

Science Center  
Research Park  
3711 Market St.  
Philadelphia, PA

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

Prepared by:  
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April 7, 2010



**[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 Angeles, 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/](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:**

Robert W. McCauley

### **Keast & Hood Co.:**

Constantine G. Doukakis

### **The Pennsylvania State University:**

Prof. M. K. Parfitt

Dr. Ali M. Memari

Dr. Andres Lepage

Prof. Robert Holland

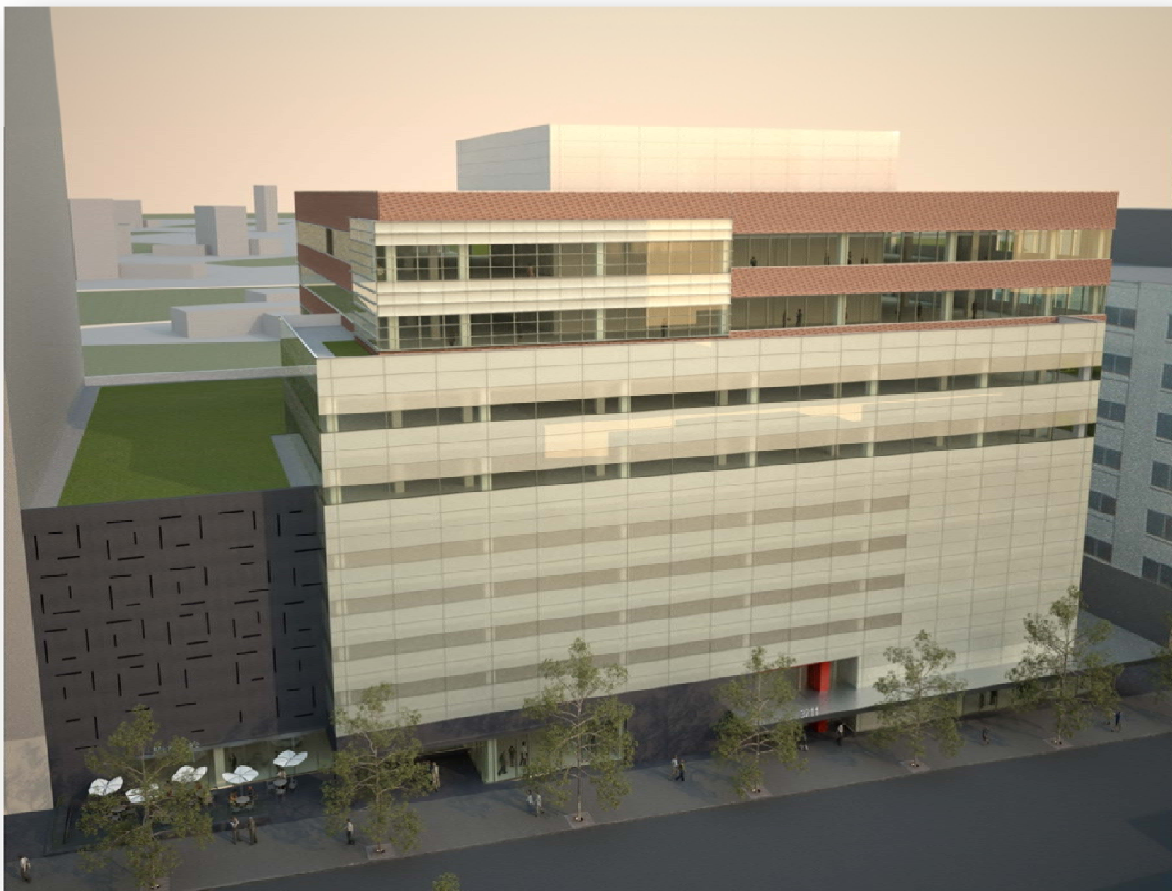
The entire AE faculty and Staff

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.

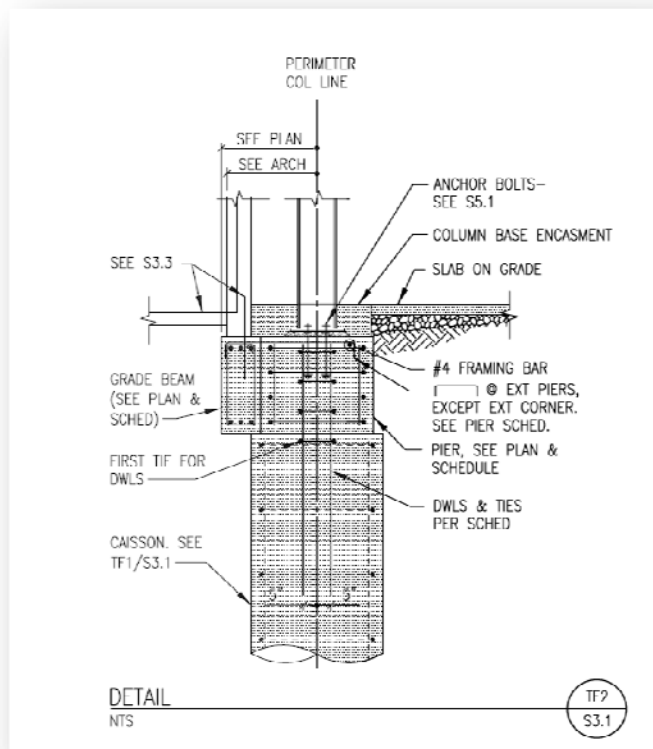
### **Telecommunications:**

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 6 inches 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 3/4" 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.



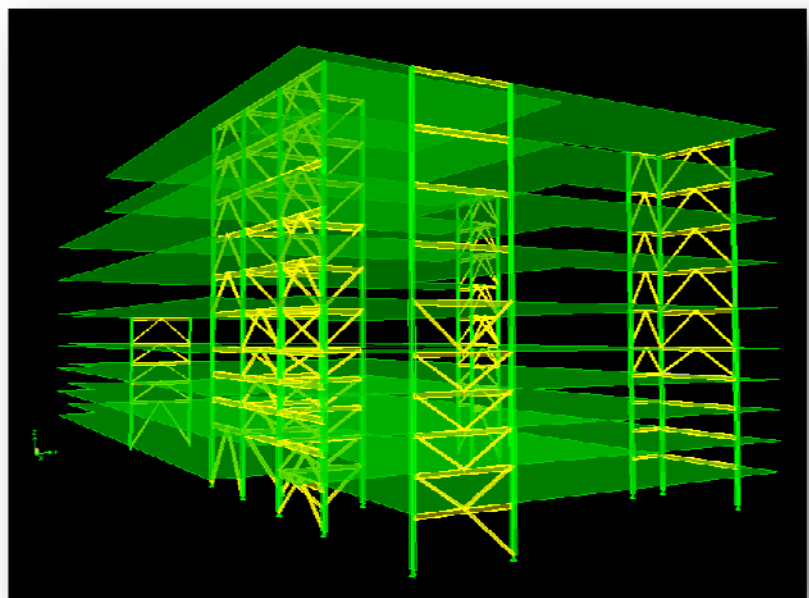


*Figure 6: Existing Parking Garage*

## **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*



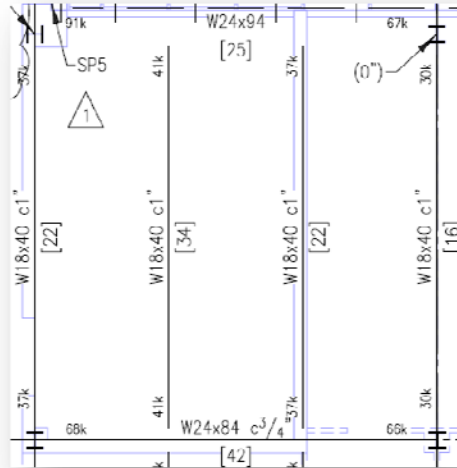


Figure 8: Typical Interior Bay

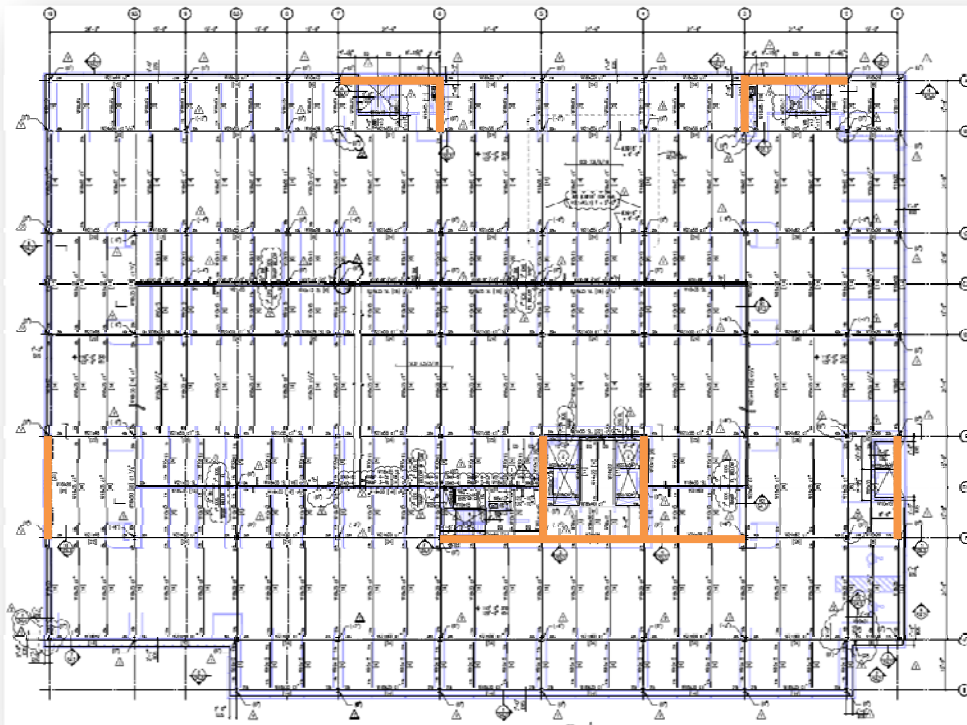


Figure 9: Typical Floor Framing Layout and Lateral Bracing Layout

# 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  $0.015h_x$  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 13<sup>th</sup> 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.  $1.4(\text{Dead})$
2.  $1.2(\text{Dead}) + 1.6(\text{Live}) + 0.5(\text{Roof Live})$
3.  $1.2(\text{Dead}) + 1.6(\text{Roof Live}) + (1.0(\text{Live}) + 0.8(\text{Wind}))$
4.  $1.2(\text{Dead}) + 1.6(\text{Wind}) + 1.0(\text{Live}) + 0.5(\text{Roof Live})$
5.  $1.2(\text{Dead}) + 1.0(\text{Seismic}) + 1.6(\text{Wind})$
6.  $0.9(\text{Dead}) + 1.6(1.6(\text{Wind}))$
7.  $0.9(\text{Dead}) + 1.0(\text{Seismic})$

*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)
<b>Windward</b>	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	
<b>Leeward</b>	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
<b>Total</b>	<b>147.5</b>				<b>419</b>	<b>305</b>		

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

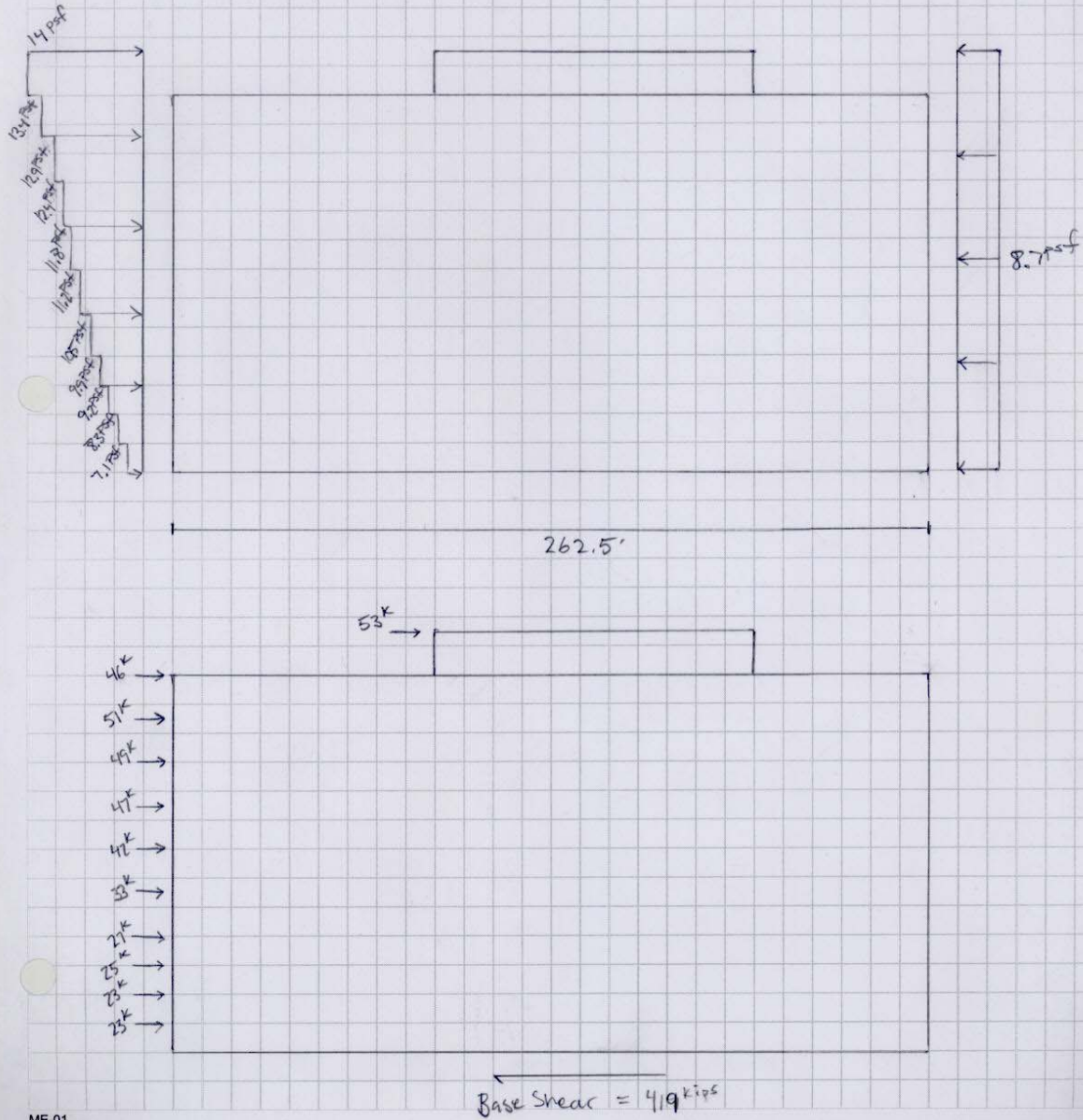


CALCULATION SHEET

PAGE 6 OF

CLIENT \_\_\_\_\_ SUBJECT Senior Thesis Prepared By \_\_\_\_\_ Date \_\_\_\_\_  
PROJECT No. Wind Analysis Reviewed By \_\_\_\_\_ Date \_\_\_\_\_

Wind Pressure  
North-South Elevation



ME-01

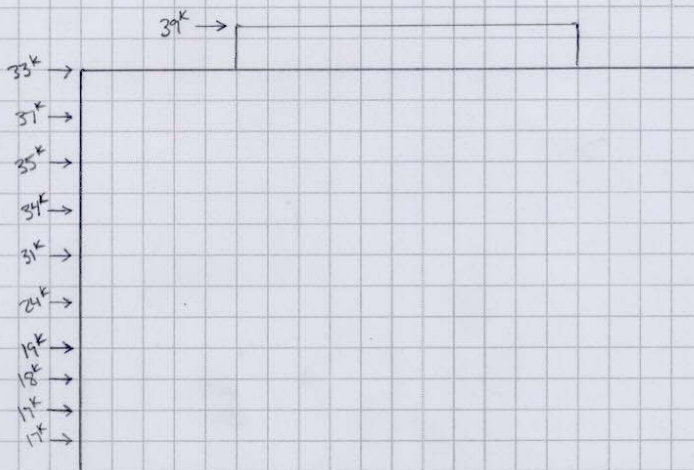
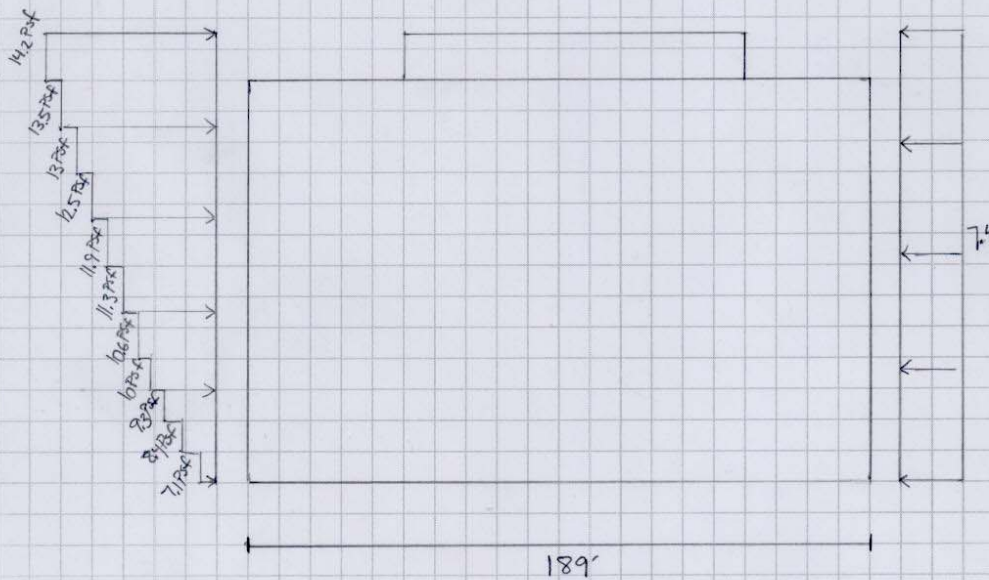


CALCULATION SHEET

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CLIENT \_\_\_\_\_ SUBJECT Senior Thesis Prepared By \_\_\_\_\_ Date \_\_\_\_\_  
PROJECT No. Wind Analysis Reviewed By \_\_\_\_\_ Date \_\_\_\_\_

Wind Pressure  
East-West Elevation



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 <sup>2</sup> )	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 <sub>penthouse level</sub>	21,509	14.6	314031.4		7.33	2960	21706.67
Roof	6,437	14.6	93980.2		7.33	728	5338.67
<b>Total</b>			<b>26056785.4</b>				<b>806244.42</b>

*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
Roof <sub>penthouse level</sub>	7180.00					
Roof	92547.00				369432	
<b>Total</b>	1910175			1569474.9	3166128	42815
<b>Total Building Weight</b>		33551.62 kips				

*Table 4: Building Weight Tabulation (continued)*

### Seismic Load Tabulation

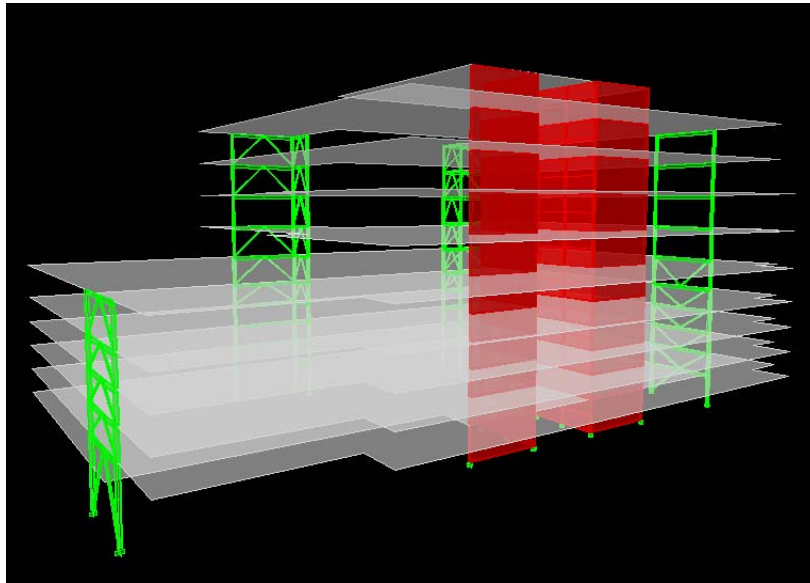
Building forces including story and base shears were calculated after the tabulation of the building weights. These forces are shown in the table below.

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
Roof <sub>penthouse level</sub>	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*

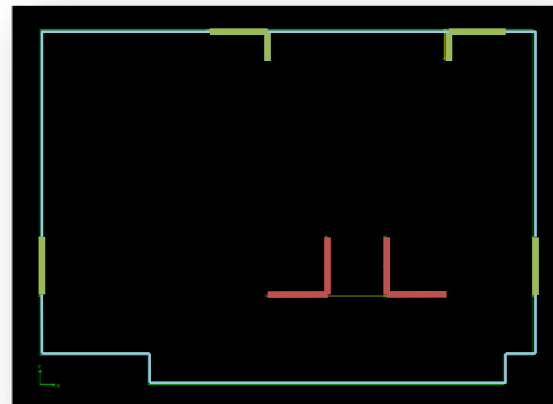
The wind calculations do not include the 1.6 factor, but even after utilizing the 1.6 factor the seismic loads still control the design of the lateral system.

## 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.



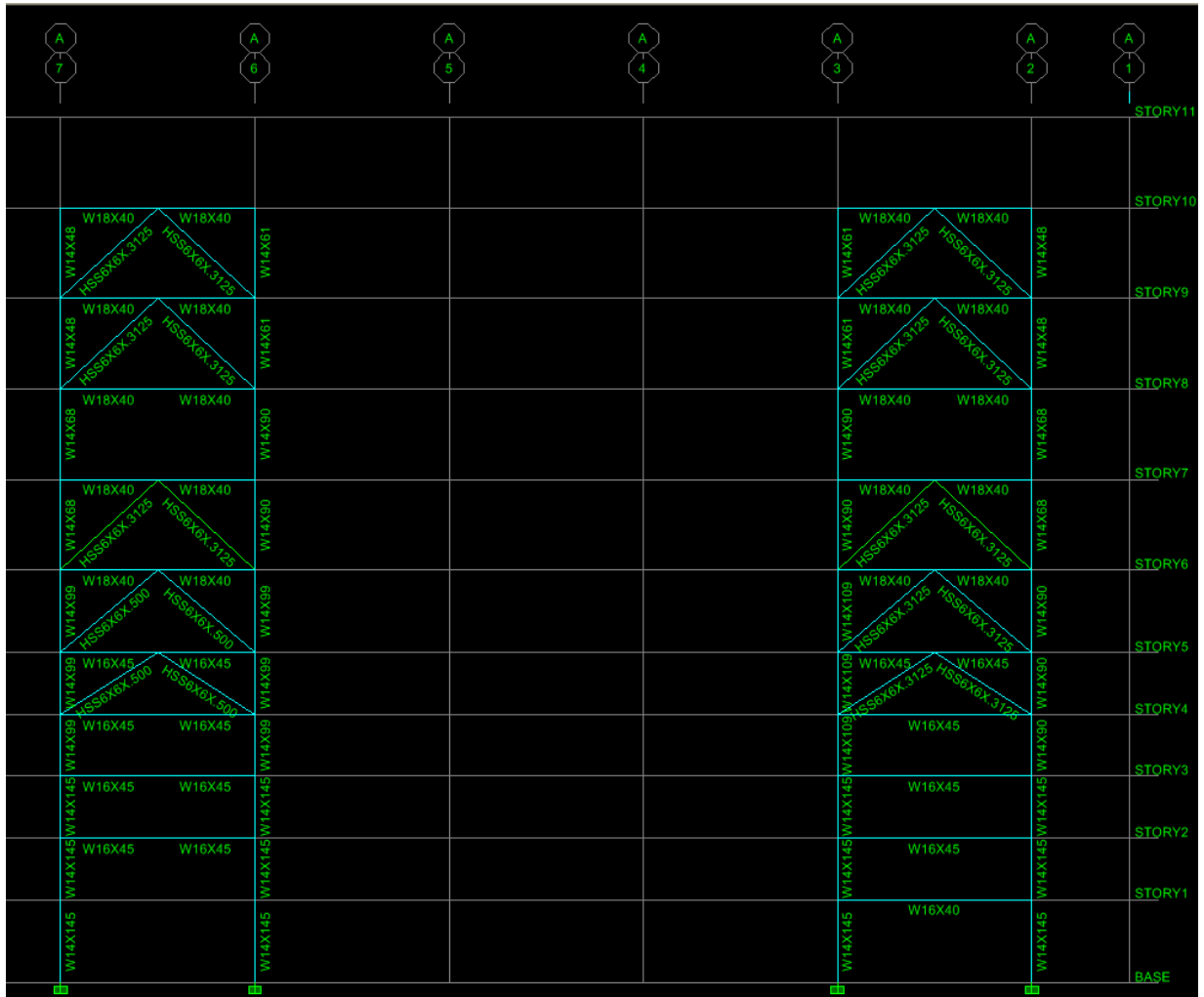
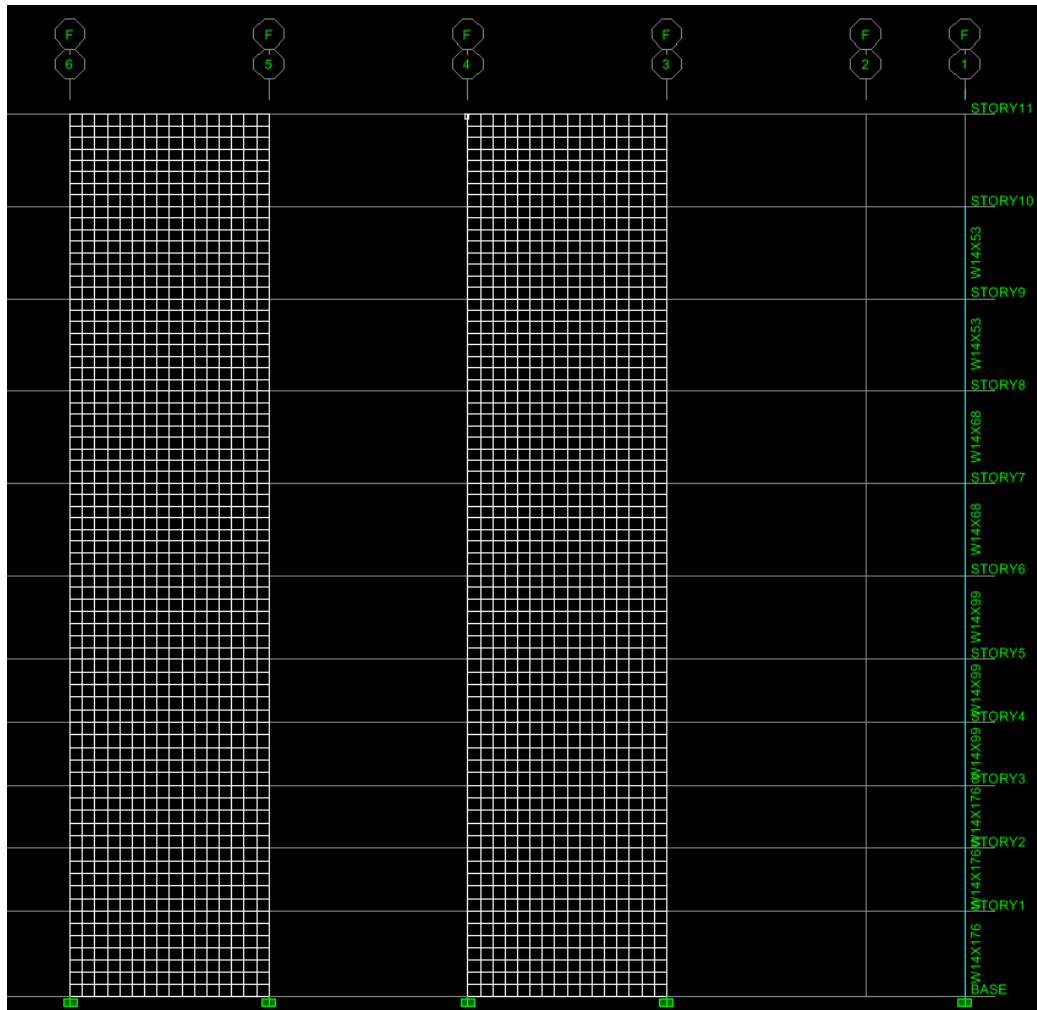
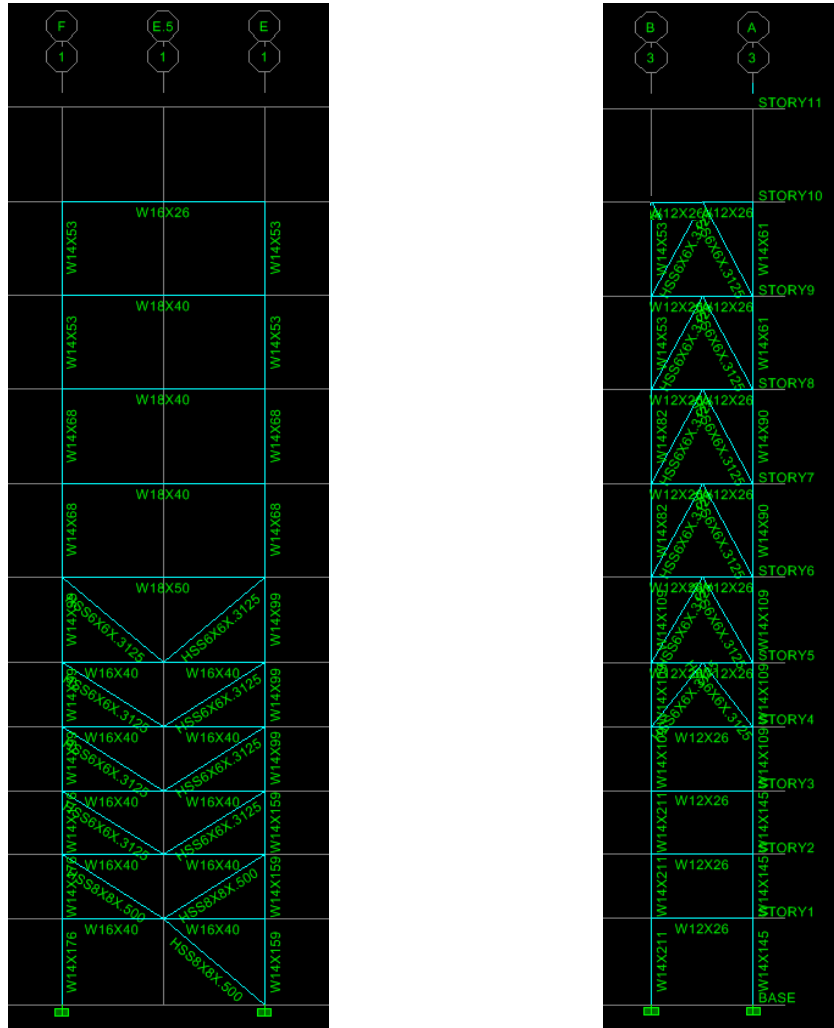


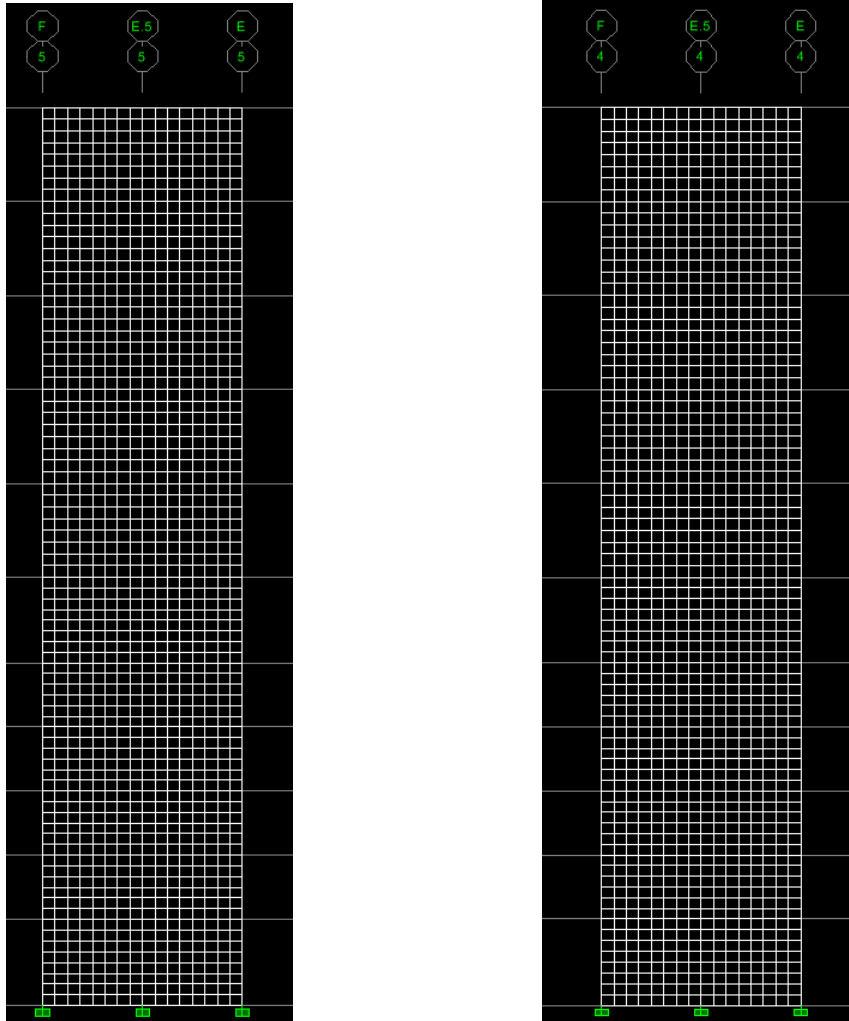
Figure 12: Brace Frames A2, 3 and A6, 7



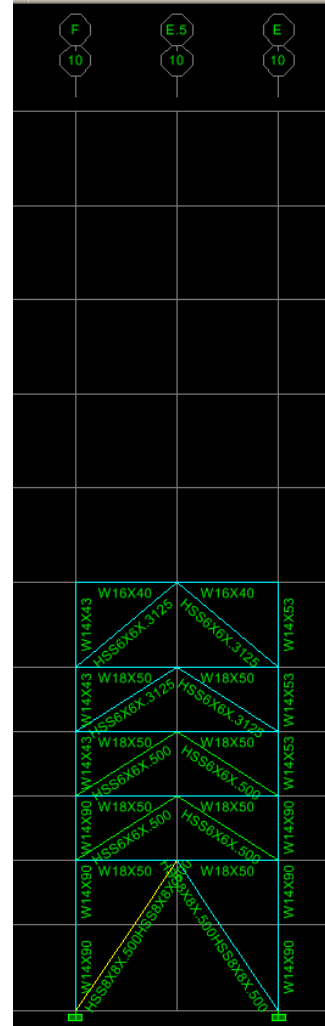
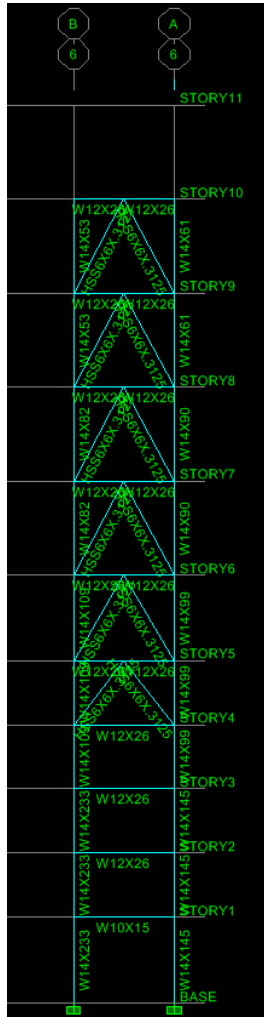
*Figure 13: Shear Walls F3, 4 and F5, 6*



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***

Overturing 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:  $\Delta_{\text{seismic}} = 0.015h_{\text{sx}}$  (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 $\Delta_{\text{seismic}} = 0.015 \cdot h_x$ (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 $\Delta_{\text{seismic}} = 0.015 \cdot h_x$ (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	
		<b>Total Steel Cost:</b>	<b>\$203,274.76</b>
		<b>Total Concrete Cost:</b>	<b>\$114,525.47</b>
<b>Total Structural Steel Cost:</b>	<b>\$500,276.86</b>	<b>Total Lateral System Cost:</b>	<b>\$317,800.23</b>
<b>Total Difference in Cost:</b>	<b>\$182,476.63</b>		

*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 be 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**.

$$LR = 2 \times 1.8 \times NFL$$

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 <sup>2</sup> -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
<b>Total</b>				0.767
<b>U=</b>			1.304	W/m <sup>2</sup> -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.