Science Center Research Park
3711 Market St.
Philadelphia, PA

The Pennsylvania State University Department of Architectural Engineering
Senior Thesis 2009-2010

Prepared by:
Zachary Yarnall

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[FINAL THESIS REPORT]

SCIENCE CENTER RESEARCH PARK

3711 MARKET STREET PHILADELPHIA, PA

GENERAL BUILDING DATA

size: 401,032 GSF
stories: 11
dates of construction: November 2006 - May 2008
project delivery method: fast-track

ARCHITECTURE

- The Science Center Research Park is an addition to the growing research/science development in the University City area.
- The building includes offices, wet labs, retail space, and a 500 car parking garage.
- Covered by glass curtain wall and a brick veneer along the Market Street facade.
- A strong modern entrance with retail spaces on the storefront curtain wall ground floor.
- The largest green roof in the city of Philadelphia

STRUCTURAL SYSTEM

- Typical 7-1/2 in. composite steel deck on steel beams
- Lateral resistance provided by concentric steel braced frame
- Ground floor is comprised of slab-on-grade, cast-in-place columns, and grade beams.
- Drilled piers are designed for end bearing upon and, where noted in the structural plans, socketing into the 20 ton-per-square foot weathered rock

M.E.P. SYSTEMS

- Cooling towers, water tanks, air-handling units, and mechanical exhaust fans are located at the penthouse level
- 480/277 volts, 3 phase, 4 wire electrical system
- Smaller motors and lighting shall be connected to 277 and 120 volts, single phase circuits as indicated in the plans
- New dual 13.2 electric service to the Building is nominal 13.2 KV from PECO

PROJECT TEAM

Owner: Wexford Science & Technology, LLC; Hanover, MD
Wexford NJ Office; Edison, NJ Science Center; Philadelphia, PA
Architect: Zimmer Gunsul Frasca Architects LLP; Los Angelas, CA
Associate Architect: Ueland Junker McCauley Nicholson LLC; Philadelphia, PA
Structural Engineer: Keast & Hood Co.; Philadelphia, PA
MEP /Fire Protection Engineers:
Vinokur-Pace Engineering Services, INC.; Jenkintown, PA
Civil Engineer: Boles, Smyth Associates, INC.; Philadelphia
Geotechnical : Site-Blauvelt Engineers; Mount Laurel, NJ
Elevator: Lerch Bates and Associates; La Rescenta, CA
Vibration/Air Quality: Rowan Williams Davies &Irwin INC.; Guelph, Ontario, Canada, Nikib8

ZACK YARNALL | STRUCTURAL OPTION

HTTP://WWW.ENGR.PSU.EDU/AE/THESIS/PORTFOLIOS/2009/ZDY5001/

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EXECUTIVE SUMMARY

The Science Center Research Park is a 401,032 GSF mixed-use building and is approximately 144 feet tall. It currently has the largest green roof in the city of Philadelphia. The building includes offices, wet labs, retail space, and a 500 car parking garage. The structure is made up of steel construction, and composite deck. Lateral support is provided by steel braced frames using HSS steel shapes for cross-bracing. The ground floor is a reinforced slab on grade with grade beams, and drilled caissons that support the buildings columns.

This report focuses on the redesign of the structure's lateral system. It was concluded that the original steel structural design was the most economical. By maintaining this type of building structure no changes had to be made to the large spans and the open floor plan. This means the architecture did not have to face any major changes. Also, included in this report are two breadth studies which are a cost and schedule study, and a blast resistant glazing study. Cost and scheduling studies were performed to compare the effects caused by the redesign of the lateral system. The blast resistance glazing was done as an educational study as of self interest.

The depth study explores the option of building the same building design in San Francisco rather than Philadelphia. Due to the location change, the seismic lateral load would be much larger and would control over the wind load. To reduce the weight of the building, light weight concrete was used in place of normal weight concrete used in the composite steel deck of the building. Vulcraft catalogs were used to design the new floor slab, and a 3D model in ETABS was used to design the preliminary lateral system. Alternate lateral systems were researched, but a dual system was chosen for design. Concentric braced frames were not used by themselves, because the lateral load was too large, and adding lateral resisting frames in different locations would have taken away from the architecture and open floor plan. The dual system includes shear walls and concentric brace frames that are used to resist the large, controlling seismic load. The shear walls are 16" in thickness and are located in place of the concentric braced frames in the core of the building. Hand calculations were performed to confirm and complete the design of the new lateral system. Through cost analysis, it was found that the new lateral system costs over $100,000 dollars less than the original design. But the construction schedule was found to be pro-longed, because construction of concrete walls is a longer process than steel erection.

At the end of this report is an appendix that contains all the calculations for the loads stated above.

ACKNOWLEDGEMENTS

The author of this report wishes to recognize and sincerely thank the following individuals for their assistance support in the completion of my thesis study.

Ueland Junker McCauley Nicholson LLC:

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Dr. Ali M. Memari

Dr. Andres Lepage

Prof. Robert Holland

The entire AE faculty and Staff

A special thanks to my family, friends, and peers for all their support and assistance.

INTRODUCTION

The Science Center Research Park is an addition to the growing research/science development in the University City area. "The Science Center is the nation's preeminent destination for early-stage life science companies across the globe", said Pradip Banerjee. The building includes offices, wet labs, retail space, and a 500 car parking garage. It is covered by glass curtain wall, stone, and a brick veneer along the Market Street facade.

This report contains a redesign of the lateral system for the Science Center Research Park building in order to gain a better understanding of the effect of large seismic loads caused by the relocation of the site to an active seismic zone (San Francisco). This report will conclude with a new lateral system design along with a study of the cost and schedule effect and a blast resistant glazing study.

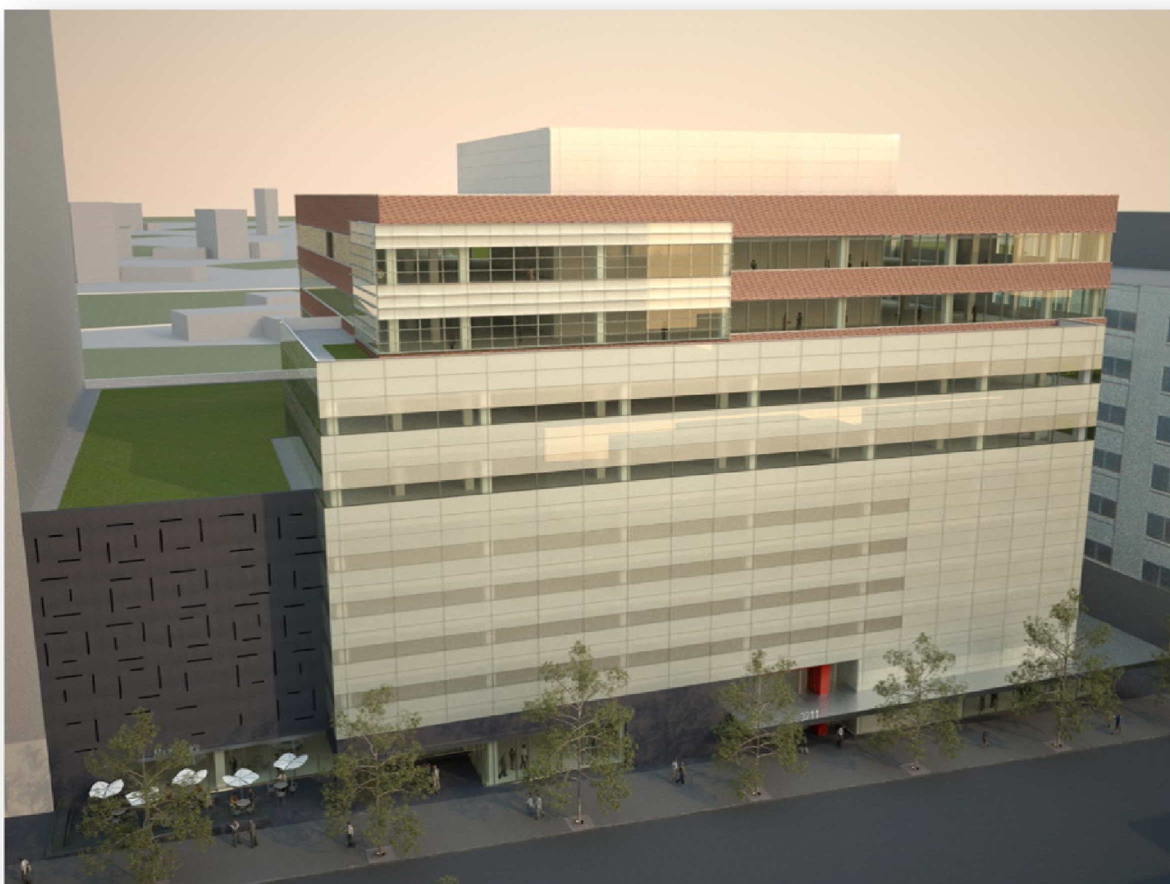


Figure 1: *Render Image provided by UJMN, LLP*

BUILDING STATISTICS

Building name: Science Center Research Park, 3711 Market Street

Location and site: 3711 Market Street Philadelphia, PA. 19104

Building Occupancy Name: Science Center and Rosetta Genomics, Inc.

Occupancy or function types (type of building): Mixed occupancies, non-separated uses

Size (total square feet): 401,032 GSF

Number of stories above grade/ total levels: 11

Applicable Codes:

Building Code (2003 IBC with Amendments by the City of Philadelphia)
Mechanical Code (Philadelphia Amendments to 2003 IMC)
Electrical Code (Philadelphia Amendments to 2003 IEC)
City of Philadelphia Plumbing Code
Fire Code (Philadelphia Amendments to 2003 IFC)
Energy Conservation Code (Philadelphia Amendments to 2003 IECC)
Pennsylvania Universal Accessibility Act

Zoning: Mixed-Use

Office/lab	B
Retail	M
Garage (enclosed parking)	S2

Figure 2: *Interior of ground entrance*

SUSTAINABILITY FEATURES:

Sustainable features include locally- manufactured materials, low-emitting interior finishes, a high performance curtain wall, and the largest green roof in the city of Philadelphia. The Science Center Research Park achieved LEED® certification.

ARCHITECTURE (DESIGN AND FUNCTIONAL COMPONENTS):

The Science Center is an addition to the growing research/science development in the University City area. The building includes offices, wet labs, retail space, and a 500 car parking garage. It is covered by curtain wall with a stone base along the Market Street facade. The mixed-use building is inviting for several different audiences with a curtain wall ground floor for retail spaces and a strong modern entrance.

BUILDING ENCLOSURE:

Clad with high performance glass curtain wall panels with aluminum mullions and a brick façade along Market Street. The storefront curtain walls on the ground floor are inviting for the retail spaces. Typical roofing system consists of steel roof decking with rigid roof insulation and a waterproof membrane. The roof also includes a state of the art 35,000 square foot PVC "Green" Roofing System by Sarnafil Inc.

Figures 3& 4: *Existing Green Roof and Entrance*

PRIMARY ENGINEERING SYSTEMS

Construction:

The construction of the Science Center Research Park started in November 2006 and ended in May 2008. The construction manager for the construction of the Science Center Research Park was Intech Construction, Inc. The delivery method was fast track.

Electrical:

The electrical system was designed by Vinokur-Pace Engineering Services, Inc, and is powered by PECO. The service voltage is 480/277 volts, 3 phase, 4 wire, but smaller motors and lighting were connected to 277 and 120 volt single phase circuits as indicated in the plans. The new dual 13.2 electric service to the Building is nominal 13.2 KV from PECO. The lines run through 4000 A buses before being distributed to main panel boards. Typical circuit panels can be found each floor to power spaces for future tenants.

Lighting:

The lighting system primarily uses fluorescent lighting fixtures throughout the building. The main spaces on floors 6 - 9 are luminated by 22 inch diameter HID industrial low bay luminaries, and each with a faceted reflector and an injection molded lens. Quartz restrike. This type of luminaire runs off of 250 watts and the quartz runs off 100 watts. The main spaces on floors ground – 5 are laminated by 18 inch diameter HID fixtures with die cast housing and an electrical ballast. This type of luminaire runs off of 150 watts. The Science Center Research Park building was LEED© certified. Therefore in order to obtain the status of a “green building”, the designers incorporated maximum day-lighting when designing the building. This gave the lighting designers an opportunity to take advantage of the sun’s light and use fewer luminaries.

Mechanical:

The mechanical system was designed by Vinokur-Pace Engineering Services, Inc. The system was designed using HVAC and exhaust requirements for a mixed-use building; which includes labs, offices, retail space and a 5 story parking garage. This means that the mechanical system is consisted of several different components. The most obvious of the components are the large exhaust fans on the roof mainly used for the lab spaces. The top floor of the building is a penthouse mainly used for mechanical space. Cooling towers, water tanks, hot water boilers, hot water unit heaters, and mechanical exhaust fans are located at the penthouse level. A main air handling unit, with power to supply 2700 CFM, can be found on the ground.

ENGINEERING SUPPORT SYSTEMS

Fire Protection:

The building is has an automatic sprinkler system which classifies the building as fully sprinkled. All exposed structural steel members are sprayed with fire protection. The structural frame, bearing walls, and floor construction have a fire resistance rating of 2 hours. The roof system and non-bearing walls have a fire resistance rating of 1 hour. Also, the building has pressurized stairwells.

Transportation:

The Science Center Research Park has a main service core adjacent to the lobby that contains 4 elevators and 1 stairwell. On the backside of the building 2 stairwells can be located that fall in accordance to egress codes. The stairwell in the main service core is the way to access the mechanical penthouse.

Telecommunications:

All the spaces in the building are supplied with outlets and receptacles. Tenants will be able to set up communication systems as needed.

EXISTING STRUCTURAL SYSTEM

FOUNDATION

The building's foundation system is composed of cast-in-place reinforce concrete grade beams and piers. Its deep foundation consists of drilled caissons that range from 3 to 5 feet in diameter, and 20 to 30 feet below grade. These caissons can carry loads up to 1900 kips depending on the size. The general thickness of the slab on grade is either 4 or 6inches depending on indication on plans, but is also 12 inches thick in some areas. Columns are also cast-in-place in some areas of the ground floor, but transfer to steel columns. All the concrete in the building has a compressive strength of 4000 psi; except for the caissons and steel column encasements have a compressive strength of 3000 psi.

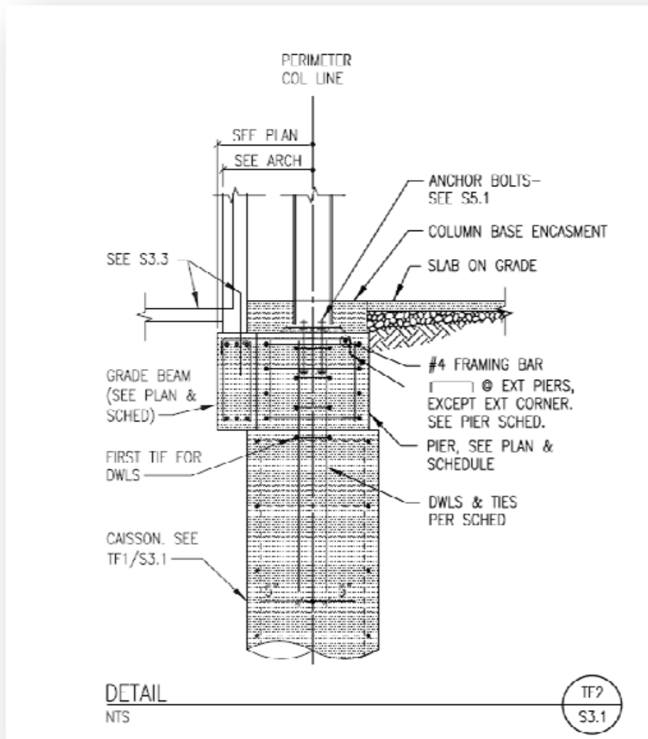


Figure 5: *Typical Caisson/foundation detail*

FLOOR SYSTEM

The floor system is a composite steel slab system on steel beams with a typical bay size of 31'6" x 31'6". The typical composite deck is composed of 6 inches of normal weight concrete and 1.5" – 18 gauge composite steel decking with ¾" studs. The floor is supported typically by W 18 x 40 beams and W 24 x 84 girders, but there are large amount of other W - shapes used. The roof consists of 1.5" – 18 gauge steel roof deck supported typically by W 16 x 26 beams and W 24 x 55 girders. Refer to typical bay layout and overall plan such as shown on page 20.

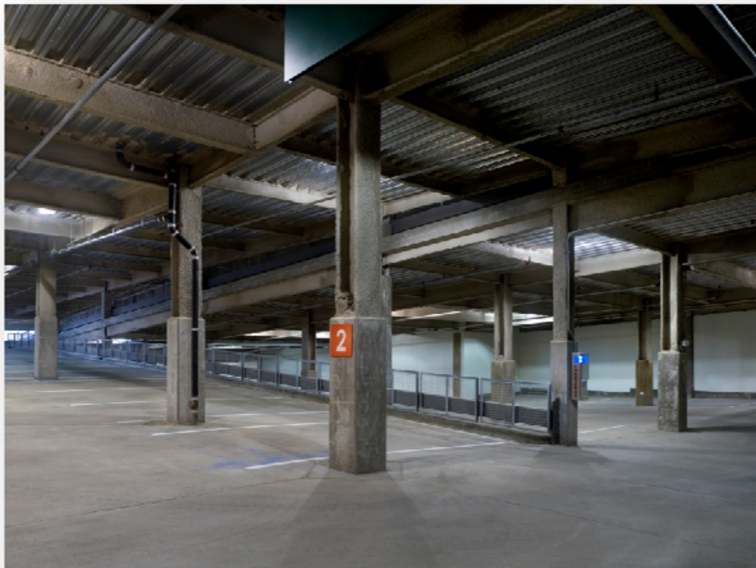


Figure 6: *Existing Parking Garage*

LATERAL SYSTEM

The building's lateral system is composed of braced frames strategically placed on each floor. Braced frames are located in the walls of the main elevator and stairwell core in the center of the building, in some exterior walls, and in the exterior walls of the penthouse. The braces are hollow structural steel members. Typical brace members are HSS 8 x 8's and HSS 6 x 6's were used, but several different sizes were used. Shear loads at the end of the beams is typically 10 kips, unless indicated otherwise on the plans. Also, column splices transmit compression forces in end bearing with a minimum of 15 kips of shear. Two bays of the braced frames in the center core connect into the buildings foundation transfer the shear load. Refer to typical braced frame layout shown in figure.

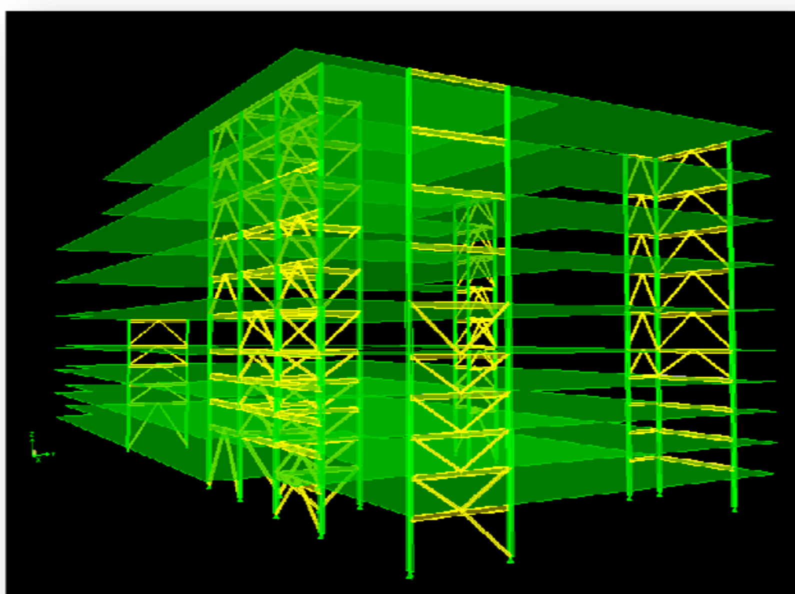


Figure 7: *3D Model of Lateral System in ETABS*

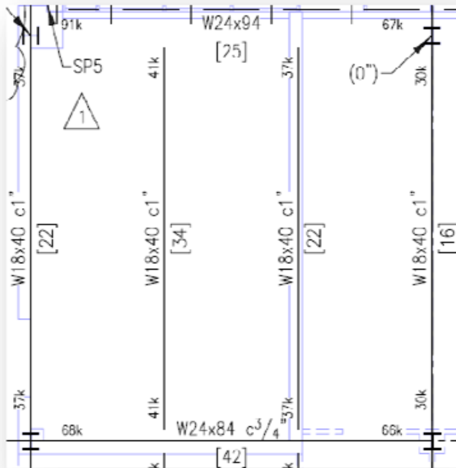


Figure 8: Typical Interior Bay

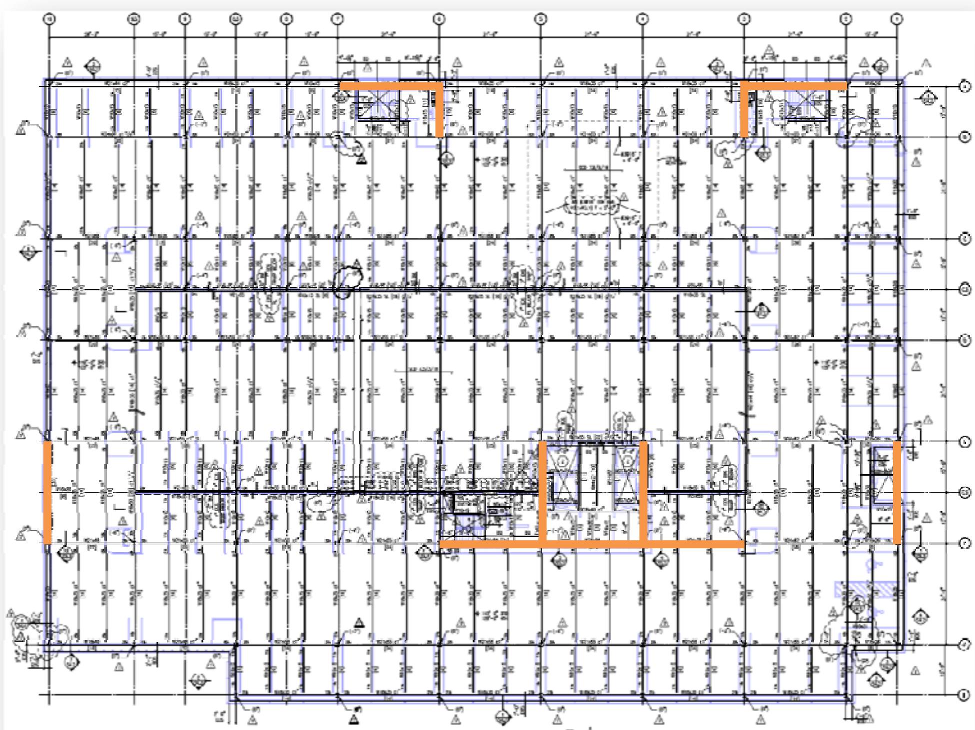


Figure 9: Typical Floor Framing Layout and Lateral Bracing Layout

PROPOSAL

PROBLEM STATEMENT

Proposed Goal: To change the site location of Science Center Research Park into an active seismic zone, and to redesign the lateral system.

The Science Center Research Park building is composed of a steel structure and a composite steel floor slab using normal weight concrete. The use of a composite steel deck is very economical choice. The lateral system is a braced frame system consisted of HSS steel shape members. The braced frames are located around the edges of the building, and also around the main elevator/stairwell core to resist lateral loads caused by wind and seismic forces. Through analysis in technical reports 1 through 3 it was found that the structural design of the building meets strength, and serviceability requirements.

Changing the structural system to a concrete structural system would increase the building weight, which would only increase the seismic load on the building. Though the cost of material would be reduced, concrete construction requires formwork and shoring. The schedule would be adjusted to a longer construction period, because the concrete needs to cure and each floor has to be shored. Although a two-way flat slab system was found to have the same span and a larger floor-to-ceiling height the size of columns would be much larger. The composite steel slab system was found to have the largest span and one of the cheapest choices out of the researched floor systems in technical report 2. Therefore, the floor system will not be changed.

PROPOSED SOLUTION

As stated above, the Science Center Research Park building will remain a steel structure with a composite steel slab floor system. Steel is the most economical choice. The Science Center Research Park building is an 11 story building and by increasing the building weight it would only create a larger base shear created by lateral forces. To reduce the building weight the change from normal weight concrete to light weight concrete is a valid option. The gravity members can be reduced in size if the change to light weight concrete is taken into account.

Proposed Option:

Relocating the Science Center Research Park building into an active seismic zone would cause change of lateral loads. A specific location and site are being researched, due to the necessity of specific seismic factors and a soils report. The lateral system will be redesigned to resist the new lateral loads. The existing lateral system is a braced frame system. Alternative lateral systems will be investigated in response to the change of controlling lateral loads. A 3D model will be created in ETABS to determine if the redesign of the lateral system is adequate.

BREADTH OPTIONS

BREADTH STUDY 1: IN-DEPTH COST AND SCHEDULE ANALYSIS (CM)

This breadth study will investigate the scheduling and cost impact of the change of the floor slab to light weight concrete and the change of the lateral system. The scheduling changes consist of the additional time that would possibly be needed. The cost of the original design will be compared to the cost of the proposed redesign. The cost will be affected by change of member sizes, the proposed lateral system, construction time and labor costs.

BREADTH STUDY 2: BUILDING ENCLOSURES

This breadth study will investigate the option of a blast-resistant glass façade. By changing the existing glass façade to blast-resistant glass façade considerations for acoustical, lighting and thermal effects have to be taken into account for. The disadvantages of the proposed changed should be investigated. A design of connections of the façade to the steel structure will be done. All Calculations will be done by hand and using current standards and codes.

DESIGN GOALS

The goal of the depth study was to determine how feasible it would be to design the same building design as 3711 Market Street in San Francisco, which is an active seismic zone. Depending on the design loads calculated for the decrease in building weight and the new lateral loads, an appropriate lateral system will be designed for the building. The composite steel deck will be redesigned using lightweight concrete to decrease the building which also will decrease the seismic load on the building. The gravity resisting steel structure will not be redesigned, but an investigation of whether the members can be reduced in size will be done. Other goals taken into consideration for the redesign are listed below.

- To limit the change of the existing column layout and typical bay size in order to keep the large open floor plan.
- To use ETABS Nonlinear v9.5 to perform preliminary designs of the lateral system and use hand calculations to finalize the design.
- To maintain the story and building drift within the serviceability standards of H/400 for wind loads and 0.015hx for seismic loads.
- To determine the effect of the material and construction costs, and the duration of construction.
- To use all necessary and current codes, and standards in the redesign of the structural system.

STRUCTURAL DEPTH

INTRODUCTION

3711 Market Street was originally designed as a steel structure with a composite steel slab system using normal weight concrete and concentric steel bracing for the lateral system. It was designed to have large spans and open space for office/lab tenants. Not only does the steel structure provide large open spaces, but it is also the most economical choice. The steel structure was chosen not to be changed for those reasons. The redesign will include the change of the normal weight concrete used for slabs to light weight concrete which will reduce the weight of the building. Also, a redesign of the lateral system was chosen to be done using a site in San Francisco rather than Philadelphia. The reason for the relocation of the building is that San Francisco is an active seismic zone, and the seismic load on the building should control. This could be thought of as simulating that an owner would want the same building design in another location. Also, self interest in the changes and effects of the change in the seismic load on the building and redesigning the lateral system. Though a alternate lateral systems were researched, the redesign of the lateral system will consist of a dual system using concentric steel bracing and shear walls in the core of building. The redesign will use the most current codes and standards where appropriate.

DESIGN PROCEDURE

The purpose of using steel construction and a composite steel deck was to maintain the 31.5' spans and the open floor plans. To decrease the weight of the building lightweight concrete was used in place of the normal weight concrete used in the original design. Live loads used to design the floor system were all current standards taken from ASCE 7-05. Vulcraft Steel Roof and Floor Deck Product Catalogs and hand calculations were used to determine the final design of the composite steel deck. The computer software ETABS Nonlinear v9.5 was used to design a preliminary lateral system and to determine if the new lateral system was adequate. Hand calculations were used to reconfirm and complete the lateral system design. Serviceability criteria for the lateral system was checked and confirmed to be okay. An investigation to determine whether gravity members can be reduced was done, but the gravity system was not redesigned due to the scope of this thesis. The hand calculations can be found in **Appendix D**.

CODE

CODE/ REFERENCES

- ASCE 7-05 *Minimum Design Loads for Buildings and Other Structures*
- IBC 2006 *International Building Code*
- ACI 318-08 *Building Code Requirements for Structural Concrete*
- AISC 13th Edition Steel Construction Manual
- Vulcraft Steel Roof and Floor Deck Product Catalog
- 2010 RSMeans Construction Cost Data
- ASTM E 1300
- ASTM F 2248

Note: The following codes and references were used in the original design and in this report. All references are up-to-date building design standards.

DRIFT CRITERIA

Allowable Building Drift = H/400

Inner-Story Drift

Wind = h/400 to h/600

Seismic = 0.015h

LOAD COMBINATIONS

The following load combinations were used in the combination of factored gravity and lateral loads. These combinations were used for the 3D model analysis done using ETABS. The four different wind load cases stated below were also used when considering these load combinations.

1. 1.4(Dead)
2. 1.2(Dead) + 1.6(Live) + 0.5(Roof Live)
3. 1.2(Dead) + 1.6(Roof Live) + (1.0(Live) + 0.8(Wind))
4. 1.2(Dead) + 1.6(Wind) 1.0(Live) + 0.5(Roof Live)
5. 1.2(Dead) + 1.0(Seismic) +1.6(Wind)
6. 0.9(Dead) + 1.6(1.6(Wind)
7. 0.9(Dead) +1.0(Seismic)

Note: The above criteria were taken from ASCE 7-05.

MATERIAL

Concrete

Slabs on grade	f'_c = 4000 psi
Slab on steel deck	f'_c = 4000 psi
Drilled caissons	f'_c = 3000 psi
Foundation walls, piers &grade beams	f'_c = 4000 psi
Steel column encasement	f'_c = 3000 psi

Structural Steel

W – Shapes	ASTM A992
Bars, rods and plates	ASTM A36 (UNO)
All other structural shapes	ASTM A36
Pipes	ASTM A53, Grade B
Cold-formed hollow structural sections (tubing)	ASTM A500, Grade B
High strength bolts	ASTM A325
Deformed bar anchors	ASTM A706 Low Carbon
Anchor rods	ASTM A36
Shear connectors (headed)	ASTM A108, Grade 1010 to 1020

GRAVITY AND DESIGN LOADS

Dead Loads

Concrete	150 pcf
Light Weight Concrete	115 pcf
Partitions	20 psf
M.E.P.	5 psf
Finishes and Misc.	3 psf
Roof Deck	2.6 psf
Rigid Insulation	4 psf

Live Loads

Corridors, Lobbies & Exits	100 psf
Labs / Offices	100 psf
Garage	40 psf
Mechanical Equip. Rooms	150 psf
Roof	30 psf

Note: The above loads were taken from ASCE 7-05, and used in the original design and in this report.

LATERAL LOADS

WIND LOADS

Wind loads were determined using ASCE 7-05 Section 6.5 which describes Method 2. The detailed analyses of the wind loads can be found in **Appendix B**. Below are summary tables including wind factors and wind loads calculated for north-south and east-west elevations.

Wind Pressure Tabulation at each level

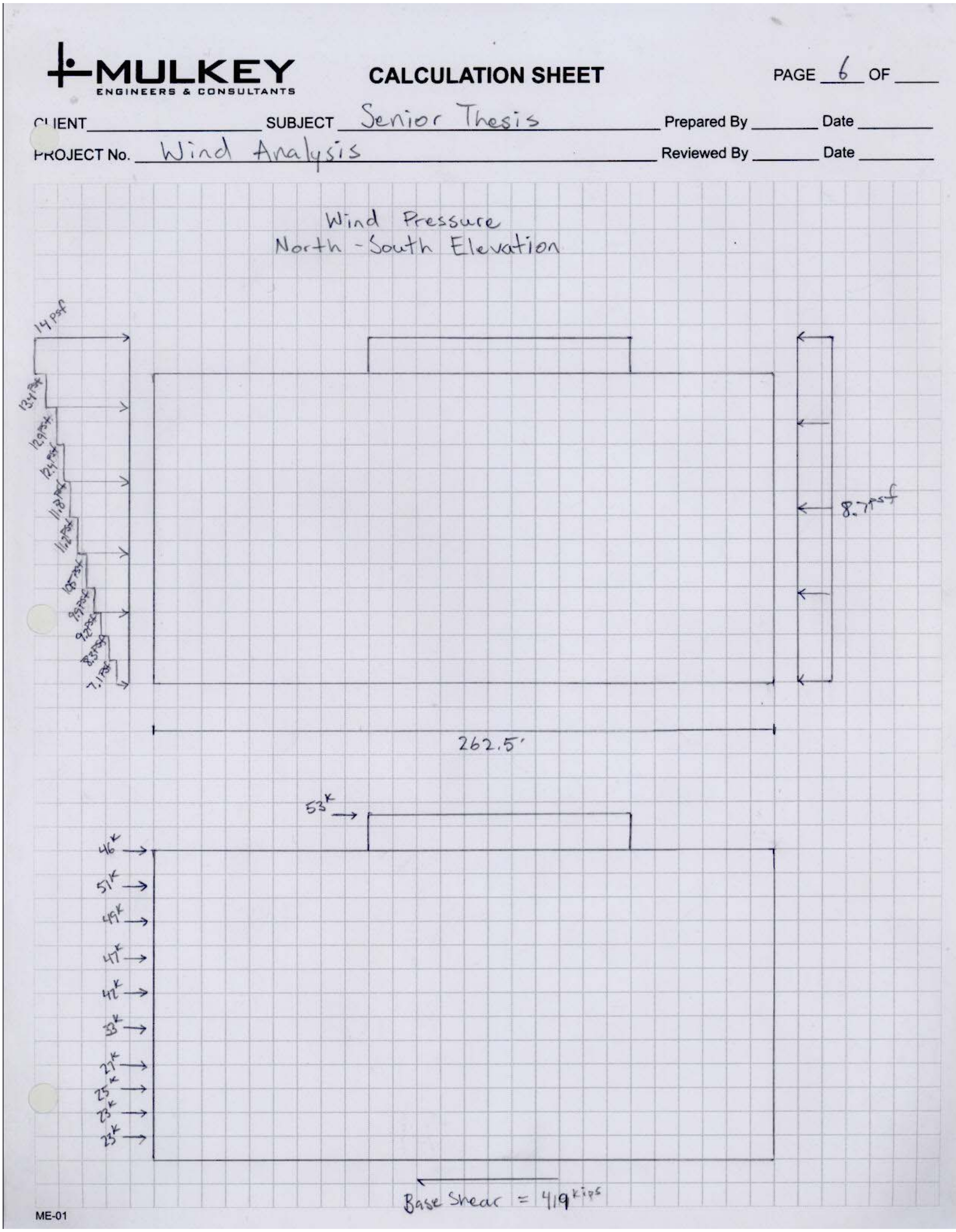
	Level	Height Above Ground (ft)	K_z	q_z	Wind Pressure	
					N-S (psf)	E-W (psf)
Windward	Pent House	147.5	1.10	19.97	14.0	14.2
	Roof Level	140.17	1.09	19.68	13.8	13.9
	T.O. Parapet	131.17	1.07	19.31	13.5	13.7
	10	125.5	1.05	19.07	13.4	13.5
	9	110.83	1.02	18.40	12.9	13.0
	8	96.17	0.98	17.67	12.4	12.5
	7	81.5	0.93	16.85	11.8	11.9
	6	66.83	0.88	15.92	11.2	11.3
	5	53.5	0.83	14.94	10.5	10.6
	4	43.5	0.78	14.09	9.9	10.0
	3	33.5	0.72	13.07	9.2	9.3
	2	23.5	0.65	11.81	8.3	8.4
	1	13.5	0.56	10.08	7.1	7.1
Leeward	All	147.5	1.10	19.97	-8.7	-7.4

Table 1: *Wind pressure tabulation at each level*

Wind Load Tabulation: Force and Shear at each level

Level	Height Above Ground (ft)	Floor Height (ft)	h/2 above	h/2 below	Wind Forces			
					Load (kips)		Shear (kips)	
					N-S	E-W	N-S	E-W
Pent House	147.5	0.00						
Roof Level	140.17	7.33	7.33	7.33	53	39	53	39
T.O. Parapet	131.17	9.00						
10	125.5	5.67	5.67	7.33	46	33	99	72
9	110.83	14.67	7.33	7.33	51	37	150	109
8	96.17	14.67	7.33	7.33	49	35	198	145
7	81.5	14.67	7.33	7.33	47	34	245	178
6	66.83	14.67	7.33	6.67	42	31	287	209
5	53.5	13.33	6.67	5	33	24	321	233
4	43.5	10.00	5.00	5	27	19	347	253
3	33.5	10.00	5.00	5	25	18	372	271
2	23.5	10.00	5.00	5	23	17	395	288
1	13.5	10.00	5.00	6.75	23	17	419	305
Total	147.5				419	305		

Table 2: *Wind Load Tabulation: Force and Shear at each level*



MULKEY
ENGINEERS & CONSULTANTS
CALCULATION SHEET
PAGE 6 OF
CLIENT
SUBJECT Senior Thesis
Prepared By
Date
PROJECT No. Wind Analysis
Reviewed By
Date
Wind Pressure
North - South Elevation
14 psf
13.4 psf
12.9 psf
12.4 psf
11.8 psf
11.2 psf
10.5 psf
9.9 psf
9.2 psf
8.3 psf
7.1 psf
8.7 psf
262.5'
53k
46k
51k
49k
47k
42k
33k
27k
25k
23k
23k
Base Shear = 419 kips
ME-01

MULKEY
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CALCULATION SHEET

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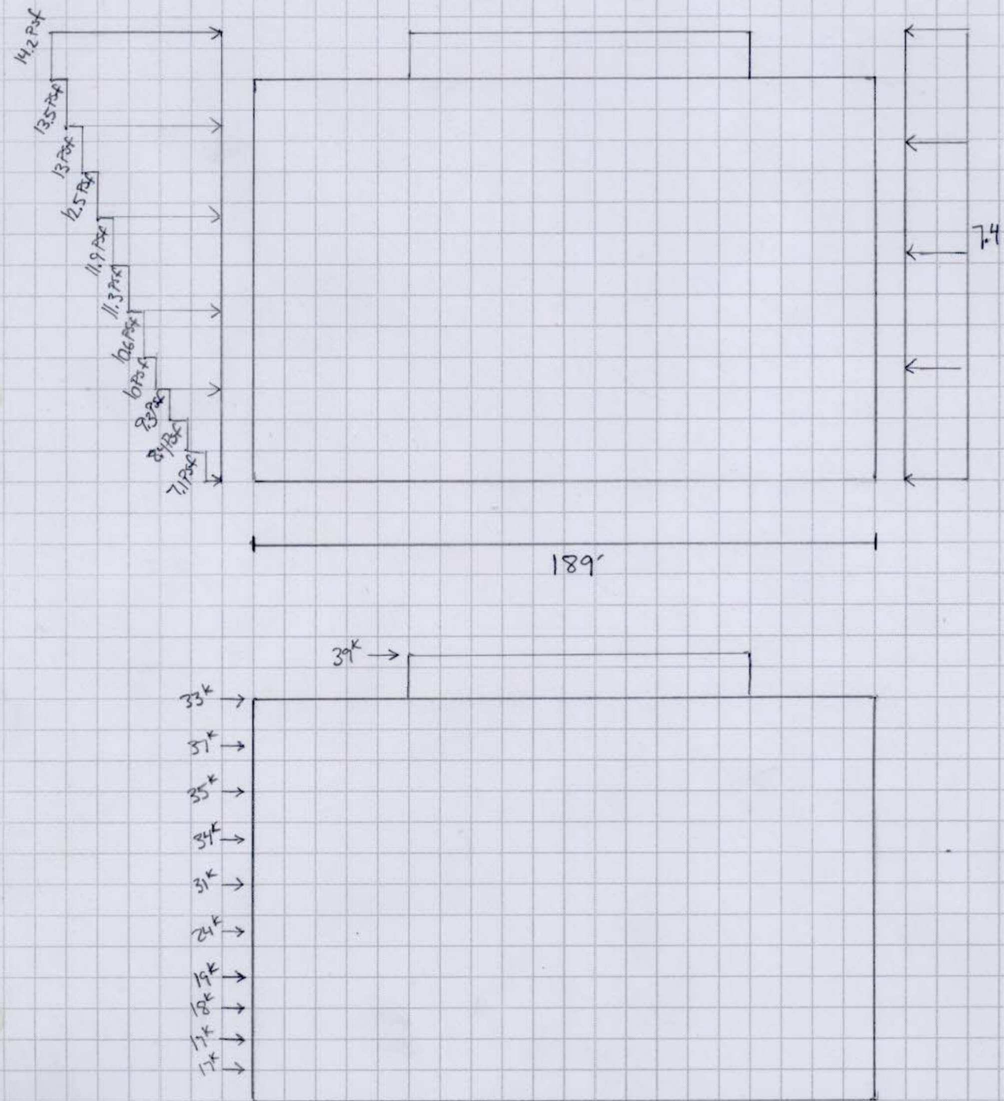
PROJECT No. Wind Analysis Reviewed By ________ Date ________

Wind Pressure
East-West Elevation



Base Shear = 305 kips

ME-01

SEISMIC LOADS

Seismic loads were determined using ASCE 7-05 chapters 11 and 12. The Equivalent Lateral Force Procedure was used for the calculation of the seismic loads. A detailed analysis of the seismic loads can be found in appendix C. Building weight was calculated for each floor of the typical steel constructed building. 3711 Market Street's building weight includes the dead loads that are listed in the tables below. The floor dead load includes the appropriate dead load loads for each floor that are listed under the "Gravity and Design Loads" on page 20. The weight of the light weight concrete composite deck is 43 psf which was taken from a Vulcraft composite deck catalog and can be found in Appendix. Floor dead load includes dead loads caused by M.E.P., partitions, and finishes which can be found under "Gravity and Design Loads". The building weight was adjusted as the design of the lateral system change in design. Seismic load calculations can be found in **Appendix C.**

Building Weight Tabulation

Floor	Floor Area (ft^2)	Floor Dead Load	Floor Weight (lbs)	h/2 above (ft)	h/2 below (ft)	Column weight/length total (plf)	Column weight= height*weight/length (lbs)
Ground						11445	
1st	33,833	71	2402143	5	6.75	11498	134743.75
2nd	50,705	71	3600055	5	5	11566	115320.00
3rd	50,705	71	3600055	5	5	7385	94755.00
4th	50,705	71	3600055	5	5	7385	73850.00
5th	40,433	71	2870743	6.67	5	7205	84958.33
6th	34,439	71	2445169	7.33	6.67	4797	83211.33
7th	34,439	71	2445169	7.33	7.33	4797	70356.00
8th	30,439	71	2161169	7.33	7.33	2960	56884.67
9th	30,439	71	2161169	7.33	7.33	2960	43413.33
Penthouse	6,437	71	457027	7.33	7.33	728	27045.33
$Roof_{penthouse\ level}$	21,509	14.6	314031.4		7.33	2960	21706.67
Roof	6,437	14.6	93980.2		7.33	728	5338.67
Total			26056785.4				806244.42

Table 3: *Building Weight Tabulation*

Floor	Approx. Beam weight (lbs)	Curtainwall (estimated length along	Curtainwall height (ft)	Curtainwall weight (height*length	Shear Wall weight	Brace Frame Weight
Ground						
1st	257591.00	913.5	10	137025	340200	4276
2nd	257591.00	913.5	10	137025	252000	7459
3rd	257591.00	913.5	10	137025	252000	4384
4th	257591.00	913.5	10	137025	252000	4384
5th	240266.00	913.5	13.33	182654.325	252000	4786
6th	178765.00	913.5	14.67	201015.675	340200	5514
7th	141120	850.5	14.67	187152.525	369432	3003
8th	141120	850.5	14.67	187152.525	369432	3003
9th	141120	819	14.67	180220.95	369432	3003
Penthouse	30240.00	378	14.67	83178.9	369432	3003
$\text{Roof}_{\text{penthouse level}}$	7180.00					
Roof	92547.00				369432	
Total	1910175			1569474.9	3166128	42815
Total Building Weight		33551.62 kips				

Table 4: *Building Weight Tabulation (continued)*

Seismic Load Tabulation

Building forces including story and base shears were calculated after the tabulation of the building weights. These forces are shown in the table below.

Level	Story Weight w_x (Kips)	Height h_x (ft)	$w_x h_x^k$	Lateral Force F_x	Story Shear V_x	Moment M_x (ft-k)
1st	3275.98	13.5	46467.70	17.9	803.32	242.10
2nd	4369.45	23.5	109029.73	42.1	785.39	988.81
3rd	4345.81	33.5	155629.31	60.1	743.31	2012.05
4th	4324.91	43.5	202114.34	78.0	683.25	3393.03
5th	3635.41	53.5	209771.08	81.0	605.25	4331.13
6th	3253.88	66.83	235530.30	90.9	524.29	6074.63
7th	3216.23	81.5	284981.75	110.0	433.40	8963.48
8th	2918.76	96.17	306137.08	118.1	323.41	11362.07
9th	2898.36	110.83	351283.61	135.6	205.27	15025.09
Penthouse	969.93	125.5	133430.98	51.5	11.47	6462.52
$\text{Roof}_{\text{penthouse level}}$	342.92	125.5	47174.61	18.2	30.99	2284.83
Roof	561.30	147.5	91031.66	35.1	0	5181.87
Total Shear				838		

Table 5: *Seismic Load Tabulation*

The wind calculations do not include the 1.6 factor, but even after utilizing the 1.6 factor the seismic loads still control the design of the lateral system.

LATERAL SYSTEM DESIGN

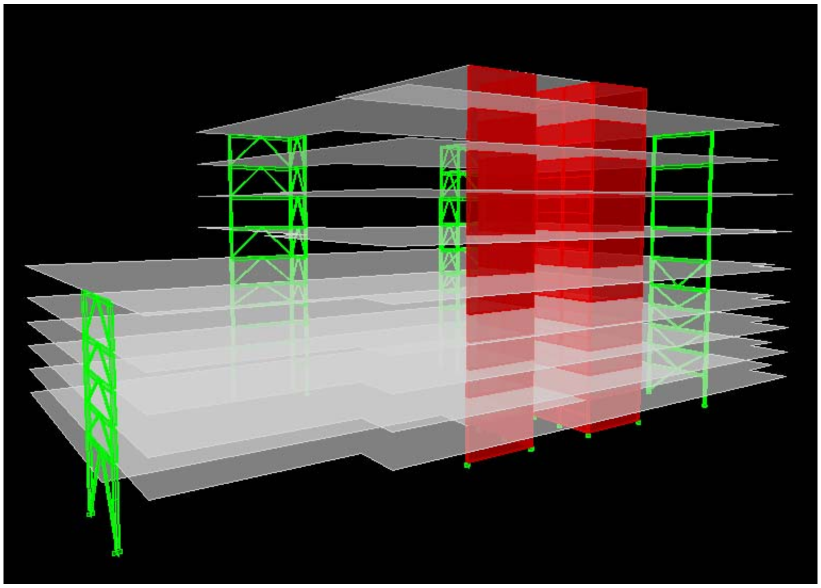


Figure 10: *Graphic of ETABS Model*

A computer model of the Science Center Research Park building was used to design the lateral system and analyze the loads applied. An ETABS model was created including only the lateral elements and diaphragms. The reason is simplicity and the reductions of possible errors. The seismic loads were applied to the center of mass. The ETABS model was used to calculate relative stiffness, seismic drifts, the center of mass and rigidity, and overturning moments. Wind drifts were assumed to be insignificant when comparing to the large lateral load caused by seismic loads. The lateral loads were assumed to be transferred through the diaphragms into the lateral frames and shear walls, and down to the base of the building.

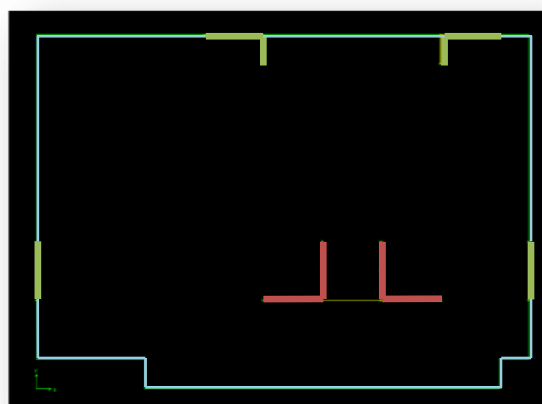


Figure 11: *Lateral System Layout*

Indicates Steel Braced Frame

Indicates Concrete Shear Walls

The new seismic lateral base shear was found to be nearly 3 times as large as the original seismic base shear. This makes sense, because the difference in seismic zones should be roughly 4 to 5 times larger. Reduction of the building weight caused the difference to be slightly less. The controlling lateral base shear was calculated to be 838 kips caused by seismic loads.

Preliminary designs consisted concentric braced frames, but were found to be insufficient to carry the lateral load applied on the building. In order to account for the large increase of lateral load, a dual system was used in the design of the new lateral system. The object of using the same locations for the lateral system was to reduce any changes in the architecture of the building and to maintain the open floor plans. Shear walls were not considered to be used on the perimeter of the building, because of the glass façade and ground floor entrances. The dual system consists of concentric braced frames in the locations around the perimeter of the building, and 4 shear walls were used in place of the existing concentric braced frames located in the core of the building. The shear walls were designed to be 16 inches in thickness. The seismic lateral load was adjusted as the lateral system changed, and was confirmed to be adequate by checking serviceability criteria.

The effect of the cost and schedule due to the redesign of the lateral system is covered in the construction management breadth. The hand calculations for the design of the shear walls can be found in **Appendix D**. The following figures contain the shear walls, concentric braced frames and the member sizes consist in the concentric braced frames.

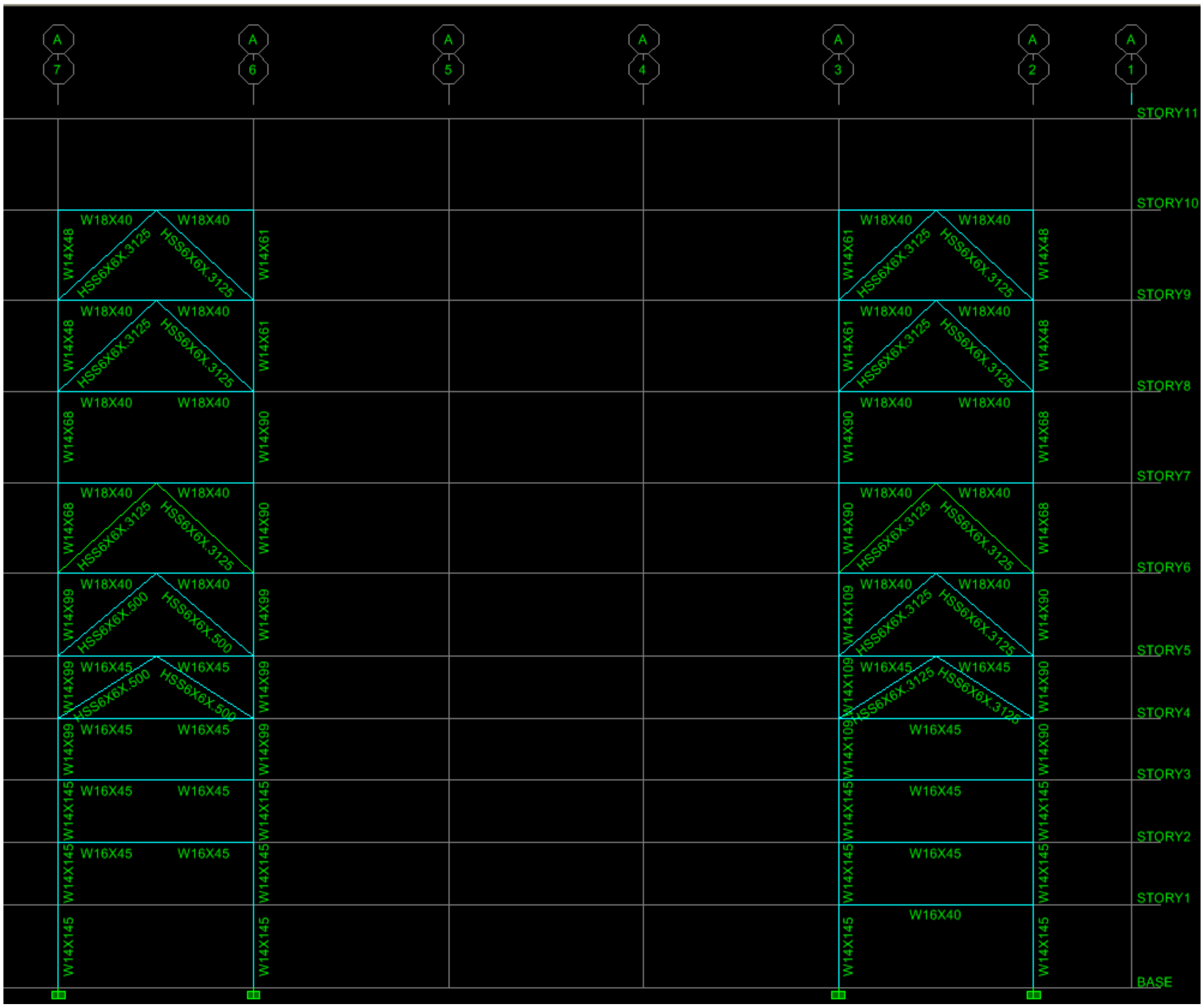


Figure 12: *Brace Frames A2, 3 and A6, 7*

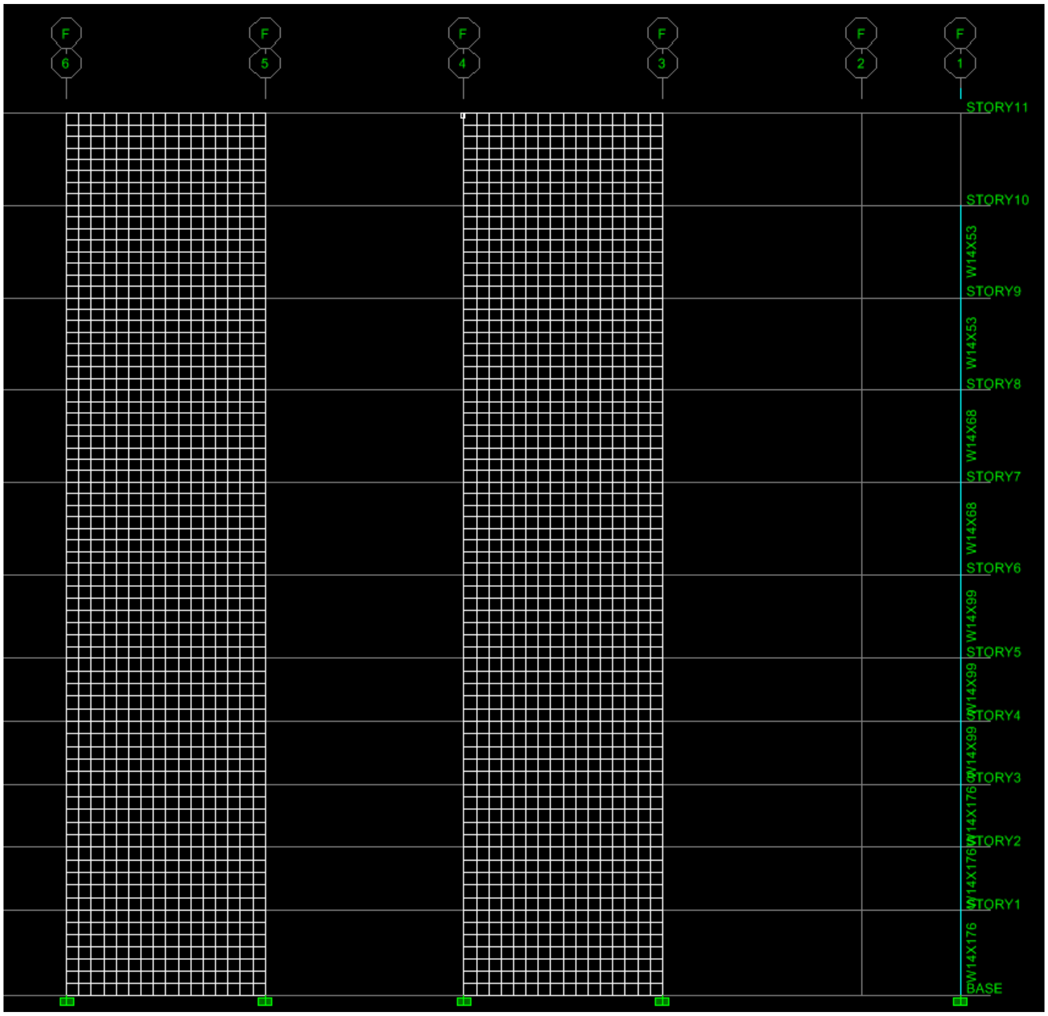
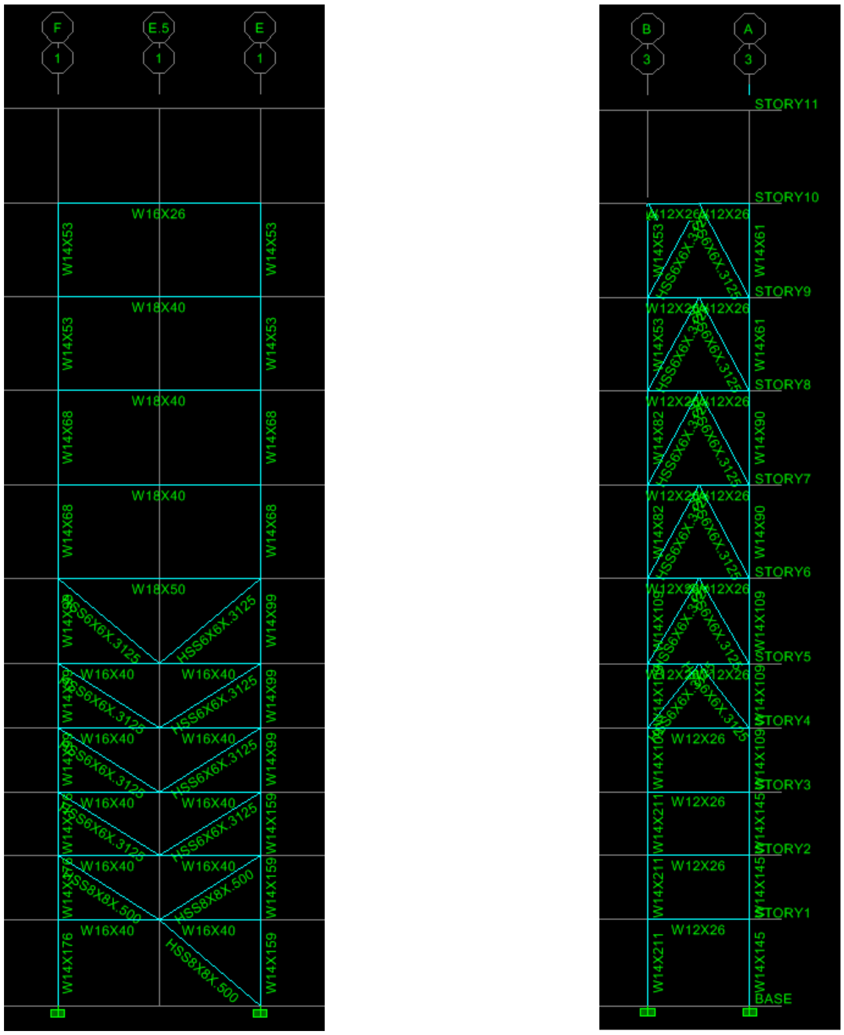
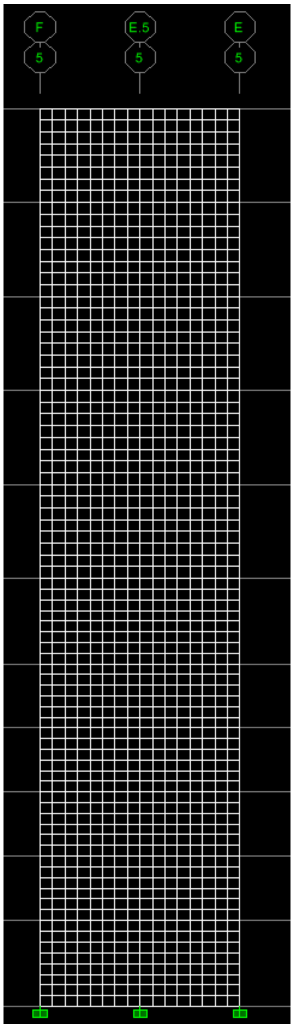
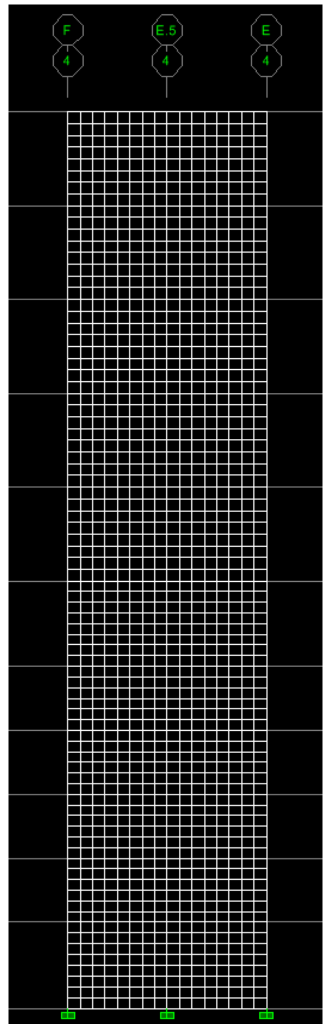


Figure 13: *Shear Walls F3, 4 and F5, 6*



Figures 14& 15: *Brace Frames 1A, B and 1E, F*





Figures 16& 17: *Shear Walls 4E, F and 5E, F*

Figures 18& 19: *Brace Frames 10E, F and 6A, B*

FOUNDATIONS

Overturning happens when the moment created by the building's self weight does not offset the moment created by lateral forces on the building. If the building's self weight does not compensate for the moment, the foundation can be designed to counteract the overturning moment. In the design of the foundation, friction from the soil can be used to assist the foundation to counteract the overturning moment.

In technical report 3 it was found that the weight of the building was sufficient enough to counteract the overturning moment. The foundations are not in the scope of this report, they will not be redesigned. Decreasing the weight of the building, and the much larger lateral load would have an effect on the design of the foundation. Also, the new site location in San Francisco has a different bedrock depth along with different types of soil which would have to be taken into consideration. This design would be to in depth, but is being considered.

SEISMIC DRIFTS

Seismic loads were determined and used in the ETABS model to determine the story drifts. Seismic drift protects against building failure/collapse unlike wind drift which is a serviceability requirement. The drift limitation for seismic drift can be calculated using this equation: $\Delta_{seismic} = 0.015h_{sx}$ (from ASCE 7-05)

The following tables are the seismic drifts, which were found to be acceptable when compared to the allowable drift values.

Controlling Seismic Drift: North-South Direction						
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift $\Delta_{seismic} = 0.015*h_x$ (in)			Total Drift (in)
11	14.67	0.52000	<	2.64060	acceptable	2.76010
10	14.67	0.48000	<	2.64060	acceptable	2.24010
9	14.67	0.42000	<	2.64060	acceptable	1.76010
8	14.67	0.35000	<	2.64060	acceptable	1.34010
7	14.67	0.29000	<	2.64060	acceptable	0.99010
6	13.33	0.24000	<	2.39940	acceptable	0.70010
5	10	0.18000	<	1.80000	acceptable	0.46010
4	10	0.13000	<	1.80000	acceptable	0.28010
3	10	0.09000	<	1.80000	acceptable	0.15010
2	10	0.06000	<	1.80000	acceptable	0.06010
1	13.5	0.03000	<	2.43000	acceptable	0.00010

Table 6: *Seismic Drift in the North-South Direction*

Controlling Seismic Drift: East-West Direction						
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift $\Delta_{seismic} = 0.015*h_x$ (in)			Total Drift (in)
11	14.67	1.01000	<	2.64060	acceptable	5.97046
10	14.67	0.98000	<	2.64060	acceptable	4.96046
9	14.67	0.90000	<	2.64060	acceptable	3.98046
8	14.67	0.80000	<	2.64060	acceptable	3.08046
7	14.67	0.64000	<	2.64060	acceptable	2.28046
6	13.33	0.51000	<	2.39940	acceptable	1.64046
5	10	0.41000	<	1.80000	acceptable	1.13046
4	10	0.32000	<	1.80000	acceptable	0.72046
3	10	0.24000	<	1.80000	acceptable	0.40046
2	10	0.16000	<	1.80000	acceptable	0.16046
1	13.5	0.09000	<	2.43000	acceptable	0.00046

Table 7: *Seismic Drift in the East-West Direction*

STRUCTURAL DEPTH SUMMARY

In conclusion, the newly design composite steel deck was tabulated to be 6 inches in total depth with 3 inches of lightweight concrete. Gravity members were found to be more than sufficient to carry the gravity loads applied, and should be considered to be reduced in size if the redesign of the structural system were to be continued. Due to not changing the steel structure and composite steel deck, no changes had to be done to the bay sizes and spans. It was an important goal for this report and it was met. The tabulated seismic load controlled in the design of the lateral system with a base shear of 838 kips. All serviceability criteria were met. Hand calculations were performed to check the design of the lateral system in ETABS which can be found in **Appendix D**. Also, hand calculations were performed to complete the design of the concrete shear walls; which includes the steel reinforcement and can be found in **Appendix D**. The composite steel deck calculations can be found in **Appendix G**.

BREADTH TOPICS

CONSTRUCTION MANAGEMENT BREADTH

The steel structure used in the design of 3711 Market Street was the most efficient choice. Therefore, the same design was used in this report, but due to the change of the site location the seismic load is much larger. Utilizing shear walls and concentric braced frames was a result of the large lateral load. The purpose of this breadth was to determine how the redesign would affect the cost and schedule of the project. It is assumed that the shear wall will cost less than a concentric braced frame, but it should cause the schedule to be longer to build the shear wall due to curing and etc. This breadth includes a detail cost and schedule analysis comparing the original lateral design to the new lateral design.

CONSTRUCTION METHODS

In most cases, the goal is to reduce the cost and duration to construct a building. But this breadth is meant to only determine the effect and the difference in the cost and schedule. By determining the difference it will be clear as to how feasible it would be to build the same design as 3711 Market Street in an active seismic zone such as San Francisco. Research was done to determine feasible this would be. The construction was assumed to be erected a floor-to-floor.

Figure 20: *Building during construction phase*

COST

A detailed cost analysis was performed on both the existing and newly design lateral system. 2010 R.S. Means Construction Cost Data was used to estimate the material and construction cost of the lateral system. When estimating the cost of the shear walls, things such as formwork, concrete, reinforcement and labor were taken into account. Estimating the cost of the concentric braced frames included the W-shapes, the hollow tubing HSS-shapes, and labor. The cost tabulation can be found in **Appendix E**.

Below in Figure it was determine that the new lateral system design would save close to $200,000 in construction.

Existing Lateral Sytem		New Lateral Sytem	
		Total Steel Cost:	$203,274.76
		Total Concrete Cost:	$114,525.47
Total Structural Steel Cost:	$500,276.86	*Total Lateral System Cost:*	$317,800.23
Total Difference in Cost:	$182,476.63		

Figure 21: *Lateral system cost comparison*

SCHEDULING

In order to build schedules for comparison of the construction duration time of labor, unit types, and amounts had to be taken into consideration. As stated before the construction method was assumed to be constructed floor-by-floor. This means that the shear walls were formed, then poured and cured per floor before the slab could constructed on the floor above. For simplicity, no schedule was actually made. The labor hours were totaled up for both the existing lateral system and the newly design lateral system. Concrete generally takes a lot more time to construct, because of reinforcing, formwork, shoring and curing. It was expected that the total duration of construction would be increased by a lot. It was determined that the new lateral system's construction duration be increased roughly by 14 weeks. Calculations of the total construction time of each system can be found in **Appendix E**.

CONSTRUCTION MANAGEMENT SUMMARY

In conclusion, the result of this breadth study was expected, but the extent of the difference in cost and construction during was intent of the study. It was determined that the new system will cost roughly $200,000 less than the existing, and it will take roughly 14 weeks longer to construct. This information would be crucial in the decision of an owner who would want this building design built in San Francisco. The goal of this breadth study were met.

BUILDING ENCLOSURE BREADTH: BLAST GLAZING

Though 3711 Market Street might not be considered at target of a terrorist attack, but that is something that cannot be planned or anticipated. The purpose of this breadth is for self education of blast resistant façade design. The structure of a building should not be the only consideration when designing a building to resist a blast or explosion. The façade should also be designed to decrease the amount of injuries or casualties during such a distressing event. The glass curtain wall along the ground floor off the main road will be designed to resist a blast load and reduce any chance of glass related injuries. The existing glass façade was designed to obtain the minimum thickness using ASTM E 1300.

Using ASTM 2248-03 a 3 second equivalent design pressure will be calculated. In order for this to be calculated a standoff distance and TNT charge mass has to be used in a procedure provided by the guide. After this is calculated, ASTM E 1300-04 can be used to design a laminated glass unit with a thickness which has a larger load resistance than the equivalent design pressure. The designed glazing will be consisted of two lites which will be laminated glass with the same thickness. Both of these lites will be assumed to fracture to be an efficient design. Also, heat strengthened glass would be helpful in the design due to its increased strength compared to normal glass. This type of glass looks better than other strengthened glass in addition. The standoff distance was assumed to be 30 feet in distance with basic information and knowledge of the site. The standoff distance and the charge weight (TNT equivalent lbs) to determine an equivalent three second blast design pressure. The charge weight was taken from **Figure 21** which is an unofficial method provided by the United States Department of Transportation.

Device	Description	Charge Weight (TNT Equiv. lbs)
	Pipe Bomb	5
	Suitcase	50
	Compact Sedan	220
	Full Size Sedan	500
	Passenger / Cargo Van	1,000
	Box Truck	4,000
	Semi-Trailer	40,000

Figure 22: *Charge Weight Guide*

The charge weight was assumed to be as large as 200 pounds of TNT, which is equivalent to a small car explosion. Utilizing charts contained in ASTM 224-03, the three second equivalent design pressure was assumed to be 10 kPa.

Along the glass curtain wall located on the first floor it was determined that the most critical pain window section would be the largest pain of glass. The largest pain of glass was found to 3.05 m x 1.22 m. The first floor's glass façade will be the only section of the building redesigned for blast resistance. Determine the load resistance includes a few variables. The glass type factor takes into account the heat strengthened glass with a value of 1.8. NFL stands for the nonfactored load determined in charts contained in ASTM E 1300-04. This is determined using the glass dimensions. The NFL charts used in this study can be found in **Appendix F**.

LR = 2 x 1.8 x NFL

Through trial and error, it was determined that a (2) ½ in. laminated insulating, heat strengthened glass unit would sufficient to resist a blast load of 200 pounds. Its load resistance was determined to be 14.4 kPa which withstand approximately 300 pound TNT equivalent.

To further this breadth study, the thermal resistance of the design glass unit was calculated using "Building Science for Building Enclosures", Straube & Burnett, 3003. This was referenced for material conductivity values. Due to lack of time actual heat loads were not calculated or evaluated for the new glass façade design. Below is **figure 23** including the thermal resistance calculations for the designed glass unit.

Material	Thickness (m)	Conductivity (W/m-K)	Conductance (W/m^2-K)	Resistance
air film	NA	NA	23	0.043
glass	0.0127	0.8	63	0.016
airspace	0.02	NA	1.75	0.571
glass	0.0127	0.8	63	0.016
air film	NA	NA	8.3	0.120
			Total	0.767
		U=	1.304	W/m^2-K

Figure 23: *Thermal resistance calculations*

CONCLUSION

The depth study explores the option of building the same building design in San Francisco rather than Philadelphia. Due to the location change, the seismic lateral load would be much larger and would control over the wind load. To reduce the weight of the building, light weight concrete was used in place of normal weight concrete used in the composite steel deck of the building. Vulcraft catalogs were used to design the new floor slab, and a 3D model in ETABS was used to design the preliminary lateral system. Alternate lateral systems were researched, but a dual system was chosen for design. Concentric braced frames were not used by themselves, because the lateral load was too large. Adding lateral resisting frames in different locations would have taken away from the architecture and the open floor plan. The dual system includes shear walls and concentric brace frames that are used to resist the large, controlling seismic load. The shear walls are 16" in thickness and are located in place of the concentric braced frames in the core of the building. Hand calculations were performed to confirm and complete the design of the new lateral system.

Through cost analysis, it was found that the new lateral system costs over $100,000 dollars less than the original design. But the construction schedule was found to be prolonged, because construction of concrete walls is a longer process than steel erection. The building enclosures study included a design of blast glazing. The curtain wall designed for blast resistance can be located on the first floor along the main road. Through research and calculation it was found that a (2) ½ in. laminated insulating, heat strengthened glass unit would sufficient to resist a blast load of 200 pounds. This size blast load is compared to a small car bomb. Thermal resistance was calculated for the glass unit, but due to lack of time the actual thermal and lighting loads were not calculated or evaluated. Also, a design of the connections for the glass unit to be supported by the steel structure was not covered in this report. The expected goals of this report were met.

APPENDIX A- EXISTING TYPICAL BAY, TYPICAL SCHEDULES AND DESIGN VALUES

DESIGN LOADS AND FACTORS

DESIGN CODE: INTERNATIONAL BUILDING CODE 2003 ED

LIVE LOAD DATA

FLOOR OR ROOF AREA	LOAD (psf)
LABS / OFFICES	100
CORRIDORS, LOBBIES & EXITS	100
GARAGE	40
MECHANICAL EQUIP ROOMS	150
ROOF	30

LIVE LOAD REDUCTION APPLIED TO:
- ☒ COLUMNS
- ☒ GIRDERS
- ☐ BEAMS
- ☐ 2-WAY SLABS

SPECIAL LOADING.

SNOW LOAD DATA

ROOF AREA	LOAD (psf)
GROUND SNOW LOAD (P_g)	30
FLAT ROOF SNOW LOAD (P_f)	23
DRIFT	VARIES

FACTOR	VALUE
SNOW EXPOSURE (C_e)	1.0
SNOW LOAD IMPORTANCE (I_s)	1.0
THERMAL FACTOR (C_t)	1.0

SPECIAL SNOW CONSIDERATIONS.

☒ GOVERNS ROOF DESIGN

WIND LOAD DATA

FACTOR	VALUE
BASIC WIND SPEED (V_{3S}) (MPH)	90
WIND IMPORTANCE (I_w)	1.0
OCCUPANCY CATEGORY	II
WIND EXPOSURE	B
INTERNAL PRESSURE COEFFICIENT	±0.18
COMPONENTS AND CLADDING WIND PRESSURE (PSF)	*VARIES

* CALCULATED PRESSURES TO BE DETERMINED BY COMPONENT AND CLADDING PROVIDER.

SPECIAL WIND CONSIDERATIONS.

☐ GOVERNS LATERAL DESIGN

EARTHQUAKE DESIGN DATA

FACTOR	VALUE
SEISMIC IMPORTANCE FACTOR (I_E)	1.0
SEISMIC USE GROUP	I
SPECTRAL RESPONSE ACCELERATION 0.2 SEC (S_S)	0.33
SPECTRAL RESPONSE ACCELERATION 1.0 SEC (S_1)	0.082
SITE CLASS	C
DESIGN SPECTRAL RESPONSE COEFFICIENT (S_{DS})	0.27
DESIGN SPECTRAL RESPONSE COEFFICIENT (S_{D1})	0.09
SEISMIC DESIGN CATEGORY	B

ANALYSIS PROCEDURE EQUIVALENT LATERAL FORCE

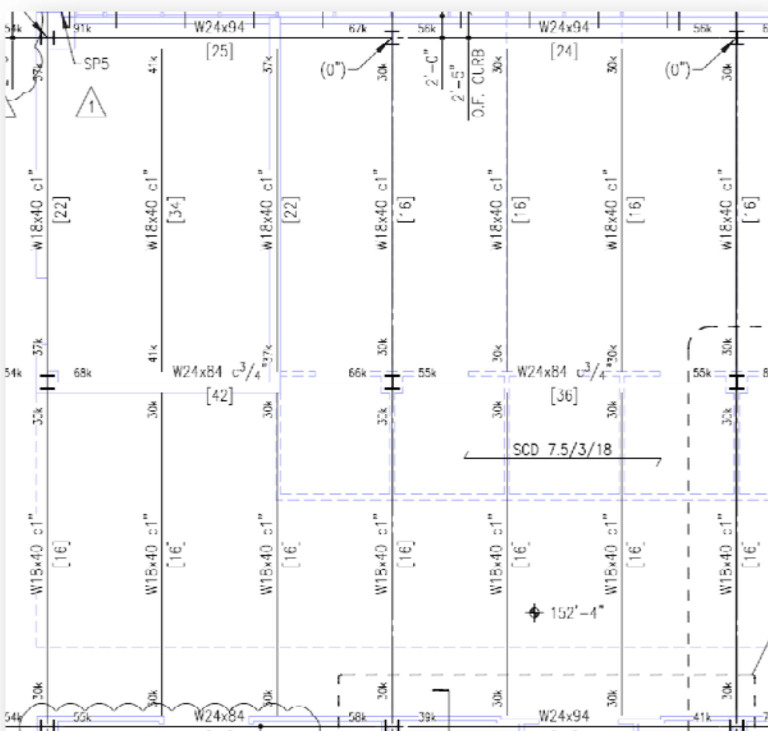
BASIC SEISMIC-FORCE-RESISTING SYSTEM: ORDINARY STEEL CONCENTRIC BRACED FRAMES

DESIGN BASE SHEAR (kips): Cs=0.021 R=3

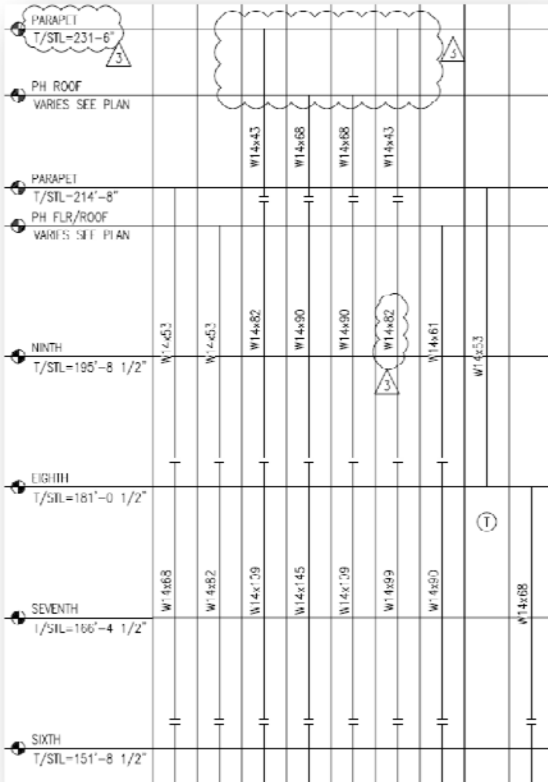
SPECIAL SEISMIC CONSIDERATIONS.

☒ GOVERNS LATERAL DESIGN

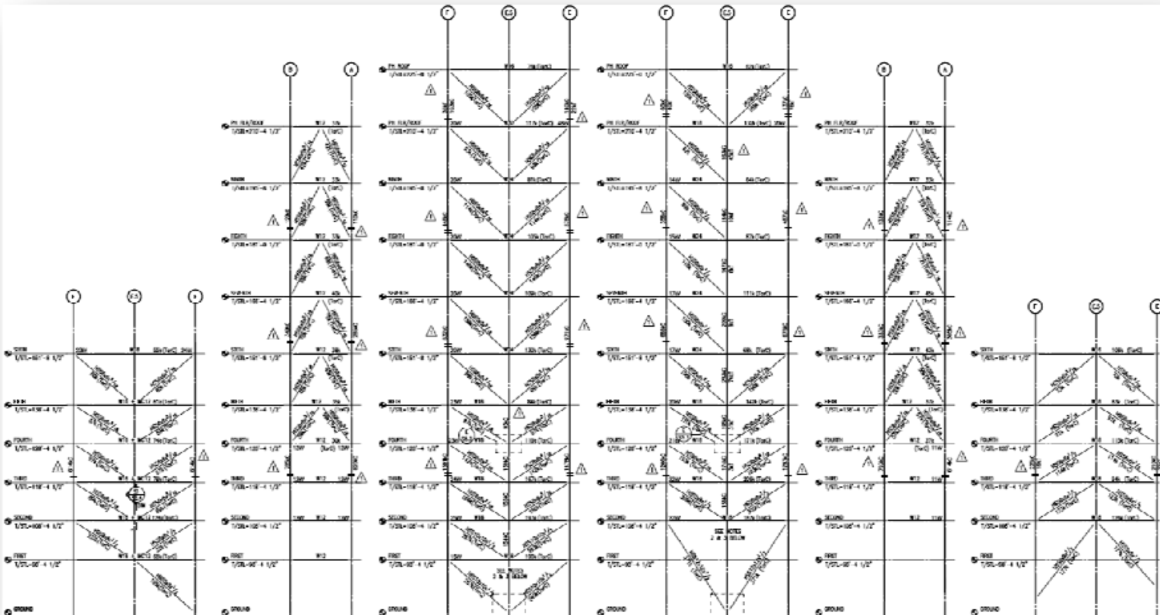
Existing Typical Bay (6th Floor)



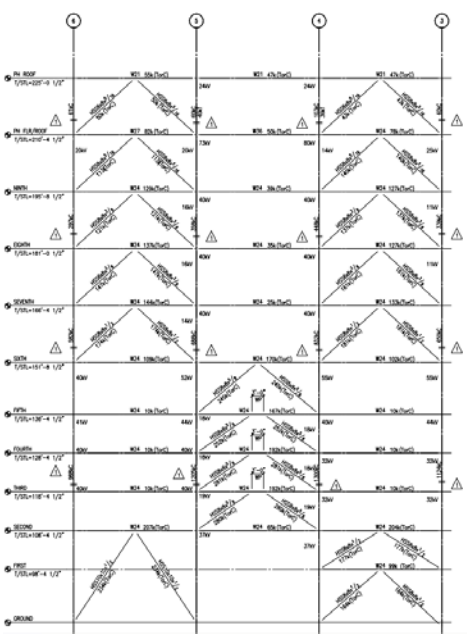
Typical Column Schedule



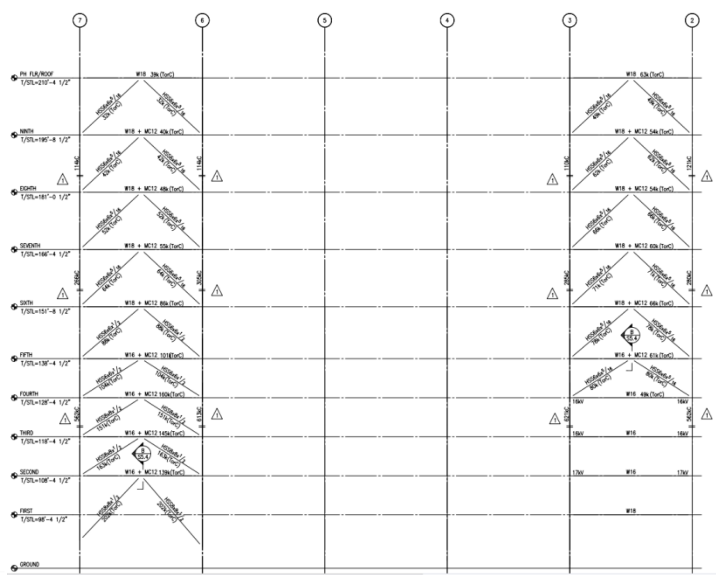
Braced Frame Schedule



ELEVATION AT LINE 1 ELEVATION AT LINE 3 ELEVATION AT LINE 4 ELEVATION AT LINE 5 ELEVATION AT LINE 6 ELEVATION AT LINE 10



ELEVATION AT LINE A



ELEVATION AT LINE F

APPENDIX B-WIND LOADS

MULKEY ENGINEERS & CONSULTANTS — CALCULATION SHEET — PAGE 1 OF ____

CLIENT ________ SUBJECT Senior Thesis Prepared By ____ Date ____
PROJECT No. Wind Analysis Reviewed By ____ Date ____

Velocity Pressures, q_z and q_h (ASCE 7-05)

- Basic Wind Speed, V = 85 mph / 38 m/s (Fig. 6-1)
- Wind Directionality factor, K_d = 0.85 for buildings (Fig. 6-4)
- Importance factor, I = 1.15 Category III (Table 6-1)
- Exposure Category B - located in urban area (6.5.6)
- Are all 5 conditions of 6.5.7.1 met? No
- Topographic factor, K_{zt} = 1.0
- Velocity Pressure Exposure Coefficients (Table 6-3 and 6-2)

  $K_z = 1.10$ ($z_g = 1200$, $\alpha = 7.0$) $K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha} = 2.01\left(\frac{147.5}{1200}\right)^{2/7} = 1.10$ (sample calculation)
  $K_h = 1.10$

- Velocity Pressure at Height z and h (refer to spread sheet)

  $q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (Equation 6-15)

  $q_z = 0.00256(1.10)(1.0)(0.85)(85)^2(1.15) = 19.89$ (sample Calculation)

Gust Effect factors, G and G_f

- Building natural frequency, n_1 (ASCE 7-05, C6.5.8, Eq. C6-17)
  $n_1 = 100/H = 100/147.5 = 0.68$ (average value)
- Damping ratio, β (ASCE 7-05, C6.5.8)
  β = 1.0 % per ISO
- Structure Dimensions
  h = 147.5′
  B = 262.5′ (N/S Elevation)
  L = 189′ (E/W Elevation)

  $n < 1$ Hz
  ∴ Structure is Flexible

ME-01

MULKEY ENGINEERS & CONSULTANTS

CALCULATION SHEET PAGE 2 OF ____

CLIENT ____ SUBJECT Senior Thesis Prepared By ____ Date ____

PROJECT No. Wind Analysis Reviewed By ____ Date ____

$g_Q = g_v = 3.4$

$g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}}$ (Eq. 6-9)

$= \sqrt{2\ln(3600(0.68))} + \frac{0.577}{\sqrt{2\ln(3600(0.68))}} = 4.097$

$\bar{z} = 0.6h = 0.6(147.5) = 88.5^{ft} > 30^{ft} = z_{min}$ (from ASCE 7-05, Fig. 6-2)

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{88.5}\right)^{1/6} = 0.255$ (c = 0.30 from ASCE 7-05, Fig. 6-2)

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{1/3} = 320\left(\frac{88.5}{33}\right)^{1/3} = 444.6$ (ℓ = 320 from ASCE 7-05, Fig 6-2)

$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$ (Eq. 6-6) $Q_{E/W} = \sqrt{\frac{1}{1+0.63\left(\frac{189+147.5}{444.6}\right)^{0.63}}} = 0.809$

$Q_{N/S} = \sqrt{\frac{1}{1+0.63\left(\frac{262.5+147.5}{444.6}\right)^{0.63}}} = 0.791$

• Basic Wind Speed, V

$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$ (Eq. 6-14) ($\bar{b} = 0.45$, $\bar{\alpha} = \frac{1}{4}$ from ASCE 7-05, Fig. 6-2)

$= (0.45)\left(\frac{88.5}{33}\right)^{1/4}(85)\left(\frac{88}{60}\right) = 71.79$

$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.68(444.6)}{71.79} = 4.21$ (Eq 6-12)

$R_n = \frac{7.47N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(4.21)}{(1+10.3(4.21))^{5/3}} = 0.057$

$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{6.45} - \frac{1}{2(6.45)^2}(1-e^{-2(6.45)}) = 0.143$

$\eta = 4.6n_1\frac{h}{\bar{V}_{\bar{z}}} = 4.6(0.68)\frac{147.5}{71.49} = 6.45$

$R_{B_1} = \frac{1}{11.49} - \frac{1}{2(11.49)^2}(1-e^{-2(11.49)}) = 0.083$ (N/S)

$\eta = 4.6n_1\frac{B}{\bar{V}_{\bar{z}}} = 4.6(0.68)\frac{262.5}{71.49} = 11.49$

$R_{B_2} = \frac{1}{8.27} - \frac{1}{2(8.27)^2}(1-e^{-2(8.27)}) = 0.114$ (E/W)

$\eta = 4.6n_1\frac{B}{\bar{V}_{\bar{z}}} = 4.6(0.68)\frac{189}{71.49} = 8.27$

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$R_{L_1} = \frac{1}{38.45} - \frac{1}{2(38.45)^2}\left(1 - e^{-2(38.45)}\right) = 0.026$ (N/S)

$\eta = 15.4 n_1 \frac{L}{\bar{V}_{\bar{z}}} = 15.4(0.68)\frac{262.5}{71.49} = 38.45$

$R_{L_2} = \frac{1}{27.69} - \frac{1}{2(27.69)^2}\left(1 - e^{-2(27.69)}\right) = 0.035$ (E/W)

$\eta = 15.4 n_1 \frac{L}{\bar{V}_{\bar{z}}} = 15.4(0.68)\frac{189}{71.49} = 27.69$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$

$R_1 = \sqrt{\frac{1}{1.0}(0.057)(0.143)(0.083)(0.53 + 0.47(0.026))} = 0.019$ (N/S)

$R_2 = \sqrt{\frac{1}{1.0}(0.057)(0.143)(0.114)(0.53 + 0.47(0.035))} = 0.023$ (E/W)

$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$ (Eq. 6-8)

$G_{f_1} = 0.925\left(\frac{1 + 1.7(0.255)\sqrt{(3.4)^2(0.791)^2 + (4.097)^2(0.019)^2}}{1 + 1.7(3.4)(0.255)}\right) = 0.876$ (N/S)

$G_{f_2} = 0.925\left(\frac{1 + 1.7(0.255)\sqrt{(3.4)^2(0.809)^2 + (4.097)^2(0.023)^2}}{1 + 1.7(3.4)(0.255)}\right) = 0.886$ (E/W)

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Buildings, Main Wind-force Resisting Systems

- The building is enclosed
- The building has a parapet
- Velocity Pressure $q_p = 19.89$ mph
- Combined net pressure coefficient, GC_{pn}

  $GC_{pn} = +1.5$ windward

  $GC_{pn} = -1.0$ leeward

- Combined net design pressure on the parapet

  $p_p = q_p GC_{pn}$ (Eq. 6-20)

  $= (19.89)(1.5) = 29.84$ (windward)

  $= (19.89)(-1) = -19.89$ (leeward)

- The building is not rigid
- Determine velocity pressure q_z for windward walls along the height of the building and q_h for leeward walls, side walls and roof
- Pressure coefficient, C_p for the walls and roof (Fig. 6-6 or 6-8)

  $L/B = \frac{262.5}{189} = 1.39$ (N/S) => $C_p = -0.5$ ($L/B = 0-1$)

  $= \frac{189}{262.5} = 0.72$ (E/W) => $C_p = -0.42$ (interpolated)

  { $L/B = 0-1$, $L/B = 2$; $C_p = -0.5$, $C_p = -0.3$ }

C_p	N/S	E/W
Windward wall	0.8	0.8
Leeward wall	-0.5	-0.42
Side wall	-0.7	-0.7

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- Determine design wind pressure, P_z

  $P_z = q_z G_z C_p$ (Eq. 6-19)

  Windward sample calculation (N/S)

  $P_z = 19.89(0.876)(0.8) = 13.9$ psf

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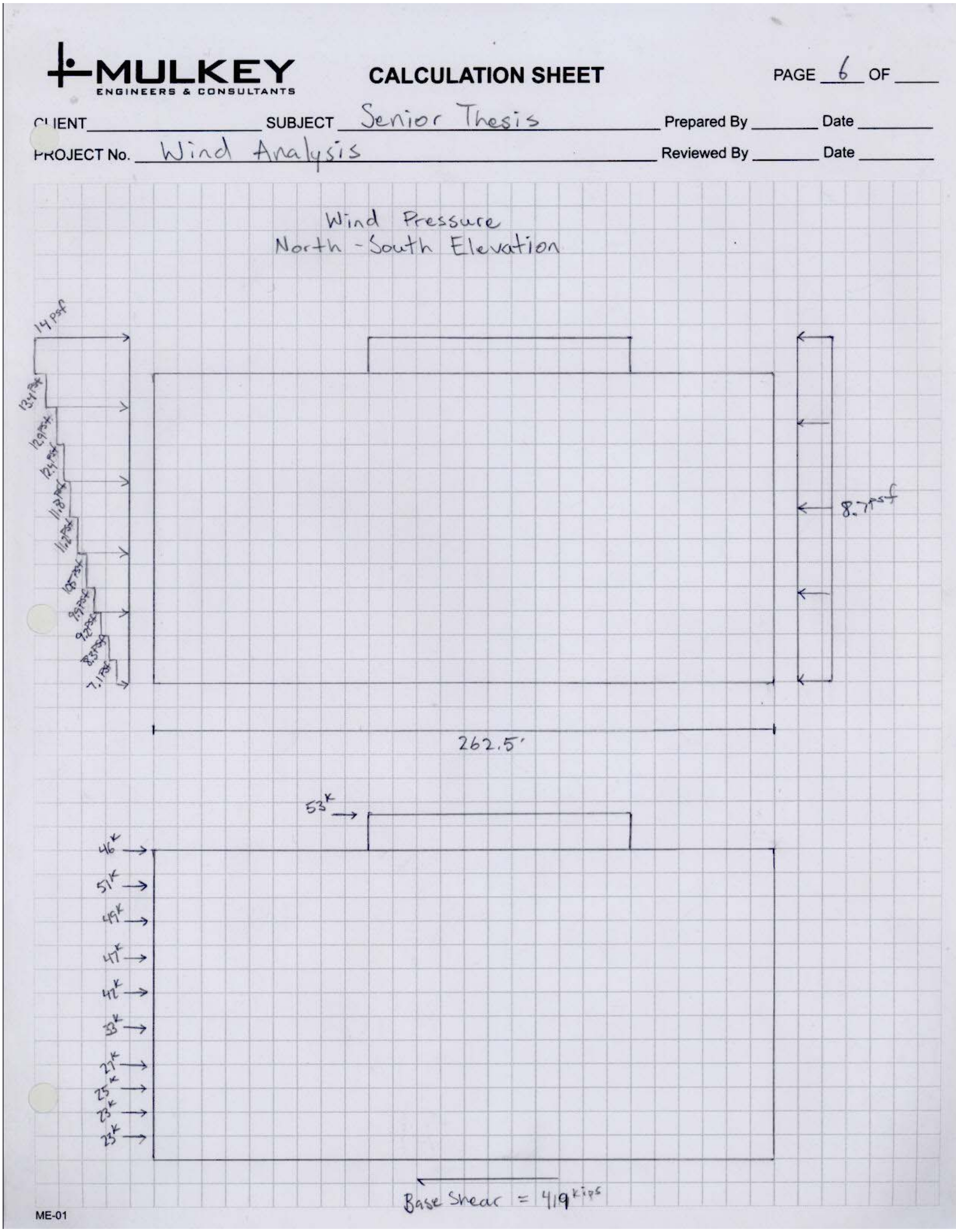
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Wind Pressure
North - South Elevation
14 psf
13.4 psf
12.9 psf
12.4 psf
11.8 psf
11.2 psf
10.5 psf
9.9 psf
9.2 psf
8.3 psf
7.1 psf
8.7 psf
262.5'
53k
46k
51k
49k
47k
42k
33k
27k
25k
23k
23k
Base Shear = 419 kips
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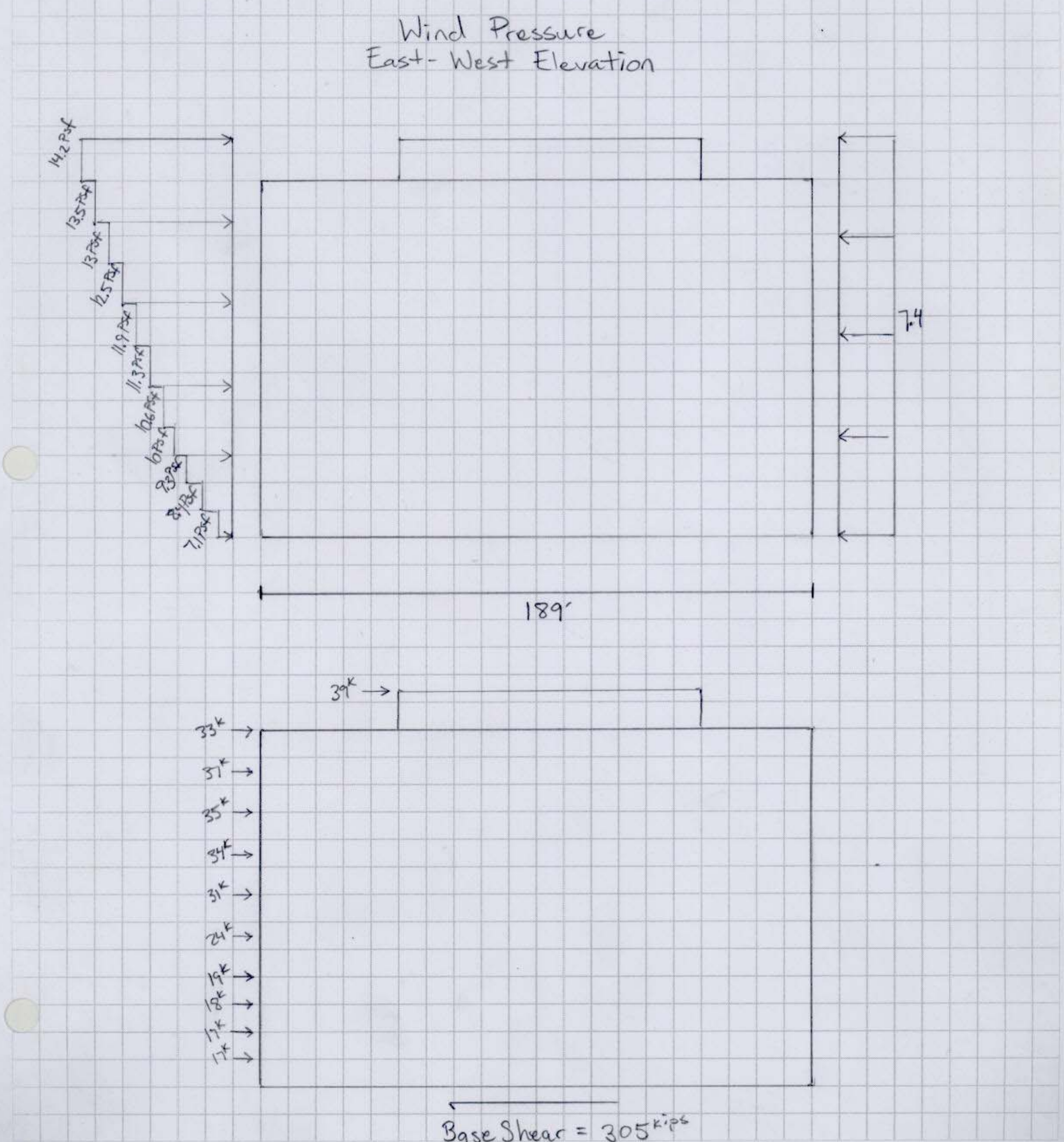

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APPENDIX C-SEISMIC LOADS

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Considerations of Seismic Design Requirements

- Not a detached one-or two-family dwelling
- Not an agricultural storage structure intended for incidental human occupancy
- Does the structure require special consideration with respect to response characteristics and environment that are not addressed in chapter 15 and for which other regulations provide criteria? No

∴ Seismic requirements of ASCE 7-05 must be considered

Seismic Ground Motion Values (These values were taken from new soils report)

Site Classification C
Occupancy Category II => I = 1.0

$S_s = 1.5 < 0.15$
∴ No
$S_1 = 0.62 \leq 0.04$

- Is the structure seismically isolated or does it have damping systems on site with $S_1 \geq 0.6$? No

∴ Determine ground motion from a ground motion hazard analysis in accordance with 21.2 (11.4.7).

$S_{MS} = F_a S_s = (1.0)(1.5) = 1.5$, $F_a = 1.0$ (value taken from soils report)

$S_{M1} = F_v S_1 = (1.3)(0.62) = 0.806$, $F_a = 1.3$ (value taken from soils report)

Determine S_{DS} and S_{D1} by Eqs. 11.4-3 and 11.4-4

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(1.5) = 1.0$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.806) = 0.537$

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Permitted Analytical Procedures

- Equivalent Lateral force procedure
  - response modification coefficient, $R = 6$
  - importance factor, $I = 1.0$

approximate fundamental period of the structure, T_a

$T_s = S_{D1}/S_{DS} = 0.537\,s$

$T_a = C_t h_n^x = 0.016(147.5)^{0.9} = 1.43\,s$

$C_t = 0.016$

$x = 0.9$

$T_L = 12$ (from Fig. 22-15) $> T_a$

Determine C_s by Eqs. 12.8-3 and 12.8-2

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \leq \frac{S_{DS}}{\left(\frac{R}{I}\right)} = 0.0626 < 0.167 \checkmark$$

$$\frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.537}{1.43\left(\frac{6}{1}\right)} = 0.0626$$

$$\frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{1}{6} = 0.167$$

Determine effective seismic weight, W in accordance with 12.7.2

$W = 33{,}552$ kips

$V = C_s W = 0.0626(33{,}552) = 2100$

$T < 0.5$ sec $\therefore k = 1$ (12.8.3)

$k = 1.019$ (interpolated)

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Determine lateral seismic force, F_x at level x by Eqs. 12.8-11 and 12.8-12

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V$$ (calculations in Excel sheet)

$$\sum_{i=1}^{n} w_i h_i^k = (33,552)(147.5)^{1.019} = 5,441,486$$

Determine seismic design story shear, V_x by Eq. 12.8-13

$$V_x = \sum_{i=x}^{n} F$$

Determine inherent torsional moment, M_t

Determine accidental torsional moment, M_{ta}

Determine the deflection δ_x at levels x by Eq. 12.8-15

$$\delta_x = \frac{C_d \delta_{xe}}{I}$$

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APPENDIX D-LATERAL DESIGN AND CHECKS

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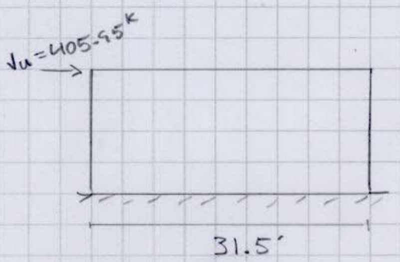

$f'_c = 4000^{psi}$

$f_y = 60{,}000^{psi}$

$\phi = 0.60$ for seismic

Check Max. Permitted Shear Strength

$V_u \le (\phi V_n)_{max} = \phi 10 \sqrt{f'_c}\, hd$

$d = 0.8 l_w = 0.8(31.5 \times 12) = 302''$

$\phi V_{n_{max}} = 0.75(10)\sqrt{3000}(16)(302)/1000 = 1836^K >$

Shear Strength by Concrete

$V_c = 2\sqrt{f'_c}\, hd = 2\sqrt{4000}(16)(302)/1000 = 611^K$

or

$V_c \le 3.3\sqrt{f'_c}\, hd + \frac{N_u d}{4 l_w}$

$\le 3.3\sqrt{4000}(16)(302)/1000 = 1008^K \Rightarrow$ controls

$V_c \le \left[0.6\sqrt{f'_c} + \frac{l_w\left(1.25\sqrt{f'_c} + 0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right] hd$

$\le \left[0.6\sqrt{4000} + \frac{378(1.25\sqrt{4000} + 0)}{96 - \frac{378}{2}}\right](16)(302)/1000 = \sout{1369}$ does not apply

Required Horizontal Shear Reinforcement

$V_u > \frac{1}{2}\phi V_c$

$\frac{1}{2}\phi V_c = \frac{1}{2}(0.60)(1008) = 302^K < 405.95^K$ ∴ Vs based on Ch. 11.9.9

389.32^K

343.86^K

443.28^K

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$V_s = \frac{A_v f_y d}{s}$

assume $s = 18''$

$q_t = \frac{A_v}{sh} \Rightarrow A_v = q_t s h = 0.0025(18)(16) = 0.72 \text{ in}^2$ minimum

$\therefore$ use $2\ \#6 @ 18''$ $A = 0.88 \text{ in}^2$

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PROJECT No. Shear Wall Vertical Reinforcement Design Reviewed By ___ Date ___

Shear Strength

ACI 318.08 § 21.9.4

"Structural walls shall not exceed V_n"

$\phi V_n = \phi A_c(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$

$\phi = 0.75$ or $\phi = 0.60$ for seismic

A_c = gross area of concrete

α_c = coefficient = 2.0 if $h_w/l_w \geq 2.0$

$\rho_t = \frac{A_v}{s \cdot h}$

s = spacing of shear reinforcement
h = thickness of wall

Sample Calculation

Ex. SW : F: 3-4

Story 1 Shear Force

$V_u = 443^k$

Vert. Reinf: (2) #6 @ 18"

$\rho_t = \frac{(2)(0.44)}{(18)(16)} = 0.00306 > 0.0025$

$A_{cv} = (378'')(16'') = 6048 \text{ in}^2$

$\phi V_n = 0.60(6048)[2\sqrt{4000} + 0.00306(60{,}000)] = 1124^k$

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F:5-6	Seismic								
Story	Seismic Shear Force (from ETABS), V_u (k)	Vertical Reinforcement	Spacing (in)	Length (in)	Thickness (in)	A_{cv} (in^2)	α_c	ρ_t	ΦV_n (k)
STORY11	17.5	(2) #6	18	378	16	6048	2	0.003056	1124
STORY10	4.94	(2) #6	18	378	16	6048	2	0.003056	1124
STORY9	25.8	(2) #6	18	378	16	6048	2	0.003056	1124
STORY8	113.44	(2) #6	18	378	16	6048	2	0.003056	1124
STORY7	124.58	(2) #6	18	378	16	6048	2	0.003056	1124
STORY6	171.88	(2) #6	18	378	16	6048	2	0.003056	1124
STORY5	207.92	(2) #6	18	378	16	6048	2	0.003056	1124
STORY4	309.81	(2) #6	18	378	16	6048	2	0.003056	1124
STORY3	348.7	(2) #6	18	378	16	6048	2	0.003056	1124
STORY2	388.84	(2) #6	18	378	16	6048	2	0.003056	1124
STORY1	389.32	(2) #6	18	378	16	6048	2	0.003056	1124

F:3-4	Seismic								
Story	Seismic Shear Force (from ETABS), V_u (k)	Vertical Reinforcement	Spacing (in)	Length (in)	Thickness (in)	A_{cv} (in^2)	α_c	ρ_t	ΦV_n (k)
STORY11	17.62	(2) #6	18	378	16	6048	2	0.003056	1124
STORY10	4.84	(2) #6	18	378	16	6048	2	0.003056	1124
STORY9	26.15	(2) #6	18	378	16	6048	2	0.003056	1124
STORY8	115.13	(2) #6	18	378	16	6048	2	0.003056	1124
STORY7	129.13	(2) #6	18	378	16	6048	2	0.003056	1124
STORY6	181.15	(2) #6	18	378	16	6048	2	0.003056	1124
STORY5	222.6	(2) #6	18	378	16	6048	2	0.003056	1124
STORY4	329.82	(2) #6	18	378	16	6048	2	0.003056	1124
STORY3	370.49	(2) #6	18	378	16	6048	2	0.003056	1124
STORY2	408.39	(2) #6	18	378	16	6048	2	0.003056	1124
STORY1	405.95	(2) #6	18	378	16	6048	2	0.003056	1124

4:E-F	Seismic								
Story	Seismic Shear Force (from ETABS), V_u (k)	Vertical Reinforcement	Spacing (in)	Length (in)	Thickness (in)	A_{cv} (in^2)	α_c	ρ_t	ΦV_n (k)
STORY11	22.82	(2) #6	18	378	16	6048	2	0.003056	1124
STORY10	12.9	(2) #6	18	378	16	6048	2	0.003056	1124
STORY9	45.64	(2) #6	18	378	16	6048	2	0.003056	1124
STORY8	155.95	(2) #6	18	378	16	6048	2	0.003056	1124
STORY7	190.5	(2) #6	18	378	16	6048	2	0.003056	1124
STORY6	204.96	(2) #6	18	378	16	6048	2	0.003056	1124
STORY5	240.92	(2) #6	18	378	16	6048	2	0.003056	1124
STORY4	284.96	(2) #6	18	378	16	6048	2	0.003056	1124
STORY3	308.44	(2) #6	18	378	16	6048	2	0.003056	1124
STORY2	316.55	(2) #6	18	378	16	6048	2	0.003056	1124
STORY1	343.86	(2) #6	18	378	16	6048	2	0.003056	1124

5:E-F	Seismic								
Story	Seismic Shear Force (from ETABS), V_u (k)	Vertical Reinforcement	Spacing (in)	Length (in)	Thickness (in)	A_{cv} (in^2)	α_c	ρ_t	ΦV_n (k)
STORY11	12.29	(2) #6	18	378	16	6048	2	0.003056	1124
STORY10	35.23	(2) #6	18	378	16	6048	2	0.003056	1124
STORY9	52.9	(2) #6	18	378	16	6048	2	0.003056	1124
STORY8	75.95	(2) #6	18	378	16	6048	2	0.003056	1124
STORY7	156.68	(2) #6	18	378	16	6048	2	0.003056	1124
STORY6	214.58	(2) #6	18	378	16	6048	2	0.003056	1124
STORY5	260.59	(2) #6	18	378	16	6048	2	0.003056	1124
STORY4	317.93	(2) #6	18	378	16	6048	2	0.003056	1124
STORY3	369.63	(2) #6	18	378	16	6048	2	0.003056	1124
STORY2	420.85	(2) #6	18	378	16	6048	2	0.003056	1124
STORY1	443.28	(2) #6	18	378	16	6048	2	0.003056	1124

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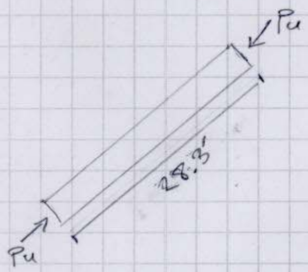
Brace Member 10: F-E.5 Floor 1



HSS 8×8½

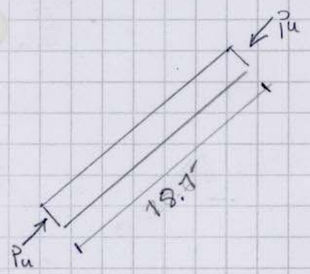
$P_u = 102.6^k$

$F_y = 68.7^k$

from Table 4-4 $\phi P_n = 231^k > 102.6^k$ ∴ Okay

$\frac{P_u}{\phi P_n} = \frac{102.6}{231} = 0.44 < 1.0$ ∴ Okay

Brace Member A: 6-7



HSS 6×6×½

$P_u = 84.3^k$

$F_y = 38.7^k$

from Table 4-4 $\phi P_n = 199^k > 84.3^k$ ∴ Okay

$\frac{P_u}{\phi P_n} = \frac{84.3}{199} = 0.42 < 1.0$ ∴ Okay

∴ Assume all lateral members are structurally sufficient

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APPENDIX E-COST AND SCHEDULE CLACULATIONS

Structural Steel Estimate										
Member Size	***Unit***	***Quantity***	***Length (LF)***	***Unit Mat'l Cost***	***Mat'l Cost***	***Unit Labor Cost***	***Labor Cost***	***Unit Equipment Cost***	***Equipment Cost***	***Total Item Cost***
Beams										
Wide Flange Shapes										
W12X26	LF	19	15.75000	$43.00	$12,868	$2.90	$868	$1.83	$548	$14,283
W16X26	LF	1	31.50000	$31.50	$992	$2.55	$80	$1.61	$51	$1,123
W16X40	LF	6	31.50000	$48.50	$9,167	$3.19	$603	$2.01	$380	$10,149
W16X45	LF	8	31.50000	$48.50	$12,222	$3.19	$804	$2.01	$507	$13,532
W16X67	LF	2	31.50000	$81.00	$5,103	$3.36	$212	$2.11	$133	$5,448
W18X40	LF	12	31.50000	$48.50	$18,333	$3.85	$1,455	$1.83	$692	$20,480
W18X50	LF	5	31.50000	$60.50	$9,529	$4.05	$638	$1.92	$302	$10,469
W18X60	LF	9	31.50000	$60.50	$17,152	$4.05	$1,148	$1.92	$544	$18,844
W21X44	LF	2	31.50000	$53.00	$3,339	$3.47	$219	$1.65	$104	$3,662
W21X57	LF	1	31.50000	$60.50	$1,906	$3.47	$109	$1.65	$52	$2,067
W24X55	LF	3	31.50000	$66.50	$6,284	$3.33	$315	$1.58	$149	$6,748
W24X62	LF	2	31.50000	$75.00	$4,725	$3.33	$210	$1.58	$100	$5,034
W24X76	LF	8	31.50000	$92.00	$23,184	$3.33	$839	$1.58	$398	$24,421
W24X84	LF	11	31.50000	$102.00	$35,343	$3.42	$1,185	$1.62	$561	$37,089
W24X94	LF	11	31.50000	$114.00	$39,501	$3.42	$1,185	$1.62	$561	$41,247
W24X103	LF	1	31.50000	$114.00	$3,591	$3.42	$108	$1.62	$51	$3,750
W30X108	LF	1	31.50000	$131.00	$4,127	$3.08	$97	$1.46	$46	$4,270
W36X135	LF	1	31.50000	$163.00	$5,135	$3.16	$100	$1.50	$47	$5,281
Braces										
Steel Tube Shapes										
HSS6X6X5/16	EA	12	43.00000	$297.00	$3,564	$47.50	$570	$30.00	$360	$4,494
HSS6X6X5/16	EA	6	37.30000	$297.00	$1,782	$47.50	$285	$30.00	$180	$2,247
HSS6X6X5/16	EA	4	41.30000	$297.00	$1,188	$47.50	$190	$30.00	$120	$1,498
HSS6X6X1/2	EA	3	37.30000	$297.00	$891	$47.50	$143	$30.00	$90	$1,124
HSS6X6X1/2	EA	1	41.30000	$297.00	$297	$47.50	$48	$30.00	$30	$375
HSS8X8X5/16	EA	10	43.00000	$297.00	$2,970	$47.50	$475	$30.00	$300	$3,745
HSS8X8X5/16	EA	5	37.30000	$297.00	$1,485	$47.50	$238	$30.00	$150	$1,873
HSS8X8X5/16	EA	2	41.30000	$297.00	$594	$47.50	$95	$30.00	$60	$749
HSS8X8X3/8	EA	4	43.00000	$645.00	$2,580	$51.00	$204	$32.00	$128	$2,912
HSS8X8X1/2	EA	4	43.00000	$645.00	$2,580	$51.00	$204	$32.00	$128	$2,912
HSS8X8X1/2	EA	5	37.30000	$645.00	$3,225	$51.00	$255	$32.00	$160	$3,640
HSS8X8X1/2	EA	1	41.50000	$645.00	$645	$51.00	$51	$32.00	$32	$728
HSS8X8X1/2	EA	1	20.75000	$645.00	$645	$51.00	$51	$32.00	$32	$728
HSS8X8X1/2	EA	2	28.30000	$645.00	$1,290	$51.00	$102	$32.00	$64	$1,456
HSS8X8X5/8	EA	3	37.30000	$645.00	$1,935	$51.00	$153	$32.00	$96	$2,184
HSS8X8X5/8	EA	1	41.30000	$645.00	$645	$51.00	$51	$32.00	$32	$728
HSS10X10X1/2	EA	1	37.30000	$1,200.00	$1,200	$53.00	$53	$33.50	$34	$1,287
HSS10X10X1/2	EA	1	41.50000	$1,200.00	$1,200	$53.00	$53	$33.50	$34	$1,287
Columns										
Wide Flange Shapes										
W14X43	LF	2	33.33000	$52.00	$1,733	$3.15	$105	$1.98	$66	$1,904
W14X48	LF	2	29.33000	$52.00	$1,525	$3.15	$92	$1.98	$58	$1,676
W14X53	LF	3	29.33000	$64.00	$1,877	$3.19	$94	$2.01	$59	$2,030
W14X61	LF	2	29.33000	$64.00	$1,877	$3.19	$94	$2.01	$59	$2,030
W14X68	LF	3	29.33000	$64.00	$1,877	$3.19	$94	$2.01	$59	$2,030
W14X68	LF	7	14.67000	$64.00	$939	$3.19	$47	$2.01	$29	$1,015
W14X74	LF	2	29.33000	$89.50	$2,625	$3.36	$99	$2.11	$62	$2,785
W14X82	LF	2	29.33000	$89.50	$2,625	$3.36	$99	$2.11	$62	$2,785
W14X82	LF	1	29.33000	$89.50	$2,625	$3.36	$99	$2.11	$62	$2,785
W14X90	LF	2	33.50000	$109.00	$3,652	$3.45	$116	$2.17	$73	$3,840
W14X90	LF	1	33.33000	$109.00	$3,633	$3.45	$115	$2.17	$72	$3,820
W14X90	LF	2	29.33000	$109.00	$3,197	$3.45	$101	$2.17	$64	$3,362
W14X90	LF	3	29.33000	$109.00	$3,197	$3.45	$101	$2.17	$64	$3,362
W14X99	LF	5	33.33000	$109.00	$3,633	$3.45	$115	$2.17	$72	$3,820
W14X109	LF	3	33.33000	$109.00	$3,633	$3.45	$115	$2.17	$72	$3,820
W14X109	LF	3	29.33000	$109.00	$3,197	$3.45	$101	$2.17	$64	$3,362
W14X120	LF	1	29.33000	$145.00	$4,253	$3.55	$104	$2.23	$65	$4,422
W14X132	LF	1	33.33000	$145.00	$4,833	$3.55	$118	$2.23	$74	$5,025
W14X145	LF	4	33.50000	$145.00	$4,858	$3.55	$119	$2.23	$75	$5,051
W14X145	LF	1	29.33000	$145.00	$4,253	$3.55	$104	$2.23	$65	$4,422
W14X159	LF	1	22.50000	$145.00	$3,263	$3.55	$80	$2.23	$50	$3,393
W14X159	LF	1	33.33000	$145.00	$4,833	$3.55	$118	$2.23	$74	$5,025
W14X176	LF	1	33.50000	$145.00	$4,858	$3.55	$119	$2.23	$75	$5,051
W14X176	LF	2	33.33000	$145.00	$4,833	$3.55	$118	$2.23	$74	$5,025
W14X193	LF	2	33.33000	$145.00	$4,833	$3.55	$118	$2.23	$74	$5,025
W14X211	LF	3	33.50000	$145.00	$4,858	$3.55	$119	$2.23	$75	$5,051
W14X233	LF	2	33.50000	$145.00	$4,858	$3.55	$119	$2.23	$75	$5,051
W14X257	LF	2	33.50000	$145.00	$4,858	$3.55	$119	$2.23	$75	$5,051
W14X283	LF	1	33.50000	$145.00	$4,858	$3.55	$119	$2.23	$75	$5,051
Subtotal Costs					$343,305		$16,453		$9,178	$368,935.74
Adjusted for Location (1.13)										$416,897.38
Design Contingency (1.5%)										$6,253.46
Escalation Contingency (3.5%)										$14,591.41
Insurance (3%)										$12,506.92
Bonds (2%)										$8,337.95
Overhead & Profit (10%)										$41,689.74
									Total Structural Steel Cost:	**$500,276.86**

Figure 1: EXISTING LATERAL SYSTEM ESTIMATE

Structural Steel Estimate										
Member Size	***Unit***	***Quantity***	***Length (LF)***	***Unit Mat'l Cost***	***Mat'l Cost***	***Unit Labor Cost***	***Labor Cost***	***Unit Equipment Cost***	***Equipment Cost***	***Total Item Cost***
Beams										
Wide Flange Shapes										
W12X26	LF	19	15.75000	$43.00	$12,868	$2.90	$868	$1.83	$548	$14,283
W16X40	LF	6	31.50000	$48.50	$9,167	$3.19	$603	$2.01	$380	$10,149
W16X45	LF	8	31.50000	$48.50	$12,222	$3.19	$804	$2.01	$507	$13,532
W18X40	LF	12	31.50000	$48.50	$18,333	$3.85	$1,455	$1.83	$692	$20,480
W18X50	LF	5	31.50000	$60.50	$9,529	$4.05	$638	$1.92	$302	$10,469
Braces										
Steel Tube Shapes										
HSS6X6X5/16	EA	10	43.00000	$297.00	$2,970	$47.50	$475	$30.00	$300	$3,745
HSS6X6X5/16	EA	5	37.30000	$297.00	$1,485	$47.50	$238	$30.00	$150	$1,873
HSS6X6X5/16	EA	3	41.30000	$297.00	$891	$47.50	$143	$30.00	$90	$1,124
HSS6X6X1/2	EA	1	37.30000	$297.00	$297	$47.50	$48	$30.00	$30	$375
HSS6X6X1/2	EA	1	41.30000	$297.00	$297	$47.50	$48	$30.00	$30	$375
HSS8X8X1/2	EA	1	37.30000	$645.00	$645	$51.00	$51	$32.00	$32	$728
HSS8X8X1/2	EA	1	20.70000	$645.00	$645	$51.00	$51	$32.00	$32	$728
HSS8X8X1/2	EA	1	28.30000	$645.00	$645	$51.00	$51	$32.00	$32	$728
Columns										
Wide Flange Shapes										
W14X43	LF	2	33.33000	$52.00	$1,733	$3.15	$105	$1.98	$66	$1,904
W14X48	LF	1	29.33000	$52.00	$1,525	$3.15	$92	$1.98	$58	$1,676
W14X53	LF	2	29.33000	$64.00	$1,877	$3.19	$94	$2.01	$59	$2,030
W14X61	LF	1	29.33000	$64.00	$1,877	$3.19	$94	$2.01	$59	$2,030
W14X68	LF	1	29.33000	$64.00	$1,877	$3.19	$94	$2.01	$59	$2,030
W14X74	LF	2	29.33000	$89.50	$2,625	$3.36	$99	$2.11	$62	$2,785
W14X82	LF	2	29.33000	$89.50	$2,625	$3.36	$99	$2.11	$62	$2,785
W14X90	LF	2	33.50000	$109.00	$3,652	$3.45	$116	$2.17	$73	$3,840
W14X90	LF	1	33.33000	$109.00	$3,633	$3.45	$115	$2.17	$72	$3,820
W14X90	LF	1	29.33000	$109.00	$3,197	$3.45	$101	$2.17	$64	$3,362
W14X99	LF	2	33.33000	$109.00	$3,633	$3.45	$115	$2.17	$72	$3,820
W14X109	LF	3	33.33000	$109.00	$3,633	$3.45	$115	$2.17	$72	$3,820
W14X145	LF	4	33.50000	$145.00	$4,858	$3.55	$119	$2.23	$75	$5,051
W14X159	LF	1	33.33000	$145.00	$4,833	$3.55	$118	$2.23	$74	$5,025
W14X176	LF	1	33.50000	$145.00	$4,858	$3.55	$119	$2.23	$75	$5,051
W14X211	LF	1	33.50000	$145.00	$4,858	$3.55	$119	$2.23	$75	$5,051
W14X233	LF	1	33.50000	$145.00	$4,858	$3.55	$119	$2.23	$75	$5,051
Subtotal Costs					$126,143		$7,302		$4,275	$137,720.03
Adjusted for Location (1.23)										$169,395.63
Design Contingency (1.5%)										$2,540.93
Escalation Contingency (3.5%)										$5,928.85
Insurance (3%)										$5,081.87
Bonds (2%)										$3,387.91
Overhead & Profit (10%)										$16,939.56

Total Steel Cost:	**$203,274.76**
Total Concrete Cost:	**$114,525.47**
Total Lateral System Cost:	**$317,800.23**

Structural Concrete Estimate											
Foundation Walls											
Item	***Size***	***Depth***	***Length***	***CY***	***Unit Mat'l***	***Mat'l Cost***	***Unit Labor Cost***	***Labor Cost***	***Unit Equip. Cost***	***Equip. Cost***	***Total Item Cost***
Normal Weight Concrete, 4000 PSI	147'-6"	3'-0"	31'-6"	516.3	$104.00	$53,690.00					$53,690.00
Item	***Size***	***Depth***	***Length***	***CY***	***Unit Mat'l***	***Mat'l Cost***	***Unit Labor Cost***	***Labor Cost***	***Unit Equip. Cost***	***Equip. Cost***	***Total Item Cost***
Placing Concrete Wall, Direct chute	147'-6"	3'-0"	31'-6"	516.3			$15.10	$7,795.38	$0.49	$252.96	$8,048.34
Item	***Size***	***Depth***	***Length***	***SFCA***	***Unit Mat'l***	***Mat'l Cost***	***Unit Labor Cost***	***Labor Cost***	***Unit Equip. Cost***	***Equip. Cost***	***Total Item Cost***
Continuous Wall Forms, plywood, 2 use	147'-6"	3'-0"	31'-6"	1002	$1.34	$1,342.01	$5.40	$5,408.10			$6,750.11
Item	***LBS/FT***	***Length***	***Quantity***	***LBS***	***Unit Mat'l***	***Mat'l Cost***	***Unit Labor Cost***	***Labor Cost***	***Unit Equip. Cost***	***Equip. Cost***	***Total Item Cost***
Wall #6 Rebar, A615 Grade 60	1.502	31'-6"	98	4637	$0.74	$3,431.14	$0.24	$1,112.80			$4,543.94
Wall #6 Rebar, A615 Grade 60	1.502	147'-6"	21	4652	$0.74	$3,442.81	$0.24	$1,116.59			$4,559.40
Subtotals						$61,905.96		$15,432.86		$252.96	$77,591.78
Adjusted for Location (1.23)											$95,437.89
Design Contingency (1.5%)											$1,431.57
Escalation Contingency (3.5%)											$3,340.33
Insurance (3%)											$2,863.14
Bonds (2%)											$1,908.76
Overhead & Profit (10%)											$9,543.79
									Total Structural Concrete Cost:		**$114,525.47**

Figure 2: NEW LATERAL SYSTEM DESIGN ESTIMATE

Existing Lateral System Construction Duration							
Member Size	***Unit***	***Quantity***	***Length (LF)***	***Crew***	***Daily Output***	***Labor Hours***	***Total Labor Hours***
Beams							
Wide Flange Shapes							
W12X26	LF	19	15.75	E2	880.00	0.064	1.22
W16X26	LF	1	31.50	E2	1000.00	0.056	0.06
W16X40	LF	6	31.50	E2	800.00	0.070	0.42
W16X45	LF	8	31.50	E2	800.00	0.070	0.56
W16X67	LF	2	31.50	E2	760.00	0.074	0.15
W18X40	LF	12	31.50	E5	960.00	0.083	1.00
W18X50	LF	5	31.50	E5	912.00	0.088	0.44
W18X60	LF	9	31.50	E5	912.00	0.088	0.79
W21X44	LF	2	31.50	E5	1064.00	0.075	0.15
W21X57	LF	1	31.50	E5	1064.00	0.075	0.08
W24X55	LF	3	31.50	E5	1110.00	0.072	0.22
W24X62	LF	2	31.50	E5	1110.00	0.072	0.14
W24X76	LF	8	31.50	E5	1110.00	0.072	0.58
W24X84	LF	11	31.50	E5	1080.00	0.074	0.81
W24X94	LF	11	31.50	E5	1080.00	0.074	0.81
W24X103	LF	1	31.50	E5	1080.00	0.074	0.07
W30X108	LF	1	31.50	E5	1200.00	0.067	0.07
W36X135	LF	1	31.50	E5	1170.00	0.068	0.07
Braces							
Steel Tube Shapes							
HSS6X6X5/16	EA	12	43.00	E2	54.00	1.037	12.44
HSS6X6X5/16	EA	6	37.30	E2	54.00	1.037	6.22
HSS6X6X5/16	EA	4	41.30	E2	54.00	1.037	4.15
HSS6X6X1/2	EA	3	37.30	E2	54.00	1.037	3.11
HSS6X6X1/2	EA	1	41.30	E2	54.00	1.037	1.04
HSS8X8X5/16	EA	10	43.00	E2	50.00	1.120	11.20
HSS8X8X5/16	EA	5	37.30	E2	50.00	1.120	5.60
HSS8X8X5/16	EA	2	41.30	E2	50.00	1.120	2.24
HSS8X8X3/8	EA	4	43.00	E2	50.00	1.120	4.48
HSS8X8X1/2	EA	4	43.00	E2	50.00	1.120	4.48
HSS8X8X1/2	EA	5	37.30	E2	50.00	1.120	5.60
HSS8X8X1/2	EA	1	41.50	E2	50.00	1.120	1.12
HSS8X8X1/2	EA	1	20.75	E2	50.00	1.120	1.12
HSS8X8X1/2	EA	2	28.30	E2	50.00	1.120	2.24
HSS8X8X5/8	EA	3	37.30	E2	50.00	1.120	3.36
HSS8X8X5/8	EA	1	41.30	E2	50.00	1.120	1.12
HSS10X10X1/2	EA	1	37.30	E2	48.00	1.167	1.17
HSS10X10X1/2	EA	1	41.50	E2	48.00	1.167	1.17
Columns							
Wide Flange Shapes							
W14X43	LF	2	33.33	E2	810.00	0.069	0.14
W14X48	LF	2	29.33	E2	810.00	0.069	0.14
W14X53	LF	3	29.33	E2	800.00	0.070	0.21
W14X61	LF	2	29.33	E2	800.00	0.070	0.14
W14X68	LF	3	29.33	E2	800.00	0.070	0.21
W14X68	LF	7	14.67	E2	800.00	0.070	0.49
W14X74	LF	2	29.33	E2	760.00	0.074	0.15
W14X82	LF	2	29.33	E2	760.00	0.074	0.15
W14X82	LF	1	29.33	E2	760.00	0.074	0.07
W14X90	LF	2	33.50	E2	740.00	0.076	0.15
W14X90	LF	1	33.33	E2	740.00	0.076	0.08
W14X90	LF	2	29.33	E2	740.00	0.076	0.15
W14X90	LF	3	29.33	E2	740.00	0.076	0.23
W14X99	LF	5	33.33	E2	740.00	0.076	0.38
W14X109	LF	3	33.33	E2	740.00	0.076	0.23
W14X109	LF	3	29.33	E2	740.00	0.076	0.23
W14X120	LF	1	29.33	E2	720.00	0.078	0.08
W14X132	LF	1	33.33	E2	720.00	0.078	0.08
W14X145	LF	4	33.50	E2	720.00	0.078	0.31
W14X145	LF	1	29.33	E2	720.00	0.078	0.08
W14X159	LF	1	22.50	E2	720.00	0.078	0.08
W14X159	LF	1	33.33	E2	720.00	0.078	0.08
W14X176	LF	1	33.50	E2	720.00	0.078	0.08
W14X176	LF	2	33.33	E2	720.00	0.078	0.16
W14X193	LF	2	33.33	E2	720.00	0.078	0.16
W14X211	LF	3	33.50	E2	720.00	0.078	0.23
W14X233	LF	2	33.50	E2	720.00	0.078	0.16
W14X257	LF	2	33.50	E2	720.00	0.078	0.16
W14X283	LF	1	33.50	E2	720.00	0.078	0.08
						Total Hours	84.34
						Weeks	2.1

New Lateral System Construction Duration

Member Size	Unit	Quantity	Length (LF)	Crew	Daily Output	Labor Hours	Total Labor Hours	
Beams								
Wide Flange Shapes								
W12X26	LF	19	15.75000	E2	880.00	0.064	1.22	
W16X40	LF	6	31.50000	E2	800.00	0.070	0.42	
W16X45	LF	8	31.50000	E2	800.00	0.070	0.56	
W18X40	LF	12	31.50000	E5	960.00	0.083	1.00	
W18X50	LF	5	31.50000	E5	912.00	0.088	0.44	
Braces								
Steel Tube Shapes								
HSS6X6X5/16	EA	10	43.00000	E2	54.00	1.037	10.37	
HSS6X6X5/16	EA	5	37.30000	E2	54.00	1.037	5.19	
HSS6X6X5/16	EA	3	41.30000	E2	54.00	1.037	3.11	
HSS6X6X1/2	EA	1	37.30000	E2	54.00	1.037	1.04	
HSS6X6X1/2	EA	1	41.30000	E2	54.00	1.037	1.04	
HSS8X8X1/2	EA	1	37.30000	E2	50.00	1.120	1.12	
HSS8X8X1/2	EA	1	20.70000	E2	50.00	1.120	1.12	
HSS8X8X1/2	EA	1	28.30000	E2	50.00	1.120	1.12	
Columns								
Wide Flange Shapes								
W14X43	LF	2	33.33000	E2	810.00	0.069	0.14	
W14X48	LF	1	29.33000	E2	810.00	0.069	0.07	
W14X53	LF	2	29.33000	E2	800.00	0.070	0.14	
W14X61	LF	1	29.33000	E2	800.00	0.070	0.07	
W14X68	LF	1	29.33000	E2	800.00	0.070	0.07	
W14X74	LF	2	29.33000	E2	760.00	0.074	0.15	
W14X82	LF	2	29.33000	E2	760.00	0.074	0.15	
W14X90	LF	2	33.50000	E2	740.00	0.076	0.15	
W14X90	LF	1	33.33000	E2	740.00	0.076	0.08	
W14X90	LF	1	29.33000	E2	740.00	0.076	0.08	
W14X99	LF	2	33.33000	E2	740.00	0.076	0.15	
W14X109	LF	3	33.33000	E2	740.00	0.076	0.23	
W14X145	LF	4	33.50000	E2	720.00	0.078	0.31	
W14X159	LF	1	33.33000	E2	720.00	0.078	0.08	
W14X176	LF	1	33.50000	E2	720.00	0.078	0.08	
W14X211	LF	1	33.50000	E2	720.00	0.078	0.08	
W14X233	LF	1	33.50000	E2	720.00	0.078	0.08	29.82

Foundation Walls								
Item	***Size***	***Depth***	***Length***	***CY***		***Finishes Labor Hours***	***Curing Hours***	
Normal Weight Concrete, 4000 PSI	147'-6"	3'-0"	31'-6"	516.25		0.01	0.291	
Item	***Size***	***Depth***	***Length***	***CY***	***Crew***	***Daily Output***	***Labor Hours***	**Total Labor Hours**
Placing Concrete Foundation Wall, Direct chute	147'-6"	3'-0"	31'-6"	516.25	C6	105	0.457	235.93
Item	***Size***	***Depth***	***Length***	***SFCA***				**Total Labor Hours**
Wall Forms, plywood, 2 use	147'-6"	3'-0"	31'-6"	1001.5	C2	345	0.139	139.21
Item	***LBS/FT***	***Length***	***Quantity***	***LBS***				**Total Labor Hours**
Wall #6 Rebar, A615 Gra	1.502	31'-6"	98	4636.674		3	10.667	49.4594
Wall #6 Rebar, A615 Gra	1.502	147'-6"	21	4652.445		3	10.667	49.62763
Finshes: assume							**Finishes Hours**	5.16
Power Screed, bull float, machine float & trowel (walk-behind)							**Curing Hours**	150.23
							Total Hours	659.44
							Weeks	16.5

APPENDIX F-BUILDING ENCLOSURES

F 2248 – 03

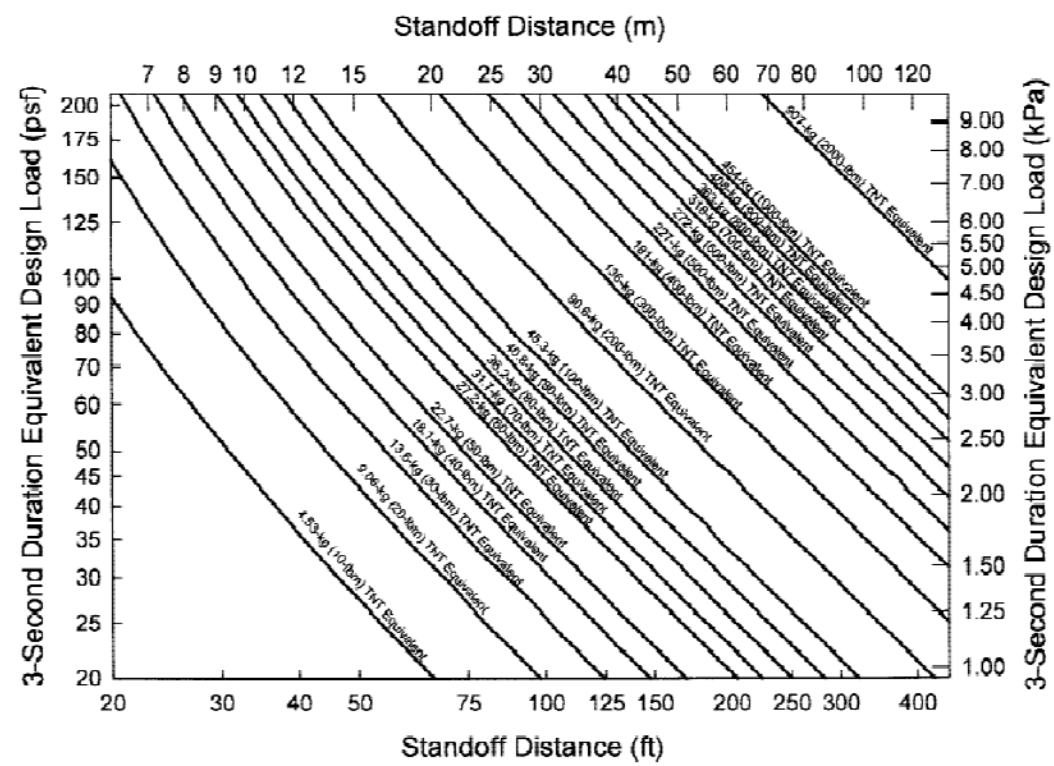

FIG. 1 Graphical Relationship Between Standoff Distance, TNT Charge Mass, and 3-Second Equivalent Design Load

E 1300 – 04ε1

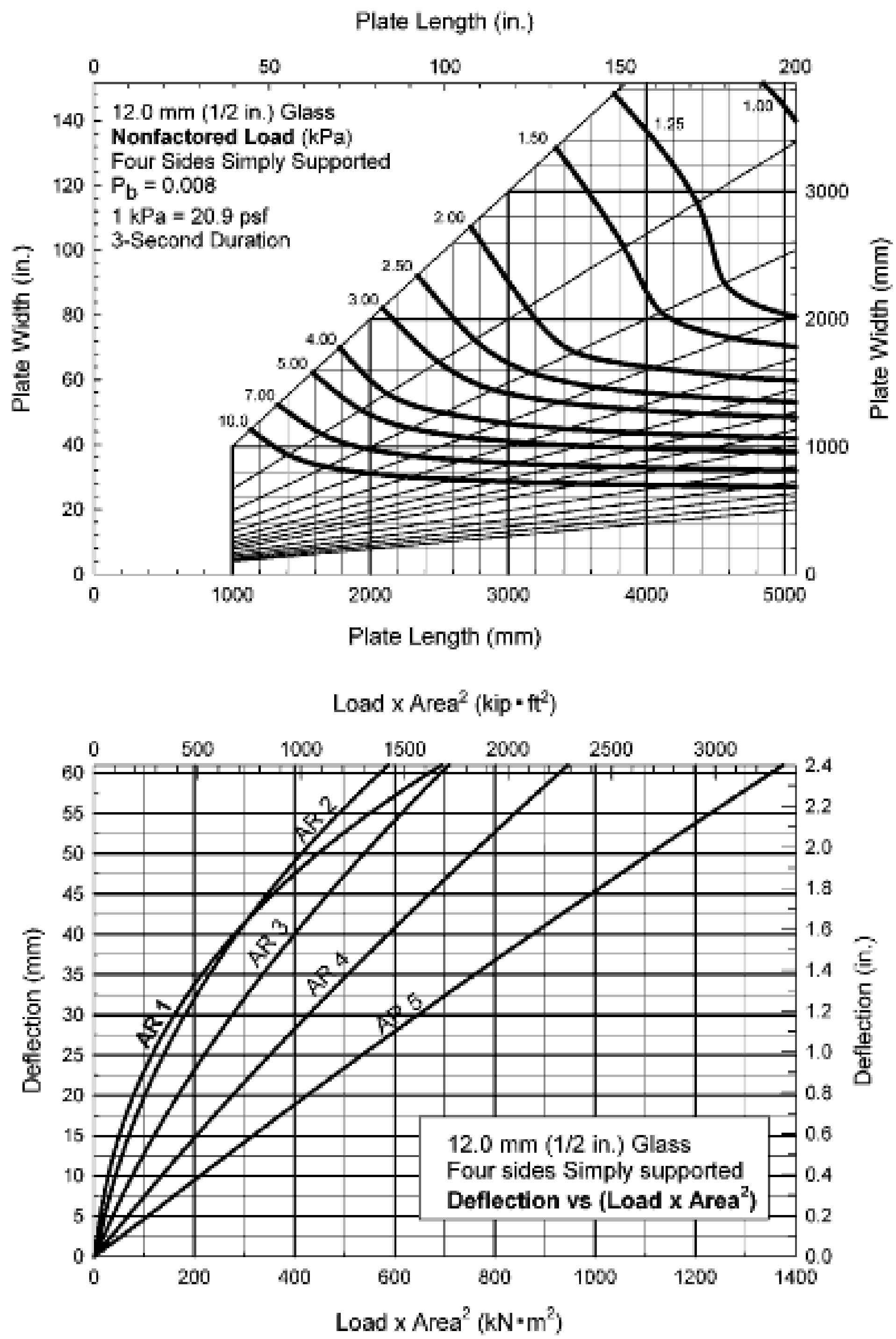

FIG. A1.9 (upper chart) Nonfactored Load Chart for 12.0 mm (½ in.) Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 12.0 mm (½ in.) Glass with Four Sides Simply Supported

APPENDIX G-COMPOSITE STEEL DECK DESIGN

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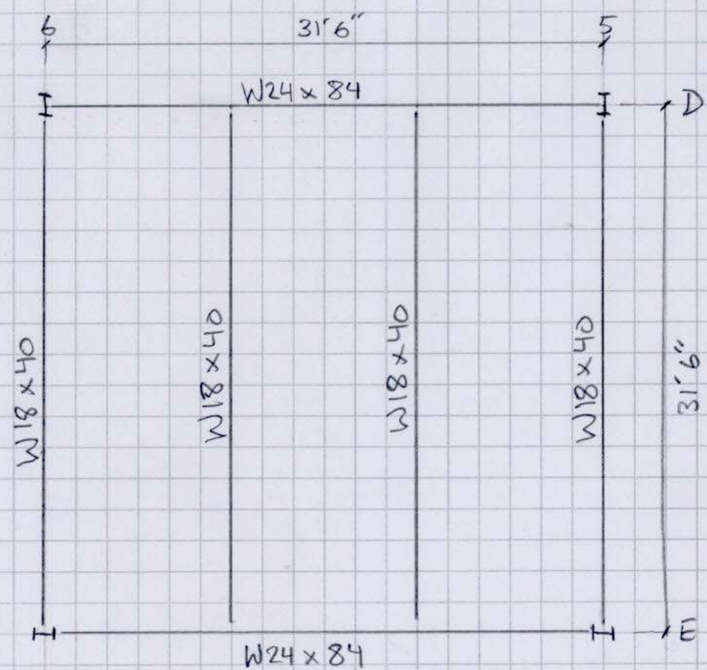
CLIENT ____ SUBJECT ____ Prepared By ____ Date ____

PROJECT No. Composite Steel Deck (Light Weight Concrete) Reviewed By ____ Date ____

Typical Interior Bay - Sixth Floor



Loads:

Dead Load$_{SLAB}$ = 43 psf (from Vulcraft Catalog)

Dead Load$_{MEP \& Finishes}$ = 8 psf

Live Load$_{offices/Labs}$ = 100 psf

Floor System:

3" x 18 Gauge Deck

3" Light Weight Concrete

ME-01

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CALCULATION SHEET

PAGE 2 OF ____

CLIENT ________ SUBJECT ________ Prepared By ________ Date ________

PROJECT No. Composite Steel Deck (Light Weight Concrete) Reviewed By ________ Date ________

Beam Spot Check

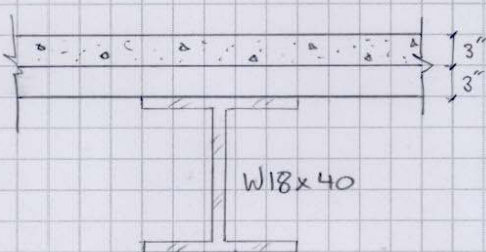
Factored Load: $1.2D + 1.6L = 1.2(80+8) + 1.6(100)$

$W_u = 266 \text{ psf}$

Trib. Width = 10.5'

$W_u = 266(10.5)/1000 = 2.79 \text{ klf}$

$M_u = \frac{W_u l^2}{8} = \frac{2.79(31.5)^2}{8} = 346 \text{ k-ft}$



$b_{eff} = \min \begin{cases} \text{spacing} = 10.5(12) = 126" \\ \frac{\text{span}}{4} = \frac{31.5(12)}{4} = 94.5" \end{cases}$ ∴ Controls

Check Deflections

$\Delta_{constr} = \frac{5 W_{conc} l^4}{384 EI}$

$W_{conc} = 43^{psf}(10.5) = 0.452 \text{ klf}$

$\Delta_{allowable} = \frac{l}{360} = \frac{31.5(12)}{360} = 1.05"$

$I_{req} = \frac{5 W_{conc} l^4}{384 \Delta_{allow} E} = \frac{5(0.452)(31.5)^4(1728)}{384(1.05)(29{,}000)} = 329 \text{ in}^4$

$I_{W18x40} = 612 \text{ in}^4 > 329 \text{ in}^4$ ∴ Okay

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CALCULATION SHEET

PAGE 3 OF ____

CLIENT ________ SUBJECT ________ Prepared By ________ Date ________

PROJECT No. Composite Steel Deck (Light Weight Concrete) Reviewed By ________ Date ________

Check Bending for construction loading

$W_{live} = 20^{psf}(10.5) = 0.21^{klf}$

$W_u = 1.2(0.452) + 1.6(0.21) = 0.878^{klf}$

$M_u = \frac{W_u l^2}{8} = \frac{0.878(31.5)^2}{8} = 109^{kft}$

$\phi M_{n\,W18x40} = 215^{kft} > 109^{kft}$

From Table 3-19 assume $\Sigma Q_n = 147^k$

$a = \frac{\Sigma Q_n}{.85 f'_c b_{eff}} = \frac{147}{0.85(4)(94.5)} = 0.458''$

$Y_2 = 7.5'' - \frac{a}{2} = 7.5 - \frac{0.458}{2} = 7.27''$ (round to 7")

W18x40 $Y_2 = 7''$ $\Sigma Q_n = 147^k$ @ PNA #7

$\phi M_n = 444^{kft} > 346^{kft}$ ∴ Okay

Check Number of Shear Studs (Table 3-21)

Shear stud diameter = 3/4"
Deck perpendicular
$f'_c = 4000^{psi}$
} $Q_n = 21.5$

studs req'd $= \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{147}{21.5}(2) = 13.7$

∴ 14 Studs required

# of studs provided = 16 over length of beam > 14 studs

∴ Okay

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CALCULATION SHEET PAGE 4 OF ____

CLIENT ____ SUBJECT ____ Prepared By ____ Date ____

PROJECT No. Composite Steel Deck (light Weight Concrete) Reviewed By ____ Date ____

Check Deflections

$Y_2 = 7'' \Rightarrow I_{LB} = 1210 \text{ in}^4$

$\Delta = \frac{5 w_u l^4}{384 E I_{LB}} = \frac{5(0.878)(31.5)^4(1728)}{384(1210)(29{,}000)} = 0.554''$

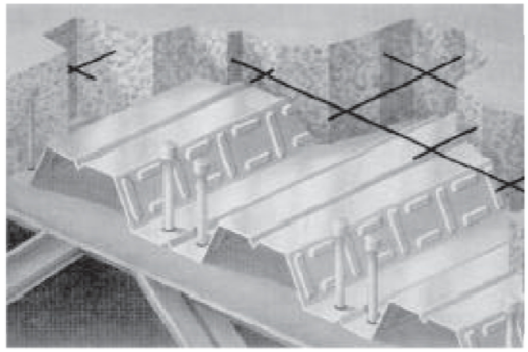
$\Delta_{allow} = \frac{l}{360} = 1.05'' > 0.554'' \therefore$ Okay

i.e. The structural beam can be reduced in size, because a W18x40 is more than sufficient for the applied load.

ME-01

SLAB INFORMATION

Total Slab Depth	Theo. Concrete Volume Yds./ 100 Sq. Ft.	Theo. Concrete Volume Cu. Ft./ Sq. Ft.	Recommended Welded Wire Fabric
5"	1.08	0.292	6x6-W1.4xW1.4
5 1/2"	1.23	0.333	6x6-W1.4xW1.4
6"	1.39	0.375	6x6-W1.4xW1.4
6 1/4"	1.47	0.396	6x6-W1.4xW1.4
6 1/2"	1.54	0.417	6x6-W2.1xW2.1
7"	1.70	0.458	6x6-W2.1xW2.1
7 1/4"	1.77	0.479	6x6-W2.1xW2.1
7 1/2"	1.85	0.500	6x6-W2.1xW2.1



(N=14) LIGHTWEIGHT CONCRETE (110 PCF)

Total Slab Depth	Deck Type	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 Span	2 Span	3 Span	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0	13'-6	14'-0	14'-6	15'-0
5"	3VLI22	9'-1	11'-6	11'-6	141	127	115	83	75	67	60	54	49	45	40				
	3VLI21	9'-10	12'-4	12'-9	153	138	125	113	82	74	67	60	54	49	45	41			
	3VLI20	10'-6	13'-0	13'-6	163	147	133	121	110	102	72	65	59	54	48	44	40		
(t=2")	3VLI19	11'-10	14'-4	14'-10	185	166	150	136	124	114	105	97	68	62	57	52	47	43	
	3VLI18	13'-0	15'-4	15'-10	244	222	204	188	174	162	151	142	133	126	119	90	85	79	75
34 PSF	3VLI17	14'-0	16'-3	16'-6	262	238	218	201	185	172	161	150	141	133	126	119	113	85	80
	3VLI16	14'-5	16'-11	16'-11	277	254	234	217	202	189	177	166	157	149	141	134	127	99	94
5 1/2"	3VLI22	8'-5	10'-6	10'-6	161	121	107	95	85	77	69	62	56	51	46	42			
	3VLI21	9'-5	11'-10	12'-2	175	157	142	105	94	84	76	69	62	56	51	47	42		
	3VLI20	10'-0	12'-6	12'-11	186	167	151	138	126	91	82	74	67	61	56	51	46	42	
(t=2 1/2")	3VLI19	11'-3	13'-9	14'-3	211	189	171	155	142	130	120	86	78	71	65	59	54	49	45
	3VLI18	12'-4	14'-8	15'-2	278	253	232	214	198	184	172	161	152	118	110	103	97	91	85
39 PSF	3VLI17	13'-4	15'-7	16'-0	299	272	248	229	211	196	183	171	161	152	143	110	103	97	91
	3VLI16	14'-0	16'-5	16'-5	316	289	267	247	230	215	202	190	179	170	161	153	146	114	107
6"	3VLI22	7'-9	9'-9	9'-9	154	136	120	107	96	86	78	70	63	57	52	47	43		
	3VLI21	9'-0	11'-4	11'-6	196	176	160	118	106	95	86	77	70	64	58	52	48	43	
	3VLI20	9'-7	12'-0	12'-5	209	188	170	155	114	103	93	84	76	69	63	57	52	47	43
(t=3")	3VLI19	10'-9	13'-3	13'-8	237	212	192	174	159	146	107	97	88	80	73	67	61	56	51
	3VLI18	11'-9	14'-2	14'-8	312	284	261	240	223	207	193	181	142	133	124	116	109	102	96
43 PSF	3VLI17	12'-9	15'-1	15'-7	335	305	279	257	237	221	206	192	181	170	132	124	116	109	102
	3VLI16	13'-5	15'-10	16'-0	354	325	299	277	258	241	226	213	201	190	181	143	135	128	121
6 1/4"	3VLI22	7'-6	9'-5	9'-5	162	143	127	113	101	91	82	74	67	60	55	50	45	41	
	3VLI21	8'-10	11'-1	11'-1	207	186	140	125	112	100	90	82	74	67	61	55	50	46	42
	3VLI20	9'-5	11'-10	12'-2	221	198	179	134	120	108	98	88	80	73	66	60	55	50	46
(t=3 1/4")	3VLI19	10'-6	13'-0	13'-6	250	224	202	184	168	154	113	102	93	84	77	70	64	59	54
	3VLI18	11'-6	13'-11	14'-5	329	300	275	253	235	218	204	191	150	140	131	122	115	108	101
46 PSF	3VLI17	12'-5	14'-10	15'-3	354	322	294	271	250	233	217	203	191	150	140	131	122	115	108
	3VLI16	13'-2	15'-6	15'-10	374	343	316	293	272	254	239	225	212	201	190	151	143	135	128
6 1/2"	3VLI22	7'-3	9'-1	9'-1	171	150	134	119	107	96	86	78	70	64	58	52	47	43	
	3VLI21	8'-7	10'-9	10'-9	218	196	147	131	117	106	95	86	78	71	64	58	53	48	44
	3VLI20	9'-2	11'-7	12'-0	232	209	189	141	127	114	103	93	84	77	70	63	58	53	48
(t=3 1/2")	3VLI19	10'-4	12'-10	13'-3	263	236	213	193	176	131	119	108	98	89	81	74	68	62	57
	3VLI18	11'-4	13'-8	14'-2	346	316	289	267	247	230	215	170	158	147	138	129	121	113	107
48 PSF	3VLI17	12'-2	14'-7	15'-0	372	338	310	285	263	245	228	214	201	158	147	138	129	121	114
	3VLI16	12'-11	15'-3	15'-7	393	360	332	308	286	268	251	236	223	211	169	159	150	142	134
7 1/4"	3VLI22	6'-7	8'-3	8'-3	196	173	153	137	122	110	99	89	81	73	66	60	55	49	45
	3VLI21	7'-10	9'-9	9'-9	216	190	169	151	135	121	109	99	90	81	74	67	61	55	50
	3VLI20	8'-8	11'-1	11'-2	267	240	182	163	146	131	118	107	97	88	80	73	66	61	55
(t=4 1/4")	3VLI19	9'-9	12'-2	12'-7	302	271	244	222	168	151	137	124	112	102	93	85	78	71	65
	3VLI18	10'-8	13'-0	13'-6	398	362	332	306	284	264	211	196	182	169	158	148	139	130	123
55 PSF	3VLI17	11'-6	13'-10	14'-4	400	388	355	327	302	281	262	245	195	181	169	158	148	139	131
	3VLI16	12'-2	14'-7	15'-1	400	400	381	353	329	307	288	271	256	207	194	183	173	163	154

Notes:
1. Minimum exterior bearing length required is 2.5 inches. Minimum interior bearing length required is 5.0 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 200 psf. Such loads often result from concentrated, dynamic, or long term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper live load limit of 250 psf.
4. Inquire about material availability of 17, 19 & 21 gage.

COMPOSITE