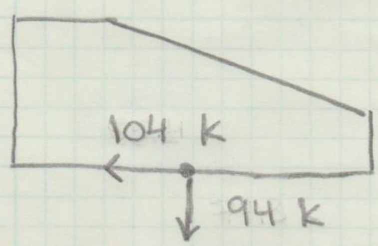
$$V_c = \frac{\beta}{r} P_u = \frac{3.25}{28.2}(192) = 22.1\ k$$

$$H_c = \frac{e_c}{r} P_u = \frac{7.25}{28.2}(192) = 49.4\ k$$

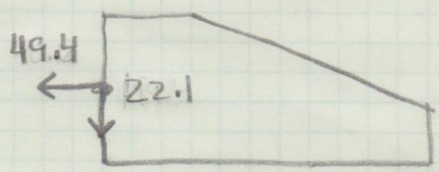
$$H_b = \frac{\alpha}{r} P_u = \frac{15.25}{28.2}(192) = 103.8 \text{ k}$$

$$V_b = \frac{e_b}{r} P_u = \frac{13.8}{28.2}(192) = 94 \text{ k}$$

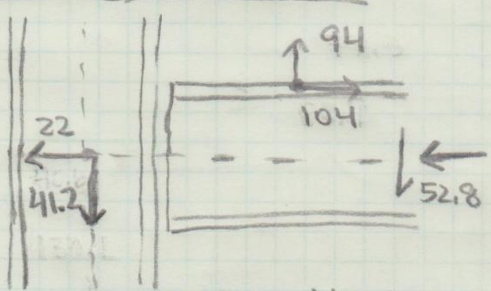
Gusset-to-Beam



Gusset-to-Column



Beam-to-Column



$$R = \frac{2.5 \text{ klf } (42.25)}{2} = 52.8$$

$$H = 126 - 104 = 22 \text{ k}$$

$$V = 94 - 52.8 = 41.2 \text{ k}$$

Column-to-Gusset

3/4"Ø A325-N (T7-1 in AISC):

$A_b = 0.442 \text{ in}^2$

$V_u = 22.1^k$

Try 4 bolts

$f_v = \frac{22.1}{4(.442)} = 12.5 \text{ ksi}$

$F'_t = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v = 1.3(89.9) - \frac{89.9}{.75(54)}(12.5)$

$= 89.1 < 90 \text{ ksi}$ OK

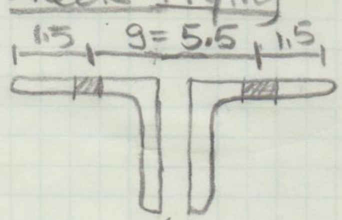
$\frac{12.5}{.75(54)} < 1.0$ OK

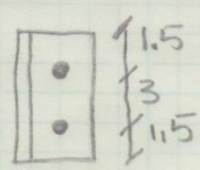
$\therefore \phi r_{nt} = .75(89.1)(.442) = 29.5 \text{ k}$

$r_{ut} = \frac{49.4}{4} = 12.4 \text{ k}$

Check Prying

* Try 2-L 4x4x3/8x6





℄ Assume gusset is 1/2" thick

$p = \begin{cases} 5.5'' = \text{gage} \\ 6/2 = 3'' \end{cases}$ min

$b = 2.5 - \frac{3/4}{2} = 2.125''$

$b' = 2.13 - 3/8 = 1.75''$

$r_{ut}\, b' = 12.4\,(1.75) = 21.7\ ''k$

$\phi M_{n1} = \dfrac{0.9(58)(3)(.375)^2}{4} = 5.5\ ''k$

$r_{ut} b' > \phi M_{n1}$ ∴ Prying will occur

Check Angle Flange:

$\phi M_{n2} = \dfrac{0.9(58)(3 - 7/8)}{4}(.375)^2 = 3.93\ ''k$

$r_{ut} = 12.4 > \dfrac{5.5 + 3.9}{1.75} = 5.37$ N.G.

Try 2-L 4x4x5/8 x6

$\phi M_{n1} = 15.3\ ''k$

∴ Prying

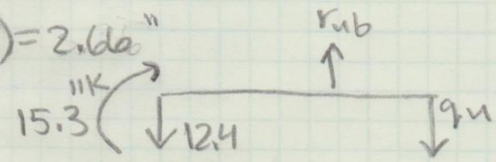
$\phi M_{n2} = 10.8\ ''k$

$\dfrac{15.3 + 10.8}{1.75} = 14.9\ ''k > r_{ut}$ OK

Check Bolts

$a = \min \begin{cases} 1.5'' \\ 1.25b = 1.25(2.125) = 2.66'' \end{cases}$

$a' = 1.5 + 3/8 = 1.88$



$q_u = \dfrac{12.4(1.75) - 15.3}{1.88} = 3.43\ k$

$r_{ub} = 3.43 + 12.4 = 15.8 < \phi r_{nt} = 29.5^k$ OK

Bolts:

$V_u = 22.1\ k$

$\frac{22.1}{17.9} = 1.23 < 4$ OK

Check Bearing:

$\phi r_{n\,Angle} = .75(58)(2.4)(.75)(5/8) = 48.9$

$\phi r_{n\,col.} = .75(2.4)(65)(.75)(.94) = 82.5$

$> 17.9k$ OK

Block Shear:

T 9.3a: 46.2^k ($L_{eh} = 1.5$)

T 9.3b: 166k ($L_{ev} = 1.25''$)

T 9.3c: 83.2 k (n=2)

$\phi r_n = 5/8\ (46.2 + 83.2) = 80.9\ k > 22.1\ k$

OK

Angle Shear Yield:

$\phi r_n = 1.0(36)(.6)(6)(5/8) = 81\ k > 22.1\ k$

OK

Angle Shear Rupture:

$\phi r_n = .75\ (.6)(58)(5/8)(6 - 2(.875))$

$= 69.3\ k > 22.1\ k$

OK

Gusset-to-Angle Welds:

min. size = 3/16"

max = 5/8 - 1/16 = 9/16

Use 3/16"

Weld Strength:

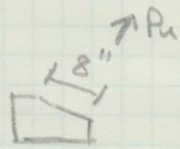
$$\phi R_n = .75(.6)(58)(.5)(6 - 2(3/16))$$
$$= 68.5\text{ k} > 22.1\text{ k} \quad \underline{\text{OK}}$$

Weld Rupture:

$$\phi R_n = 2(3)(1.392)(5.625) = 47\text{ k} > 22.1\text{ k}$$
$$\underline{\text{OK}}$$

Check Gusset:

Tension Yielding: *Assume



$$.9(36)(.5)(8) = 130\text{ k} < 192\text{ k} \quad \text{N.G.}$$

Increase width.

$$.9(36)(.5)(w) \geq 192$$
$$w \geq 11.85 \rightarrow \text{Make it } 12" \quad \underline{\text{OK}}$$

Rupture

$$\phi R_n = .75(58)(12)(.5) = 261 > 192$$
$$\underline{\text{OK}}$$

Weld to HSS

$$\text{min} = 3/16"$$
$$l \geq 4"$$

$$192 = C C_1 \phi D l$$

T 8-4:

$a = 0 \quad C = 3.71$

$$192 \leq 1.0(3.71)(.75)(l)(3)(2)$$
$$l \geq 11.5"$$

\# weld groups

Make $l = 12"$

Gusset-to-Beam:

Weld entire length of gusset

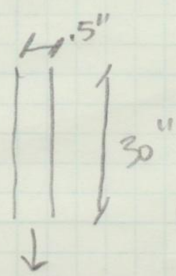
$\ell = 30''$

transverse

Shear strength $= 1.394(3)(30)(2)(1.5)$

$= 376.4\ k > 94k$ OK

Tension



$k = \frac{.5}{30} = 0.017 \approx 0$

$a = 0$

$C = 3.71$

$\phi r_n = 3.71(1.0)(3)(.75)(30)$

$= 250.4 > 104$ OK

Local Web Yielding:

$C_u = 94k$

$\phi R_n = 1.0 F_{yb}(5k_{design} + \ell_{gusset})t_{wb}$

$k_{design} = 1.7$

$\phi R_n = 1.0(50)(5(1.7) + 30)(.61)$

$= 1174.3\ k > 94\ k$ OK

Beam-to-Column:

$V_u = 41.2$; $\frac{41.2}{17.9} = 2.3 \rightarrow$ Use 3 bolts

$T_u = \frac{22}{29.8} = .74$ ∴ Use 3 bolts ($3/4''\phi$)

Bolt Bearing

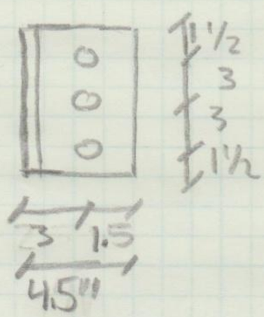
$\phi r_{n_{Angle}} = .75(58)(2.4)(.75)(t_a) \geq 41.2$

$\phi r_{n_{girder}} = .75(2.4)(65)(.75)(.61) \geq 41.2$

$t_a \geq 0.53'' \longrightarrow$ Use $t_p = 5/8''$

$\phi r_{n_g} = 53.5$ k OK

Angle length = $3+3+1.5+1.5 = 9''$



Check Block Shear

T 9.3a = 46.2
T 9.3b = 121
T 9.3c = 139

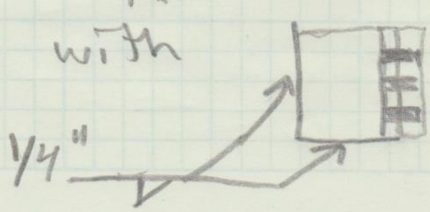
$1/4(46.2+121) = 41.8k > 41.2$ k
OK

Use L 5x3x1/2

From T 10-12,

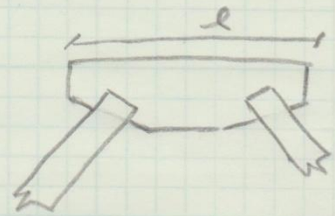
$\phi R_n = 53.2 > 41.2$ k OK

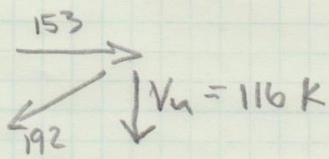
with



weld $\phi r_n = 54.8$k
OK

<u>Determine top Gusset length</u>





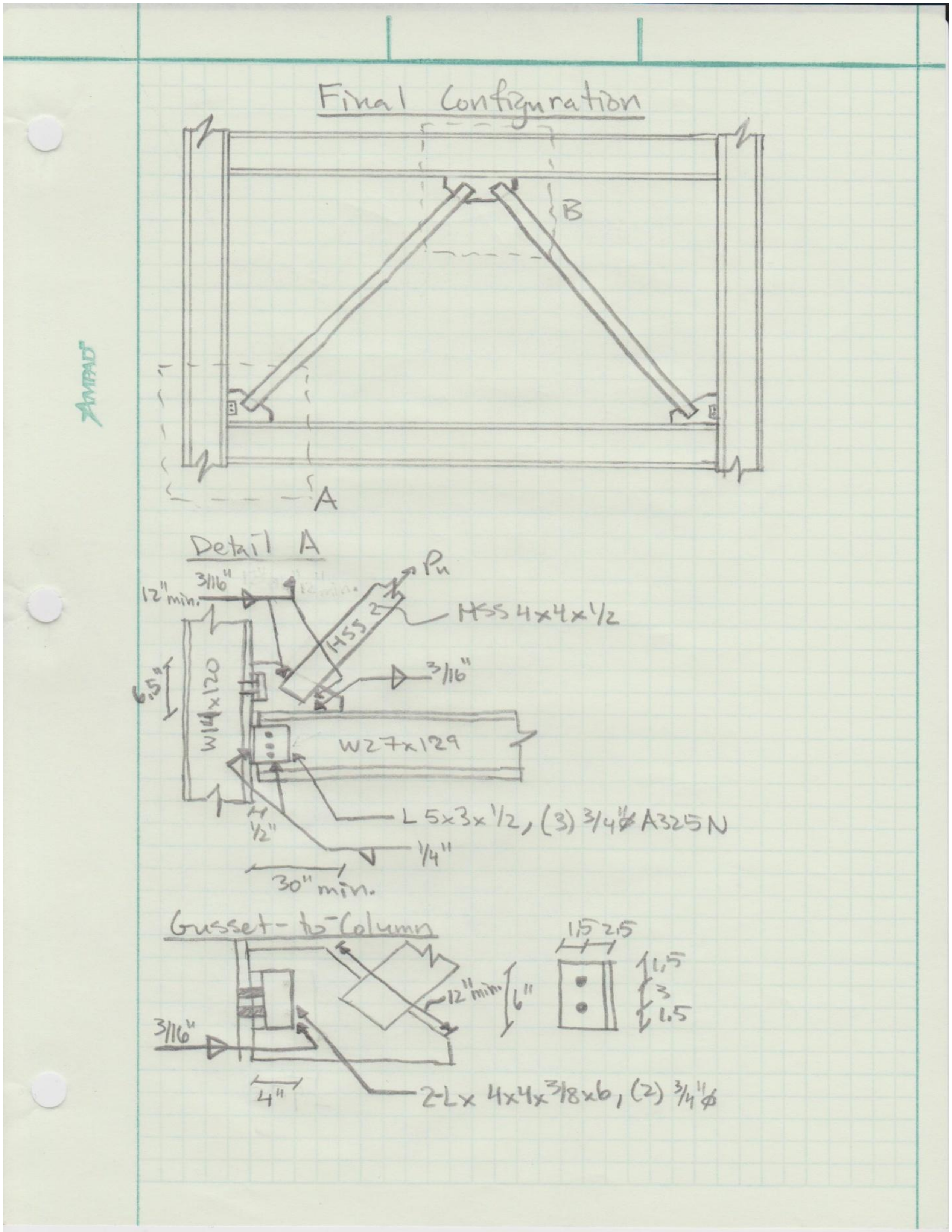
l_{min} for tension $= \dfrac{\phi r_n}{3.71(1.0)(.75)(3)}$ ← #16ths

$= 18.33"$

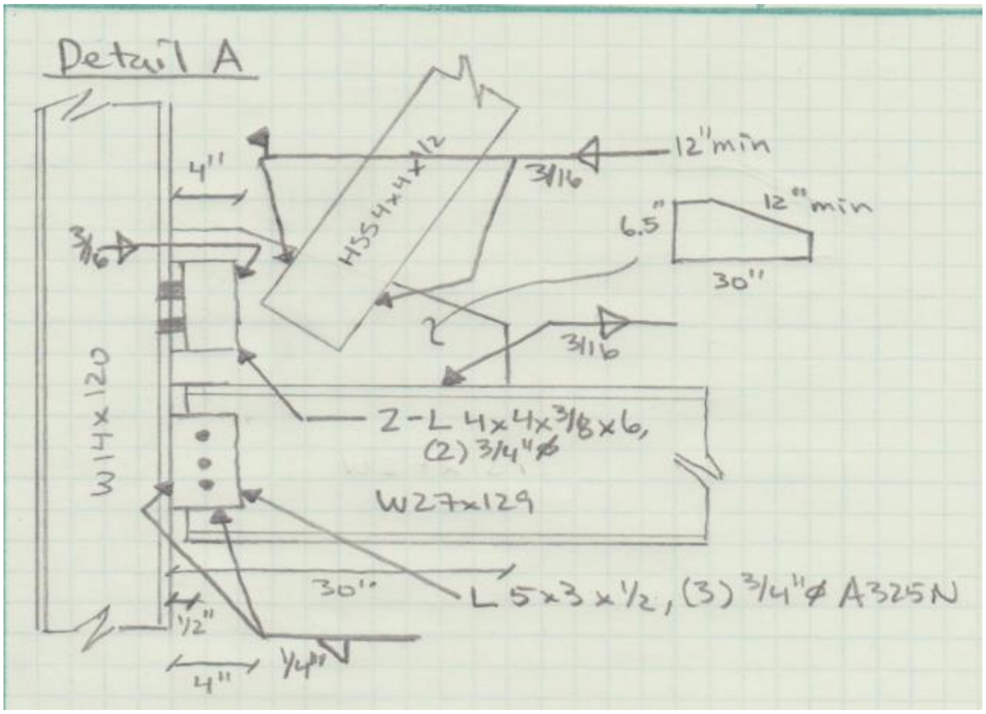
l_{min} for shear $= \dfrac{116}{1.394(3)(2)(1.5)}$ ← transverse

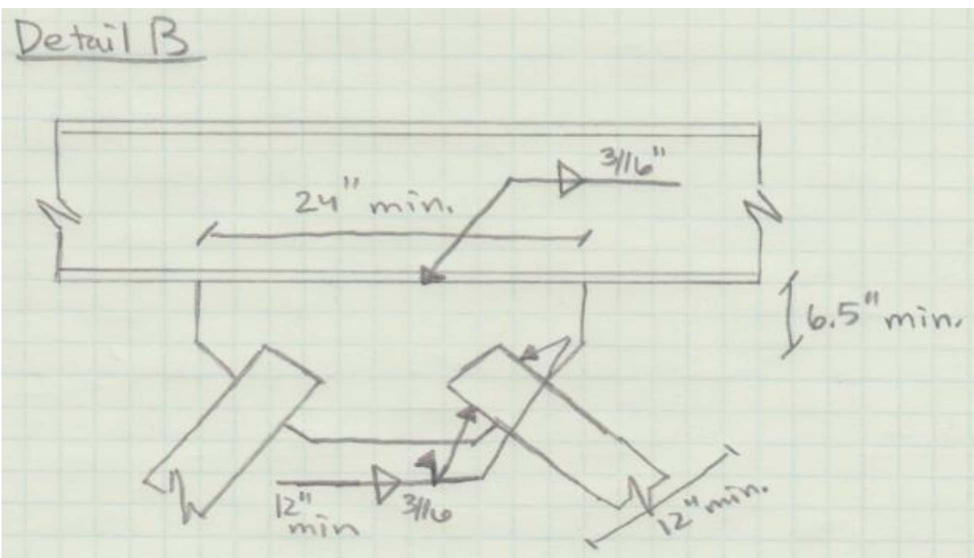
$= 9.25$

Make $l = 24"$



Final Configuration
B
A
Detail A
Pu
3/16"
12" min.
HSS 4x4x1/2
HSS
3/16"
6.5"
W14x120
W27x129
L 5x3x1/2, (3) 3/4"⌀ A325N
1/2"
1/4"
30" min.
Gusset-to-Column
1.5 2.5
1.5
3
1.5
12" min.
6"
3/16"
4"
2-L x 4x4x3/8x6, (2) 3/4"⌀


Detail A
4"
3/16
12"min
3/16
HSS 4x4x1/2
6.5"
12"min
30"
3/16
2-L 4x4x3/8x6,
(2) 3/4"⌀
W14x120
W27x129
30"
L 5x3x1/2, (3) 3/4"⌀ A325N
1/2"
1/4"
4"


Detail B
3/16"
24" min.
6.5" min.
12"
min
3/16
12" min.